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October 17, 2018
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Revision 0

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

RE: Seismic Impact Zones
Active Ash Pond 2
EPA Final Coal Combustion Residuals (CCR) Rule
TVA Johnsonville Fossil Plant
New Johnsonville, Tennessee

1.0 PURPOSE

As described in 40 CFR § 257.63(a), an owner or operator of an existing CCR surface impoundment is required to demonstrate that the unit is not located in seismic impact zones unless the unit meets certain requirements. This letter documents Stantec's certification that Active Ash Pond 2 at the TVA Johnsonville Fossil Plant (JOF) 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 Active Ash Pond 2 meets the requirements set forth in 40 CFR § 257.63(a).

3.0 QUALIFIED PROFESSIONAL ENGINEER CERTIFICATION

I, Stephen H. Bickel, being a Professional Engineer in good standing in the State of Tennessee, do hereby certify, to the best of my knowledge, information, and belief:

1. that the information contained in this certification is prepared in accordance with the accepted practice of engineering;

2. that the information contained herein is accurate as of the date of my signature below; and

3. that the TVA Johnsonville Active Ash Pond 2 meets the requirements specified in 40 CFR § 257.63(a).

SIGNATURE	[signature]	DATE 10/16/2018

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ATTACHMENTS: Seismic Impact Zones Demonstration

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STATE OF TENNESSEE

Seismic Impact Zone Demonstration

Active Ash Pond 2
Johnsonville Fossil Plant
New Johnsonville, Humphreys County, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga Tennessee

Prepared by:
Stantec Consulting Services Inc.
Lexington, Kentucky

October 16, 2018
Revision 0

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1.0 INTRODUCTION

On April 17, 2015, EPA published the “Disposal of Coal Combustion Residuals (CCR) from Electric Utilities” final rule in the Federal Register. The Tennessee Valley Authority (TVA) contracted Stantec Consulting Services Inc. (Stantec) to evaluate the Active Ash Pond 2 at the Johnsonville Fossil Plant (JOF) 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. The objective of this report is to determine if the JOF Active Ash Pond 2 meets the requirements for seismic impact zones.

1.2 UNIT DESCRIPTION

JOF is located on a 685-acre reservation in New Johnsonville, Humphreys County, Tennessee. The plant is on the east bank of Kentucky Lake reservoir, approximately 12 miles west from Waverly, Tennessee, and approximately 70 miles west from Nashville, Tennessee.

Figure 1: Site Vicinity Map

Referring to Figure 1, the Unit is centered approximately 2,000 feet west from the plant's powerhouse. It was created by placing fill and then building an approximate two-mile-long perimeter dike, on an area within the former Tennessee River floodplain (now inundated by Kentucky Lake), to enclose approximately 90 acres. The perimeter dike varies from 25 to 35 feet in height.

The Unit has been in operation since 1970. It formerly received sluiced fly ash and bottom ash and plant process water. It also received stormwater runoff pumped from the Coal Yard Drainage Basin. The last coal fired generating units were shut down in December 2017, therefore the Unit no longer receives fly ash or bottom ash.

TVA has determined that the Unit is a CCR Surface Impoundment and therefore subject to the final rule.

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.

3.0 DEMONSTRATION

Active Ash Pond 2 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 Johnsonville 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 7.02 M_w (moment magnitude) event with a peak horizontal acceleration of 0.214g 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 Active Ash Pond 2 at the new spillway site 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.

Table 1. Generalized Subsurface Conditions (adapted from Stantec 2016)

Materials	Approximate Elevation (feet)	General Consistency/Density
Upper Clay Dike – lean clay, lean clay with sand, and lean clay with gravel	378 to 390	Medium stiff to very stiff
Bottom Ash - silty sand with gravel and silt	Below Upper Clay Dike (bridging layer over Ash)	Medium dense to dense
Ash – silty sand with gravel and silt	Below Bottom Ash	Very Loose
Lower Clay Dike – lean clay, lean clay with sand, lean clay with gravel, and silt	370 to 378	Medium stiff to very stiff
Fill – lean clay, sandy lean clay, lean clay with gravel, silt, and silt with sand	334 to 370	Soft to medium stiff
Alluvial Clay and Silt – lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, silt, and silt with gravel	320 to 334	Very soft to medium stiff
Alluvial Sand and Gravel – silty sand, silty sand with gravel, poorly graded sand with or without silt and gravel, well graded sand with or without silt and gravel, and poorly graded gravel with or without silt and sand	Below 320	Medium dense to 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 100 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 liquefaction is expected in the ash at the spillway structures during the design earthquake. As such, the post-earthquake slope stability analysis did consider liquefied soil strengths in this material. Refer to Appendix A for details of the liquefaction assessment.

3.4 SEISMIC SLOPE STABILITY AND DISPLACEMENT ANALYSES

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

The results indicate stable slopes for the post-earthquake conditions, while permanent displacements (i.e., deformations) 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. Sliding failure mechanisms were evaluated along two potential slip planes: one between the precast inlet units and the slab-on-grade and the other between the slab-on-grade and underlying soil. As part of the sliding analyses, elongation of the HDPE pipe was checked using the net sliding force to ensure a reasonable displacement. Overturning was assessed based on the location of the resultant force on the base of the precast inlet units. Bearing failures were also assessed based on a trapezoidal or triangular pressure distribution depending on the eccentricity of the applied vertical load.

The results indicate the structure is stable for the failure mechanisms discussed above utilizing some resistance (less than 10%) of the tensile capacity of the outlet pipes in the analyses. 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, slide, or overturn during the seismic event, assuming some resistance from the outlet pipes which are connected to the structure. The expected embankment deformation might cause the flexible HDPE outlet pipes to bend or pinch, but this would not release additional water above what is discharged during normal pond operations. The analyses show that the outlet structure will remain 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 the structure will be identified and remediation measures will be implemented to repair the damage and modify operations, if necessary. Implementation of the EAP will reduce risk of a progressive failure and harmful release of CCR.

4.0 CONCLUSION

Based on this assessment, Active Ash Pond 2 at JOF meets the requirements of §257.63 of the EPA Final CCR Rule for seismic impact zones.

5.0 REFERENCES

Geocomp (2016). “Initial Seismic Safety Factor Assessment, EPA Final CCR Rule, TVA Johnsonville Fossil Plant Active Ash Pond 2, New Johnsonville, Tennessee,” Prepared for Tennessee Valley Authority, October.

Stantec (2010a). “Report of Geotechnical Exploration and Slope Stability Evaluation, Ash Disposal Areas 2 and 3 (Active Ash Disposal Area), Johnsonville Fossil Plant, New Johnsonville, Tennessee,” Prepared for Tennessee Valley Authority, April.

Stantec (2010b). “Plans for Construction. Spillway Replacement Project, Ash Disposal Area No. 2, Work Plan 3 (JOF-090515-WP-3). Johnsonville Fossil Plant, New Johnsonville, Humphreys County, Tennessee,” Prepared for Tennessee Valley Authority, April.

Stantec (2011). 10W505 R2. “Ash Disposal Area No. 2 – Existing Spillway Closure Project Work Plan 4 (JOF-100407-WP-4), Johnsonville Fossil Plant.” Prepared for Tennessee Valley Authority. April 19.

Stantec (2016). “Initial Static Safety Factor Assessment, Active Ash Pond 2, EPA Final CCR Rule, TVA Johnsonville Fossil Plant, Humphreys County, Tennessee,” Prepared for Tennessee Valley Authority, October.

APPENDIX A
GEOTECHNICAL ANALYSES

Tennessee Valley Authority Johnsonville Fossil Plant Humphreys County, Tennessee	**Seismic Impact Zones** **Slope Stability Analysis**

Calculation Package N/A

Stantec

Exhibit
Seismic Impact Zones

Purpose:

- Evaluate the slope stability of the dike at the spillway location for Active Ash Pond 2, considering pseudostatic and post-earthquake load conditions.

Methods:

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

Results:

- Liquefaction is anticipated in the Ash for the design earthquake.
- The factor of safety for post-earthquake load conditions is adequate.
- The estimated displacement for the pseudostatic load condition (with liquefaction in the Ash) is less than about 2 feet.

Calculation Performed by: Stantec Consulting Services Inc.; August 14, 2018

Prepared by: Stephen Karpenko	*Reviewed by:* Jeffrey S. Dingrando, PE, PG

Revisions: N/A

TVA JOHNSONVILLE ACTIVE 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 Active 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).

The following sections present the data and calculations performed for a refined analysis for the embankment at the Active Ash Pond 2 spillway structure.

2. SUMMARY OF DRILLING AND LAB TESTING RESULTS

The following geotechnical explorations performed at the Active Ash Plant have collected data that were used as a basis for this analysis:

- Stantec 2010
- Geocomp 2016a and 2016b
- Stantec 2016
- ConeTec 2018

The explorations consisted of advancing geotechnical borings and/or Cone Penetration Test (CPT) soundings. A list of the existing borings used herein is presented in Table 1 and shown on Plates 1 through 5 (see Attachment A).

Table 1. As-Drilled Borings and Soundings Locations

Boring	Type	Total Depth (ft)	Coordinates		Ground Surface Elevation (ft)	Source
			Northing (ft)	Easting (ft)		
STN-B-5	SPT	31.5	598,852.56	1,409,642.08	389.9	Stantec (2010)
STN-B-6	SPT	31.5	598,911.82	1,409,640.12	389.9	Stantec (2010)
STN-B-7	SPT	31.5	598,973.53	1,409,636.42	390.1	Stantec (2010)
STN-B-8	SPT	31.5	599,039.79	1,409,640.20	389.9	Stantec (2010)
STN-B-9	SPT	31.5	599,129.97	1,409,640.48	389.7	Stantec (2010)
STN-B-10	SPT	31.5	599,391.63	1,409,727.39	389.1	Stantec (2010)
STN-HC	SPT	61.5	599,345.93	1,409,646.07	389.5	Stantec (2010)

Boring	Type	Total Depth (ft)	Coordinates		Ground Surface Elevation (ft)	Source
			Northing (ft)	Easting (ft)		
STN-HM	SPT	46.5	599,331.00	1,409,595.58	377.9	Stantec (2010)
STN-HT	SPT	51.5	599,308.41	1,409,545.23	363.1	Stantec (2010)
STN-AC-PZ	SPT	26.0	603,133.83	1,410,898.66	391.6	Stantec (2010)
STN-BC-PZ	SPT	26.0	602,314.45	1,410,988.89	392.4	Stantec (2010)
STN-FC-PZ	SPT	26.0	598,892.27	1,410,076.24	389.8	Stantec (2010)
STN-GC-PZ	SPT	27.0	598,696.58	1,409,758.99	389.8	Stantec (2010)
STN-HC-PZ	SPT	27.0	599,314.96	1,409,635.77	390.0	Stantec (2010)
STN-IC-PZ	SPT	32.0	600,086.63	1,409,629.83	390.1	Stantec (2010)
STN-JC-PZ	SPT	26.0	600,825.25	1,409,856.37	390.0	Stantec (2010)
STN-KC-PZ	SPT	26.0	601,483.81	1,410,099.66	390.5	Stantec (2010)
JOF-B-2A	SPT	97.5	623,697.73	1,379,502.95	392.7	Geocomp (2016)
JOF-B-2B	SPT	74.3	623,696.28	1,379,573.90	371.0	Geocomp (2016)
JOF-C-2A	SPT	99.1	622,832.82	1,379,600.69	392.8	Geocomp (2016)
JOF-C-2B	SPT	76.8	622,824.28	1,379,662.63	370.6	Geocomp (2016)
JOF-E-2A	SPT	98.9	620,872.69	1,378,948.54	390.9	Geocomp (2016)
JOF-E-2B	SPT	75.0	620,845.71	1,379,021.93	365.4	Geocomp (2016)
JOF-K-2A	SPT	76.0	622,821.21	1,378,562.35	377.5	Geocomp (2016)
JOF-27	CPT	63.7	599,533.00	1,409,942.90	389.7	Stantec (2016)
JOF-28	CPT	53.3	599,382.90	1,409,722.30	388.6	Stantec (2016)
AAP2-CPT-01	CPT	51.2	620,854.04	1,378,313.83	390.0*	ConeTec (2018)
AAP2-CPT-02	CPT	62.8	620,960.84	1,378,611.78	390.0*	ConeTec (2018)
AAP2-CPT-03	CPT	57.3	621,344.39	1,378,729.03	390.0*	ConeTec (2018)
VST-01	VST	45.0	620,860.17	1,378,370.15	390.0*	ConeTec (2018)
VST-02	VST	46.0	620,923.72	1,378,646.52	390.0*	ConeTec (2018)
VST-03	VST	40.0	621,383.88	1,378,756.46	390.0*	ConeTec (2018)

SPT = Standard Penetration Test; CPT = Cone Penetration Test; VST = Vane Shear Test; * Estimated Coordinates

3. CROSS SECTION GEOMETRY

A cross section was generated perpendicular to the embankment at the spillway structure location (Section 1) as shown in Plate 6. Eight soil regions were characterized based on available data. The boundaries between each region were derived from the following sources.

