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October 9, 2018
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Revision 0

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

**RE: Seismic Impact Zones
Ash Pond 2
EPA Final Coal Combustion Residuals (CCR) Rule
TVA Shawnee Fossil Plant
West Paducah, Kentucky**

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 Ash Pond 2 at the TVA Shawnee Fossil Plant (SHF) 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, Don W. Fuller II, being a Professional Engineer in good standing in the State of Kentucky, do hereby certify, to the best of my knowledge, information, and belief:

1. that the information contained in this certification is prepared in accordance with the accepted practice of engineering;
2. that the information contained herein is accurate as of the date of my signature below;
and
3. that the TVA Shawnee Ash Pond 2 meets the requirements specified in 40 CFR § 257.63(a).

Re: **Seismic Impact Zones**
Ash Pond 2
EPA Final Coal Combustion Residuals (CCR) Rule
TVA Shawnee Fossil Plant
West Paducah, Kentucky

SIGNATURE 

DATE 10/9/2018

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ATTACHMENTS: Seismic Impact Zones Demonstration



Seismic Impact Zone Demonstration

Ash Pond 2
Shawnee Fossil Plant
Paducah, McCracken County,
Kentucky



Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

Prepared by:
Stantec Consulting Services Inc.
Lexington, Kentucky

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SEISMIC IMPACT ZONE DEMONSTRATION – SHF ASH POND 2

Introduction
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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 Ash Pond 2 at the Shawnee Fossil Plant (SHF) 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 by October 17,2018 to demonstrate that the unit is not located in a seismic impact zone unless the unit meets certain requirements.

1.2 UNIT DESCRIPTION

SHF is a coal-fired, electric-generating plant. The plant is located in McCracken County, Kentucky, along the south shore of the Ohio River near river mile 946, just east of the confluence of Little Bayou Creek with the Ohio River.

Ash Pond 2 is approximately 100 acres in area and is enclosed by a perimeter dike system that is approximately 9,200 feet in total length. In 1971, the first phase of Ash Pond 2, consisting of dikes constructed to a crest elevation of 340 feet, was constructed and put into service. In 1979, the dikes were raised 10 feet using upstream construction methods (constructing inwardly over sluiced ash). The overall constructed height of the perimeter dike system now varies from approximately 20 to 25 feet. Dike slopes are approximately 2.5H:1V to 3H:1V (Stantec 2010a).

Ash Pond 2 at SHF meets the EPA definition of a CCR surface impoundment because it is a manmade area designed to hold an accumulation of CCR and liquids and is used to treat, store, or dispose of CCR.

During 2012, the existing unsupported riser spillway in Ash Pond 2 was replaced by new concrete stoplog spillways. The new spillways were constructed of cast-in-place concrete with high-density polyethylene (HDPE) outlet pipes installed through the embankment. These pipes discharge through a concrete headwall at the downstream end. The pipes are encased in concrete through the crest of the embankment and a sand filter diaphragm is installed just downstream of the encasement. These measures serve to counteract potential seepage through the embankment along the outlet pipes. Following operation of the new spillways, the existing spillway pipes were abandoned by cleaning, inspecting, and filling pipe sections beneath the perimeter dike using flowable sand-cement grout (Stantec 2012).

SEISMIC IMPACT ZONE DEMONSTRATION – SHF ASH POND 2

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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.*

While the word "resist" in the above language is not defined in the EPA Final Rule, 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. at 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.

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3.0 DEMONSTRATION

Ash Pond 2 at SHF 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 the 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 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 Shawnee 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 7.53 M_w (moment magnitude) event with a peak horizontal acceleration of 0.756g 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 Ash Pond 2 is summarized in the table below. A more in-depth review is found in Stantec (2010a, 2016). Specific details of subsurface conditions at the spillway structures are included with the geotechnical analysis in Appendix A.

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Table 1. Generalized Subsurface Conditions (Stantec 2016)

Materials	Approximate Elevation	General Consistency/Density
Upper Dike fill – lean clay, lean clay with sand, and sandy lean clay	El. 340 to El. 351	Stiff to very stiff with isolated medium zones
Lower Dike fill – lean clay, lean clay with sand, poorly graded sand with silt	Native Ground Surface to El. 340	Medium stiff to very stiff / medium dense
Native clay and/or native silt – lean clay, lean clay with sand, sandy lean clay, sandy silt, and silt with potential interbedded lower sands	Below Lower Dike	Soft to very stiff
Native sand and Native Sand and Gravel - poorly graded sand, poorly graded sand with silt, poorly graded sand with gravel, poorly graded sand with silt and gravel, silty sand, well grade gravel with silt and sand	Below native clays and silts	Medium dense to very dense with isolated very loose to loose zones
Sluiced fly ash and bottom ash – silty sand with gravel and silt with sand	Between Upper Dike and native soils (upstream construction)	Very soft to medium stiff / loose to medium dense

Relative to the height of the perimeter dike system and the thickness of the foundation soils, the depth to rock is significant (in excess of 300 feet deep). As such, the location of the top of bedrock has no influence on the slope stability failure mode of interest.

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 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.

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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 displacement resistance due to the spillway structures.

The results indicate stable slopes for the post-earthquake conditions, while permanent displacements (ie.e. deformation) are expected for the pseudostatic condition. The predicted displacements that would impact the spillway inlet structures and/or spillway pipes are roughly two feet or less. 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.

The stability of the outlet structure was evaluated for overturning and sliding failure mechanisms. The results indicate the structure is stable for overturning failures. The structure is stable for sliding failures using some resistance (less than 20%) of the tensile capacity of the outlet pipes in the analyses. The outlet structure has six 30" DR17 HDPE pipes connected by a flange structure embedded in the structure, with each pipe encased in concrete below the crest of the embankment. Because the concrete structure is relatively light, and the outlet pipe system is robust, utilizing some sliding resistance from the outlet structures is a reasonable assumption to predict actual performance during the design seismic event. Structural calculations are included in Appendix B.

3.6 ANALYSES DISCUSSION

The geotechnical analyses indicate that deformations of the perimeter dike at the outlet structure of up to two feet are expected. The structural analyses indicate that the outlet structure is unlikely to rotate or slide during the seismic event, assuming some resistance from the outlet pipes which are connected to the structure. Due to the expected deformation of the embankment, the outlet pipes may warp or pinch, but this would not cause release of additional water above what is released during normal operations of the pond. The analyses show that the outlet structure should still be connected to the outlet pipes during and following the seismic event so an uncontrolled release of water is not expected. While there is a potential for minor damage to the system, it is not anticipated to result in a harmful release of CCR under current operational conditions.

Additionally, this facility has an Emergency Action Plan (EAP) that will be activated following a seismic event. Inspection of the facility and outlet structures will be performed. Minor damage to

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the structure will be identified and remediation measures will be implemented to repair the damage and modify operations, if necessary. The implementation of the EAP will reduce the potential for a progressive failure and harmful release of CCR.

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4.0 CONCLUSION

Based on this assessment, Ash Pond 2 located at SHF meets the requirements of §257.63 of the EPA Final CCR Rule for seismic impact zone.

SEISMIC IMPACT ZONE DEMONSTRATION – SHF ASH POND 2

References
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5.0 REFERENCES

Geocomp (2016). "Initial Seismic Safety Factor Assessment, EPA Final CCR Rule, TVA Shawnee Fossil Plant Ash Pond 2, West Paducah, Kentucky," Prepared for Tennessee Valley Authority, October.


Stantec Consulting Services Inc. (2010a). "Report of Geotechnical Exploration and Slope Stability Evaluation, Ash Pond 1 & 2 and Consolidated Waste Dry Stack, Shawnee Fossil Plant, Paducah, Kentucky," Prepared for Tennessee Valley Authority, July.

Stantec Consulting Services Ins. (2010b). "Plans for Construction. Spillway Replacement Project, Ash Disposal Area No. 2, Pond B (Ash Stilling Pond), Work Plan 5 (SHF-1005-4-WP-5). Shawnee Fossil Plant, West Paducah, McCracken County, Kentucky," Prepared for Tennessee Valley Authority, May.

Stantec (2012). "Construction Certification Report, Ash Disposal Area No. 2, Spillway Replacement Project, Work Plan 5 (SHF-100504-WP-5), Shawnee Fossil Plant, McCracken County, Kentucky" Prepared for Tennessee Valley Authority. January 6.

Stantec (2016). "Initial Static Safety Factor Assessment, Ash Pond 2, EPA Final CCR Rule, TVA Shawnee Fossil Plant, McCracken County, Kentucky," Prepared for Tennessee Valley Authority, October.

APPENDIX A GEOTECHNICAL ANALYSES

Tennessee Valley Authority Shawnee Fossil Plant McCracken County, Kentucky	Seismic Impact Zones Slope Stability Analysis
<i>Calculation Package</i> N/A	
	<p style="text-align: center;">Exhibit Seismic Impact Zones</p>
<p><u><i>Purpose:</i></u></p> <ul style="list-style-type: none"> • Evaluate the slope stability of the dike at the spillway location for Ash Pond 2, considering pseudostatic and post-earthquake load conditions. 	
<p><u><i>Methods:</i></u></p> <ul style="list-style-type: none"> • Liquefaction screening • Spencer’s method of slices • Newmark analysis (simplified seismic displacement analysis) 	
<p><u><i>Results:</i></u></p> <ul style="list-style-type: none"> • 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 2 feet. 	
<i>Calculation Performed by:</i> Stantec Consulting Services Inc.; March 20, 2018	
<i>Prepared by:</i> Enrique Farfan, PhD, PE	<i>Reviewed by:</i> Jeffrey S. Dingrando, PE, PG
<i>Revisions:</i> N/A	

TVA SHAWNEE ASH POND 2

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 Ash Pond 2 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 (see 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 Ash Pond 2 spillway structure.

2. SUMMARY OF DRILLING AND LAB TESTING RESULTS

During May and June of 2017, three standard penetration borings test (SPT) and four seismic cone penetration tests (SCPTu) were performed near the spillway structure located at the north-west side of the Ash Pond 2 (see Plate 1 in Attachment A). As-drilled boring locations were surveyed by TVA. CPT locations were approximated relative to the boring locations. Table 1 summarize the coordinates for the listed borings.

Boring logs, boring layout, CPT results, 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 and CPT Locations (Stantec, 2018)

Boring	Total Depth (ft)	Coordinates		Surface Elevation (NGVD29) (ft)
		Northing (Plant Local) (ft)	Easting (Plant Local) (ft)	
SHF-SS-1	55.5	317,233.7	1,112,535.7	351.7
SHF-SS-2	43.0	317,362.0	1,112,617.3	352.5
STN-AQ-207	39.5	317,365.9	1,112,732.4	351.0
CPT-1	55.9	317,356*	1,112,615*	351.6*
CPT-2	45.4	317,231*	1,112,534*	352.5*
CPT-3	56.8	317,369*	1,112,731*	351.0*

CPT-4	9.7	317,392*	1,112,714*	350.0*
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* Estimated

Selected soil samples from borings SHF-SS-1, SHF-SS-2 and STN-AQ-207 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	18
ASTM D 4318 Method A – Atterberg Limits	17
ASTM D 422 – Particle Size Analysis (sieve + hydrometer)	17

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 2010a) that were considered in the preliminary slope stability analysis (see Plate 1). A list of the existing borings is presented in Table 3.

One important finding of the drilling and laboratory testing program was the characterization of the material designated “Fly Ash – Sluiced”. In the 2017 borings, in the vicinity of the spillways and beneath the adjacent divider dike, this horizon was typically found to consist of a mixture of lean clay, fly ash, and bottom ash. Although this finding is somewhat unexpected, it is consistent with 2010 boring logs in the same vicinity. Laboratory index tests typically classify this material as lean clay, which has important implications with regard to liquefaction potential when compared to typical fly ash (non-plastic silt). See Section 5 for more information regarding liquefaction potential.

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

Boring	Total Depth (ft)	Coordinates		Ground Surface Elevation (ft)
		Northing (ft)	Easting (ft)	
STN-1-SP	24.0	317,284.7*	1,112,573.7*	351.7
STN-2-SP	26.5	317,293.2	1,112,579.6	351.5
STN-3-SP	26.5	317,302.4	1,112,586.2	351.7
STN-4-SP	26.5	317,251.8	1,112,552.3	351.7

* Estimated

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. Seven 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.

- Spillway Replacement Project (Stantec 2012) Profile - Baseline B (Drawing No. 10W505-08).
- Geotechnical Exploration (Stantec 2010b) Stability Section B-B' (Drawing No. 10W501-05).
- Subsurface information available from Borings STN-4 & 5, STN-1-SP to STN-3-SP (Drawing No. 10W505-14) included in the Spillway Replacement Project (Stantec 2012).

The geometry of the cross section developed for this analysis is depicted in Plates 4 and 5 along with SPT and CPT data.

4. MATERIAL PARAMETERS

The soil parameters for this analysis were developed based on the site-specific seismic assessment performed by Geocomp (2015) for Ash Pond 2, the slope stability evaluation performed by Stantec (2010b) for Ash Pond 2, 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 (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. Unsaturated soils (e.g., most of the Upper Dike – Lean Clay) do not require this strength reduction, and thus these seismic strengths are equal to the static undrained strengths. The Bottom Ash layer is considered free draining; therefore, a strength reduction was not applied the seismic load cases.

Based on the results of the preliminary slope stability analyses, the seismic strengths of the Upper Dike, Lower Dike, and Native Clay were found to be important for the inboard failure mode. Therefore, the strength parameters for these materials were refined based on the SPT and CPT data obtained from adjacent borings to the spillway structure (see Plate 2). The average SPT field blowcount (N value) for the Upper Dike is approximately 19 and for the Lower Dike is approximately 17, which corresponds to a very stiff soil consistency. The average SPT blowcount for the Native Clay is approximately 11, which corresponds to a stiff soil consistency. CPT tip and sleeve resistance and shear wave velocities for the Native Clay suggest a very stiff soil consistency. The refined soil strengths were derived using correlations between undrained shear strength and SPT blowcounts (NAVFAC, 1982). The relative lack of new field data in the Fly Ash – Sluiced prevented a refinement of its shear strength, so the strength from the preliminary analysis was used.

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. The use of the bilinear envelope is conservative, but at low normal stresses it can be overly conservative and can, in fact, control the analyses for shallow slope failures such as those considered herein. As such, for the Upper Dike, Lower Dike, and Native Clay, the refined strengths (which are based on data specific to the spillway vicinity) are define in terms of the seismic (i.e., reduced) undrained strengths. The use of a bilinear strength envelope for the Fly Ash – Sluiced may be conservative for this scenario, but the modeled failure mode (see Section 6) is judged to be representative.

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)
Upper Dike-Lean Clay	130	30	200	2000	0
Lower Dike-Lean Clay	130	26	130	2000	0
Native Clay	128	28	110	1600	0
Native Clay (Soft to medium stiff)	128	0	28	500	0
Native Sand	130	0	32	0	32
Fly Ash – Sluiced	85	0	26	400	10
Bottom Ash	105	0	38	0	38

Table 5. Summary of seismic shear strength parameters

Soil Layers	Unit Weight (pcf)	Drain Strength Parameters		Undrained Strength Parameters	
		c' (psf)	ϕ' (degrees)	c_{EQ} (psf)	ϕ_{EQ} (degrees)
Upper Dike-Lean Clay	130	N/A	N/A	2000	0
Lower Dike-Lean Clay	130	N/A	N/A	1600	0
Native Clay	128	N/A	N/A	1280	0
Native Clay (Soft to medium stiff)	128	0	28	400	0
Native Sand	130	0	32	-	27
Fly Ash – Sluiced	85	0	26	337	8
Bottom Ash	105	0	38	-	38

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 (SHF-SS-1, SHF-SS-2 and STN-AQ-207) and CPTs (CPT-1, CPT-2, CPT-3 and CPT-4). The clay-like soils are not susceptible to cyclic softening. An interesting finding of the new borings was the presence of clay-like soils and/or mixtures of clay and bottom ash or fly ash within the horizon that was expected to be sluiced ash. 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 (2015) report, peak ground acceleration (PGA) on rock for Shawnee Fossil Plant is 0.756g (PGA_{ROCK}). A seismic coefficient (k_h) of $\frac{1}{2}$ of PGA_{ROCK} ($k_h = 0.378g$) 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 Ash Pond 2, elevation 344.5 ft.
- The phreatic surface was assumed to vary linearly within the embankment, between the headwater (344.5 ft) and an assumed tailwater elevation of 330 ft. Elevation 330 ft corresponds to the outboard toe of the upper dike fill, which is a conservative assumption. For an inboard slope failure, this conservative assumption has little impact on the results.

- An appropriate tension crack was specified based on the Upper Dike soil properties.
- Any potential resistance from the spillway structure itself is neglected in the slope stability analysis.

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

Table 6. Summary of seismic slope stability results

Load Case	Soil Strengths	Calculated FS
Pseudostatic	Seismic	0.8
Post-Earthquake	Seismic	3.2

The pseudostatic analysis resulted in a factor of safety (FS) less than 1, therefore excessive permanent embankment displacements may be possible at the spillway location. 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 Shawnee Fossil Plant site by Geocomp (2015) 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 0.3g. The accelerations imposed by the design earthquake are then compared to the yield acceleration, Considering the seven time histories, the largest permanent displacement was 1.7 feet (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 3.2 was calculated for the inboard slope stability of the Ash Pond 2 perimeter dike at the spillway structure location. Also considering seismic shear strengths, the pseudostatic factor of safety was less than one; therefore, a Newmark analysis was performed to estimate permanent displacement due to the design earthquake. The Newmark analysis indicates a permanent displacement of 1.7 feet or less. In summary, the expected seismic displacement is roughly 2 feet or less.

9. REFERENCES

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ATTACHMENT A

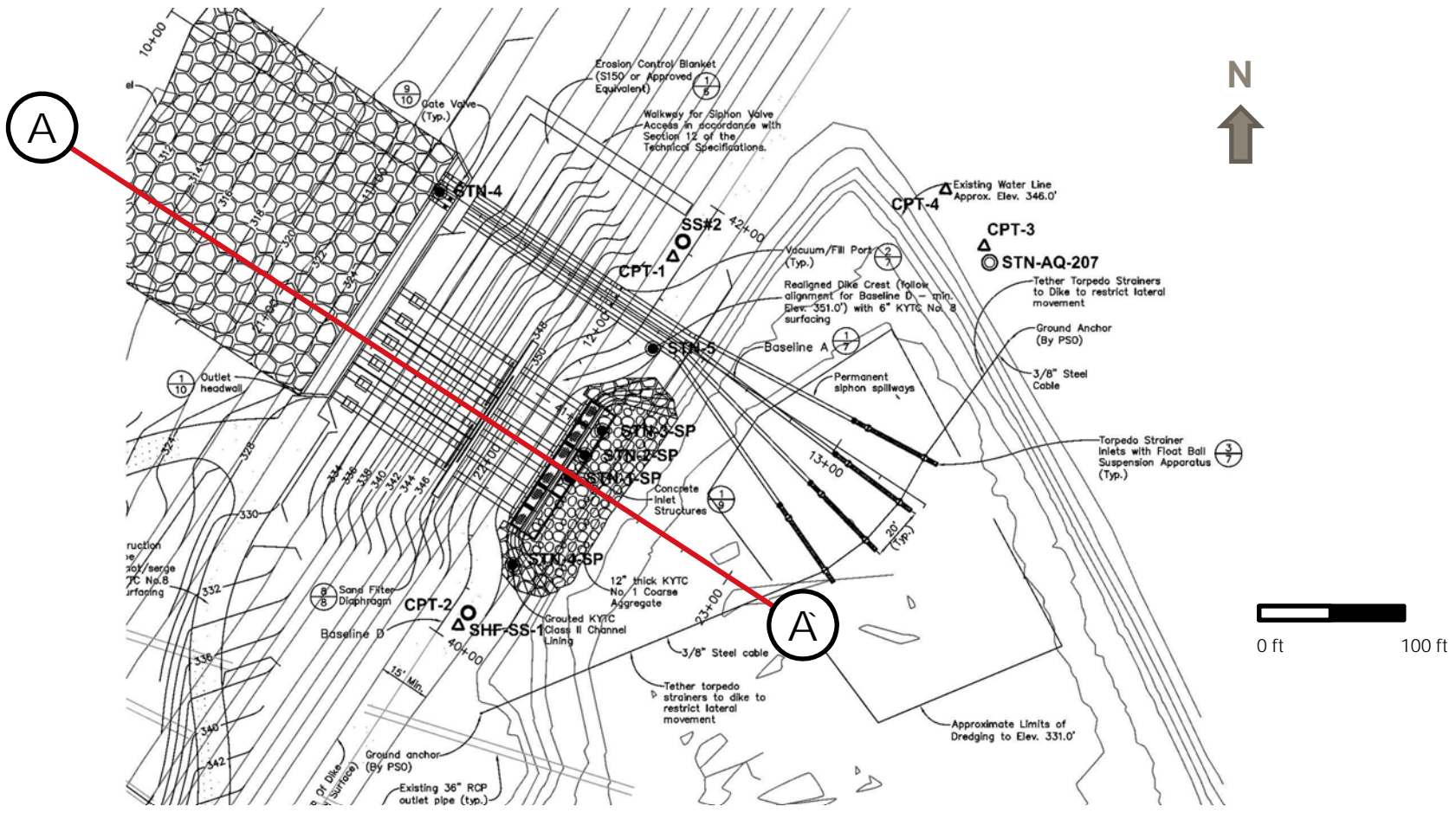
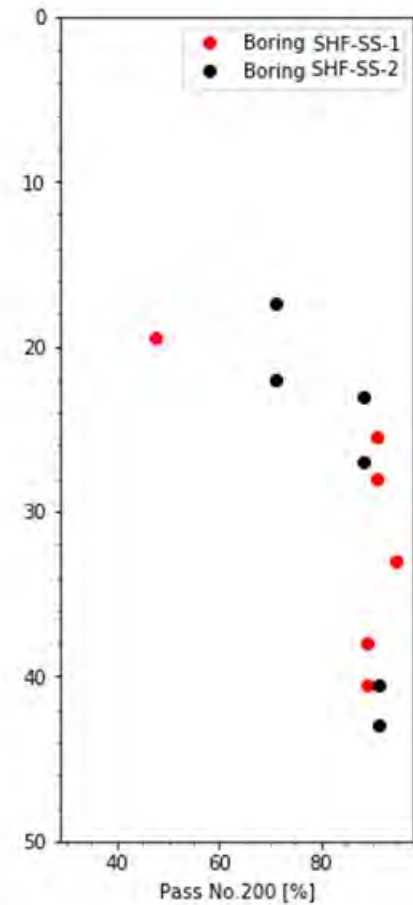
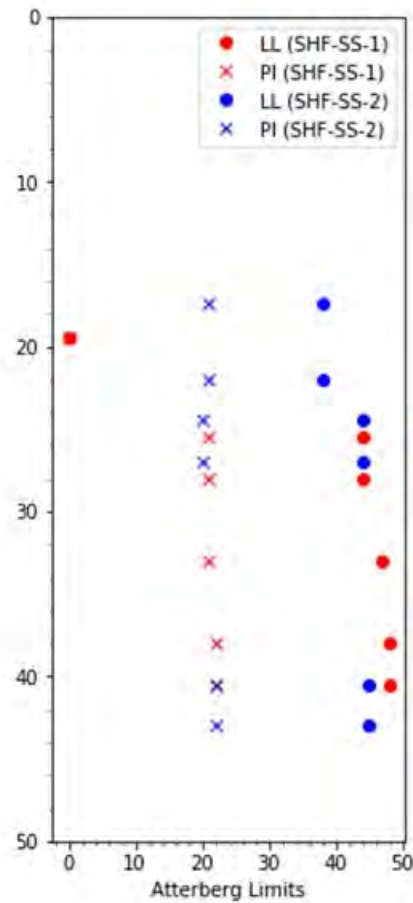
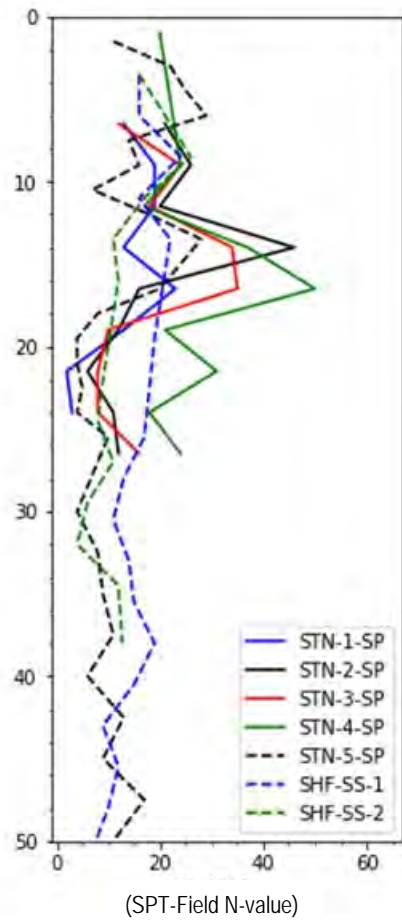
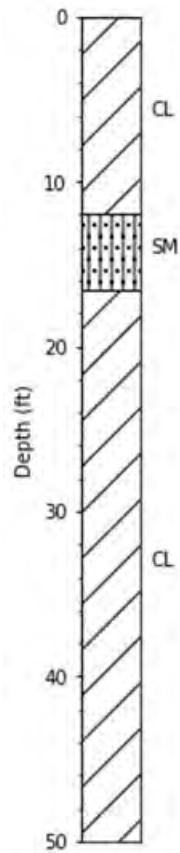