- Spillway Replacement Project (Stantec 2011) Profile - Baseline A (Drawing No. 10W502-05).
- Geotechnical Exploration (Stantec 2010) Stability Section H-H' (Drawing No. XXWXXX-11).
- Ash ground line available from 2018 bathymetric survey data of Active Ash Pond 2 (Stantec 2018).

The geometry of the cross section developed for this analysis is depicted in Plate 8.

4. MATERIAL PARAMETERS

The soil parameters for this analysis were developed based on the following studies for Active Ash Pond 2: slope stability evaluation performed by Stantec (2010), the site-specific seismic assessment performed by Geocomp (2016a), the Static Safety Factor Assessment performed by Stantec (2016), and Cone Penetration Testing (CPT) soundings performed by ConeTec in 2018. Soil parameters were adjusted based on field and laboratory data that 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 Table 2 and Table 3.

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 Table 2 and Table 3, respectively. The Bottom Ash, Alluvial Sand and Gravel, and the Rip Rap layers are considered free draining; therefore, a strength reduction was not applied in the seismic load cases.

Based on the results of preliminary slope stability analyses, the seismic strengths of the Upper Clay Dike, Bottom Ash, and Ash were found to be important for the inboard failure mode. Therefore, the static and seismic strength parameters for these materials was reviewed and updated from the Stantec (2010) analysis. The Upper Clay Dike strengths were updated based on reassessment of the Stantec (2010) data and consideration of Geocomp (2016a) data. The Bottom Ash parameters were consistent with those used by Geocomp (2016b) and are generally derived based on CPT data. The drained strength for the Ash was estimated using historical data, typical values, and published correlations using SPT N-values (Stantec 2010). The undrained strength for the Ash was based on CPT and vane shear data obtained by ConeTec in 2018. The remaining material parameters used in the analysis are from the Stantec (2010) and Stantec (2016) analyses.

Table 3 presents the strength parameters used in the seismic analyses. For the Lower Clay Dike, Fill, and Alluvial Clay and Silt, 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 Clay Dike, and Ash, the refined undrained seismic strengths are used in the analysis. The free draining materials Bottom Ash, Alluvial Sand and Gravel, and Rip Rap used the static undrained strengths (which are equal to the static drained strengths) in the analysis.

Table 2. 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 Clay Dike	125	200	35	800	25
Lower Clay Dike	125	100	29	210	19
Fill	124	50	29	470	17
Alluvial Clay and Silt	124	100	30	520	17
Alluvial Sand and Gravel	120	0	30	0	30
Ash	100	0	22	900	0
Bottom Ash	125	0	35	0	35
Rip Rap	100	0	38	0	38

Table 3. Summary of seismic shear strength parameters

Soil Layers	Unit Weight (pcf)	Drained Strength Parameters		Undrained Strength Parameters	
		c' (psf)	ϕ' (degrees)	c_{EQ} (psf)	ϕ_{EQ} (degrees)
Upper Clay Dike	125	N/A	N/A	640	20
Lower Clay Dike	125	100	29	170	15
Fill	124	50	29	380	14
Alluvial Clay and Silt	124	100	30	420	14
Alluvial Sand and Gravel	120	0	30	0	30
Ash	100	N/A	N/A	720	0
Bottom Ash	125	0	35	0	35
Rip Rap	100	0	38	0	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 borings (STN-B-5, STN-B-6, STN-B-7, STN-B-8, STN-B-9, STN-B-10, STN-HC, STN-HM, and STN-HT) and CPTs (AAP-CPT-01, AAP-CPT-02, and AAP-CPT-03). The clay-like soils are not susceptible to cyclic softening. Based on SPT blowcount corrections/normalizations and liquefaction triggering criteria from Boulanger and Idriss (2014), Bottom Ash and Alluvial Sand and Gravel materials classify as sand-like soils and are generally too dense to liquefy, regardless of the size of the design earthquake. The Ash material classifies as sand-like and is expected to liquefy during the design earthquake. Detailed calculations are provided in Attachment C.

In summary, the Ash in the vicinity of the spillway is susceptible to liquefaction and is expected to liquefy during the design earthquake. Liquefied (i.e., residual) strength for the Ash was selected based on review of remolded undrained shear strengths from vane shear tests (ConeTec 2018) and the use of published correlations with the SPT and CPT data. The vane shear tests provide material-specific, direct measurements of shear strength in the Ash, whereas the SPT/CPT data is utilized in conjunction with residual strength correlations typically intended for natural, sand-like soils. As such, greater weight was given to the vane shear data, although results from the SPT/CPT correlations still influenced the selected value. For post-earthquake stability analysis, the residual strengths in Table 4 are used for the liquefied materials and the seismic strengths from Table 3 are used for the other materials.

Table 4. Summary of residual shear strength parameters

Liquefied Soil Layers	Residual Strength
Ash	150 psf

6. SLOPE STABILITY ANALYSIS

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

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

- Based on the Geocomp (2016) report, peak ground acceleration (PGA) on rock for Johnsonville Fossil Plant is 0.214g (PGA_{ROCK}). A seismic coefficient (k_h) of ½ of PGA_{ROCK} (k_h = 0.107g) 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 Active Ash Pond 2, elevation 384.3 ft.
- The phreatic surface was assumed to vary linearly within the embankment, between the headwater (384.3 ft) and an assumed tailwater elevation of 359 ft.
- An appropriate tension crack was specified based on the Upper Clay Dike soil properties.
- Any potential resistance from the spillway structure itself is neglected in the slope stability analysis.

Table 5 summarizes the load cases considered in this analysis and the soil properties applied for each case (Attachment D).

Table 5. Summary of seismic slope stability results

Load Case	Soil Strengths	Calculated FS
Pseudostatic	Seismic	N/A[1]
Post-Earthquake	Seismic/Residual	1.2

[1] Pseudostatic Analyses are generally not valid when significant soil liquefaction is predicted.

Pseudostatic analyses are acceptable for screening-level evaluations of embankment dams, where the computed factors of safety can be used to judge if more detailed analyses are warranted. The results are generally invalid when significant soil liquefaction is predicted, therefore no factor of safety is reported in Table 5. However, the permanent displacements due to shaking were estimated, see Section 7.

7. SIMPLIFIED SEISMIC DISPLACEMENT ANALYSIS

Permanent displacements due to shaking were estimated using a simplified, sliding block analysis (i.e., Newmark 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 Johnsonville Fossil Plant site by Geocomp (2016a) 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).

Typically, a key assumption in a Newmark analysis is that significant liquefaction is not predicted. Where liquefaction occurs but the slope remains stable (i.e., post-earthquake factor of safety greater than or equal to 1), a sliding block analysis may be used to roughly estimate displacements. The results are less reliable in this case, however, because the ground motions on the slip surface are computed without modeling the effects of liquefaction or cyclic softening. For simplicity, the residual strength is conservatively assumed to be acting in the liquefied soils throughout the shaking.

The yield acceleration (i.e., kh value at which FS=1.0) was determined to be 0.016g. The accelerations imposed by the design earthquake are then compared to the yield acceleration. Considering the seven time histories, the largest permanent displacement was 1.9 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 the Ash will liquefy due to the design earthquake. Considering reduced and liquefied seismic shear strengths, a post-earthquake FS of 1.2 was calculated for the inboard slope stability of the Ash Pond 2 perimeter dike at the spillway structure location. Also considering seismic shear strengths (included liquefied strength of the Ash), a Newmark analysis was performed to estimate permanent displacement due to the design earthquake. The Newmark analysis indicates a permanent displacement of 1.9 feet or less. In summary, the expected seismic displacement is roughly 2 feet or less.

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ATTACHMENT A

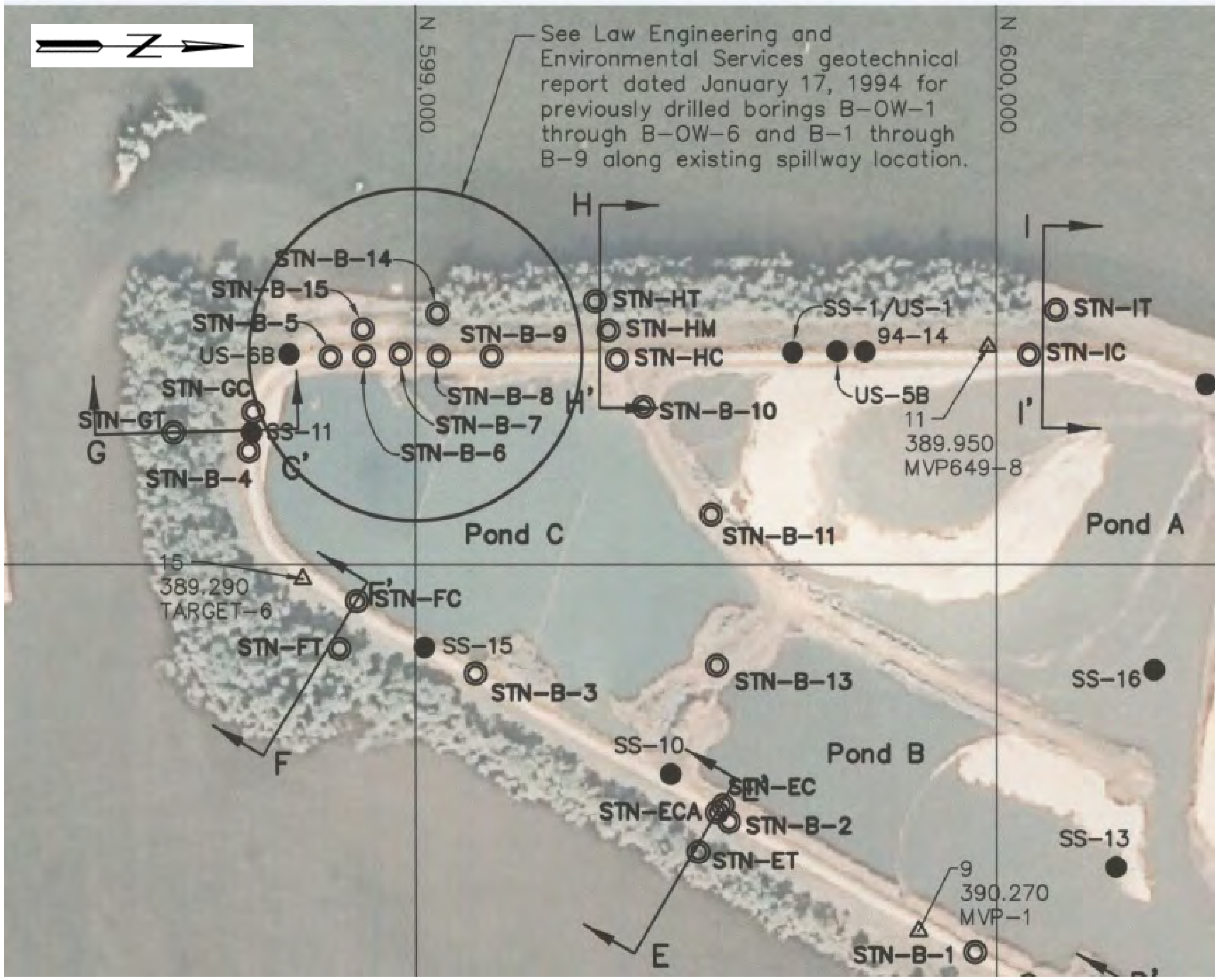


Plate 1
Stantec (2010) Boring Location Plan

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Notes
1. Borings and CPT locations are approximated