Plate 1
Boring Location Plan

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
 Shawnee Fossil Plant Ash Pond 2
 McCracken County, Kentucky

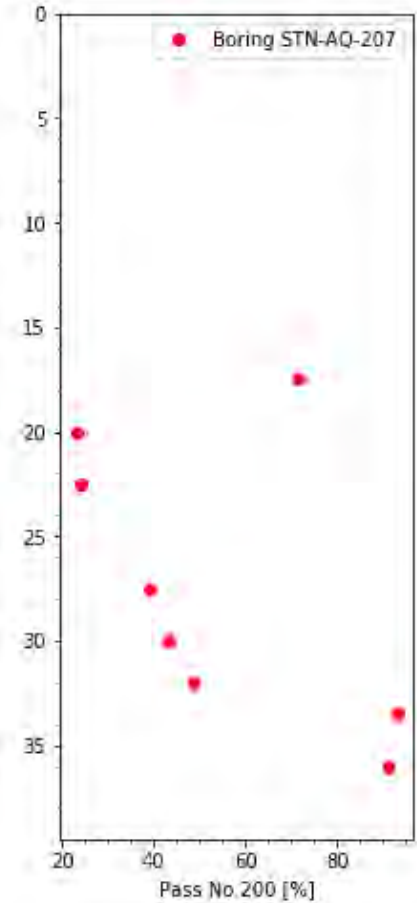
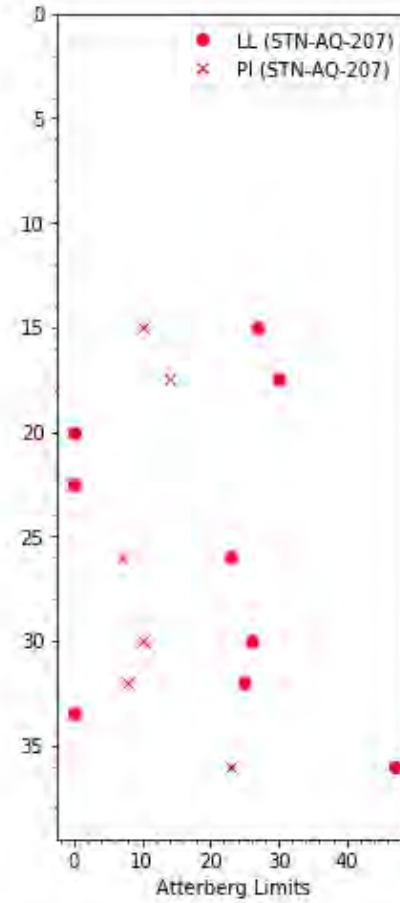
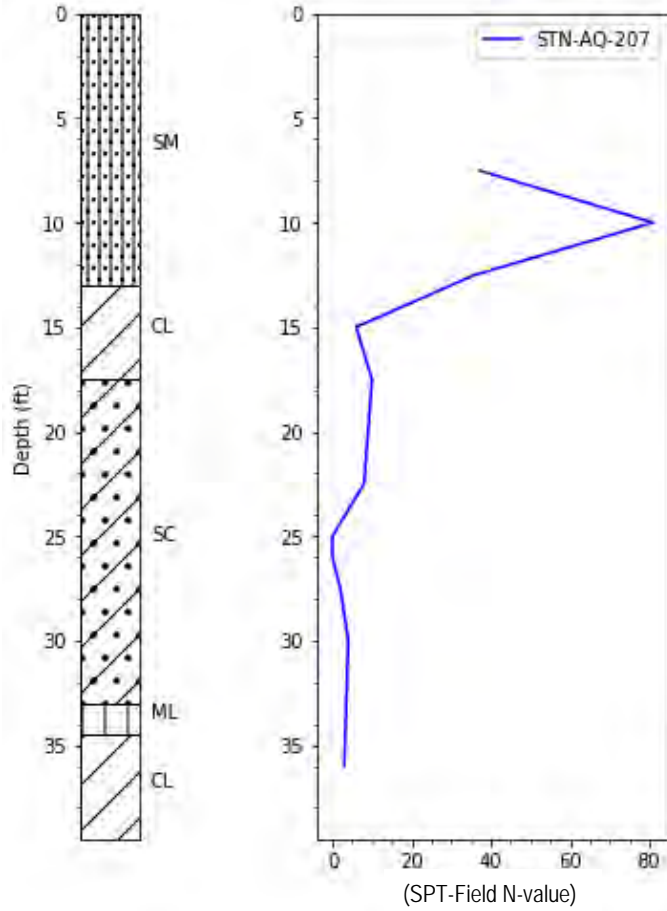
Notes
 1. Borings and CPT locations are approximated





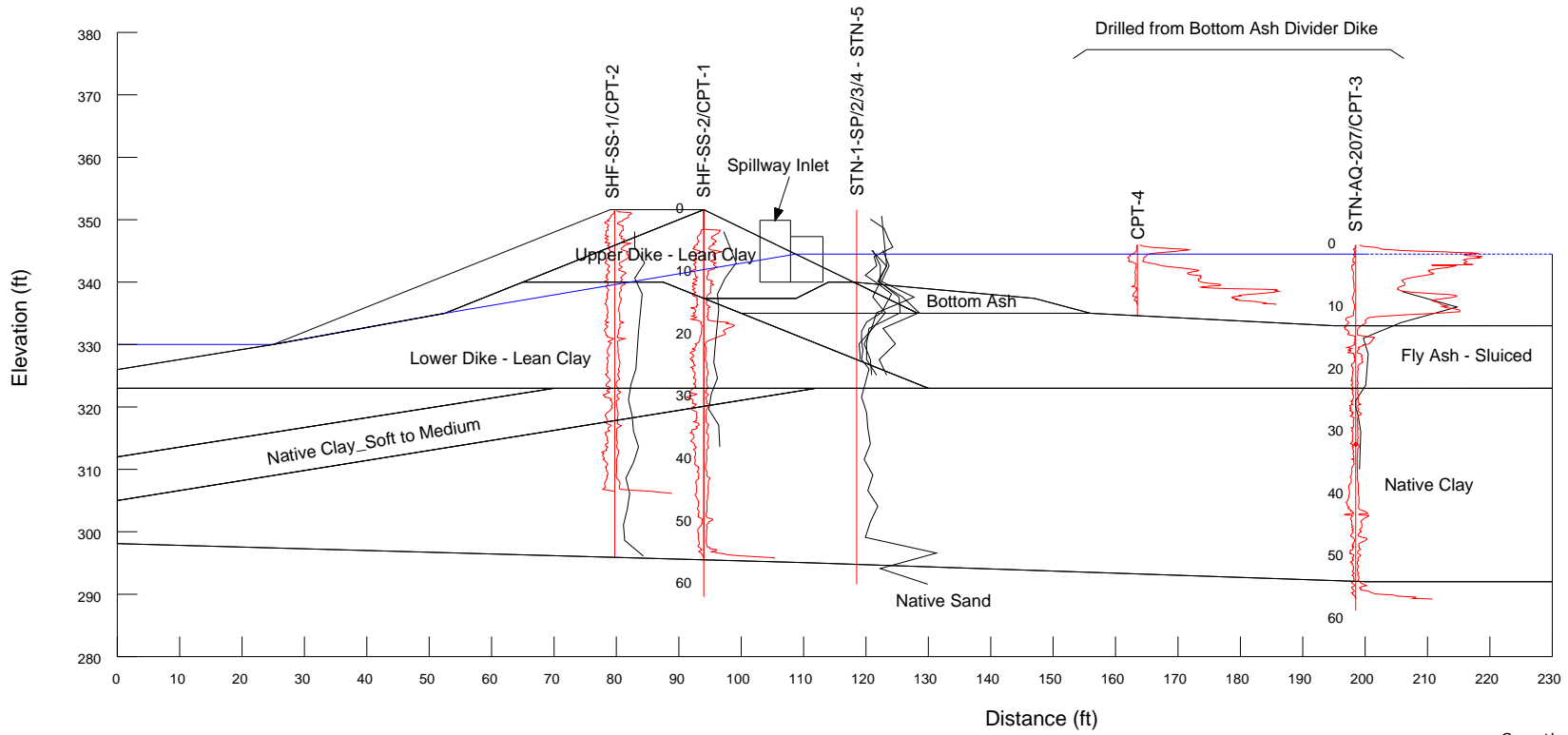
**Plate 2
Soil Data**

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Shawnee Fossil Plant Ash Pond 2
McCracken County, Kentucky



**Plate 3
Soil Data**

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Shawnee Fossil Plant Ash Pond 2
McCracken County, Kentucky



Section A-A

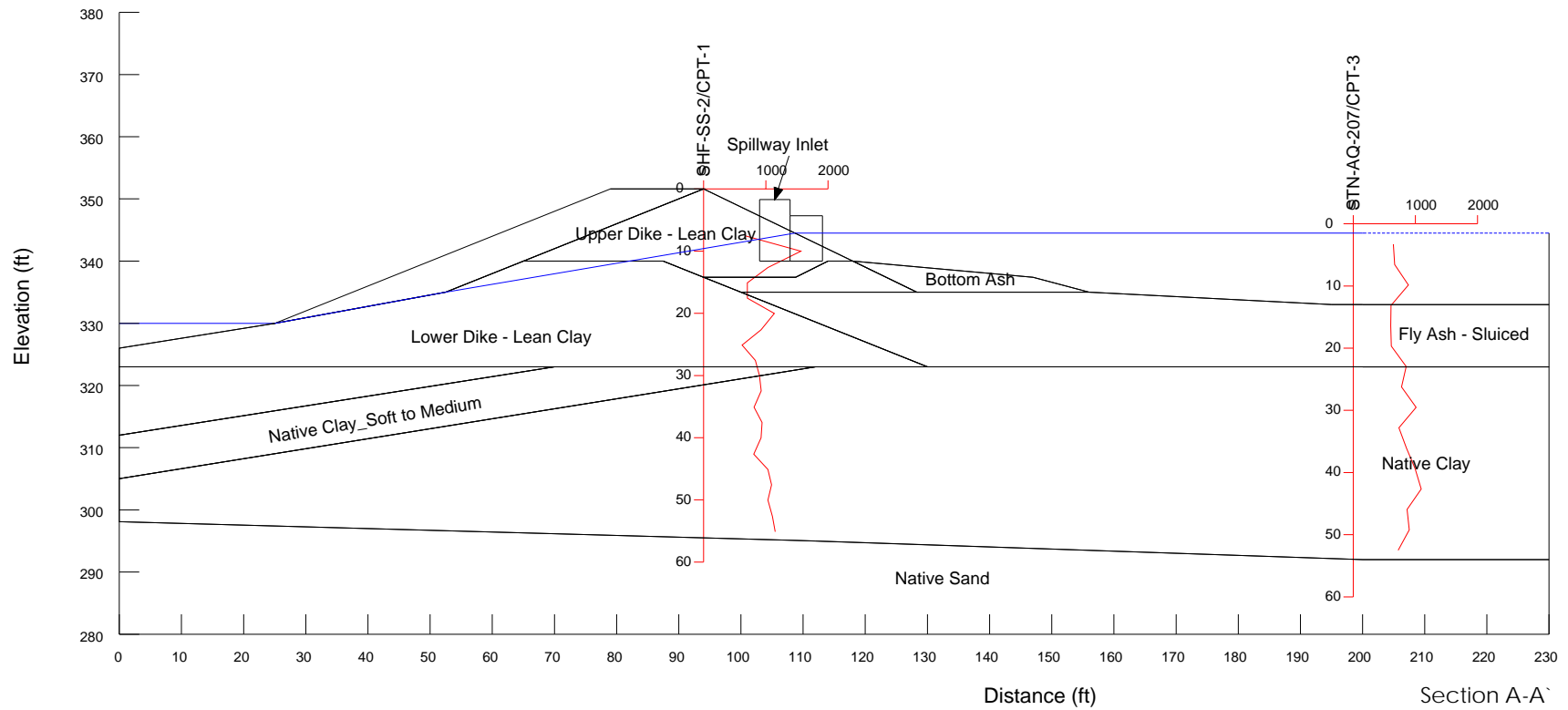
Legend

- Modeled phreatic surface
- SPT blow-counts (N)
- CPT tip resistance (qt)

Plate 4
Embankment Profile with SPT and CPT data

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Shawnee Fossil Plant Ash Pond 2
McCracken County, Kentucky





Legend

- Modeled phreatic surface
- ~ Shear Wave Velocity (m/sec)

Plate 5
Embankment Profile with SPT and CPT data

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Shawnee Fossil Plant Ash Pond 2
McCracken County, Kentucky



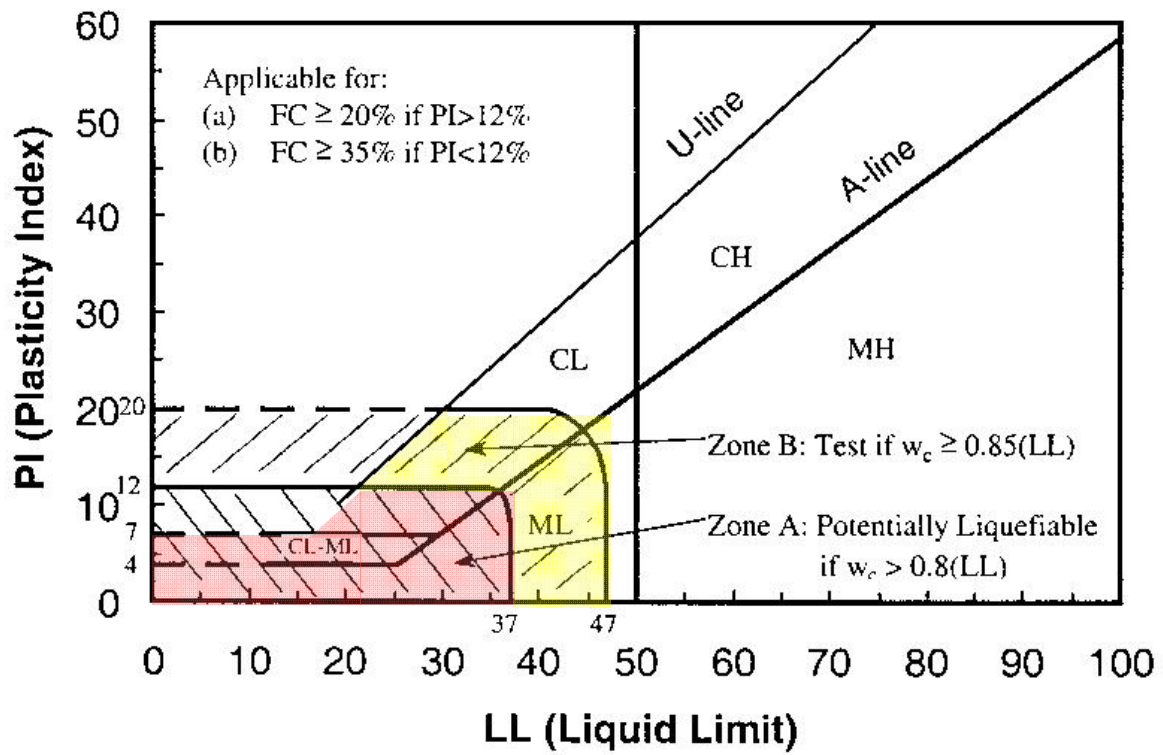
ATTACHMENT B

CLAM - Clay-like Laboratory-Based Assessment of Materials

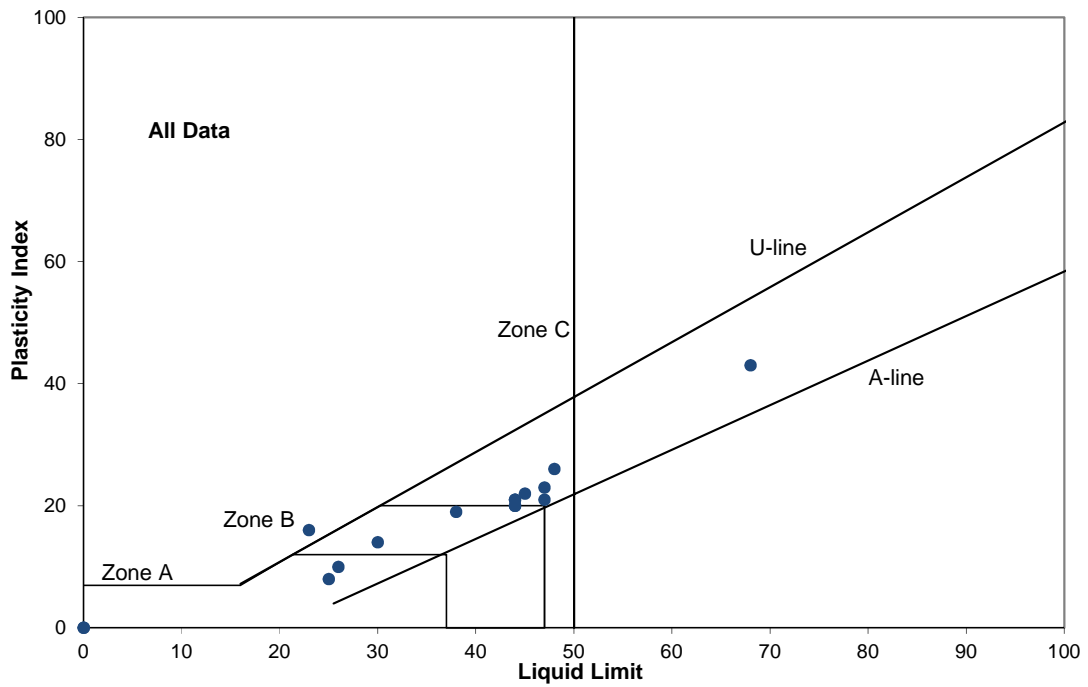
Stantec Project Number:	175567301
Project Name:	TVA Shawnee Ash Pond 2 Seismic Impact Zones: Phase 2
Material:	Native Clay

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)																													
Using Criteria published by Seed et al (2003)																													
Using Criteria published by Idriss and Boulanger (2008)																													
Using criteria published by MSHA (2010)																													
Overall Judgement based on 3 methods (sand-like or clay-like)																													
Elevation at Midpoint (ft)	Lab ID	Boring	Depth(s) (ft)	Soil Classification	NMC (w _c) (%)	% Passing #200	% Passing #40	LL	PI	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)	Overall Judgement based on 3 methods (sand-like or clay-like)
										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					
		SHF-SS-1	25.5	CL	23.3	90.9	97.9	44	21	-1	-1	-1	-1	-1	-1	-1	-1	44	21	-1	21	-1	21	-1	21	-1	Clay-like		
		SHF-SS-1	28	CL	23.3	90.9	97.9	44	21	-1	-1	-1	-1	-1	-1	-1	-1	44	21	-1	21	-1	21	-1	21	-1	Clay-like		
		SHF-SS-1	33	CL	24.6	94.8	99	47	21	-1	-1	-1	-1	-1	-1	-1	-1	47	21	-1	21	-1	21	-1	21	-1	Clay-like		
		SHF-SS-1	38	CL	23.2	85.9	95.9	38	19	-1	-1	38	19	-1	-1	38	19	-1	-1	-1	19	-1	19	-1	19	-1	Clay-like		
		SHF-SS-1	40.5	CL	27	88.9	94.1	48	26	-1	-1	-1	-1	-1	-1	-1	-1	48	26	-1	26	-1	26	-1	26	-1	Clay-like		
		SHF-SS-2	24.5	CL	21	88.2	97.4	44	20	-1	-1	44	20	-1	-1	44	20	-1	-1	-1	20	-1	20	-1	20	-1	Clay-like		
		SHF-SS-2	27	CL	21	88.2	97.4	44	20	-1	-1	44	20	-1	-1	44	20	-1	-1	-1	20	-1	20	-1	20	-1	Clay-like		
		SHF-SS-2	40.5	CL	25	86.5	89.4	68	43	-1	-1	-1	-1	-1	-1	-1	-1	68	43	-1	43	-1	43	-1	43	-1	Clay-like		
		STN-AQ-207	15	CL	17.8	91.1	95.9	45	22	-1	-1	-1	-1	-1	-1	-1	-1	45	22	-1	22	-1	22	-1	22	-1	Clay-like		
		STN-AQ-207	17.5	CL	19.9	71.7	93.4	30	14	-1	-1	30	14	-1	-1	30	14	-1	-1	-1	14	-1	14	-1	14	-1	Clay-like		
		STN-AQ-207	27.5	SC-SM	18.3	39.1	65	23	16	-1	-1	23	16	-1	-1	23	16	-1	-1	-1	16	-1	16	-1	16	-1	Clay-like		
		STN-AQ-207	30	SC	19.3	43.1	62.3	26	10	26	10	-1	-1	-1	-1	-1	-1	-1	-1	-1	10	-1	10	-1	10	-1	Clay-like		
		STN-AQ-207	32	SC	23.7	48.6	76.3	25	8	25	8	-1	-1	-1	-1	-1	-1	-1	-1	-1	8	-1	8	-1	8	-1	Sand-like		
		STN-AQ-207	36	CL	31.8	91.1	97	47	23	-1	-1	-1	-1	-1	-1	-1	-1	47	23	-1	23	-1	23	-1	23	-1	Clay-like		

Note: NP = Non-Plastic

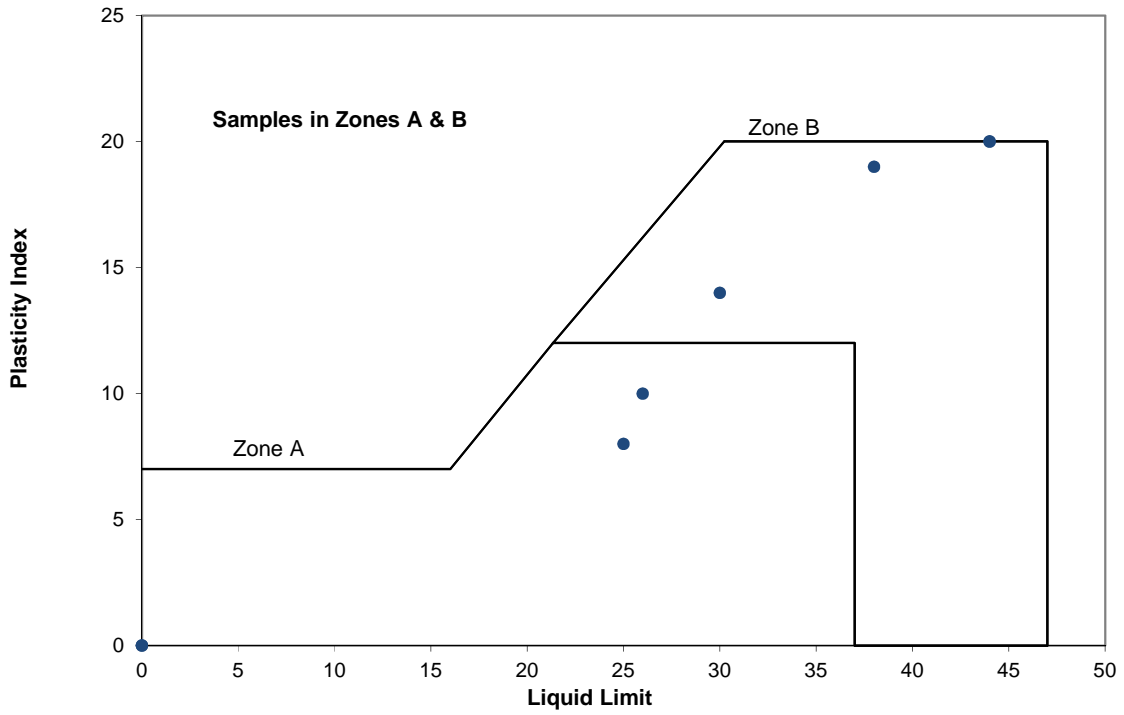


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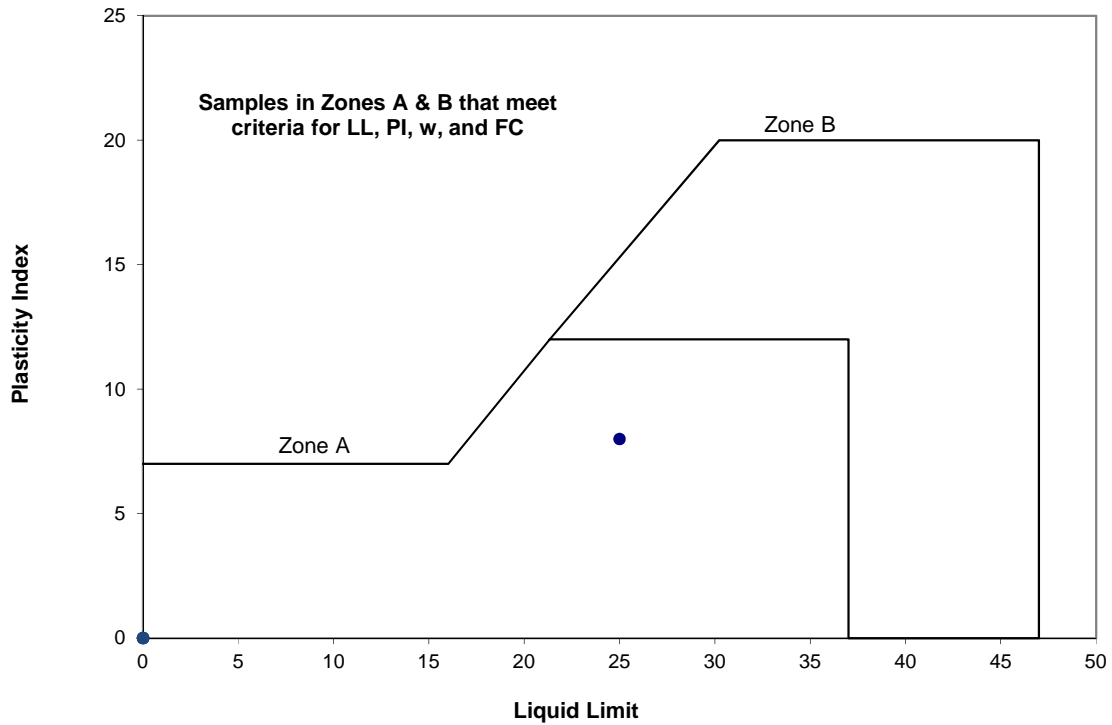


(b)

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

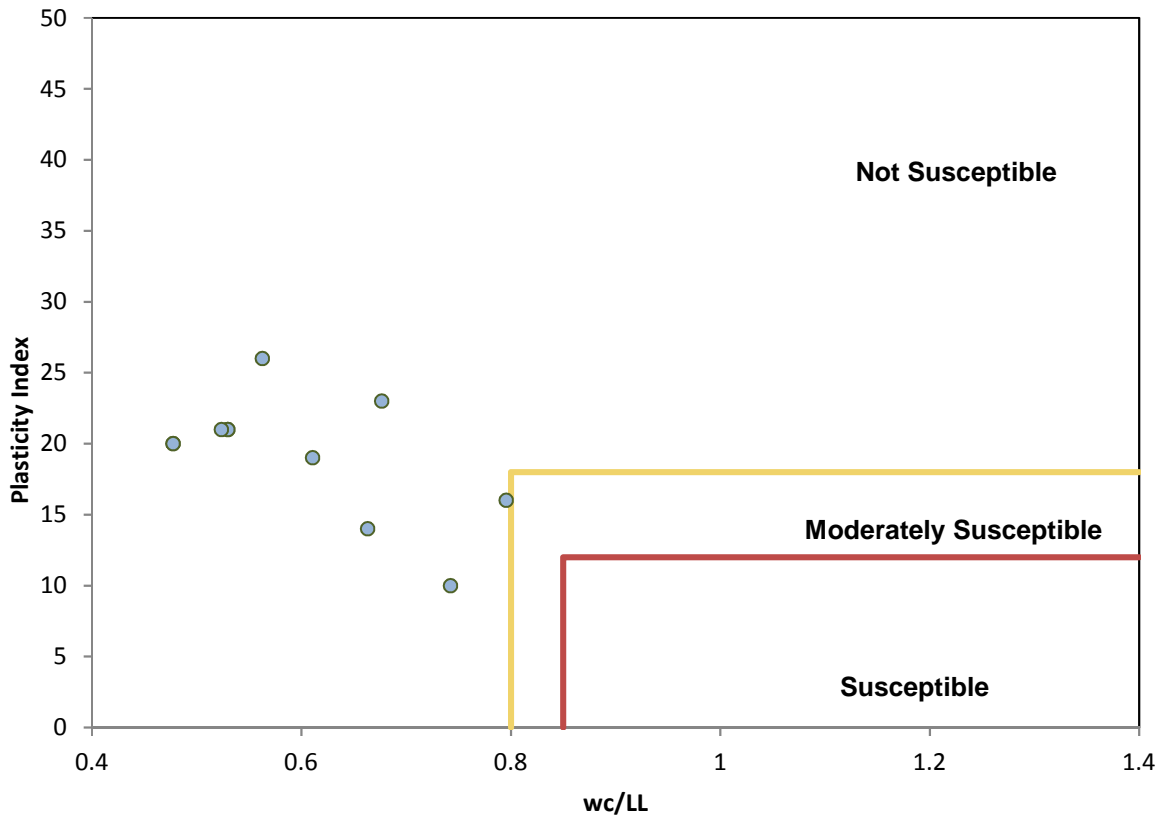
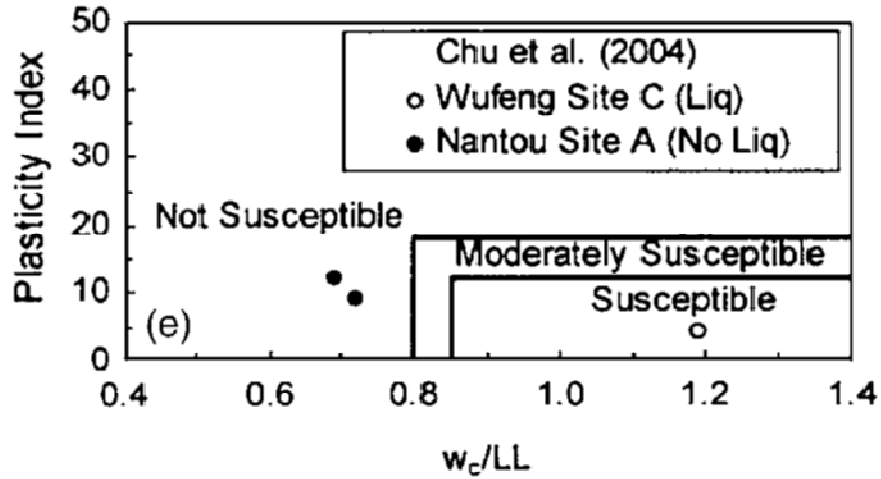


(c)



(d)

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

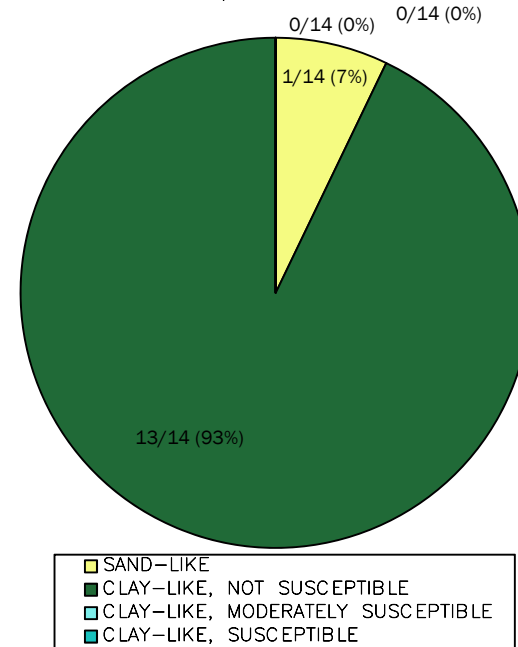


Screening Criteria for Assessing Liquefaction in Fine Grained Soils (Bray and Sancio 2006)

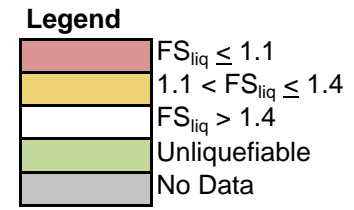
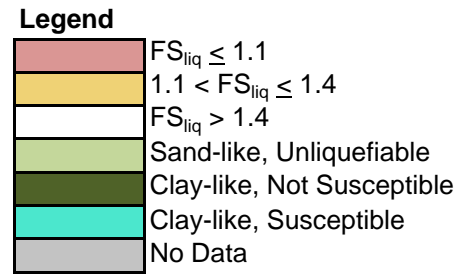
Native Clay, Fine-grained Screening and Cyclic Softening Susceptibility Summary

Soil Classification	Clay-like vs. Sand-like	Clay-like Susceptibility	Sample Count	Sample Percent
CL			11	79%
	Clay-like		11	100%
		Not Susceptible	11	100%
SC			2	14%
	Clay-like		1	50%
		Not Susceptible	1	100%
	Sand-like		1	50%
		Sand-like	1	100%
SC-SM			1	7%
	Clay-like		1	100%
		Not Susceptible	1	100%
Grand Total			14	100%

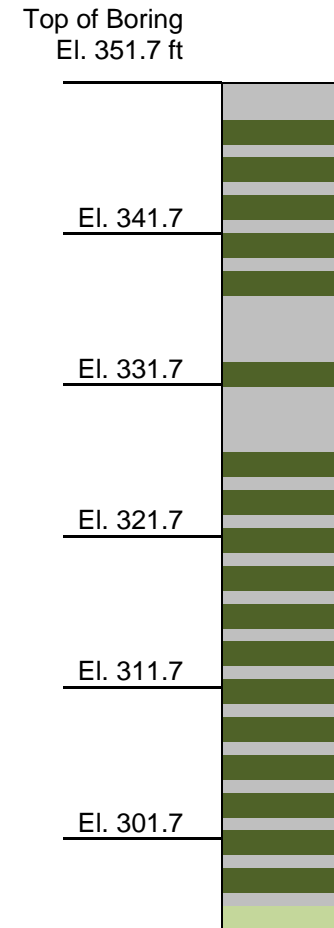
NATIVE CLAY, LABORATORY DATA



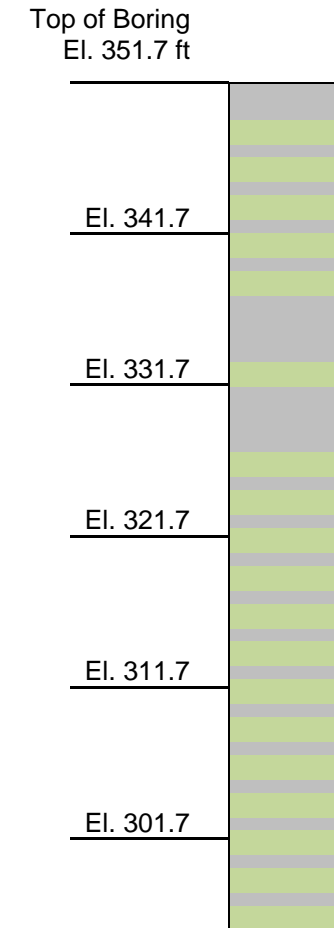
ATTACHMENT C



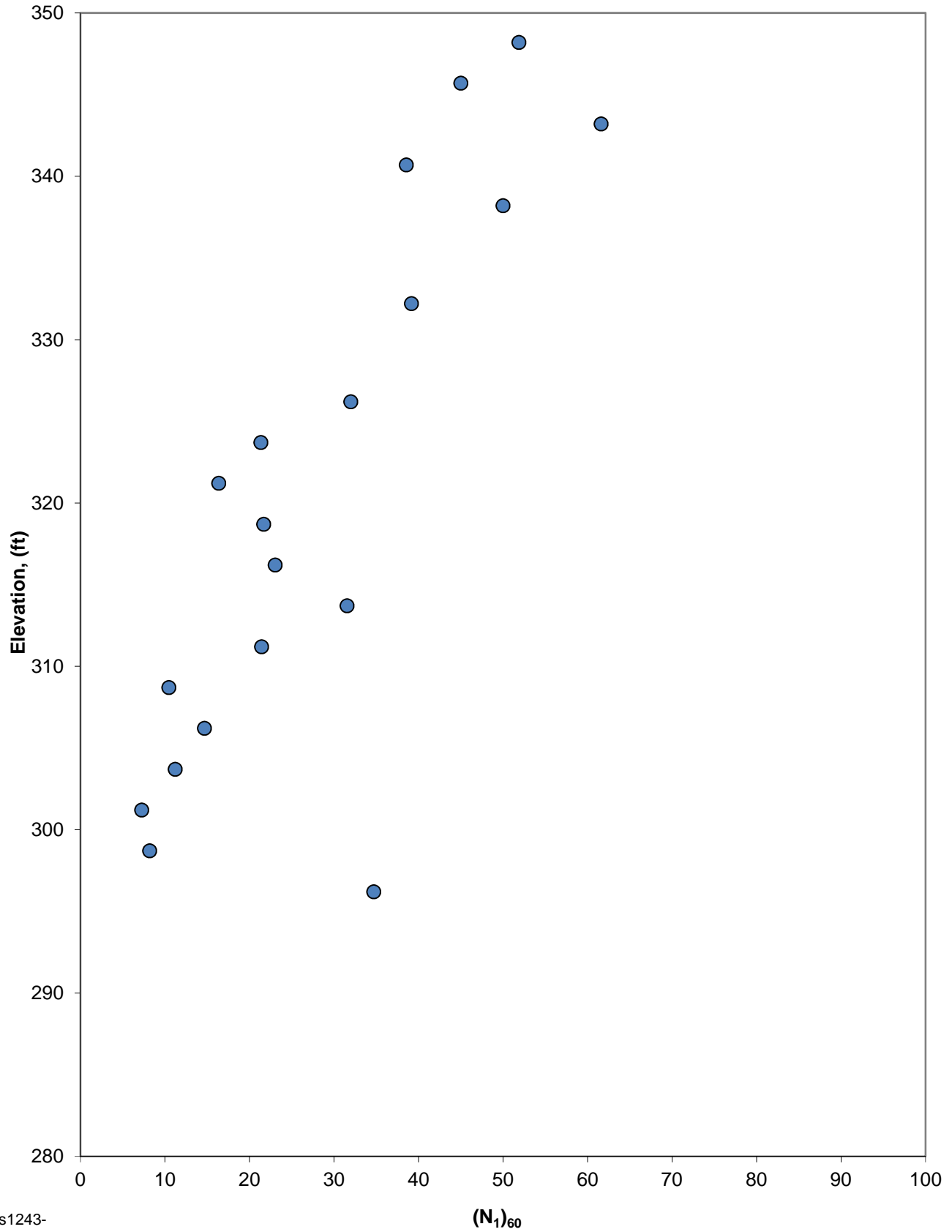
STN-SS-1, Liquefaction Triggering Results

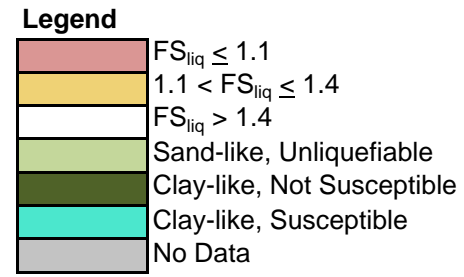


STN-SS-1, Liquefaction Triggering Results



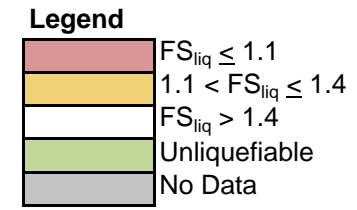
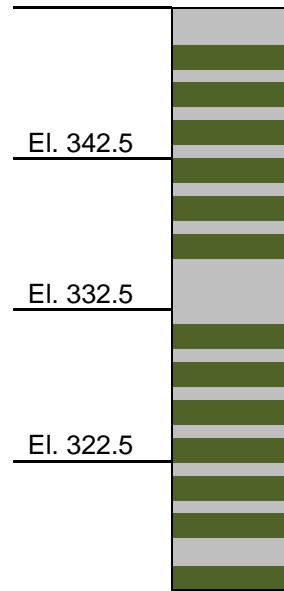
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Simplified Stress-Based Approach





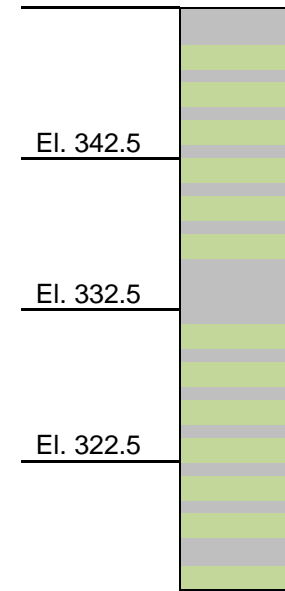
STN-SS-2, Liquefaction Triggering Results

Top of Boring
El. 352.5 ft

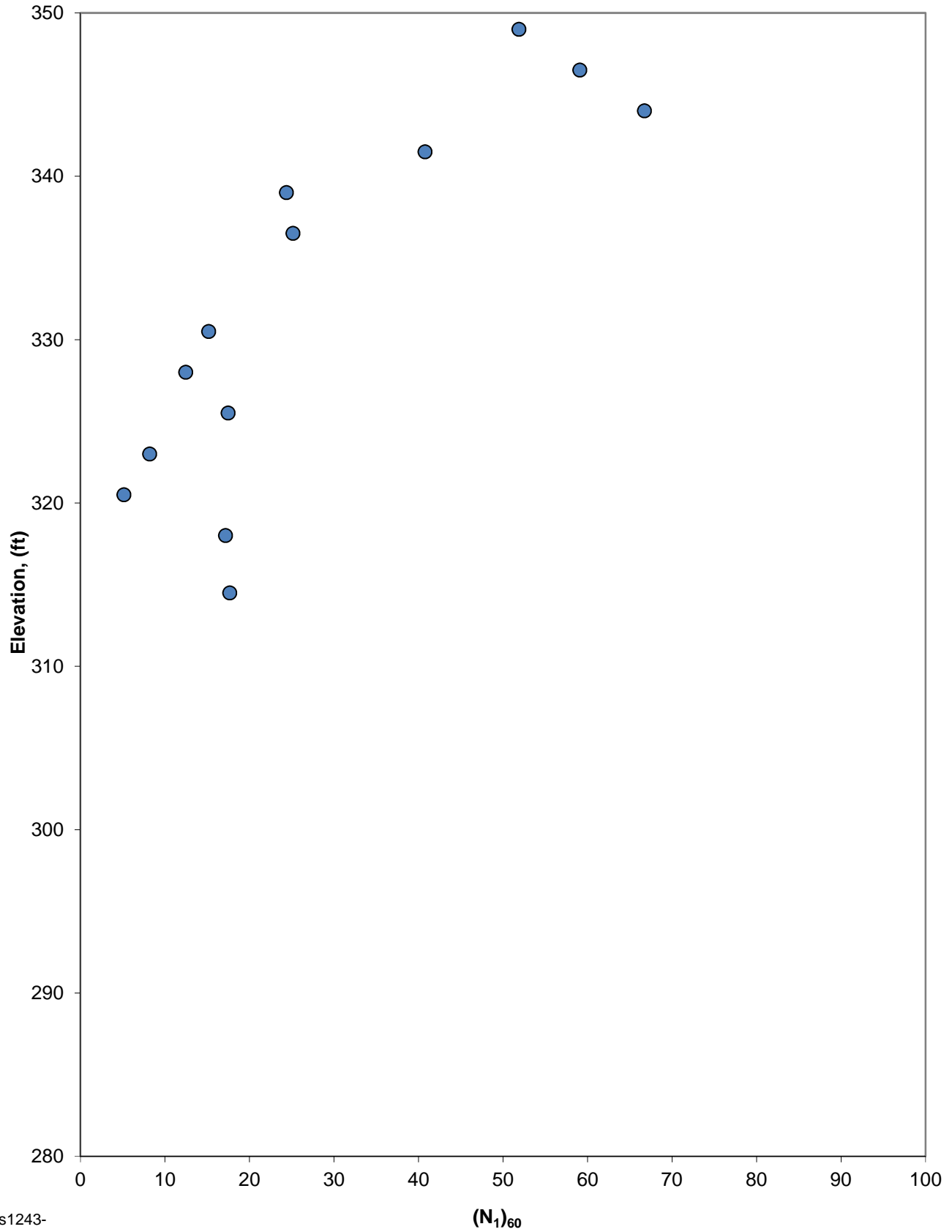


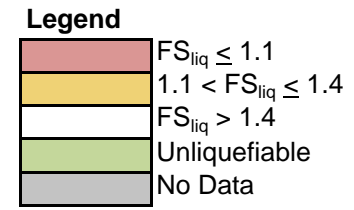
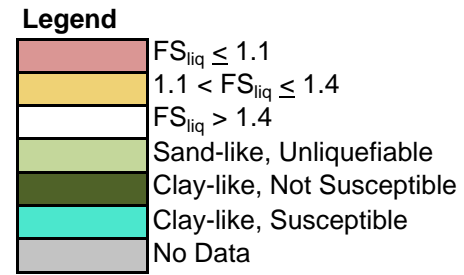
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Top of Boring
El. 352.5 ft