Stantec

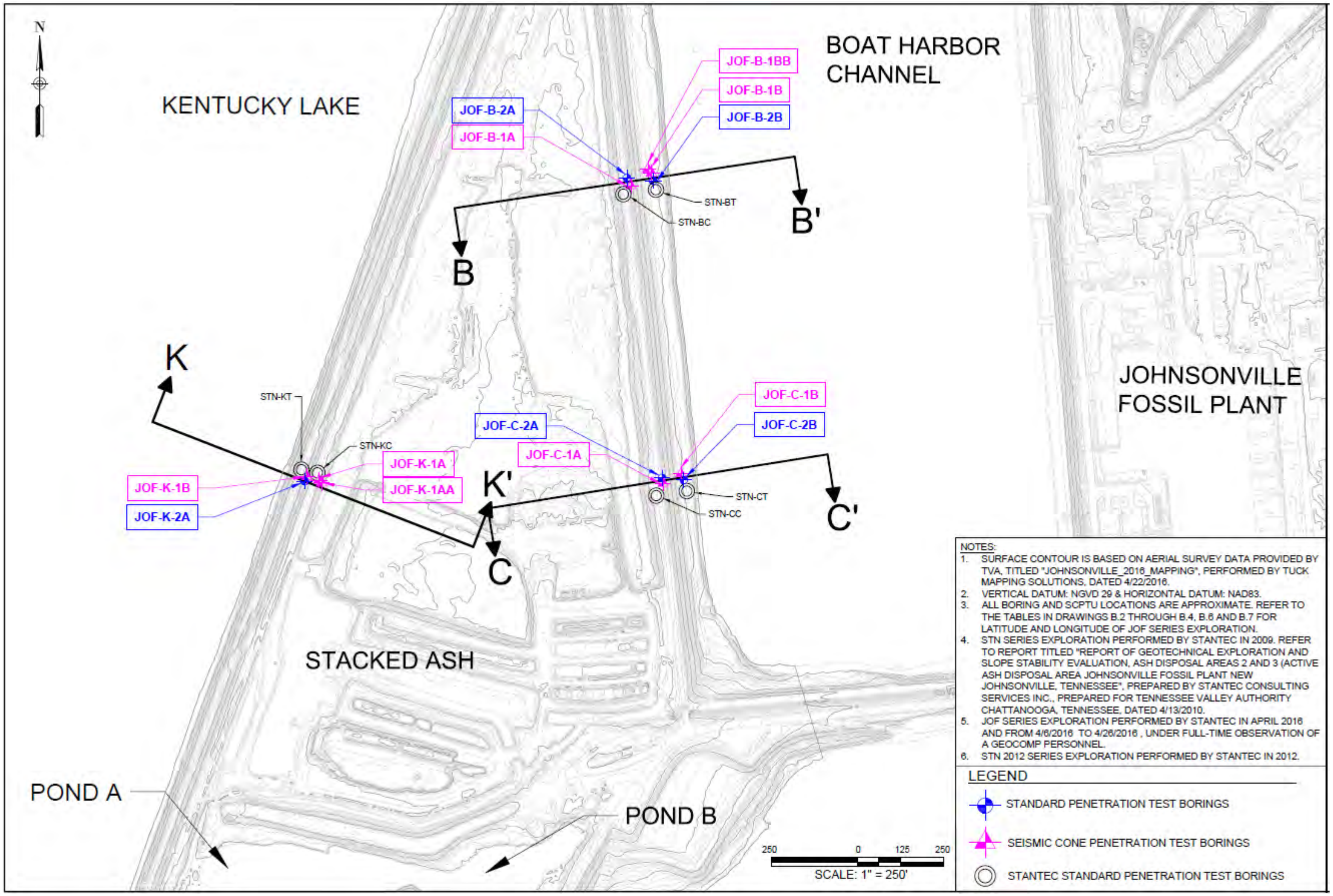


Plate 2
Geocomp (2016) Boring Location Plan

Notes
1. Borings and CPT locations are approximated

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Stantec

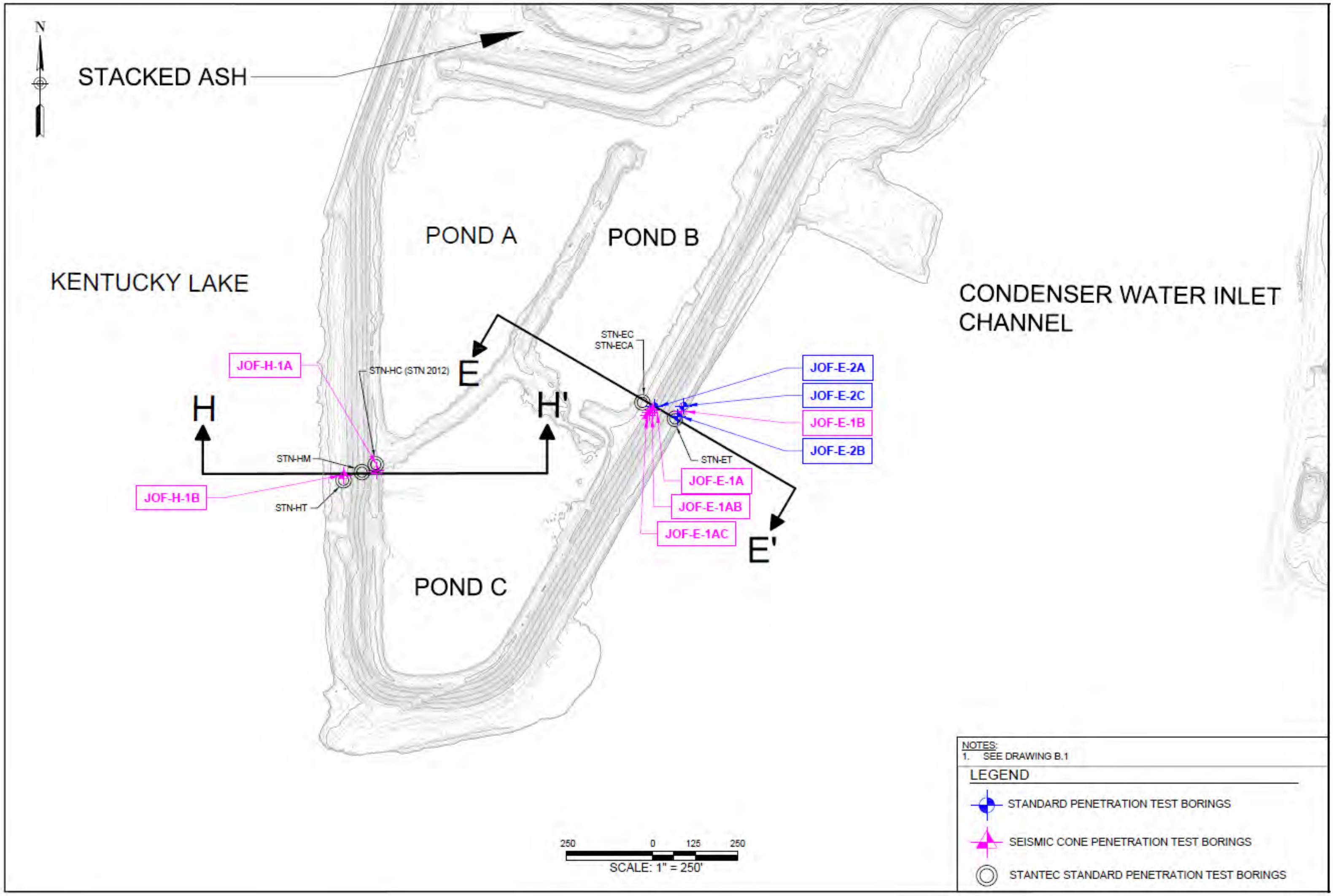


Plate 3
Geocomp (2016) Boring Location Plan

Notes
1. Borings and CPT locations are approximated

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Stantec

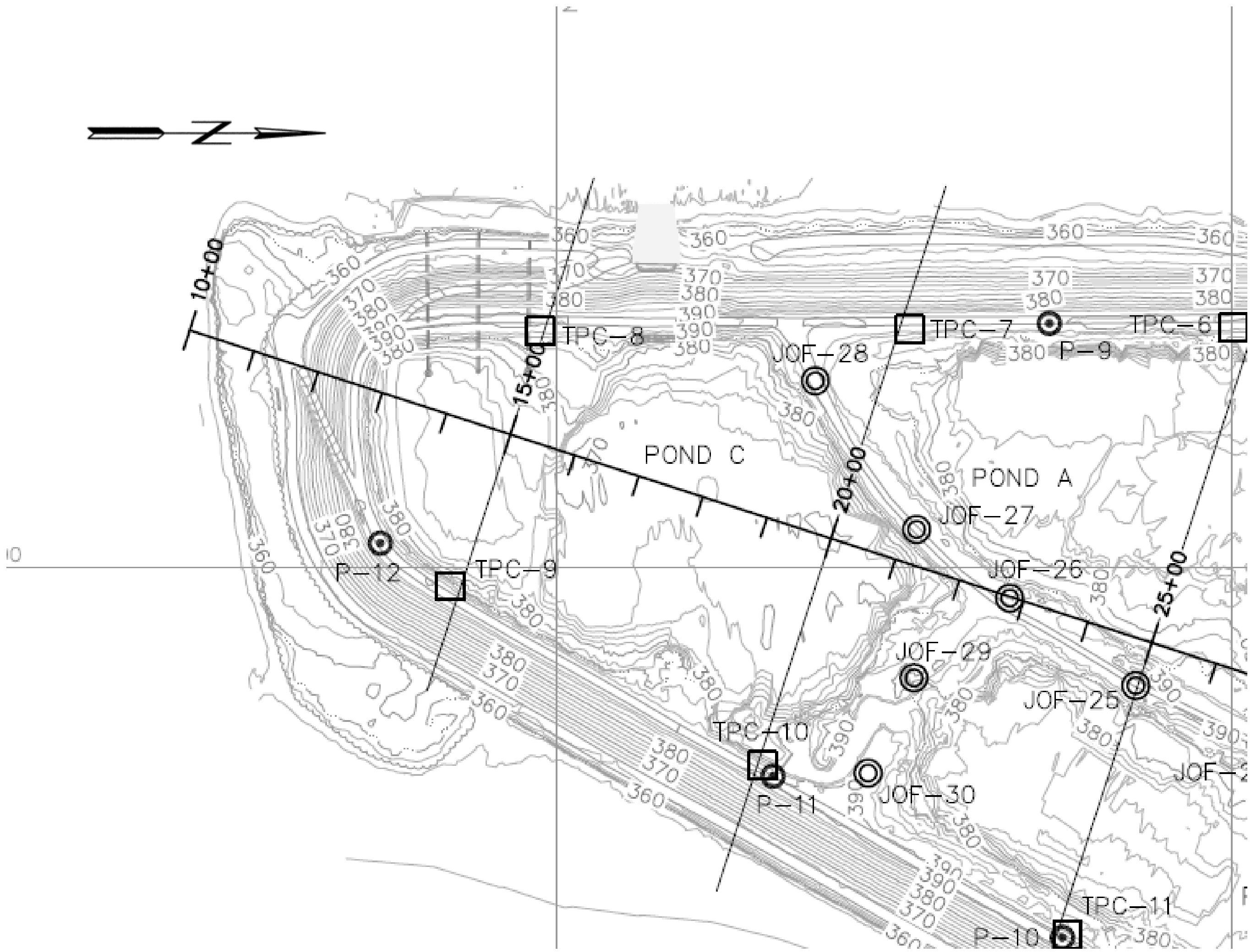


Plate 4
Stantec (2016) Boring Location Plan

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Stantec

Notes
1. Borings and CPT locations are approximated

Plate 5
ConeTec (2018), CPT and VST Locations

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Notes
1. CPT and VST locations are approximated