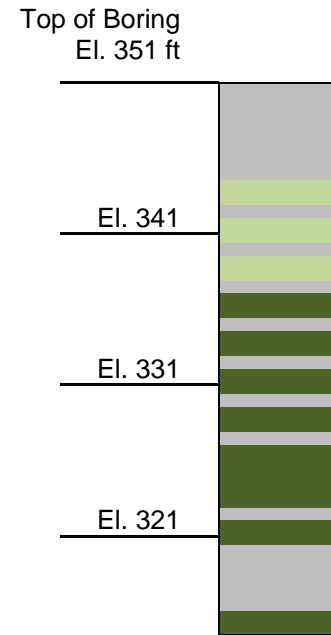


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Simplified Stress-Based Approach

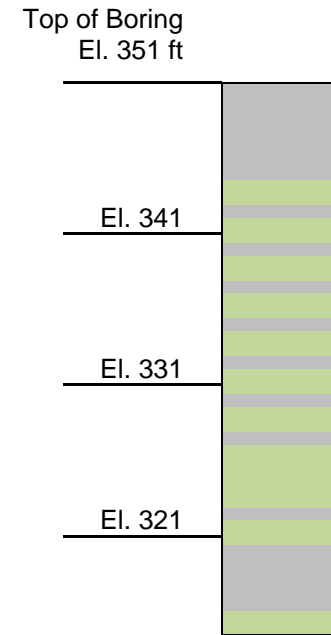




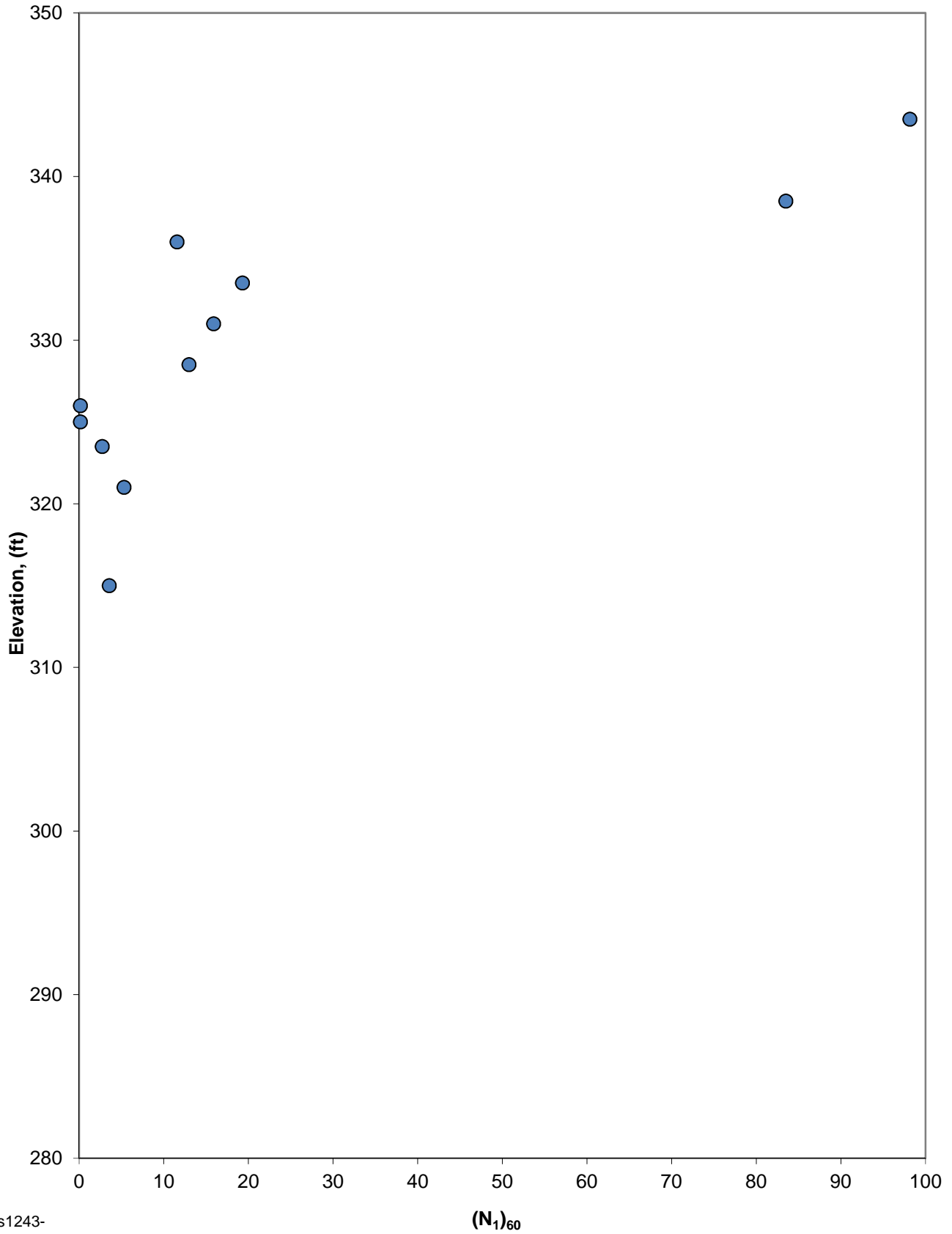
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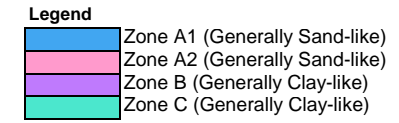
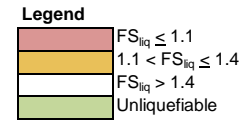
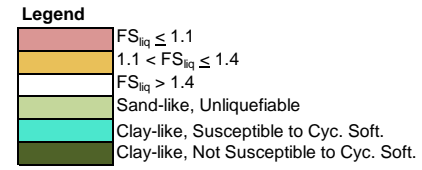


STN-AQ-207, Liquefaction Triggering Results

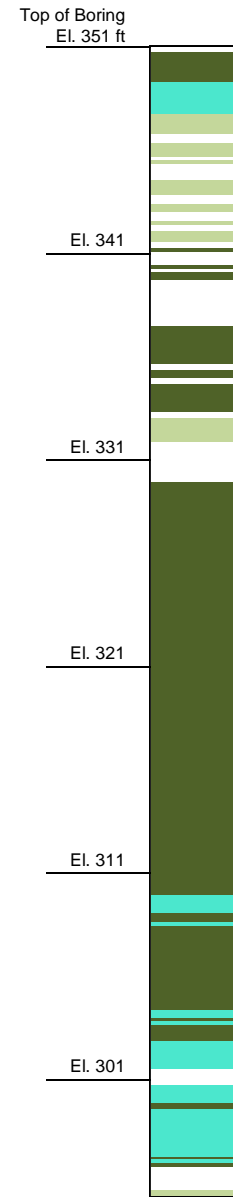


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Simplified Stress-Based Approach

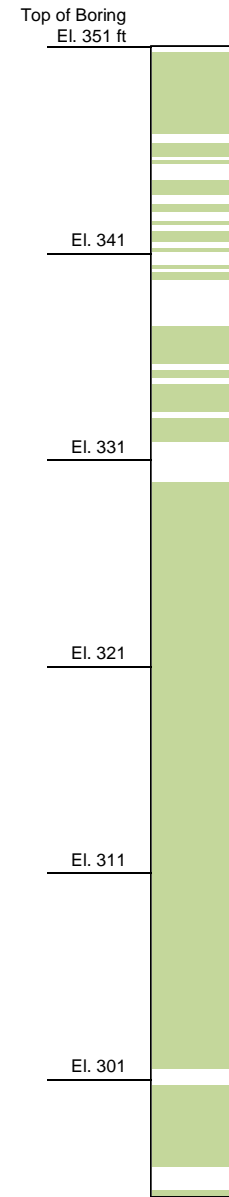




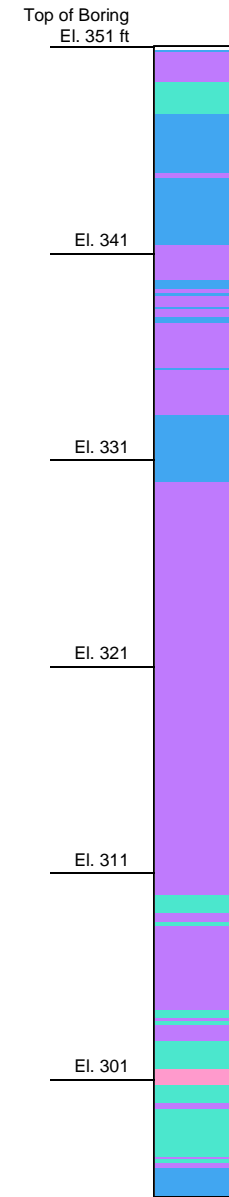
CPT-1, Liquefaction Triggering Results



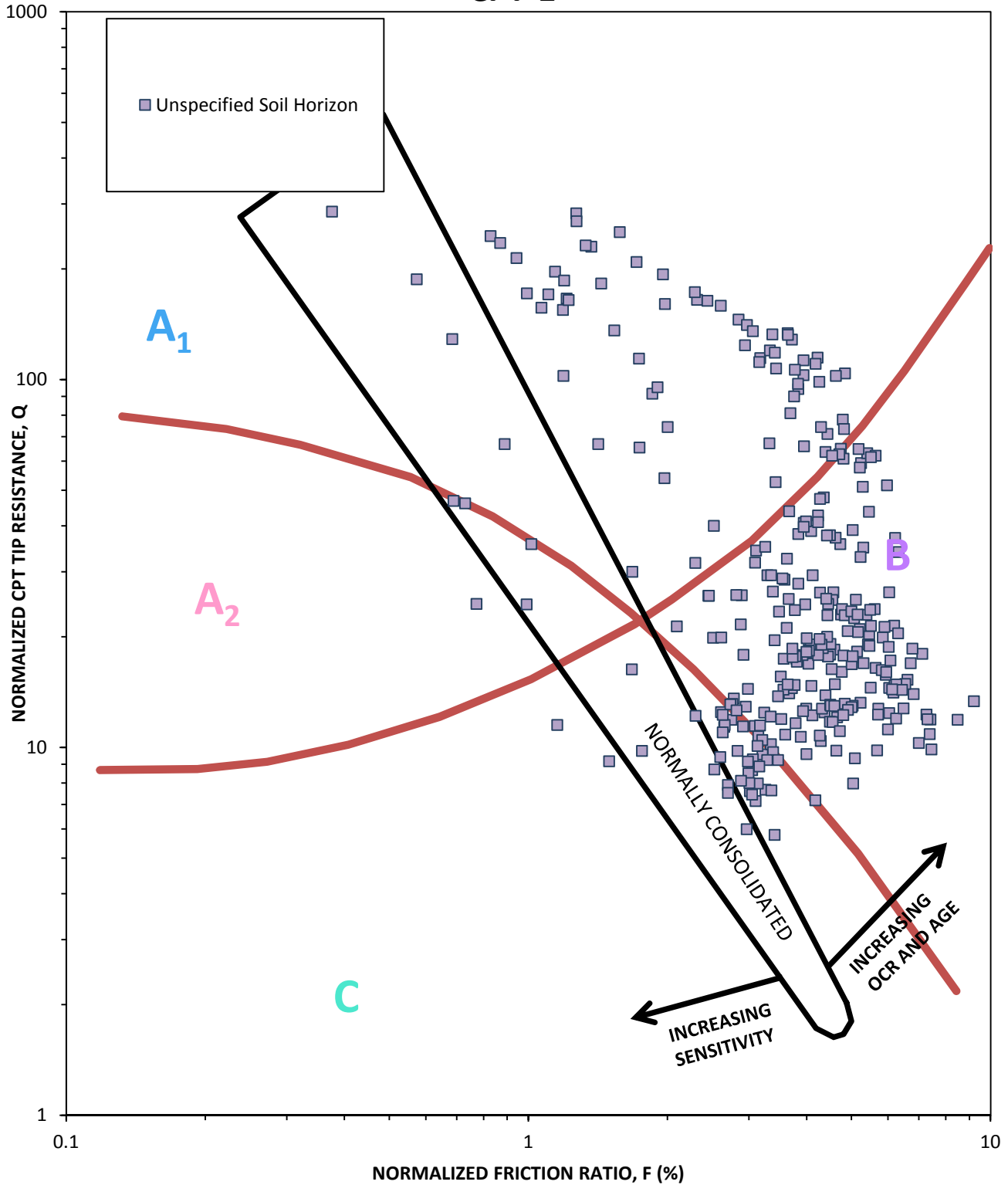
CPT-1, Liquefaction Triggering Results



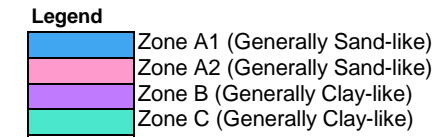
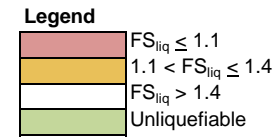
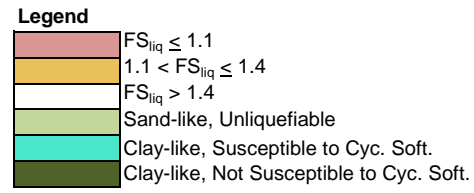
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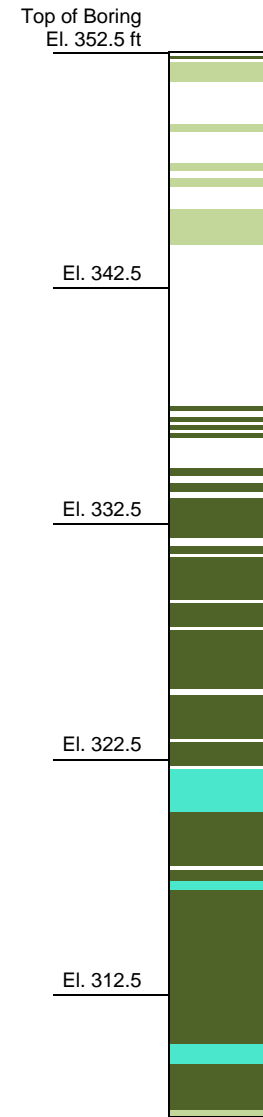
CPT-1



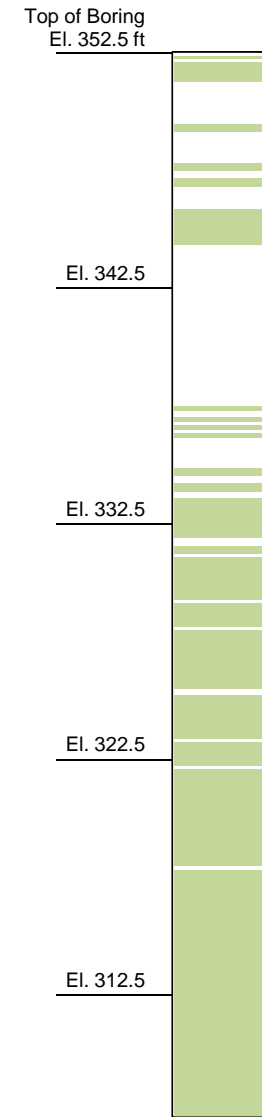
Adapted from "Discussion of 'Liquefaction Potential of Silts from CPTu'" (Robertson 2008). Zone A₁ and Zone A₂ samples are generally sand-like. Zone B samples are generally clay-like and not susceptible to cyclic softening. Zone C samples are generally clay-like and susceptible to cyclic softening. Note that the transition from clay-like to sand-like in this chart does not exactly match the liquefaction calculations herein; that is, some transitional samples in Zone B and Zone C will be treated as sand-like and analyzed for liquefaction if the calculations indicate as such.



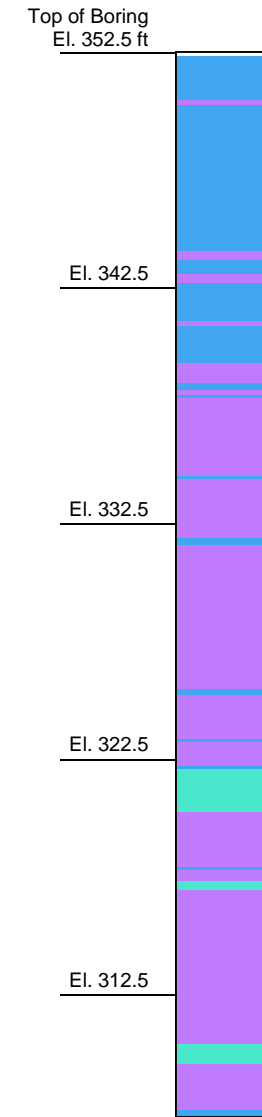
CPT-2, Liquefaction Triggering Results



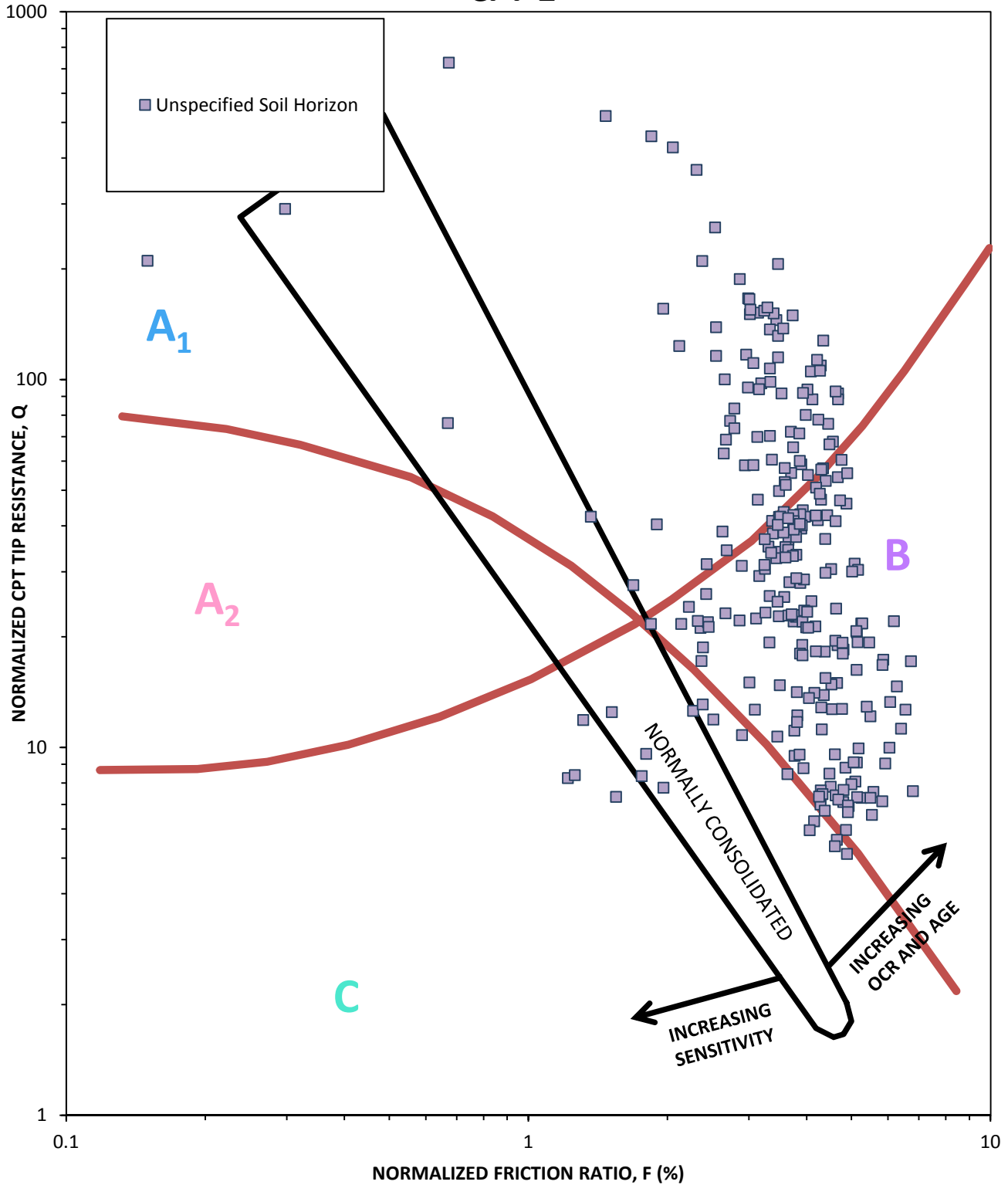
CPT-2, Liquefaction Triggering Results



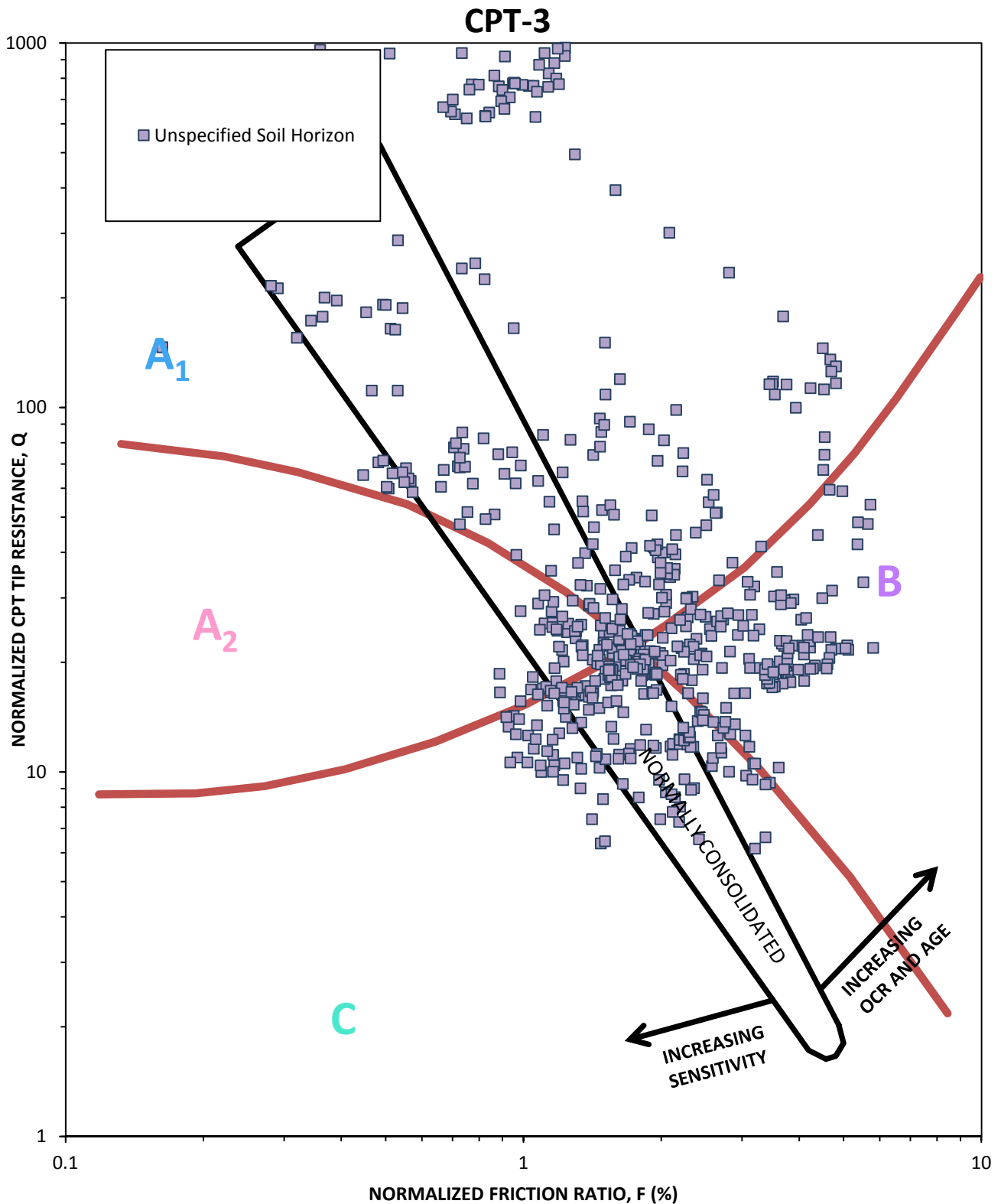
CPT-2, Robertson Soil Type



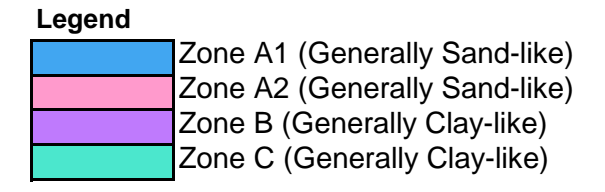
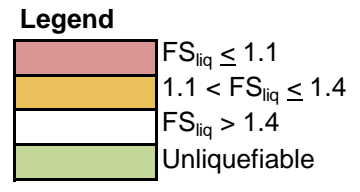
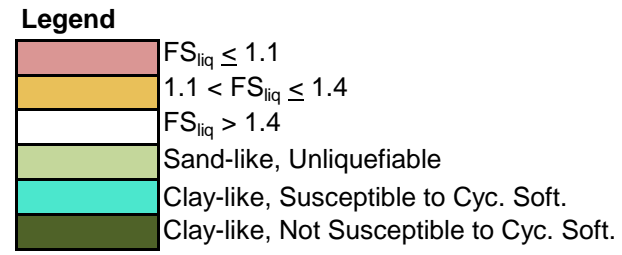
CPT-2



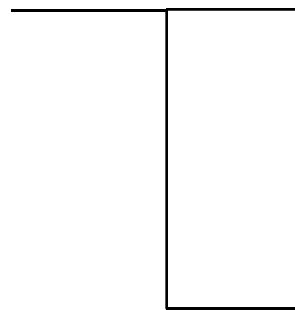
Adapted from "Discussion of 'Liquefaction Potential of Silts from CPTu'" (Robertson 2008). Zone A_1 and Zone A_2 samples are generally sand-like. Zone B samples are generally clay-like and not susceptible to cyclic softening. Zone C samples are generally clay-like and susceptible to cyclic softening. Note that the transition from clay-like to sand-like in this chart does not exactly match the liquefaction calculations herein; that is, some transitional samples in Zone B and Zone C will be treated as sand-like and analyzed for liquefaction if the calculations indicate as such.



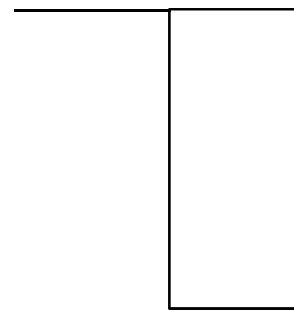
Adapted from "Discussion of 'Liquefaction Potential of Silts from CPTu'" (Robertson 2008). Zone A₁ and Zone A₂ samples are generally sand-like. Zone B samples are generally clay-like and not susceptible to cyclic softening. Zone C samples are generally clay-like and susceptible to cyclic softening. Note that the transition from clay-like to sand-like in this chart does not exactly match the liquefaction calculations herein; that is, some transitional samples in Zone B and Zone C will be treated as sand-like and analyzed for liquefaction if the calculations indicate as such.



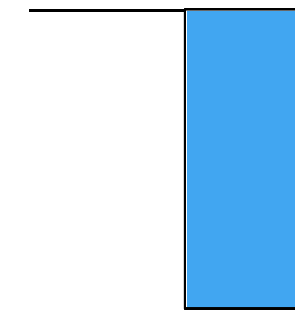
CPT-4, Liquefaction Triggering Results

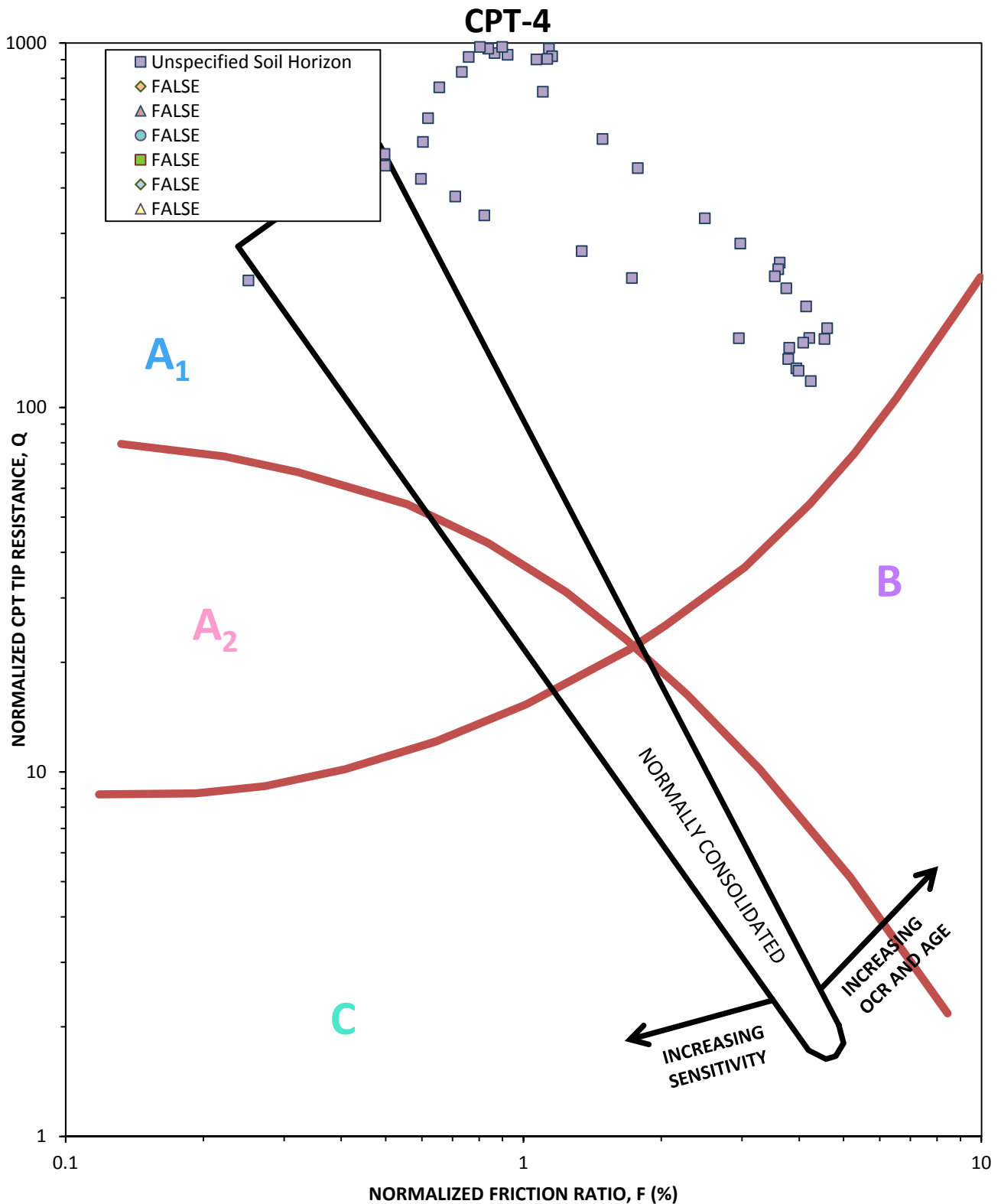


CPT-4, Liquefaction Triggering Results



CPT-4, Robertson Soil Type





Adapted from "Discussion of 'Liquefaction Potential of Silts from CPTu'" (Robertson 2008). Zone A₁ and Zone A₂ samples are generally sand-like. Zone B samples are generally clay-like and not susceptible to cyclic softening. Zone C samples are generally clay-like and susceptible to cyclic softening. Note that the transition from clay-like to sand-like in this chart does not exactly match the liquefaction calculations herein; that is, some transitional samples in Zone B and Zone C will be treated as sand-like and analyzed for liquefaction if the calculations indicate as such.

ATTACHMENT D

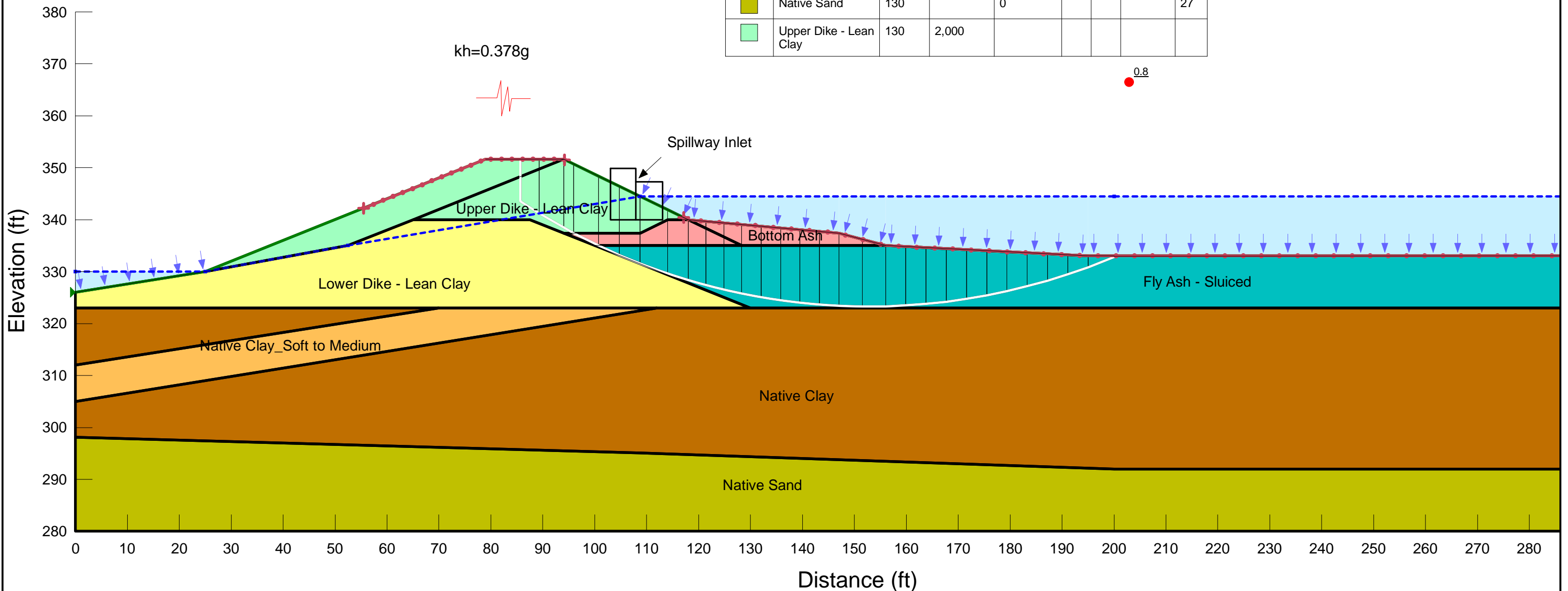


Shawnee Fossil Plant
 McCracken County, Kentucky
 Shawnee Fossil Plant, Profile - Section A-A'

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.

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)	Phi' (°)
Light Red	Bottom Ash	105		0				38
Teal	Fly Ash - Sluiced	100		0	26	8	921.5	
Yellow	Lower Dike - Lean Clay	130	1,600					
Brown	Native Clay	130	1,280					
Light Orange	Native Clay_Soft to Medium	128		0	28	0	752	
Greenish Yellow	Native Sand	130		0				27
Light Green	Upper Dike - Lean Clay	130	2,000					



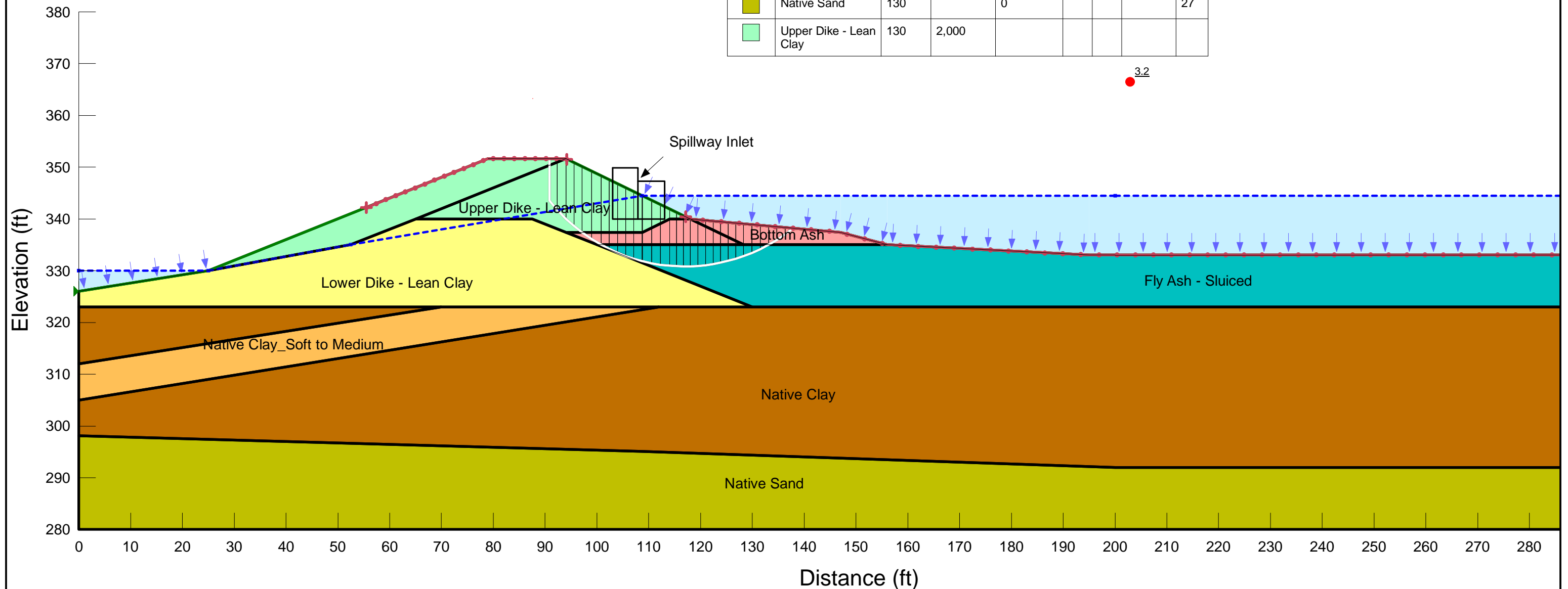


Shawnee Fossil Plant
 McCracken County, Kentucky
 Shawnee Fossil Plant, Profile - Section A-A'

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.

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)	Phi' (°)
Light Red	Bottom Ash	105		0				38
Teal	Fly Ash - Sluiced	100		0	26	8	921.5	
Yellow	Lower Dike - Lean Clay	130	1,600					
Brown	Native Clay	130	1,280					
Light Orange	Native Clay_Soft to Medium	128		0	28	0	752	
Greenish-Yellow	Native Sand	130		0				27
Light Green	Upper Dike - Lean Clay	130	2,000					



ATTACHMENT E

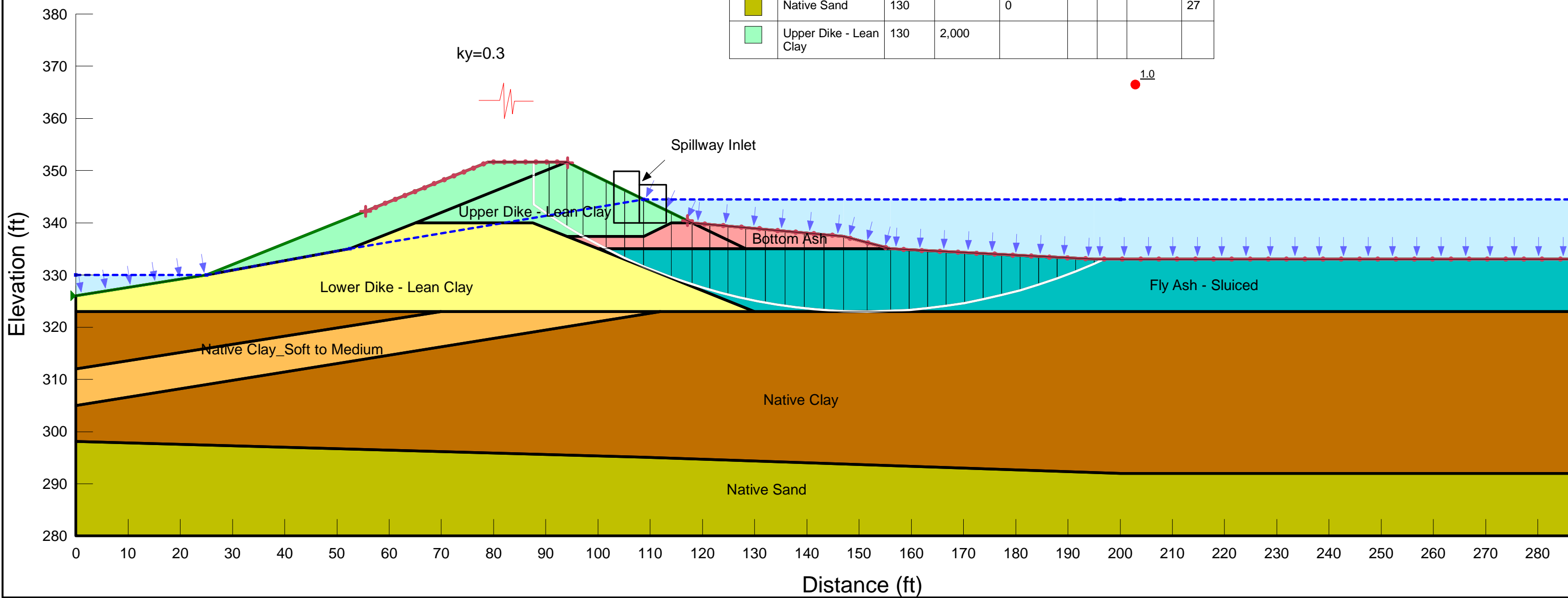


Shawnee Fossil Plant
 McCracken County, Kentucky
 Shawnee Fossil Plant, Profile - Section A-A'

Pseudostatic Slope Stability Analysis
 Yield Acceleration

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)	Cohesion' (psf)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)	Phi' (°)
Light Red	Bottom Ash	105		0				38
Teal	Fly Ash - Sluiced	100		0	26	8	921.5	
Yellow	Lower Dike - Lean Clay	130	1,600					
Brown	Native Clay	130	1,280					
Light Orange	Native Clay_Soft to Medium	128		0	28	0	752	
Greenish-Yellow	Native Sand	130		0				27
Light Green	Upper Dike - Lean Clay	130	2,000					

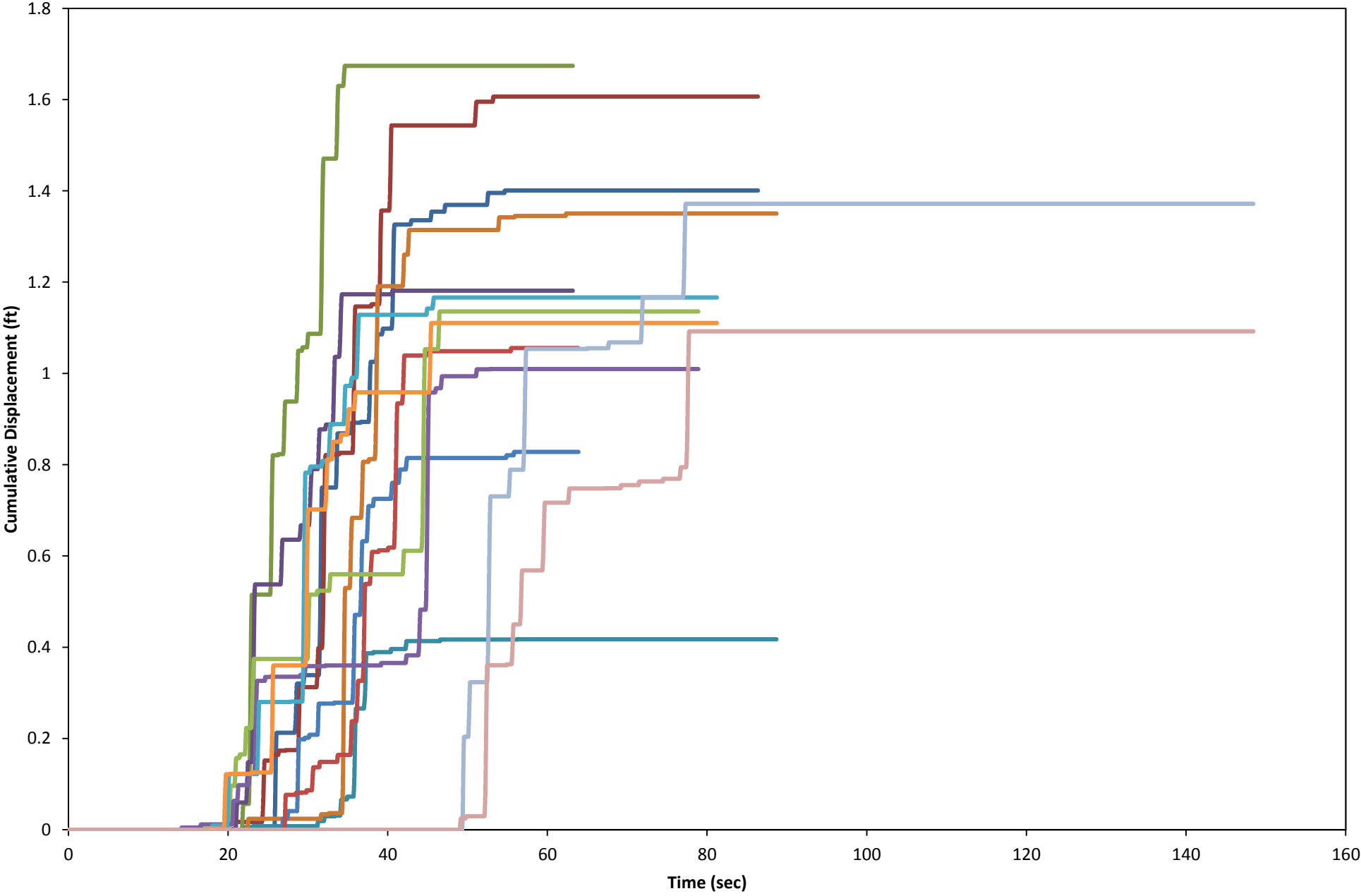


Facility	SHF	
Section/Group	A	
Profile Name	seism_normal	
Top of Profile Elevation	347.4	ft
Depth to water table during EQ	2.9	ft
Surcharge Pressure	0	psf