Stantec

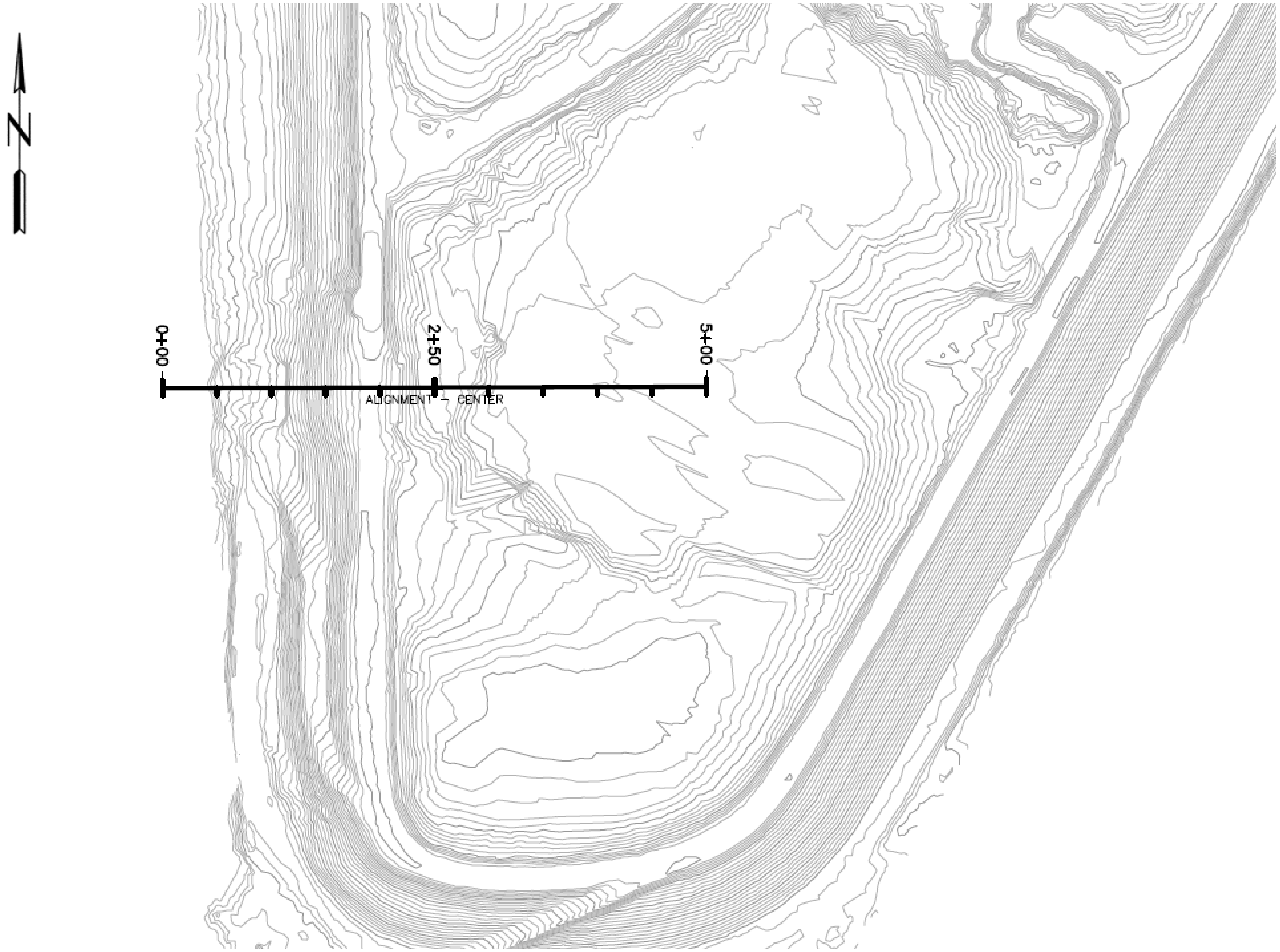


Plate 6
Spillway Cross Section 1

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Stantec

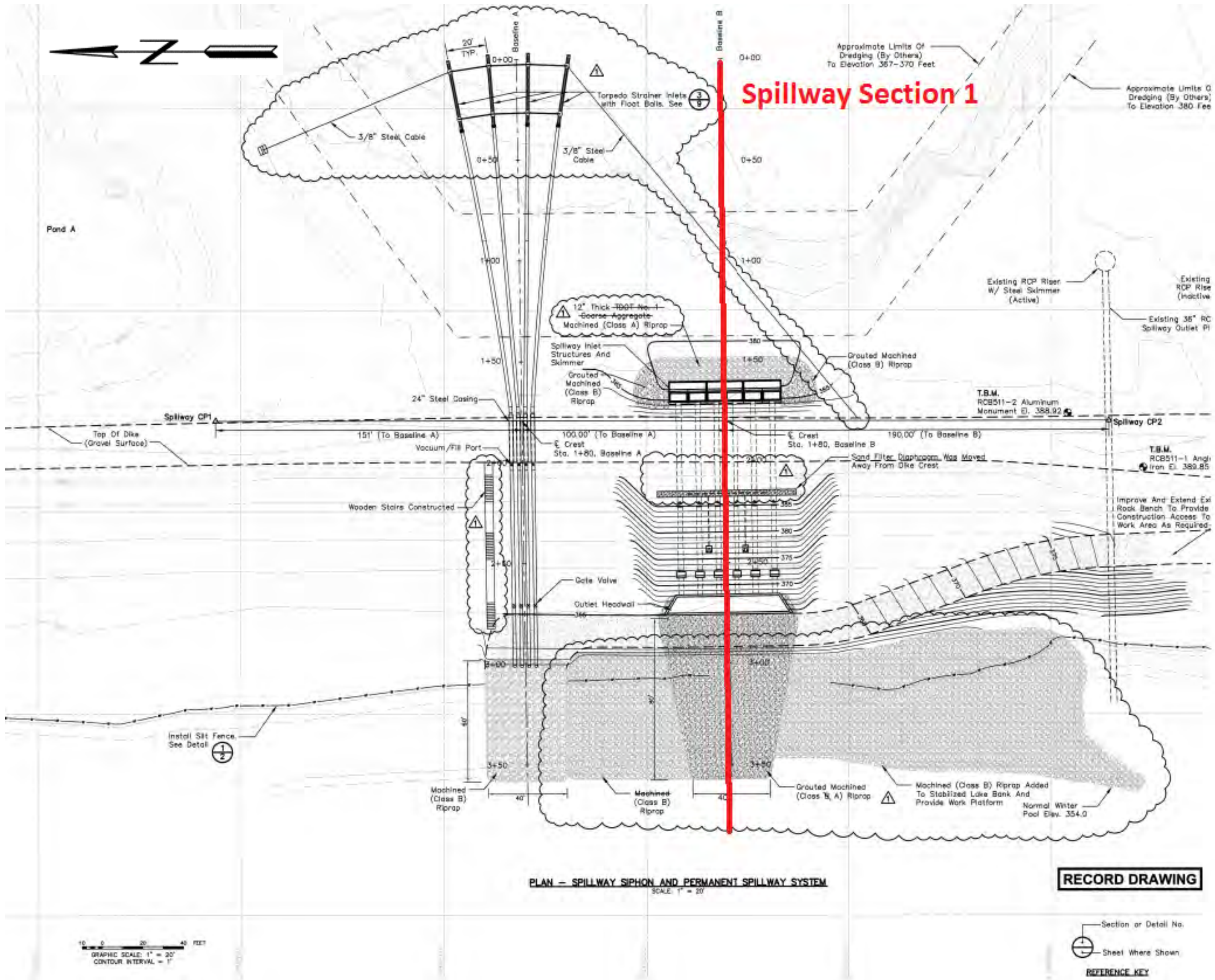


Plate 7
Spillway Cross Section 1

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Stantec

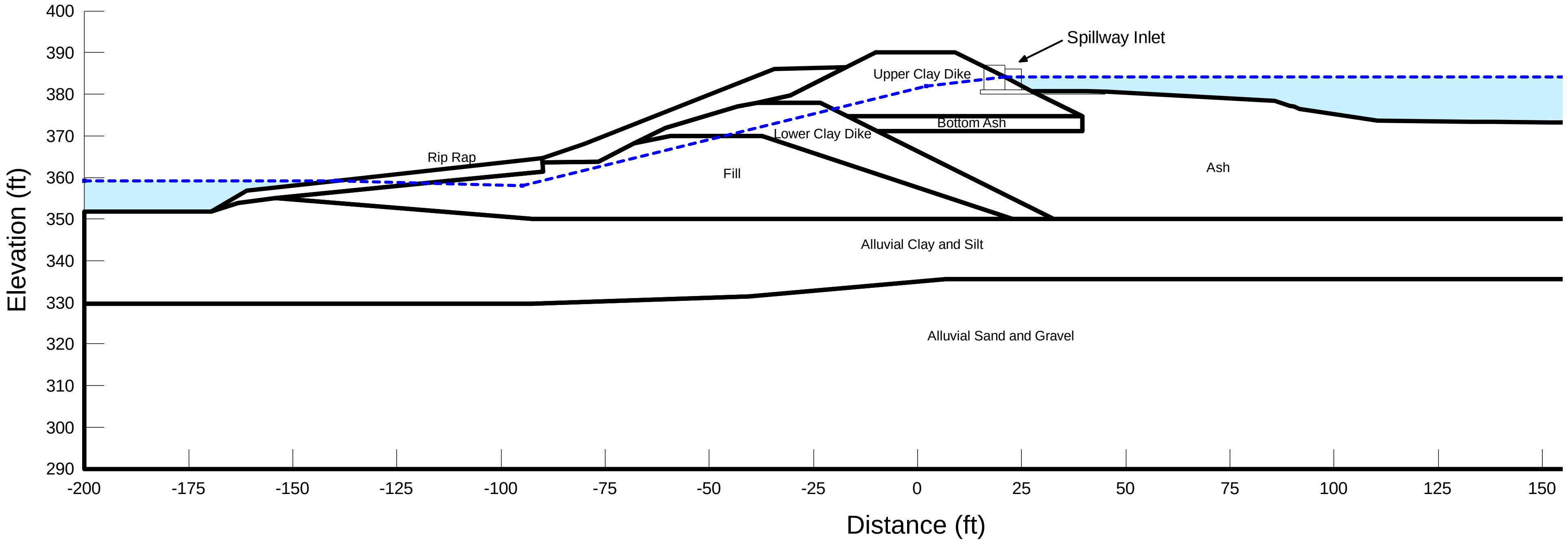


Plate 8
Boring Location Plan

TVA CCR Rule: Location Restrictions, Seismic Impact Zones
Johnsonville Fossil Plant Ash Pond 2
Humphreys County, Tennessee