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 ₀	Mean Effective Stress at midpoint (psf)	Plasticity Index	Name (for Strata)	Unit Weight (pcf)	Modulus Reduction (G/G _{max}) Model	Damping Model	Thickness (ft)	Soil Type (for Strata)	Shear Wave Velocity (fps)
0	2.72	Upper Dike Lean Clay	Soil	4.80E+06	0.5	117.9	20	Upper Dike Lean Clay 1	130	I&Z, p'=0-400, PI=15-20	I&Z, p'=0-400, PI=15-20	2.72	Upper Dike Lean Clay 1	1090
2.72	2.9	Upper Dike Lean Clay	Soil	4.80E+06	0.5	243.5	20	Upper Dike Lean Clay 2	130	I&Z, p'=0-400, PI=15-20	I&Z, p'=0-400, PI=15-20	0.18	Upper Dike Lean Clay 2	1090
2.9	5.62	Upper Dike Lean Clay	Soil	4.80E+06	0.5	312.6	20	Upper Dike Lean Clay 3	130	I&Z, p'=0-400, PI=15-20	I&Z, p'=0-400, PI=15-20	2.72	Upper Dike Lean Clay 3	1090
5.62	8.34	Upper Dike Lean Clay	Soil	4.80E+06	0.5	435.2	20	Upper Dike Lean Clay 4	130	I&Z, p'=400-800, PI=15-20	I&Z, p'=400-800, PI=15-20	2.72	Upper Dike Lean Clay 4	1090
8.34	8.5	Upper Dike Lean Clay	Soil	4.80E+06	0.5	500.1	20	Upper Dike Lean Clay 5	130	I&Z, p'=400-800, PI=15-20	I&Z, p'=400-800, PI=15-20	0.16	Upper Dike Lean Clay 5	1090
8.5	12.25	Bottom Ash	Soil	7.34E+06	0.5	557.0	0	Bottom Ash 6	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	3.75	Bottom Ash 6	1500
12.25	12.4	Bottom Ash	Soil	7.34E+06	0.5	612.3	0	Bottom Ash 7	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	0.15	Bottom Ash 7	1500
12.4	14.15	Fly Ash - Sluiced	Soil	1.29E+06	0.5	627.6	10	Fly Ash - Sluiced 8	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.75	Fly Ash - Sluiced 8	700
14.15	15.9	Fly Ash - Sluiced	Soil	1.29E+06	0.5	654.0	10	Fly Ash - Sluiced 9	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.75	Fly Ash - Sluiced 9	700
15.9	17.65	Fly Ash - Sluiced	Soil	1.29E+06	0.5	680.4	10	Fly Ash - Sluiced 10	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.75	Fly Ash - Sluiced 10	700
17.65	19.4	Fly Ash - Sluiced	Soil	1.29E+06	0.5	706.7	10	Fly Ash - Sluiced 11	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.75	Fly Ash - Sluiced 11	700
19.4	21.15	Fly Ash - Sluiced	Soil	1.29E+06	0.5	733.1	10	Fly Ash - Sluiced 12	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.75	Fly Ash - Sluiced 12	700
21.15	22.9	Fly Ash - Sluiced	Soil	1.29E+06	0.5	759.5	10	Fly Ash - Sluiced 13	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.75	Fly Ash - Sluiced 13	700
22.9	24.4	Fly Ash - Sluiced	Soil	1.29E+06	0.5	784.0	10	Fly Ash - Sluiced 14	85	I&Z, p'=400-800, PI=0-10	I&Z, p'=400-800, PI=0-10	1.5	Fly Ash - Sluiced 14	700
24.4	26.14	Native Clay	Soil	1.94E+06	0.5	833.3	20	Native Clay 15	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 15	698
26.14	27.88	Native Clay	Soil	1.94E+06	0.5	909.4	20	Native Clay 16	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 16	698
27.88	29.62	Native Clay	Soil	1.94E+06	0.5	985.5	20	Native Clay 17	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 17	698
29.62	31.36	Native Clay	Soil	1.94E+06	0.5	1061.6	20	Native Clay 18	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 18	698
31.36	33.1	Native Clay	Soil	1.94E+06	0.5	1137.7	20	Native Clay 19	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 19	698
33.1	34.84	Native Clay	Soil	1.94E+06	0.5	1213.8	20	Native Clay 20	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 20	698
34.84	36.58	Native Clay	Soil	1.94E+06	0.5	1289.9	20	Native Clay 21	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 21	698
36.58	38.32	Native Clay	Soil	1.94E+06	0.5	1366.0	20	Native Clay 22	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 22	698
38.32	40.06	Native Clay	Soil	1.94E+06	0.5	1442.1	20	Native Clay 23	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 23	698
40.06	41.8	Native Clay	Soil	1.94E+06	0.5	1518.2	20	Native Clay 24	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 24	698
41.8	43.54	Native Clay	Soil	1.94E+06	0.5	1594.3	20	Native Clay 25	128	I&Z, p'=800-1600, PI=15-20	I&Z, p'=800-1600, PI=15-20	1.74	Native Clay 25	698
43.54	45.28	Native Clay	Soil	1.94E+06	0.5	1670.4	20	Native Clay 26	128	I&Z, p'=1600-2500, PI=15-20	I&Z, p'=1600-2500, PI=15-20	1.74	Native Clay 26	698
45.28	47.02	Native Clay	Soil	1.94E+06	0.5	1746.5	20	Native Clay 27	128	I&Z, p'=1600-2500, PI=15-20	I&Z, p'=1600-2500, PI=15-20	1.74	Native Clay 27	698
47.02	48.76	Native Clay	Soil	1.94E+06	0.5	1822.6	20	Native Clay 28	128	I&Z, p'=1600-2500, PI=15-20	I&Z, p'=1600-2500, PI=15-20	1.74	Native Clay 28	698
48.76	50.5	Native Clay	Soil	1.94E+06	0.5	1898.7	20	Native Clay 29	128	I&Z, p'=1600-2500, PI=15-20	I&Z, p'=1600-2500, PI=15-20	1.74	Native Clay 29	698
50.5	52.24	Native Clay	Soil	1.94E+06	0.5	1974.8	20	Native Clay 30	128	I&Z, p'=1600-2500, PI=15-20	I&Z, p'=1600-2500, PI=15-20	1.74	Native Clay 30	698
52.24	53	Native Clay	Soil	1.94E+06	0.5	2029.4	20	Native Clay 31	128	I&Z, p'=1600-2500, PI=15-20	I&Z, p'=1600-2500, PI=15-20	0.76	Native Clay 31	698
53	56.83	Continental Deposits	Soil	1.00E+07	0.5	2141.3	0	Continental Deposits 32	137	I&Z, p'=1600-2500, PI=0	I&Z, p'=1600-2500, PI=0	3.83	Continental Deposits 32	1535
56.83	60.66	Continental Deposits	Soil	1.00E+07	0.5	2331.8	0	Continental Deposits 33	137	I&Z, p'=1600-2500, PI=0	I&Z, p'=1600-2500, PI=0	3.83	Continental Deposits 33	1535
60.66	64.49	Continental Deposits	Soil	1.00E+07	0.5	2522.2	0	Continental Deposits 34	137	I&Z, p'=2500-5000, PI=0	I&Z, p'=2500-5000, PI=0	3.83	Continental Deposits 34	1535
64.49	67.4	Continental Deposits	Soil	1.00E+07	0.5	2689.8	0	Continental Deposits 35	137	I&Z, p'=2500-5000, PI=0	I&Z, p'=2500-5000, PI=0	2.91	Continental Deposits 35	1535
67.4	71.23	McNairy and Clayton Formations	Soil	1.02E+07	0.5	2861.3	21	McNairy and Clayton Formations 36	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 36	1535
71.23	75.06	McNairy and Clayton Formations	Soil	1.02E+07	0.5	3059.4	21	McNairy and Clayton Formations 37	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 37	1535
75.06	78.89	McNairy and Clayton Formations	Soil	1.02E+07	0.5	3257.5	21	McNairy and Clayton Formations 38	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 38	1535
78.89	82.72	McNairy and Clayton Formations	Soil	1.02E+07	0.5	3455.7	21	McNairy and Clayton Formations 39	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 39	1535
82.72	86.55	McNairy and Clayton Formations	Soil	1.02E+07	0.5	3653.8	21	McNairy and Clayton Formations 40	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 40	1535
86.55	90.38	McNairy and Clayton Formations	Soil	1.02E+07	0.5	3852.0	21	McNairy and Clayton Formations 41	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 41	1535
90.38	94.21	McNairy and Clayton Formations	Soil	1.02E+07	0.5	4050.1	21	McNairy and Clayton Formations 42	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 42	1535
94.21	98.04	McNairy and Clayton Formations	Soil	1.02E+07	0.5	4248.2	21	McNairy and Clayton Formations 43	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 43	1535
98.04	101.87	McNairy and Clayton Formations	Soil	1.02E+07	0.5	4446.4	21	McNairy and Clayton Formations 44	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 44	1535
101.87	105.7	McNairy and Clayton Formations	Soil	1.02E+07	0.5	4644.5	21	McNairy and Clayton Formations 45	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 45	1535
105.7	109.53	McNairy and Clayton Formations	Soil	1.02E+07	0.5	4842.7	21	McNairy and Clayton Formations 46	140	I&Z, p'=2500-5000, PI=20-25	I&Z, p'=2500-5000, PI=20-25	3.83	McNairy and Clayton Formations 46	1535
109.53	113.36	McNairy and Clayton Formations	Soil	1.02E+07	0.5	5040.8	21	McNairy and Clayton Formations 47	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 47	1535
113.36	117.19	McNairy and Clayton Formations	Soil	1.02E+07	0.5	5238.9	21	McNairy and Clayton Formations 48	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 48	1535
117.19	121.02	McNairy and Clayton Formations	Soil	1.02E+07	0.5	5437.1	21	McNairy and Clayton Formations 49	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 49	1535
121.02	124.85	McNairy and Clayton Formations	Soil	1.02E+07	0.5	5635.2	21	McNairy and Clayton Formations 50	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 50	1535
124.85	128.68	McNairy and Clayton Formations	Soil	1.02E+07	0.5	5833.3	21	McNairy and Clayton Formations 51	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 51	1535
128.68	132.51	McNairy and Clayton Formations	Soil	1.02E+07	0.5	6031.5	21	McNairy and Clayton Formations 52	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 52	1535
132.51	136.34	McNairy and Clayton Formations	Soil	1.02E+07	0.5	6229.6	21	McNairy and Clayton Formations 53	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 53	1535
136.34	140.17	McNairy and Clayton Formations	Soil	1.02E+07	0.5	6427.8	21	McNairy and Clayton Formations 54	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 54	1535
140.17	144	McNairy and Clayton Formations	Soil	1.02E+07	0.5	6625.9	21	McNairy and Clayton Formations 55	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 55	1535
144	147.83	McNairy and Clayton Formations	Soil	1.02E+07	0.5	6824.0	21	McNairy and Clayton Formations 56	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 56	1535
147.83	151.66	McNairy and Clayton Formations	Soil	1.02E+07	0.5	7022.2	21	McNairy and Clayton Formations 57	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 57	1535
151.66	155.49	McNairy and Clayton Formations	Soil	1.02E+07	0.5	7220.3	21	McNairy and Clayton Formations 58	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 58	1535
155.49	159.32	McNairy and Clayton Formations	Soil	1.02E+07	0.5	7418.5	21	McNairy and Clayton Formations 59	140	I&Z, p'=5000-7500, PI=20-25	I&Z, p'=5000-7500, PI=20-25	3.83	McNairy and Clayton Formations 59	1535
159.32	163.15	McNairy and Clayton Formations	Soil	1.02E+07	0.5	7616.6	21	McNairy and Clayton Formations 60	140	I&Z, p'=7500-10000, PI=20-25	I&Z, p'=7500-10000, PI=20-25	3.83	McNairy and Clayton Formations 60	1535

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)
163.15	166.98	McNairy and Clayton Formations	Soil	1.02E+07	0.5	7814.7	21	McNairy and Clayton Formations_61	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_61	1535
166.98	170.81	McNairy and Clayton Formations	Soil	1.02E+07	0.5	8012.9	21	McNairy and Clayton Formations_62	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_62	1535
170.81	174.64	McNairy and Clayton Formations	Soil	1.02E+07	0.5	8211.0	21	McNairy and Clayton Formations_63	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_63	1535
174.64	178.47	McNairy and Clayton Formations	Soil	1.02E+07	0.5	8409.2	21	McNairy and Clayton Formations_64	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_64	1535
178.47	182.3	McNairy and Clayton Formations	Soil	1.02E+07	0.5	8607.3	21	McNairy and Clayton Formations_65	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_65	1535
182.3	186.13	McNairy and Clayton Formations	Soil	1.02E+07	0.5	8805.4	21	McNairy and Clayton Formations_66	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_66	1535
186.13	189.96	McNairy and Clayton Formations	Soil	1.02E+07	0.5	9003.6	21	McNairy and Clayton Formations_67	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_67	1535
189.96	193.79	McNairy and Clayton Formations	Soil	1.02E+07	0.5	9201.7	21	McNairy and Clayton Formations_68	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_68	1535
193.79	197.62	McNairy and Clayton Formations	Soil	1.02E+07	0.5	9399.8	21	McNairy and Clayton Formations_69	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_69	1535
197.62	201.45	McNairy and Clayton Formations	Soil	1.02E+07	0.5	9598.0	21	McNairy and Clayton Formations_70	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_70	1535
201.45	205.28	McNairy and Clayton Formations	Soil	1.02E+07	0.5	9796.1	21	McNairy and Clayton Formations_71	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_71	1535
205.28	209.11	McNairy and Clayton Formations	Soil	1.02E+07	0.5	9994.3	21	McNairy and Clayton Formations_72	140	I&Z, p'=7500-10000, Pi=20-25	I&Z, p'=7500-10000, Pi=20-25	3.83	McNairy and Clayton Formations_72	1535
209.11	212.94	McNairy and Clayton Formations	Soil	1.02E+07	0.5	10192.4	21	McNairy and Clayton Formations_73	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_73	1535
212.94	216.77	McNairy and Clayton Formations	Soil	1.02E+07	0.5	10390.5	21	McNairy and Clayton Formations_74	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_74	1535
216.77	220.6	McNairy and Clayton Formations	Soil	1.02E+07	0.5	10588.7	21	McNairy and Clayton Formations_75	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_75	1535
220.6	224.43	McNairy and Clayton Formations	Soil	1.02E+07	0.5	10786.8	21	McNairy and Clayton Formations_76	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_76	1535
224.43	228.26	McNairy and Clayton Formations	Soil	1.02E+07	0.5	10985.0	21	McNairy and Clayton Formations_77	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_77	1535
228.26	232.09	McNairy and Clayton Formations	Soil	1.02E+07	0.5	11183.1	21	McNairy and Clayton Formations_78	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_78	1535
232.09	235.92	McNairy and Clayton Formations	Soil	1.02E+07	0.5	11381.2	21	McNairy and Clayton Formations_79	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_79	1535
235.92	239.75	McNairy and Clayton Formations	Soil	1.02E+07	0.5	11579.4	21	McNairy and Clayton Formations_80	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_80	1535
239.75	243.58	McNairy and Clayton Formations	Soil	1.02E+07	0.5	11777.5	21	McNairy and Clayton Formations_81	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_81	1535
243.58	247.41	McNairy and Clayton Formations	Soil	1.02E+07	0.5	11975.6	21	McNairy and Clayton Formations_82	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_82	1535
247.41	251.24	McNairy and Clayton Formations	Soil	1.02E+07	0.5	12173.8	21	McNairy and Clayton Formations_83	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_83	1535
251.24	255.07	McNairy and Clayton Formations	Soil	1.02E+07	0.5	12371.9	21	McNairy and Clayton Formations_84	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_84	1535
255.07	258.9	McNairy and Clayton Formations	Soil	1.02E+07	0.5	12570.1	21	McNairy and Clayton Formations_85	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_85	1535
258.9	262.73	McNairy and Clayton Formations	Soil	1.02E+07	0.5	12768.2	21	McNairy and Clayton Formations_86	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_86	1535
262.73	266.56	McNairy and Clayton Formations	Soil	1.02E+07	0.5	12966.3	21	McNairy and Clayton Formations_87	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_87	1535
266.56	270.39	McNairy and Clayton Formations	Soil	1.02E+07	0.5	13164.5	21	McNairy and Clayton Formations_88	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_88	1535
270.39	274.22	McNairy and Clayton Formations	Soil	1.02E+07	0.5	13362.6	21	McNairy and Clayton Formations_89	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_89	1535
274.22	278.05	McNairy and Clayton Formations	Soil	1.02E+07	0.5	13560.8	21	McNairy and Clayton Formations_90	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_90	1535
278.05	281.88	McNairy and Clayton Formations	Soil	1.02E+07	0.5	13758.9	21	McNairy and Clayton Formations_91	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_91	1535
281.88	285.71	McNairy and Clayton Formations	Soil	1.02E+07	0.5	13957.0	21	McNairy and Clayton Formations_92	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_92	1535
285.71	289.54	McNairy and Clayton Formations	Soil	1.02E+07	0.5	14155.2	21	McNairy and Clayton Formations_93	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_93	1535
289.54	293.37	McNairy and Clayton Formations	Soil	1.02E+07	0.5	14353.3	21	McNairy and Clayton Formations_94	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_94	1535
293.37	297.2	McNairy and Clayton Formations	Soil	1.02E+07	0.5	14551.5	21	McNairy and Clayton Formations_95	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_95	1535
297.2	301.03	McNairy and Clayton Formations	Soil	1.02E+07	0.5	14749.6	21	McNairy and Clayton Formations_96	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_96	1535
301.03	304.86	McNairy and Clayton Formations	Soil	1.02E+07	0.5	14947.7	21	McNairy and Clayton Formations_97	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_97	1535
304.86	308.69	McNairy and Clayton Formations	Soil	1.02E+07	0.5	15145.9	21	McNairy and Clayton Formations_98	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_98	1535
308.69	312.52	McNairy and Clayton Formations	Soil	1.02E+07	0.5	15344.0	21	McNairy and Clayton Formations_99	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_99	1535
312.52	316.35	McNairy and Clayton Formations	Soil	1.02E+07	0.5	15542.1	21	McNairy and Clayton Formations_100	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_100	1535
316.35	320.18	McNairy and Clayton Formations	Soil	1.02E+07	0.5	15740.3	21	McNairy and Clayton Formations_101	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_101	1535
320.18	324.01	McNairy and Clayton Formations	Soil	1.02E+07	0.5	15938.4	21	McNairy and Clayton Formations_102	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.83	McNairy and Clayton Formations_102	1535
324.01	327.4	McNairy and Clayton Formations	Soil	1.02E+07	0.5	16125.2	21	McNairy and Clayton Formations_103	140	I&Z, p'=10000-17000, Pi=20-25	I&Z, p'=10000-17000, Pi=20-25	3.39	McNairy and Clayton Formations_103	1535
327.4	Half-Space	Mississippian Limestone	Half-Space	4.15E+08	0.5	Half-Space	0	Bedrock	165	NA, Half-Space	Damping = 0.5%	Half Space	Bedrock	9000

Newmark Displacement Plot: SHF-Spillway-DesignE-Source-normal-ib



APPENDIX B

STRUCTURAL ANALYSES



Project: SHF Seismic Demo.
Job No.: 175555010
Location: Ash Pond 2
Contents: Seismic Stability

Calc. by: JEH
Date: 6/6/2018
Check by: JTP
Date: 6/15/2018

SHF SEISMIC STABILITY

ANALYSIS PROCEDURE

I Stability

- a) Stability analysis was performed for the entire inlet structure considering loading in 2 dimensions.
- b) Origin: the origin about which all moments are taken is the bottom of footing, inlet side of the structure
- c) Sign Convention: Positive vertical loads are upwards, positive horizontal loads are to the from the inlet side to the outlet side, positive moments are counter-clockwise.
- d) Loading for stability was as follows:
 - 1 - *Structure Weight* : includes concrete inlet boxes and steel grating on top, as well as miscellaneous steel components used for attachment of pipes, etc. Miscellaneous steel and grating weight was assumed as described below.
 - 2 - *Steel Skimmer Weight* : the skimmer weight was taken from the 2012 Stantec design calculations and placed at the location of the corrugated steel, since this is where the majority of the weight acts.
 - 3 - *Hydrostatic Loading* : the depth of the water in the pool was assumed to be 344.5 which is approximately the top of stoplog crest. The depth of groundwater was also taken to be 344.5 for the purpose of this calculation.
 - 4 - *Static Earth Pressure* : static earth pressure was computed using the Rankine active coefficient K_a for soil on the outlet side assuming tensile crack closure and the at-rest coefficient k_0 for soil on the inlet side.
 - 5 - *Dynamic Earth Pressure*: dynamic earth pressure was computed using three methods. Two methods (namely: Mononobe-Okabe and the General Wedge methods) came up with errors due to the fact that the ground acceleration is relatively high for this site. Therefore the simple, yet conservative Seed-Whitman method (see calculations below for more detail) was used to compute dynamic earth pressures. Dynamic earth pressures were resolved into horizontal and vertical (downward) components with respect to the wall friction angle δ .
 - 6 - *Inertial Force*: the weight of the entire structure (concrete and steel) plus the weight of the soil and water above the footing were multiplied by the maximum horizontal acceleration $MHA = kh$.
 - 7 - *Hydrodynamic Force*: the hydrodynamic force is used to simulate the water above the soil on the pond-side of the structure as it is mobilized during the earthquake event.
 - 8 - *Pipe Connection Capacity*: the capacity of the connecting pipes were used to provide lateral and overturning support to the structure.
- e) Stability Assessment: the structure was assessed for sliding stability and overturning. Friction and static earth pressure on the inlet side were used as a resistive forces, while all other lateral loads were used as sliding forces according to the sign convention (see item b above). Overturning was assessed based on the location of the resultant force on the base.

II Bearing Pressure

Bearing Pressure was computed as follows: $P/A + M/S$, where A is the total area of the base, S is the section modulus of the base, P is the resultant force and $M = P \times e$. "e" is the eccentricity of the resultant force with respect to the center of the base.

III Elongation of HDPE Pipe

The elongation of the HDPE pipe was checked using the net sliding force to ensure a reasonable displacement.



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SHF SEISMIC STABILITY

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Project: SHF Seismic Demo.
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SHF SEISMIC STABILITY

STRUCTURE LAYOUT

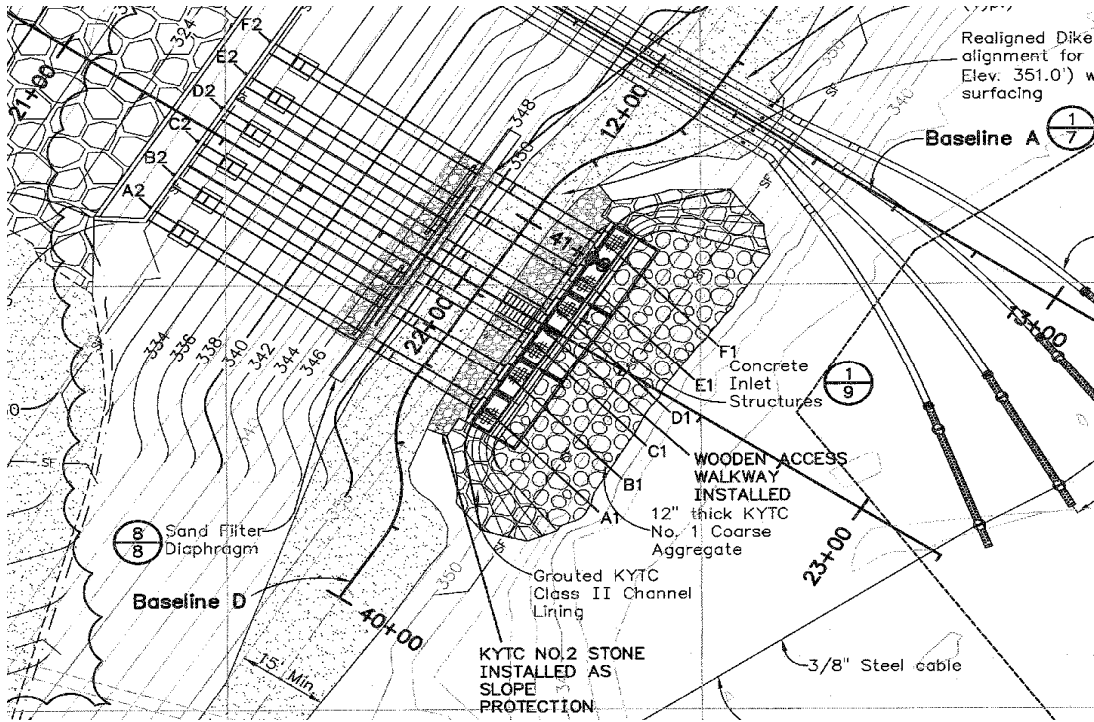


FIGURE 1 - Plan of Spillway (Stantec 2012)

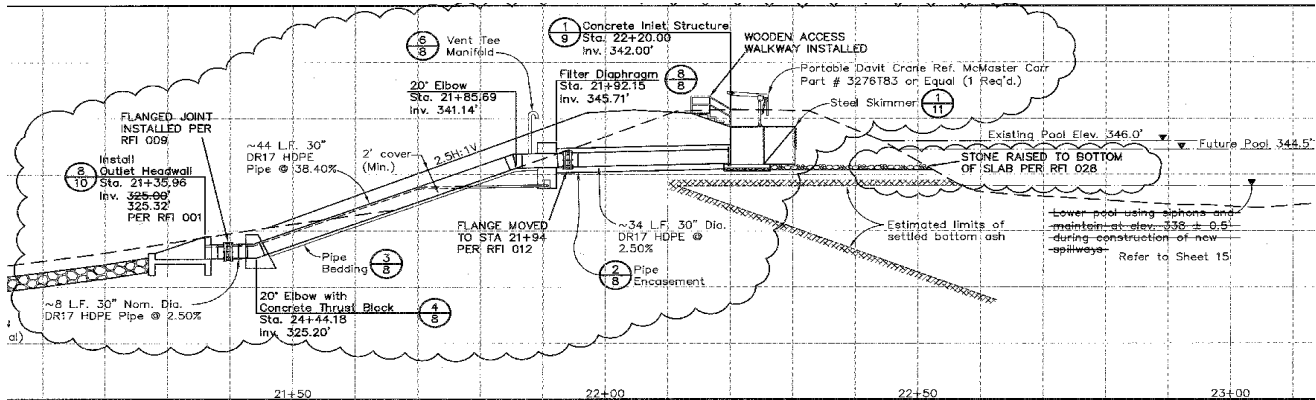


FIGURE 2 - Profile of Spillway (Stantec 2012)



Project: SHF Seismic Demo.
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SHF SEISMIC STABILITY

STRUCTURE GEOMETRY AND WEIGHT

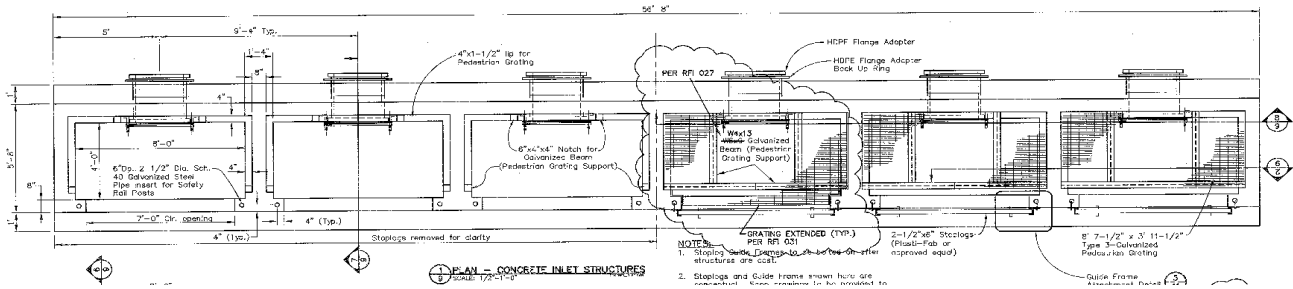


FIGURE 3 - Inlet Plan (Stantec 2012)

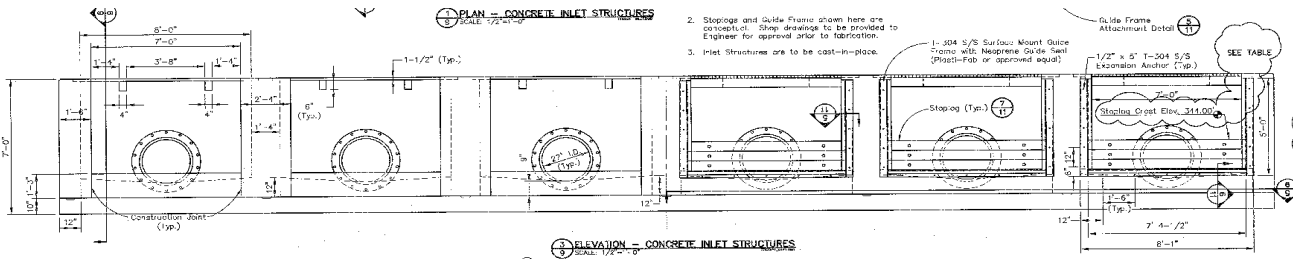


FIGURE 4 - Inlet Elevation (Stantec 2012)

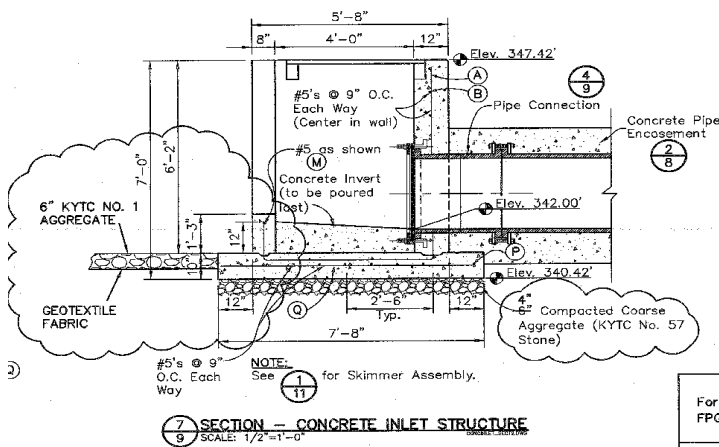


FIGURE 5 - Inlet Cross Section (Stantec 2012)

FOOTING	
thickness:	0.83 ft
width:	7.67 ft
length:	56.67 ft
INTERIOR WALLS	
thickness:	1.33 ft
width:	4.00 ft
height:	6.17 ft
number:	5
EXTERIOR SIDE WALLS	
thickness:	1.00 ft
width:	4.00 ft
height:	6.17 ft
number:	2
INTERIOR FRONT WALL STUBS	
thickness:	0.67 ft
width:	2.33 ft
height:	6.17 ft
number:	5



Project: SHF Seismic Demo.
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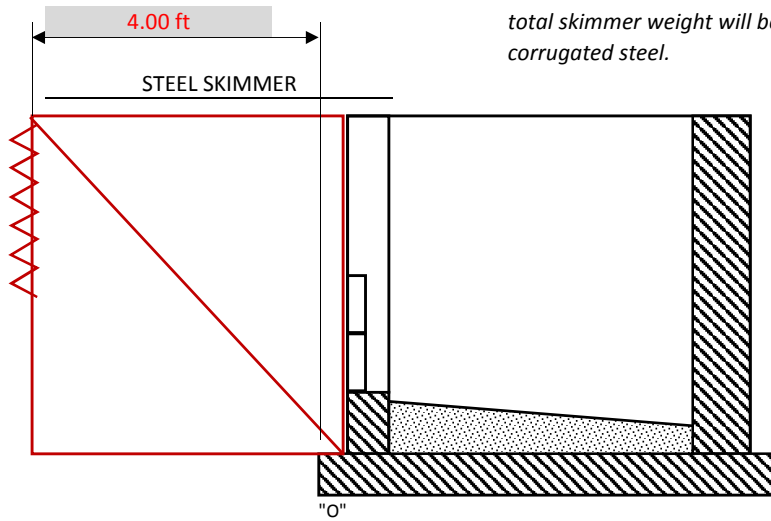
Calc. by: JEH
 Date: 6/6/2018
 Check by: JTP
 Date: 6/15/2018

SHF SEISMIC STABILITY

STEEL GRATING	
assumed weight*:	262.9 lb
number:	6
LESS PIPE VOLUME	
diameter:	2.50 ft
area:	4.91 ft ²
backwall thickness:	1.00 ft
number:	6
MISCELLANEOUS STEEL AND STOP LOGS	
assumed weight:	2000.0 lb
<i>*based on 7 lb/ft² assumed and 4'-4" x 8'-8" area.</i>	
Unit Weight (wc):	150 pcf
TOTAL STRUCTURE WEIGHT	
concrete volume:	1171.1 cf
concrete weight:	175.7 k
steel weight:	3.58 k
Total Weight (Ws):	179.24 k

@ 4.406 ft from "O" ↓

EXTERIOR FRONT WALL STUBS	
thickness:	0.67 ft
width:	1.50 ft
height:	6.17 ft
number:	2
BACKWALL	
thickness:	1.00 ft
height:	6.17 ft
length:	56.67 ft
FRONT WEIR	
thickness:	0.67 ft
width:	7.00 ft
height:	1.25 ft
number:	6
CONCRETE INVERT	
plan view area:	32.00 ft ²
average thickness:	0.94 ft
number:	6



The steel skimmer is attached as shown below. The location of the total skimmer weight will be assumed to act coincidentally with the corrugated steel.

Steel Skimmer Weight: 2.50 k per 2-box unit → 6 boxes
 Skimmer weight taken from Stantec Design Calculations (2012), page 122.

Total Skimmer Weight: 7.50 k @ -4.00 ft from "O" ↓

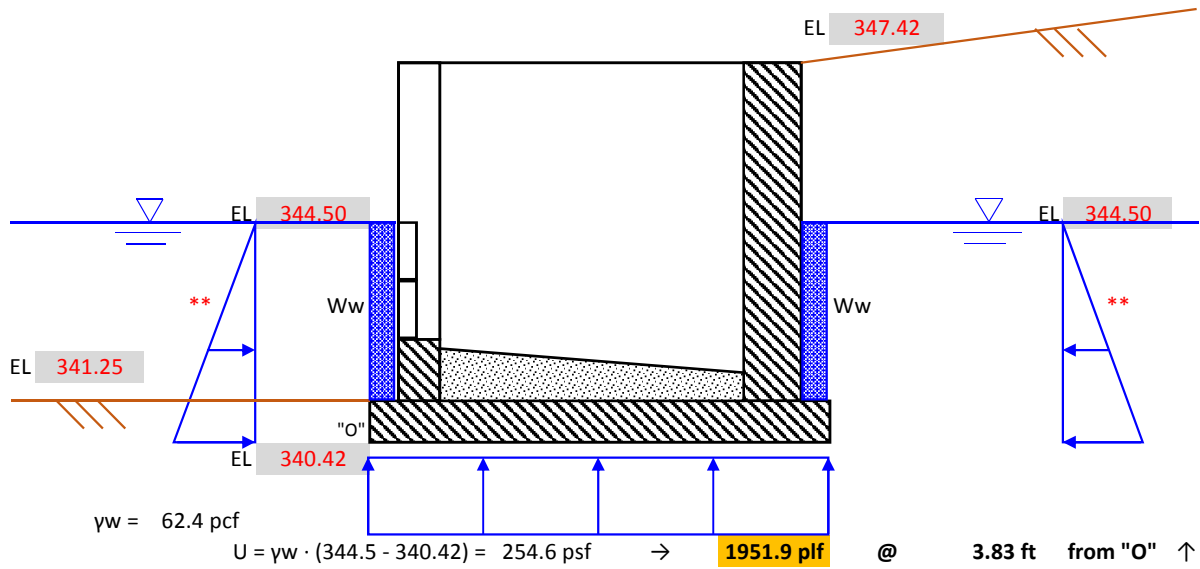


Project: SHF Seismic Demo.
 Job No.: 175555010
 Location: Ash Pond 2
 Contents: Seismic Stability

Calc. by: JEH
 Date: 6/6/2018
 Check by: JTP
 Date: 6/15/2018

SHF SEISMIC STABILITY

HYDROSTATIC LOADING



**Lateral loads from water on sides of structure will cancel each other. Therefore, they have been neglected.

$W_w = (344.5 - 341.25) \cdot 1\text{ft ledge} \cdot \gamma_w = 202.8 \text{ plf}$
 @ 0.500 ft from "O" ↓
 @ 7.167 ft from "O" ↓

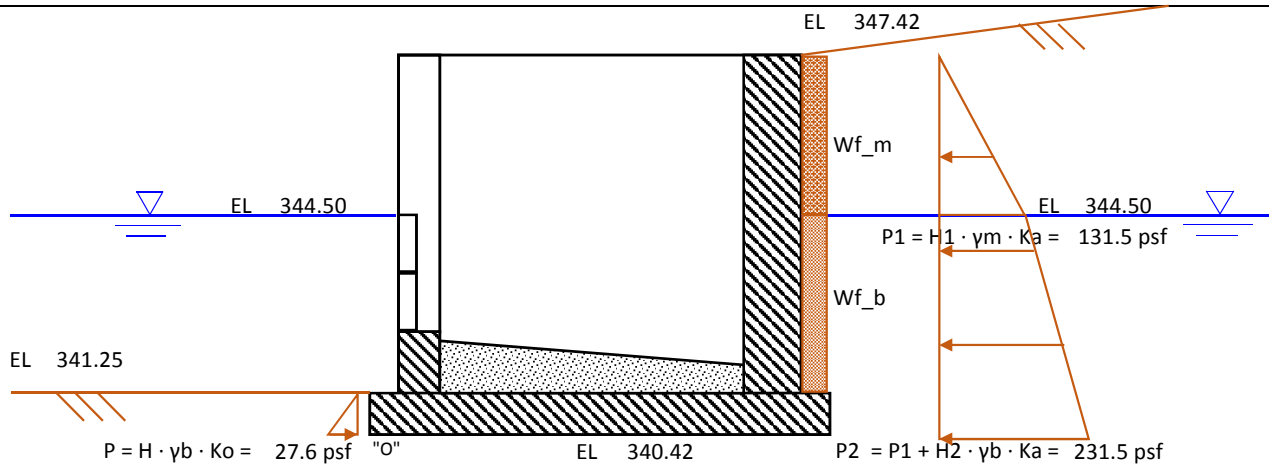


Project: SHF Seismic Demo.
 Job No.: 175555010
 Location: Ash Pond 2
 Contents: Seismic Stability

Calc. by: JEH
 Date: 6/6/2018
 Check by: JTP
 Date: 6/15/2018

SHF SEISMIC STABILITY

STATIC EARTH LOADING



Backfill Soil Properties:

$\phi = 28.0^\circ$ internal friction angle
 $\theta = 0.0^\circ$ wall inclination
 $\delta = 18.0^\circ$ wall friction angle
 $\beta = 12.4^\circ$ slope of backfill

$\gamma_m = 115.0 \text{ pcf}$
 $\gamma_{sat} = 125.0 \text{ pcf}$
 $\gamma_b = \gamma_{sat} - \gamma_w = 62.6 \text{ pcf}$
 Cohesion (c) = 500.0 psf Stantec Design Calcs. (2012)

$$K_a = \cos \beta \frac{\cos \beta - \sqrt{(\cos \beta)^2 - (\cos \phi)^2}}{\cos \beta + \sqrt{(\cos \beta)^2 - (\cos \phi)^2}} = 0.392$$

(Assume tensile crack closure and Rankine active earth pressure)

$$K_0 = (1 - \sin \phi) = 0.531$$

Inlet Side Lateral Soil Load (Ph): **11.44 plf** @ 0.277 ft from "O" →
 Outlet Side Lateral Soil Load (Ph): **200.0 plf** @ 7.67 ft from "O" ↓
911.0 plf @ 2.512 ft from "O" ←

Weight of Soil on Right footing

$Wf_m = \gamma_m \cdot 1\text{ft} \cdot (347.42 - 344.5)$: **335.8 plf** @ 7.167 ft from "O" ↓
 $Wf_b = \gamma_b \cdot 1\text{ft} \cdot (344.5 - 341.25)$: **203.5 plf** @ 7.167 ft from "O" ↓

SHF SEISMIC STABILITY

DYNAMIC EARTH LOADING

Maximum Horizontal Acceleration (kh): **0.724g** See Maximum Horizontal Acceleration calc. kv = neglect

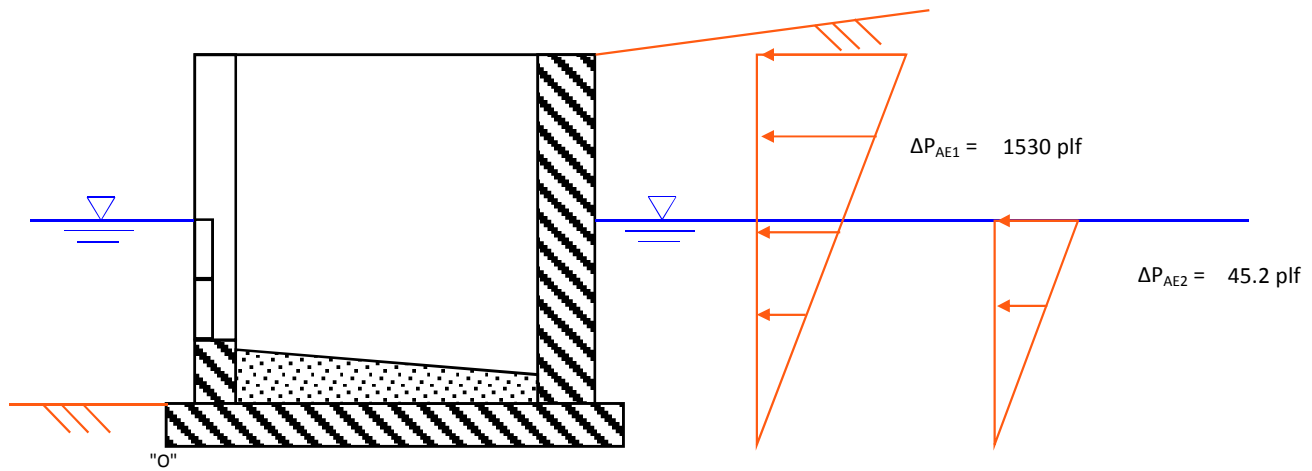
Following Seed-Whitman (1970) Publication:

$$\Delta K_{AE} = \frac{3}{4}k_h = 0.543$$

$$\Delta P_{AE1} = 0.5\Delta K_{AE}\gamma_m h^2 = 1530 \text{ plf}$$

$$\Delta P_{AE3} = 0.5\Delta K_{AE}(\gamma_{sat} - \gamma_m)h_s^2 = 45.2 \text{ plf}$$

$\gamma_m = 115.0 \text{ pcf}$ moist unit weight
 $\gamma_{sat} = 125.0 \text{ pcf}$ saturated unit weight
 $\gamma_b = \gamma_{sat} - \gamma_w = 62.6 \text{ pcf}$ bouyant unit weight
 $h = (EL_{top} - EL_b) = 7.0 \text{ ft}$ height of wedge
 $h_s = (EL_{gw} - EL_b) = 4.08 \text{ ft}$ height of wedge below water



Total Dynamic Earth Force: **1575 plf** @ **4.150 ft** from "O"

Select Method and Resolve Dynamic Forces into Vertical and Horizontal Components

SELECT METHOD:

1: SEED-WHITMAN

$\delta = 18.0^\circ$ wall friction angle

Vertical and horizontal components were resolved assuming the dynamic earth force computed above acts at an angle δ from the horizontal, such that the vertical component acts downward.