Stantec

ATTACHMENT B

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

Stantec Project Number:	175568235
Project Name:	TVA Johnsonville Ash Pond 2 Seismic Impact Zones
Material	Alluvial Clay and Silt

Note: NP = Non-Plastic

Elevation at Midpoint (ft)	Lab ID	Boring	Depth(s) (ft)	Soil Classification	NMC (w_c) (%)	% Passing #200	% Passing #40	LL	PI
331.7	SPT-9	JOF-B-2A	60.0'- 62.0'	CL	28.3	82.7	82.7	37	17
328.4	ST-7	JOF-B-2A	41.0'- 43.5'	CL	29.5	96.8	96.8	43	23
346.9	ST-8	JOF-B-2A	44.5'- 47.0'	CL	29.3	97.8	97.8	52	30
334.4	ST-12	JOF-B-2A	57.5'- 59.0'	CL	26.6	87.5	87.5	39	21
328.4	ST-13	JOF-B-2A	63.0'- 65.5'	CL	31.1	57.1	57.1	31	13
344.8	ST-6	JOF-B-2B	25.0'- 27.5'	CL	26.6	97	97	44	21
341.3	ST-7	JOF-B-2B	28.5'- 31.0'	CL	25.2	94.2	94.2	38	19
337.8	ST-8	JOF-B-2B	32.0'- 34.5'	CL	25.6	95.7	95.7	41	22
331.5	ST-10	JOF-B-2B	38.5'- 40.5'	CL	27.5	82.1	82.1	31	13
340.5	SPT-9	JOF-C-2A	51.5'- 53.0'	CL	30.5	97.8	97.8	42	22
355.0	ST-7	JOF-C-2A	36.5'- 39.0'	CL	26.8	58.4	58.4	36	19
351.5	ST-8	JOF-C-2A	40.0'- 42.5'	CL	24.8	86.7	86.7	42	23
344.5	ST-10	JOF-C-2A	47.0'- 49.5'	CL	31	99.9	99.9	45	25
334.0	ST-13	JOF-C-2A	57.5'- 60.0'	CL	25.6	91.9	91.9	29	11
350.3	ST-4	JOF-C-2B	19.0'- 21.5'	CL	30.5	88.5	88.5	43	27
343.3	ST-6	JOF-C-2B	26.0'- 28.5'	CL	28.8	99.6	99.6	38	20
339.8	ST-7	JOF-C-2B	29.5'- 32.0'	CL	29.4	99.6	99.6	40	21
332.8	ST-9	JOF-C-2B	36.5'- 39.0'	CL	25.9	85.8	85.8	30	11
347.1	SPT-7	JOF-E-2A	43.0'- 44.5'	CL	23.7	92.4	92.4	41	22
353.6	ST-8	JOF-E-2A	36.0'- 38.5'	CL	26.4	98.5	98.5	46	21
338.6	ST-11	JOF-E-2A	51.0'- 53.5'	CL	23.3	79.9	79.9	32	17
335.1	ST-12	JOF-E-2A	54.5'- 57.0'	CL	24.5	78.9	78.9	34	16
345.1	ST-5	JOF-E-2B	19.0'- 21.5'	CL	23.8	93.2	93.2	39	21
338.6	ST-7	JOF-E-2B	26.0'- 27.5'	CL	22.7	81.3	81.3	38	17
335.6	ST-8	JOF-E-2B	28.5'- 31.0'	CL	23.6	78.8	78.8	33	14
338.0	SPT-1	JOF-K-2A	38.5'- 40.5'	CL	26.7	98.9	98.9	32	12
335.0	SPT-2	JOF-K-2A	41.5'- 43.5'	CL	27.1	85.1	85.1	34	16
344.3	ST-1	JOF-K-2A	32.5'- 34.0'	CL	24.9	97.4	97.4	42	21
341.5	ST-2	JOF-K-2A	35.0'- 37.0'	CL	24.6	96.2	96.2	38	19

Sand-like versus Clay-like Behavior (-1 indicates result does not meet criteria)

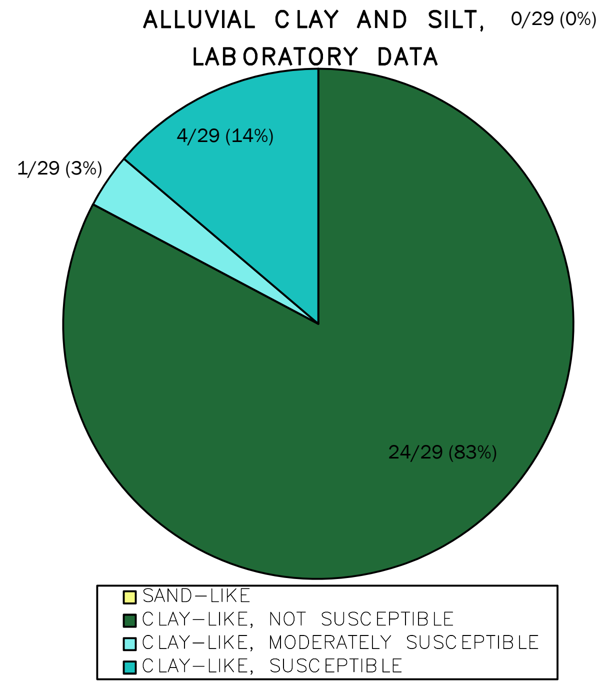
		Using Criteria published by Seed et al (2003)										Using Criteria published by Idriss and Boulanger (2008)		Using criteria published by MSHA (2010)			
		Meets criteria for sand-like behavior		In Zone B		in B with w >= .85LL		Meets criteria for clay-like behavior				Meets criteria for sand-like behavior	Meets criteria for clay-like behavior	Meets criteria for sand-like behavior	Meets criteria for clay-like behavior	Borderline soils (treat as sand-like)	
Lab ID	Boring	LL in Zone A (see plot)	PI in Zone A (see plot)	LL	PI	LL	PI	LL in Zone B (see plot)	PI in Zone B (see plot)	LL in Zone C (see plot)	PI in Zone C (see plot)	PI < 7	PI >= 7	PI <= 7	P40>=35%, P200>=20%, and PI>=10	7 < PI < 10, or does not meet P40 or P200	Overall Judgement based on 3 methods (sand-like or clay-like)
SPT-9	JOF-B-2A	-1	-1	37	17	-1	-1	37	17	-1	-1	-1	17	-1	17	-1	**Clay-like**
ST-7	JOF-B-2A	-1	-1	-1	-1	-1	-1	-1	-1	43	23	-1	23	-1	23	-1	**Clay-like**
ST-8	JOF-B-2A	-1	-1	-1	-1	-1	-1	-1	-1	52	30	-1	30	-1	30	-1	**Clay-like**
ST-12	JOF-B-2A	-1	-1	-1	-1	-1	-1	-1	-1	39	21	-1	21	-1	21	-1	**Clay-like**
ST-13	JOF-B-2A	-1	-1	31	13	31	13	31	13	-1	-1	-1	13	-1	13	-1	**Clay-like**
ST-6	JOF-B-2B	-1	-1	-1	-1	-1	-1	-1	-1	44	21	-1	21	-1	21	-1	**Clay-like**
ST-7	JOF-B-2B	-1	-1	38	19	-1	-1	38	19	-1	-1	-1	19	-1	19	-1	**Clay-like**
ST-8	JOF-B-2B	-1	-1	-1	-1	-1	-1	-1	-1	41	22	-1	22	-1	22	-1	**Clay-like**
ST-10	JOF-B-2B	-1	-1	31	13	31	13	31	13	-1	-1	-1	13	-1	13	-1	**Clay-like**
SPT-9	JOF-C-2A	-1	-1	-1	-1	-1	-1	-1	-1	42	22	-1	22	-1	22	-1	**Clay-like**
ST-7	JOF-C-2A	-1	-1	36	19	-1	-1	36	19	-1	-1	-1	19	-1	19	-1	**Clay-like**
ST-8	JOF-C-2A	-1	-1	-1	-1	-1	-1	-1	-1	42	23	-1	23	-1	23	-1	**Clay-like**
ST-10	JOF-C-2A	-1	-1	-1	-1	-1	-1	-1	-1	45	25	-1	25	-1	25	-1	**Clay-like**
ST-13	JOF-C-2A	29	11	-1	-1	-1	-1	-1	-1	-1	-1	-1	11	-1	11	-1	**Clay-like**
ST-4	JOF-C-2B	-1	-1	-1	-1	-1	-1	-1	-1	43	27	-1	27	-1	27	-1	**Clay-like**
ST-6	JOF-C-2B	-1	-1	38	20	-1	-1	38	20	-1	-1	-1	20	-1	20	-1	**Clay-like**
ST-7	JOF-C-2B	-1	-1	-1	-1	-1	-1	-1	-1	40	21	-1	21	-1	21	-1	**Clay-like**
ST-9	JOF-C-2B	30	11	-1	-1	-1	-1	-1	-1	-1	-1	-1	11	-1	11	-1	**Clay-like**
SPT-7	JOF-E-2A	-1	-1	-1	-1	-1	-1	-1	-1	41	22	-1	22	-1	22	-1	**Clay-like**
ST-8	JOF-E-2A	-1	-1	-1	-1	-1	-1	-1	-1	46	21	-1	21	-1	21	-1	**Clay-like**
ST-11	JOF-E-2A	-1	-1	32	17	-1	-1	32	17	-1	-1	-1	17	-1	17	-1	**Clay-like**
ST-12	JOF-E-2A	-1	-1	34	16	-1	-1	34	16	-1	-1	-1	16	-1	16	-1	**Clay-like**
ST-5	JOF-E-2B	-1	-1	-1	-1	-1	-1	-1	-1	39	21	-1	21	-1	21	-1	**Clay-like**
ST-7	JOF-E-2B	-1	-1	38	17	-1	-1	38	17	-1	-1	-1	17	-1	17	-1	**Clay-like**
ST-8	JOF-E-2B	-1	-1	33	14	-1	-1	33	14	-1	-1	-1	14	-1	14	-1	**Clay-like**
SPT-1	JOF-K-2A	32	12	-1	-1	-1	-1	-1	-1	-1	-1	-1	12	-1	12	-1	**Clay-like**
SPT-2	JOF-K-2A	-1	-1	34	16	-1	-1	34	16	-1	-1	-1	16	-1	16	-1	**Clay-like**
ST-1	JOF-K-2A	-1	-1	-1	-1	-1	-1	-1	-1	42	21	-1	21	-1	21	-1	**Clay-like**
ST-2	JOF-K-2A	-1	-1	38	19	-1	-1	38	19	-1	-1	-1	19	-1	19	-1	**Clay-like**

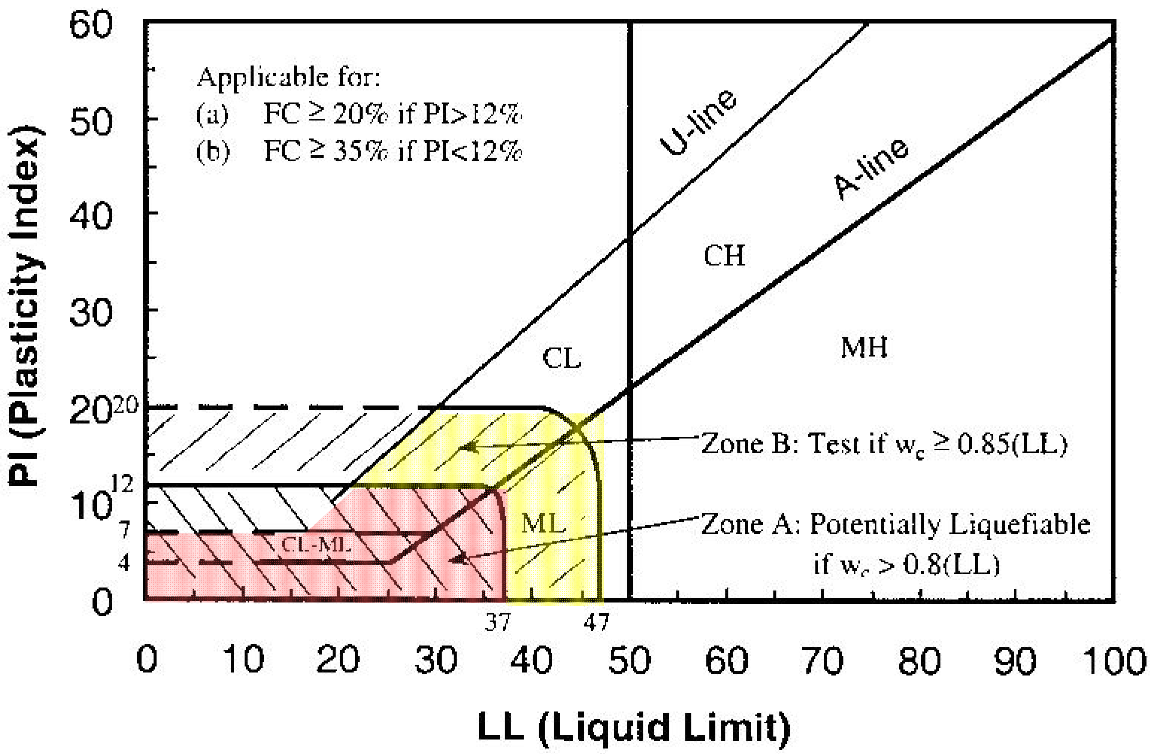
Susceptibility of Clay-like Soils to Cyclic Softening (-1 indicates result does not meet criteria)

		Using Criteria published by Seed et al (2003)		Using Criteria published by Bray and Sancio (2006)						
		Meets all criteria for B (clay-like and potentially liquefiable, -2 indicates zone A but susceptible, -3 indicates not applicable due to fines content		Clay-like soil is susceptible (must meet both)		Clay-like soil is not susceptible (must meet one or both)		Clay-like soil is moderately susceptible		
Lab ID	Boring	LL	PI	w_c/LL >= 0.85	PI <= 12	w_c/LL < 0.80	PI > 18	Intermediate w_c/LL (see plot)	Intermediate PI (see plot)	Overall Judgement based on 2 methods (susceptibility)
SPT-9	JOF-B-2A	-1	-1	-1.00	-1	0.76	17	-1.00	-1	**Not Susceptible**
ST-7	JOF-B-2A	-1	-1	-1.00	-1	0.69	23	-1.00	-1	**Not Susceptible**
ST-8	JOF-B-2A	-1	-1	-1.00	-1	0.56	30	-1.00	-1	**Not Susceptible**
ST-12	JOF-B-2A	-1	-1	-1.00	-1	0.68	21	-1.00	-1	**Not Susceptible**
ST-13	JOF-B-2A	31	13	-1.00	-1	-1.00	-1	1.00	13	**Susceptible**
ST-6	JOF-B-2B	-1	-1	-1.00	-1	0.60	21	-1.00	-1	**Not Susceptible**
ST-7	JOF-B-2B	-1	-1	-1.00	-1	0.66	19	-1.00	-1	**Not Susceptible**
ST-8	JOF-B-2B	-1	-1	-1.00	-1	0.62	22	-1.00	-1	**Not Susceptible**
ST-10	JOF-B-2B	31	13	-1.00	-1	-1.00	-1	0.89	13	**Susceptible**
SPT-9	JOF-C-2A	-1	-1	-1.00	-1	0.73	22	-1.00	-1	**Not Susceptible**
ST-7	JOF-C-2A	-1	-1	-1.00	-1	0.74	19	-1.00	-1	**Not Susceptible**
ST-8	JOF-C-2A	-1	-1	-1.00	-1	0.59	23	-1.00	-1	**Not Susceptible**
ST-10	JOF-C-2A	-1	-1	-1.00	-1	0.69	25	-1.00	-1	**Not Susceptible**
ST-13	JOF-C-2A	-2	-2	0.88	11	-1.00	-1	-1.00	-1	**Susceptible**
ST-4	JOF-C-2B	-1	-1	-1.00	-1	0.71	27	-1.00	-1	**Not Susceptible**
ST-6	JOF-C-2B	-1	-1	-1.00	-1	0.76	20	-1.00	-1	**Not Susceptible**
ST-7	JOF-C-2B	-1	-1	-1.00	-1	0.74	21	-1.00	-1	**Not Susceptible**
ST-9	JOF-C-2B	-2	-2	0.86	11	-1.00	-1	-1.00	-1	**Susceptible**
SPT-7	JOF-E-2A	-1	-1	-1.00	-1	0.58	22	-1.00	-1	**Not Susceptible**
ST-8	JOF-E-2A	-1	-1	-1.00	-1	0.57	21	-1.00	-1	**Not Susceptible**
ST-11	JOF-E-2A	-1	-1	-1.00	-1	0.73	17	-1.00	-1	**Not Susceptible**
ST-12	JOF-E-2A	-1	-1	-1.00	-1	0.72	16	-1.00	-1	**Not Susceptible**
ST-5	JOF-E-2B	-1	-1	-1.00	-1	0.61	21	-1.00	-1	**Not Susceptible**
ST-7	JOF-E-2B	-1	-1	-1.00	-1	0.60	17	-1.00	-1	**Not Susceptible**
ST-8	JOF-E-2B	-1	-1	-1.00	-1	0.72	14	-1.00	-1	**Not Susceptible**
SPT-1	JOF-K-2A	-2	-2	-1.00	-1	-1.00	-1	0.83	12	**Moderately Susceptible**
SPT-2	JOF-K-2A	-1	-1	-1.00	-1	0.80	16	-1.00	-1	**Not Susceptible**
ST-1	JOF-K-2A	-1	-1	-1.00	-1	0.59	21	-1.00	-1	**Not Susceptible**
ST-2	JOF-K-2A	-1	-1	-1.00	-1	0.65	19	-1.00	-1	**Not Susceptible**

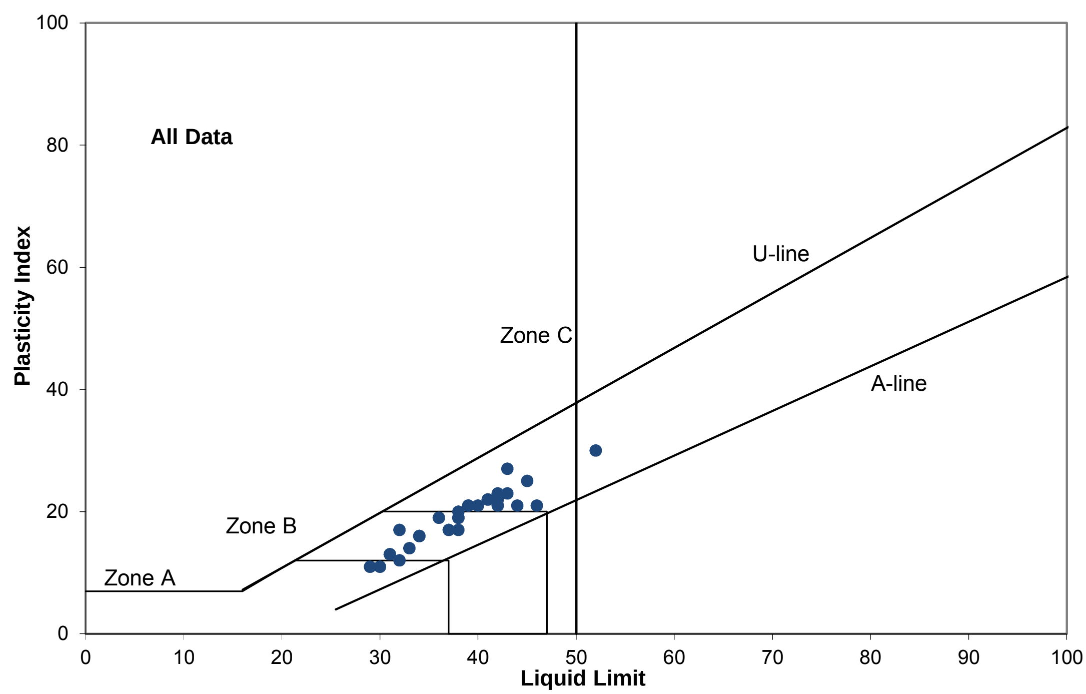
Alluvial Clay and Silt, Fine-grained Screening and Cyclic Softening Susceptibility Summary

Soil Classification	Clay-like vs. Sand-like	Clay-like Susceptibility	Sample Count	Sample Percent
CL			**29**	**100%**
	Clay-like		29	100%
		Moderately Susceptible	1	3%
		Not Susceptible	24	83%
		Susceptible	4	14%
Grand Total			**29**	**100%**



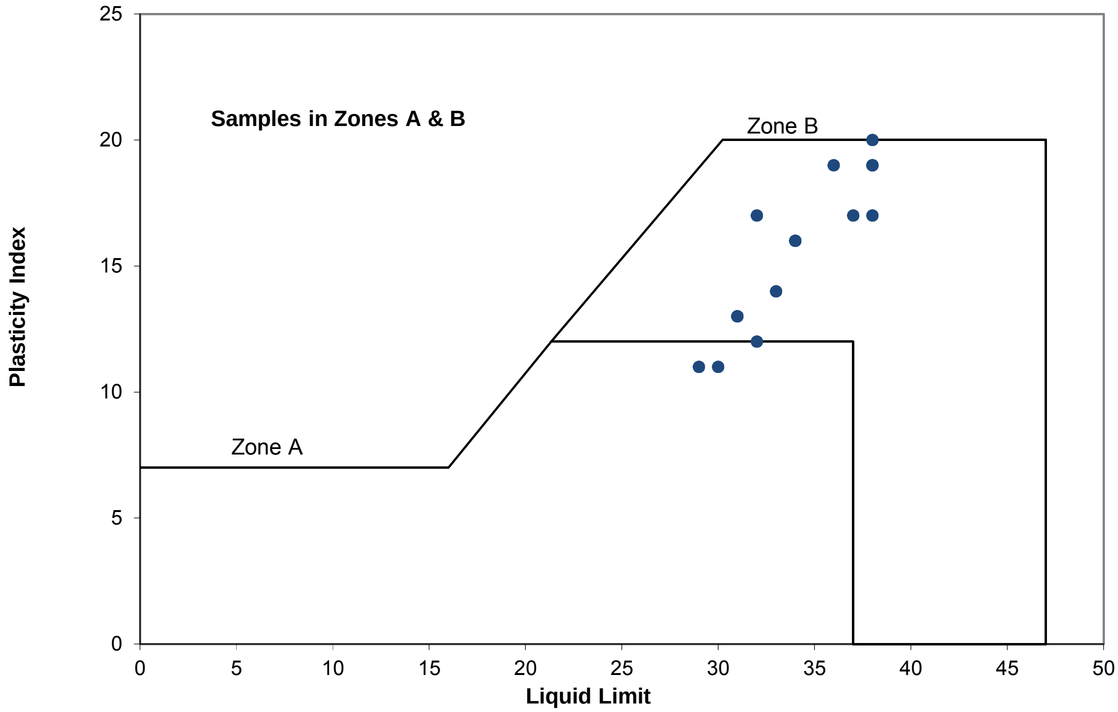


(a)