$\sin(\delta) = 0.309$
 $\cos(\delta) = 0.951$

Vertical Dynamic Lateral Force:	487 plf	@	7.667 ft from "O"	↓	
Horizontal Dynamic Lateral Force:	1498 plf	@	4.150 ft from "O"	←	428.0019
Static Lateral Earth Force (Outlet Side):	200 plf	@	7.667 ft from "O"	↓	
	911 plf	@	2.512 ft from "O"	←	



Project: SHF Seismic Demo.
Job No.: 175555010
Location: Ash Pond 2
Contents: Seismic Stability

Calc. by: JEH
Date: 6/6/2018
Check by: JTP
Date: 6/15/2018

SHF SEISMIC STABILITY

INERTIAL FORCE

Inertial Force of Entire Structure with Soil and Water above Footing

$$W_t = W_s + \text{Skimmer} + (W_{f_m} + W_{f_b} + 2 \cdot W_w) \cdot \text{base length} = 240.28 \text{ k}$$

$$P_i = W_t \cdot k_h = 173.97 \text{ k} \quad @ \quad 2.373 \text{ ft} \quad \text{from "O"} \quad \leftarrow$$

USACE EM 1110-2-2502
[3-101]

Inertial Force of Backwall Segment with Soil and Water above Footing, Directly Adjacent to Wall (for backwall reinforcement check)

$$W_t = w_c \cdot (\text{backwall width}) \cdot (\text{backwall height}) \cdot (\text{backwall thickness}) + (W_{f_m} + W_{f_b} + W_w) \cdot (\text{backwall width}) =$$

$$W_t = 16.14 \text{ k}$$

$$P_i = W_t \cdot k_h = 11.687 \text{ k}$$

Distribute as uniform pressure against backwall: 0.207 ksf

HYDRODYNAMIC FORCE DUE TO WATER ABOVE GROUND LEVEL

$$P_E = \left(\frac{2}{3}\right) C_E k_h h^2 = 0.260 \text{ klf} \quad @ \quad 2.133 \text{ ft} \quad \text{from "O"} \quad \leftarrow$$

USACE EM 1110-2-2502
[3-102] Figure 3-39

CE = 0.051 EM 1110-2-2502, (Section 3-26.e) allows for the use of 0.051 for CE. This will be used since the earthquake period (T) is not known.

$$h = (344.5 - 341.25) = 3.25 \text{ ft}$$

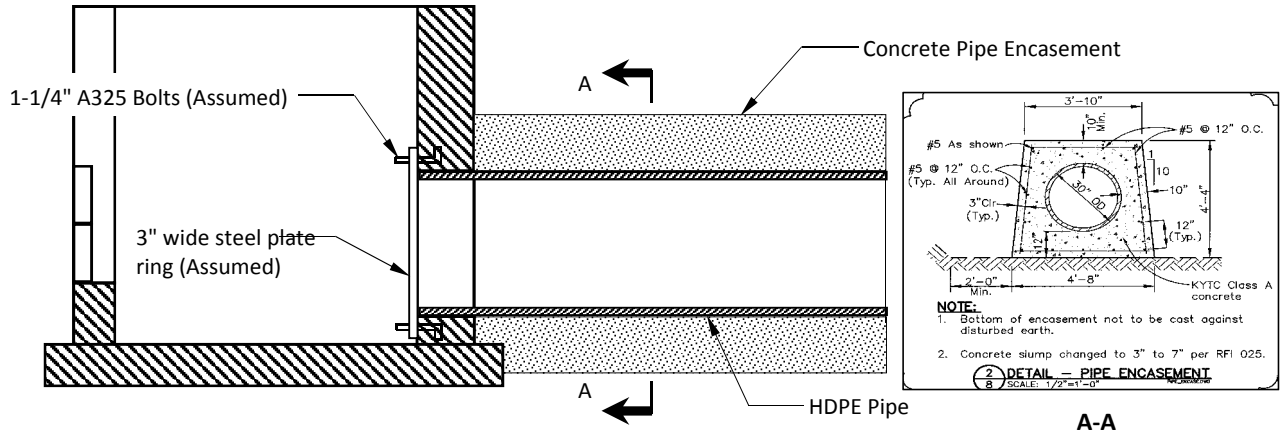


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SHF SEISMIC STABILITY

CONNECTING PIPE CAPACITY



Tensile Capacity of HDPE Pipe

Tensile Strength:	3.60 ksi
Area of Pipe:	166 in ²

[JMM HDPE Product Specification]

[JMM HDPE Product Specification]

For 6 Pipe Connections:

Tensile Capacity of HDPE Pipe [(0.5) (3.6 ksi) (166 in²)]: 298.80 k

→ 1793 k



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SHF SEISMIC STABILITY

SUMMATION OF FORCES AND MOMENTS

Select Load Condition: **Normal Pool w/ MDE**

Description	Force	Direction	Arm	Moment
<i>Vertical</i>				
Structure Weight	179.24 k	↓	4.41 ft	-790 k-ft
Weight of Skimmer	7.50 k	↓	-4.00 ft	30 k-ft
Water Weight (Outlet Side)	11.49 k	↓	7.17 ft	-82 k-ft
Water Weight (Inlet Side)	11.49 k	↓	0.50 ft	-6 k-ft
Uplift	110.61 k	↑	3.83 ft	424 k-ft
Static Earth Pressure (Outlet Side)	11.33 k	↓	7.67 ft	-87 k-ft
Vertical Dynamic Lateral Earth	27.58 k	↓	7.67 ft	-211 k-ft
TOTAL VERTICAL (P)	138.04 k	↓	--	--
<i>Horizontal</i>				
Static Earth Pressure (Outlet Side)	51.62 k	←	7.67 ft	396 k-ft
Dynamic Earth Pressure on Outlet Side	84.89 k	←	4.15 ft	352 k-ft
Inertial Force of Structural Wedge	173.97 k	←	2.37 ft	413 k-ft
Hydrodynamic Force Due to Water on Inlet Side	14.734 k	←	2.13 ft	31.4 k-ft
TOTAL HORIZONTAL (V)	325.21 k	←	--	--
<i>Resistance</i>				
Static Earth Pressure (Inlet Side)	0.65 k	→	0.277 ft	-0.2 k-ft
Base Resistance ($\tan(d) \cdot P + c \cdot (\text{base area})$)	79.70 k	→		KYTC 57 Stone
Resistance from Pipe (Max)	1792.80 k	→		
Resistance from Pipe (Req)	244.86 k	→		
TOTAL RESISTANCE (F)	1873.14 k	→	--	--
TOTAL MOMENT (M)	--	--	--	-184 k-ft

Required Sliding Factor of Safety: 1.1

Sliding Factor of Safety (F/V): 5.760 OK

Eccentricity ($e = M/P - b/2$): 2.501 ft ≤ $b/2 = 3.833$ ft OK *measured from center of base*



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SHF SEISMIC STABILITY

BEARING PRESSURES

Area of Base: 434.4 ft² Max Allowable Soil Pressure: 1.733 ksf
Section Modulus of Base: 555.1 ft³ Stantec Design Calcs. (2012) (50% increase per EM1110-2-2100 Section 3.10)

q_{max}	1.219 ksf	OK
q_{min}	0.000 ksf	OK

(positive denotes compression)

ELONGATION OF HDPE PIPE

Net tension on 1 pipe:	40.81 k
Tensile stress on 1 pipe (σ):	0.246 ksi
Elastic Modulus of HDPE (E_H):	120 ksi

Strain in HDPE ($\epsilon = \sigma/E_H$):	0.0020487
Length of Horizontal Pipe to apply strain to (L):	34.0 ft

Total Elongation ($\epsilon \cdot L \cdot 12$):	0.836 in
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DISTRIBUTED PRESSURES ON WALL

Project: Shawnee Fossil Plant - SIZ

Load Case: LC1 - 1.2(D+F) + 1.6H + 1.0E

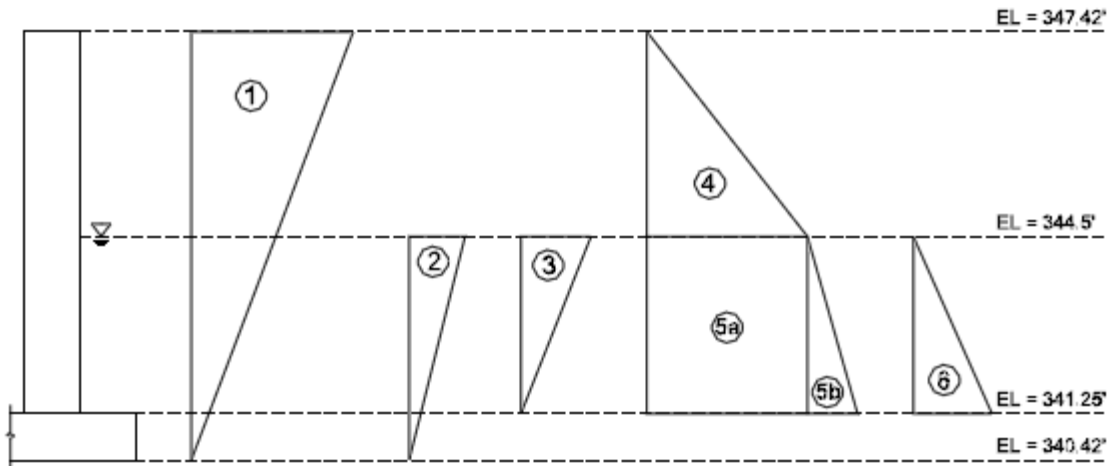
Input By : PRS

Date : 06/08/2018

Section Location: Back Wall

Input Checker: JTP

Date: 6/15/2018

Load Diagram (acting towards wall)


Refer to file named "SHF Seismic Stability" for resultant forces

1. Dynamic Soil Pressure for Moist Soil

Resultant Force: $P_1 := 1498\text{plf}$
Distributed Height: $h_1 := 347.42\text{ft} - 340.42\text{ft} = 7\text{-ft}$
Distributed Pressure: $w_1 := \frac{2 \cdot P_1}{h_1^2} = 61.14 \cdot \frac{\text{psf}}{\text{ft}}$
SAP2000 Input:

Joint Pattern: Seismic

Load Pattern: Seismic

2. Additional Dynamic Soil Pressure for Saturated Soil

Resultant Force: $P_2 := 45.2\text{plf}$
Distributed Height: $h_2 := 344.5\text{ft} - 340.42\text{ft} = 4.08\text{-ft}$
Distributed Pressure: $w_2 := \frac{2 \cdot P_2}{h_2^2} = 5.43 \cdot \frac{\text{psf}}{\text{ft}}$
SAP2000 Input:

Joint Pattern: Seismic

Load Pattern: Seismic

3. Hydrodynamic Load:

Resultant Force: $P_3 := 260\text{plf}$ acting at $h_3 := 2.133\text{ft}$

SAP2000 Input:

Calculate Equivalent Triangular Pressure Distribution:

Joint Pattern: Seismic_Hydro

Moment: $M_3 := P_3 \cdot h_3 = 554.58 \cdot \frac{\text{ft} \cdot \text{lbf}}{\text{ft}}$

Load Pattern: Seismic

Distributed Height: $h_3 := 344.5\text{ft} - 341.25\text{ft} = 3.25 \cdot \text{ft}$

Resultant Force Equation: $P_3 = W_3 \cdot h_3 / 2$

Moment Equation: $M_3 = P_3 \cdot (2h_3 / 3 + 0.83\text{ft})$

Equivalent Pressure at Top: if $M_3 = M_3$: $W_3 := \frac{M_3}{\frac{1}{2}h_3 \cdot \left(\frac{2}{3}h_3 + 0.83\text{ft}\right)} = 113.89 \cdot \text{psf}$

Equivalent Dist. Pressure: $w_3 := \frac{W_3}{h_3} = 35.04 \cdot \frac{\text{psf}}{\text{ft}}$

4. Lateral Earth Pressure: Moist Soil

Moist Soil Unit Weight: $\gamma_m := 115\text{pcf}$

SAP2000 Input:

Active Earth Pressure Coefficient: $K_a := 0.392$

Joint Pattern: Soil

Distributed Height: $h_4 := 347.42\text{ft} - 344.5\text{ft} = 2.92 \cdot \text{ft}$

Load Pattern: Soil

Distributed Pressure: $w_4 := \gamma_m \cdot K_a = 45.08 \cdot \frac{\text{psf}}{\text{ft}}$

5a. Lateral Earth Pressure: Saturated Soil - Pressure due to Surcharge

Distributed Height: $h_5 := 344.5\text{ft} - 341.25\text{ft} = 3.25 \cdot \text{ft}$

SAP2000 Input:

Distributed Pressure: $w_{5a} := w_4 \cdot h_4 = 131.63 \cdot \text{psf}$

Applied as uniform pressure

Load Pattern: Soil

5b. Lateral Earth Pressure: Saturated Soil - Triangular Pressure Distribution

Saturated Soil Unit Weight: $\gamma_{\text{sat}} := 125\text{pcf}$

SAP2000 Input:

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

Joint Pattern: Soil_2

Distributed Pressure: $w_{5b} := (\gamma_{\text{sat}} - \gamma_w) \cdot K_a = 24.54 \cdot \frac{\text{psf}}{\text{ft}}$

Load Pattern: Soil

6. Hydrostatic Pressure

Distributed Height: $h_6 := 344.5\text{ft} - 341.25\text{ft} = 3.25 \cdot \text{ft}$

SAP2000 Input:

Distributed Pressure: $w_6 := \gamma_w = 62.4 \cdot \frac{\text{psf}}{\text{ft}}$

Joint Pattern: Hydro

Load Pattern: Hydro

SAP2000 MODEL OUTPUTS

PROJECT: SHAWNEE FOSSIL PLANT - SIZ

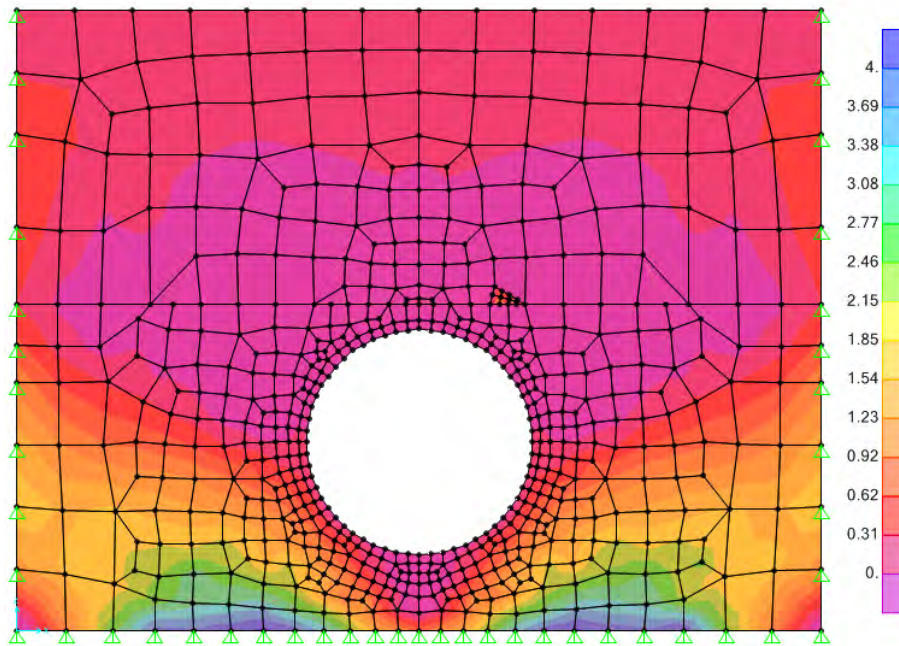
Input By: PRS

Checker: JTP

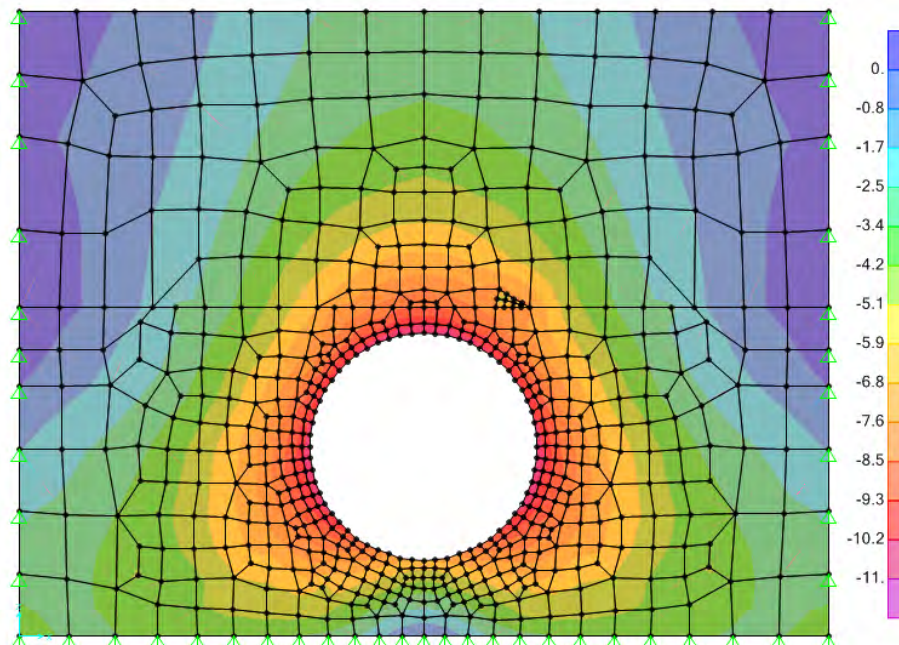
Date: 06/08/2018

Date: 6/15/2018

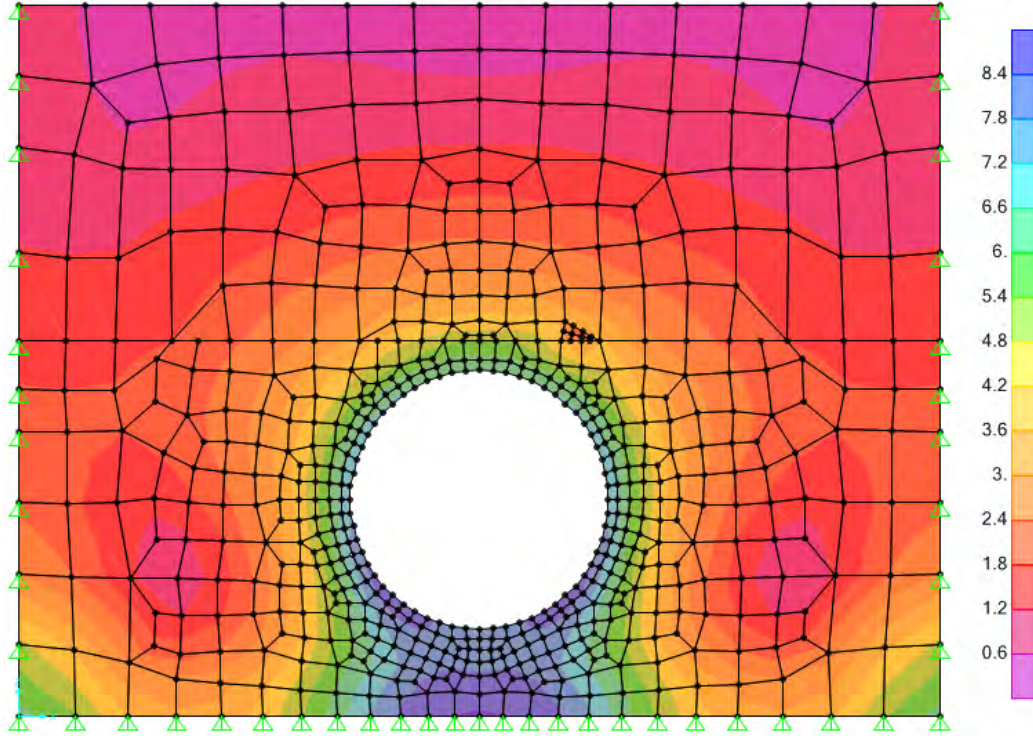
$M_{\max} = 4.08$ ft-kip (Service $M_{\max} = 3.37$ ft-kip)



$M_{\min} = -11.09$ ft-kip (Service $M_{\min} = 9.09$ ft-kip)



$V_{max} = 8.46 \text{ kip}$



Section Cut 6in from Base – used to determine total shear along the cut plane:

TABLE: Section Cut Forces - Design						
OutputCase	P	V2	V3	T	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
S1	-8.329	-0.00139	-17.543	-1.831	10.325	1.734
LC1	-9.995	-0.00167	-21.031	-2.138	12.271	2.081
S2	-8.329	-0.00139	-17.543	-1.831	10.325	1.734
LC2	-7.496	-0.00125	-14.858	-1.285	8.332	1.561

REINFORCEMENT DESIGN FOR WALL SECTIONS

Project: Shawnee Fossil Plant - SIZ

Load Case: LC1 - 1.2(D+F) + 1.6H + 1.0E

Section Location: Back Wall

Input By : PRS

Input Checker: JTP

Date : 06/08/2018

Date: 6/15/2018

Assumptions:

1. This sheet calculates flexural and shear capacities for a wall cross section per ACI 350-06
2. Controlling shear and moments are from a SAP2000 Model of the Structure

1.0 PROVIDED DATA AND PARAMETERS

Section Information

- Wall Width: $b_s := 8\text{ft}$
- Member Thickness: $h := 12\text{in}$
- Width of Analysis Section: $b_w := 1\text{ft}$

Concrete and Reinforcement Properties

- Concrete Strength: $f_c := 4000\text{psi}$
- Steel Yield Strength: $f_y := 60\text{-ksi}$
- Moment Reduction Factor: $\phi_m := 0.9$ ACI 350-06 - 9.3.2.1
- Shear Reduction Factor: $\phi_v := 0.75$ ACI 350-06 - 9.3.2.3
- Correction Factor Related to Unit Weight of Concrete: $\lambda := 1.0$ ACI 350-06 - 11.7.4.3

Flexural Reinforcement Details

- Flexural Bar Diameter: $d_b := 0.625\text{in}$ #5 bar
- Flexural Bar Area: $A_b = 0.31\cdot\text{in}^2$
- Reinforcement Spacing: $s_w := 9\text{in}$
- Concrete Cover : $\text{cov} := 0.5(h - d_b) = 5.69\cdot\text{in}$ (reinforcement centered in wall)

Shrinkage & Temperature Reinforcement Details

- S&T Bar Area: $A_{st} := 0.31\text{in}^2$ #5 bar
- S&T Reinforcement Spacing: $s_{st} := 9\text{in}$

2.0 DETERMINE CRITICAL MOMENT AND SHEAR (over width b_s for shear and b_w for moment)

Controlling Loads:

- Load Factor:

Load factors from ACI 350-06, 9.2.1, factors already included in SAP2000 Model

- Controlling Service Moment: $M_{ser} := 9.09 \text{ft} \cdot \text{kip}$

- Controlling Factored Shear: $V_f := 21.031 \text{kip}$ Tributary to Wall Width

- Controlling Factored Moment: $M_u := 11.09 \text{ft} \cdot \text{kip}$

- Ratio of Factored to Service Moments: $\gamma_{flex} := \max\left(1.0, \frac{M_u}{M_{ser}}\right) = 1.2$

- For the Extreme Load Case the Environmental Durability Factor does not apply (ACI 350-06 - 21.2.1.8.a)

3.0 SHEAR AND MOMENT CAPACITY DESIGN (ACI 350-06)
Factored Shear Strength (ACI 350 - 11.3.2, 11.5.5):

- Gross Cross Sectional Area: $A_g := h \cdot b_w = 144 \cdot \text{in}^2$

- Dist. from Comp. Face to CL of Tension Reinforcement: $d := h - \left(\text{cov} + \frac{d_b}{2}\right) = 6 \cdot \text{in}$

- Concrete Shear Strength: $\phi V_c := \phi_v \cdot 2 \cdot \text{psi} \cdot \sqrt{\left(\frac{f_c}{\text{psi}}\right)} \cdot \lambda \cdot d \cdot b_s = 54.64 \cdot \text{kip}$ ACI 350 - 11.3.2.3

- Design Factored Shear: $V_u := V_f = 21 \cdot \text{kip}$

- Shear Capacity Demand Ratio: $PR_V := \frac{\phi V_c}{V_u} = 2.60$

$\text{if}(PR_V \geq 1.0, "OK", "NG") = "OK"$

Factored Moment Strength: ACI 350 - 10.2

- Stress Block Factor: $\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \leq 4000\text{psi} \\ \left[0.85 - 0.05 \left(\frac{f'_c - 4000\text{psi}}{1000\text{psi}} \right) \right] & \text{if } 4000\text{psi} < f'_c < 8000\text{psi} \\ 0.65 & \text{if } f'_c \geq 8000\text{psi} \end{cases} = 0.85$
 ACI 350 - 10.2.7.3

- Area of Moment Reinforcement: $A_s := \frac{A_b \cdot b_w}{s} = 0.41 \cdot \text{in}^2$

- Dist. from Comp. Face to N.A.: $c := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b_w} = 0.71 \cdot \text{in}$ ACI 350 - 10.2.7.1, 10.2.7.2

- Strain in Steel for Concrete Crushing: $\epsilon_s := 0.003 \cdot \left(\frac{d - c}{c} \right) = 0.022$ > 0.005, Tension controlled, **OK**
 ACI 350 - 10.3.4

- Available Moment Strength: $\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d - \frac{\beta_1 \cdot c}{2} \right) = 10.49 \cdot \text{kip} \cdot \text{ft}$

- Controlling Factored Moment: $M_u = 11.09 \cdot \text{kip} \cdot \text{ft}$

- Moment Capacity Demand Ratio: $PR_M := \frac{\phi M_n}{M_u} = 0.95$ $\text{if}(PR_M \geq 1.0, "OK", "NG") = "NG"$

Note: Even though the available moment strength, $\phi M_n = 10.49 \text{ ft} \cdot \text{kip}$, is less than the Factored Moment, $M_u = 11.09 \text{ ft} \cdot \text{kip}$, this design is considered acceptable because difference is less than 10%, and this elastic design procedure is known to be conservative.