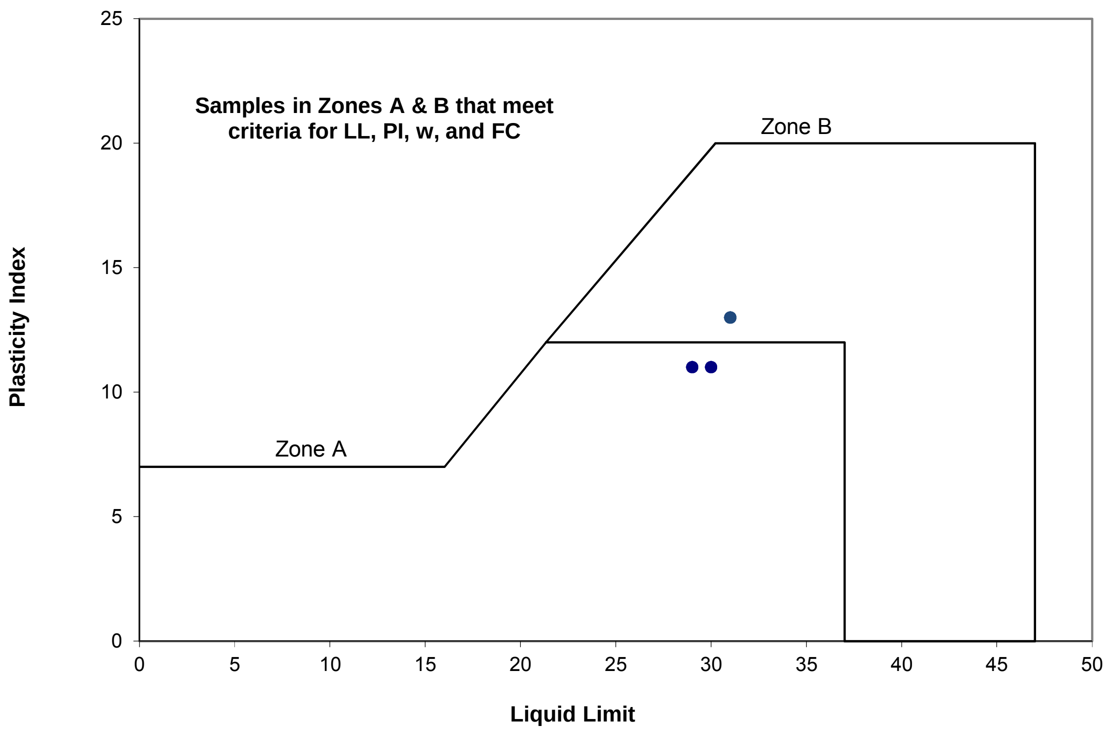


(b)

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

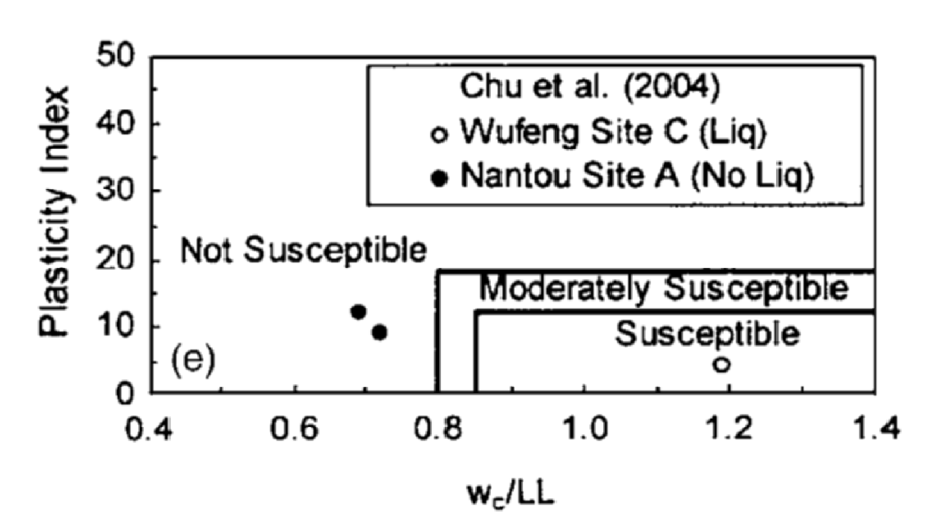


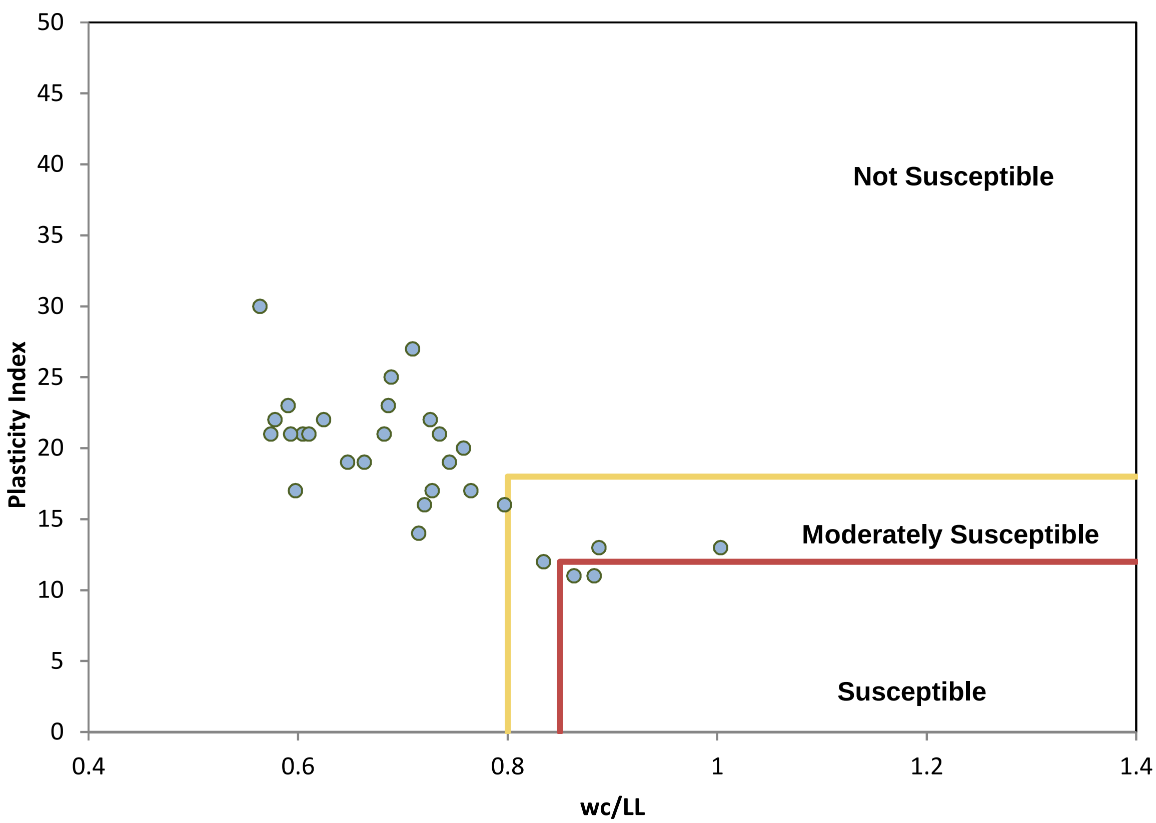
(c)



(d)

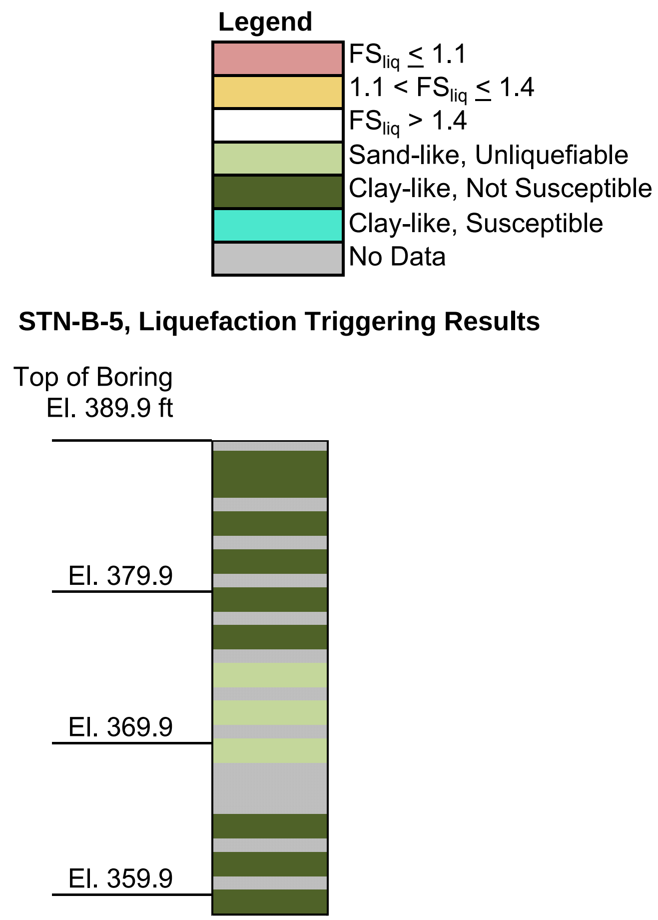
Screening Criteria for Liquefiable Fine-Grained Soils (Seed et al. 2003)

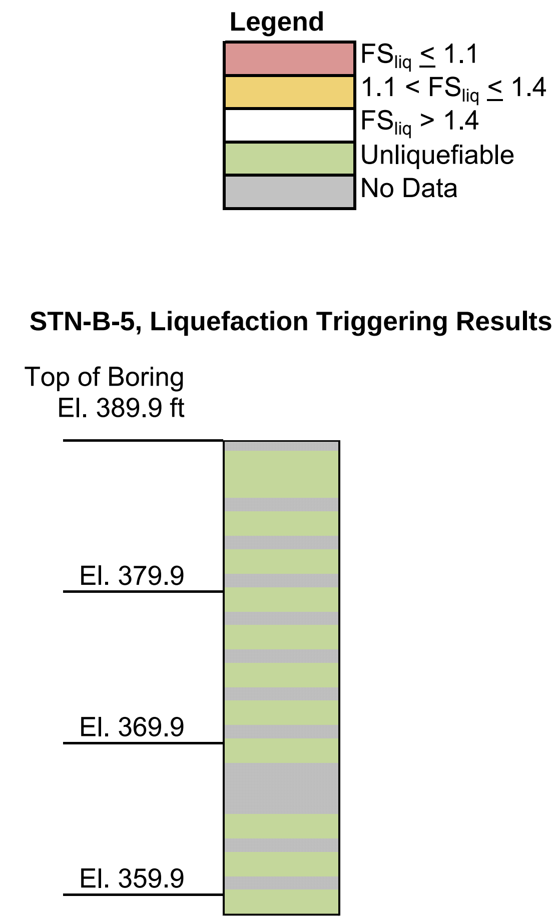




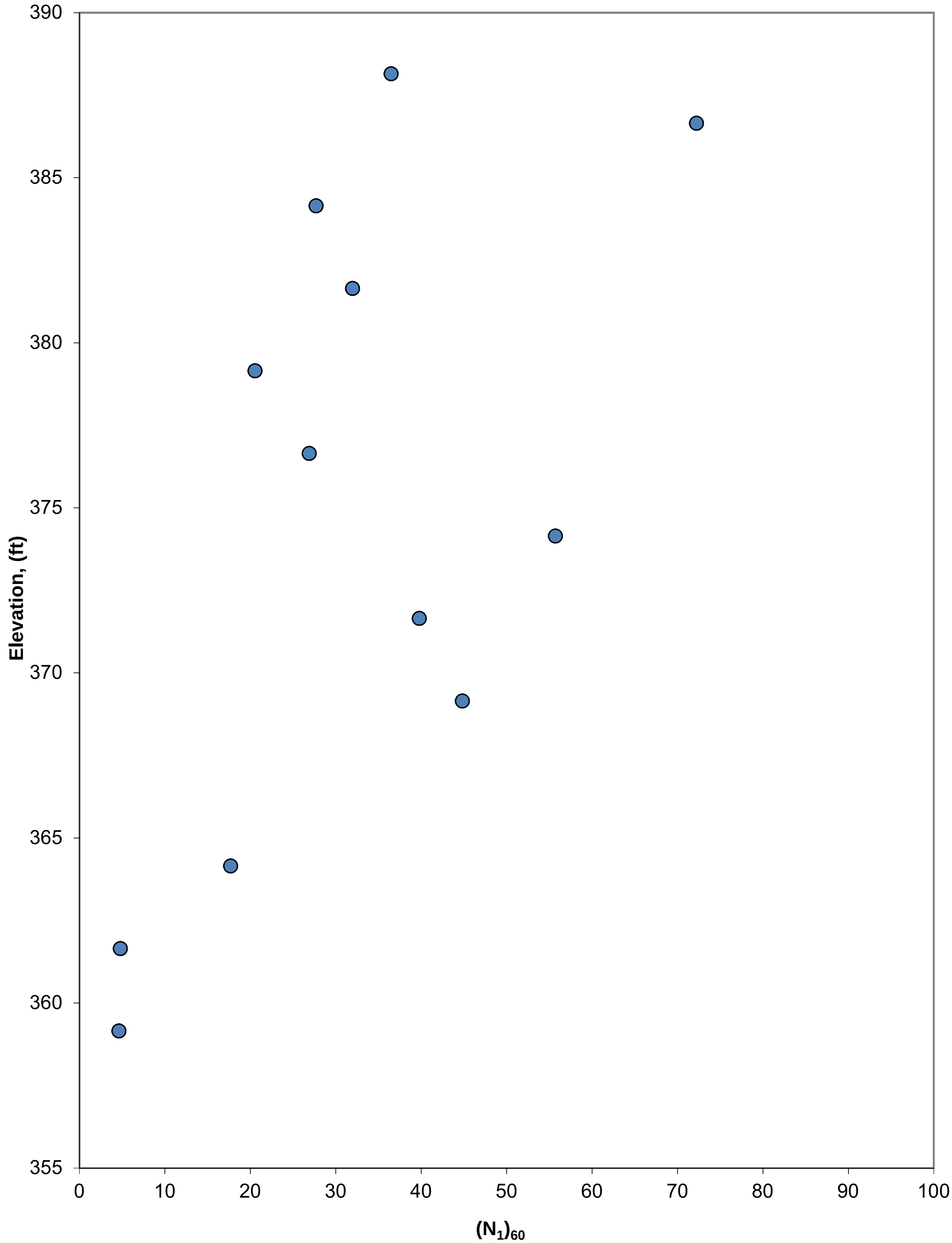
Screening Criteria for Assessing Liquefaction in Fine Grained Soils (Bray and Sancio 2006)

ATTACHMENT C

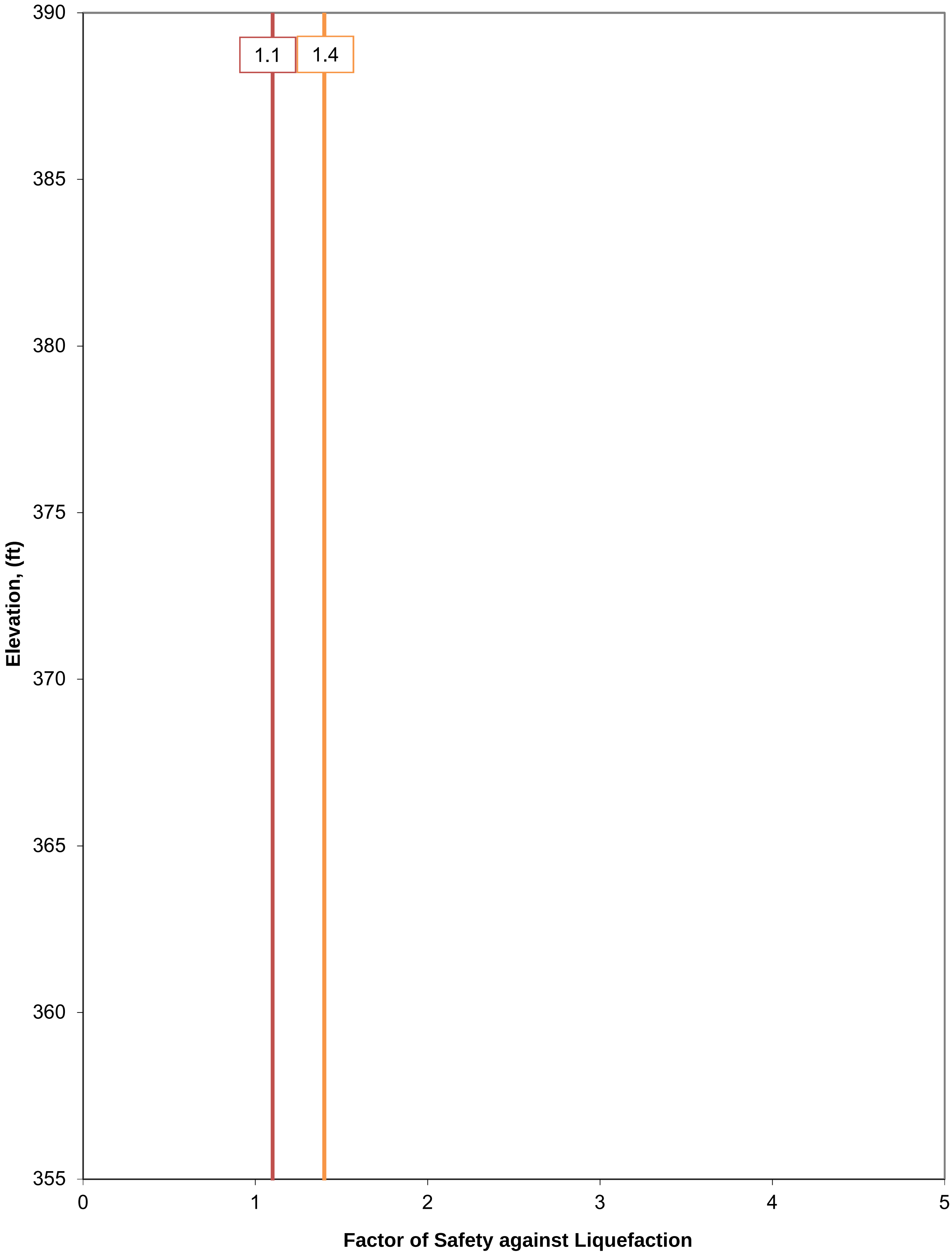

Legend
$FS_{liq} \le 1.1$
$1.1 < FS_{liq} \le 1.4$
$FS_{liq} > 1.4$
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-B-5, Liquefaction Triggering Results
Top of Boring
El. 389.9 ft
El. 379.9
El. 369.9
El. 359.9

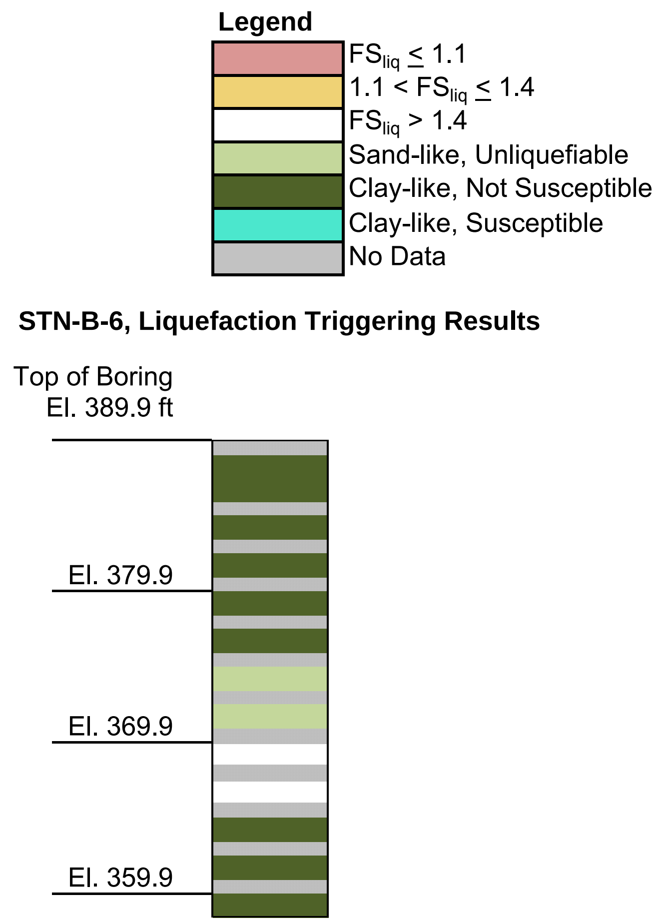

Legend
$FS_{liq} \le 1.1$
$1.1 < FS_{liq} \le 1.4$
$FS_{liq} > 1.4$
Unliquefiable
No Data
STN-B-5, Liquefaction Triggering Results
Top of Boring
El. 389.9 ft
El. 379.9
El. 369.9
El. 359.9

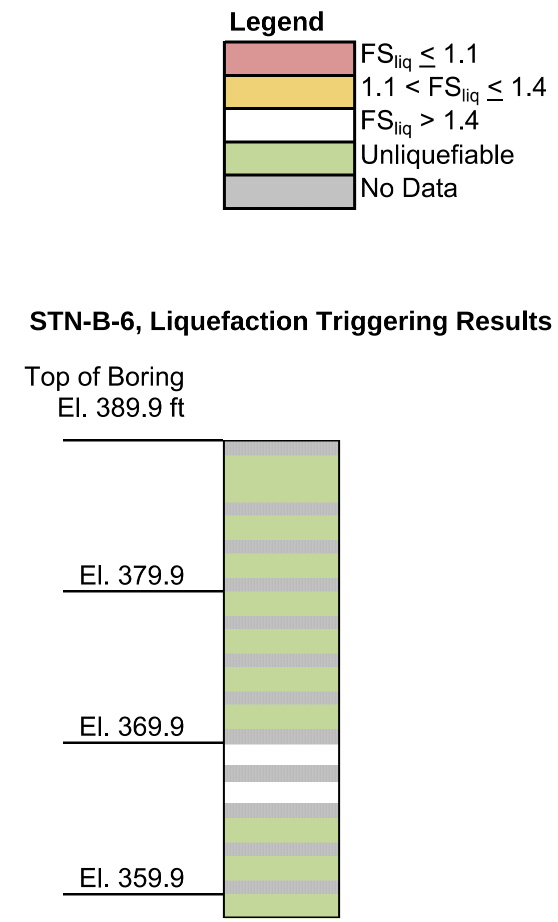
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-5, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



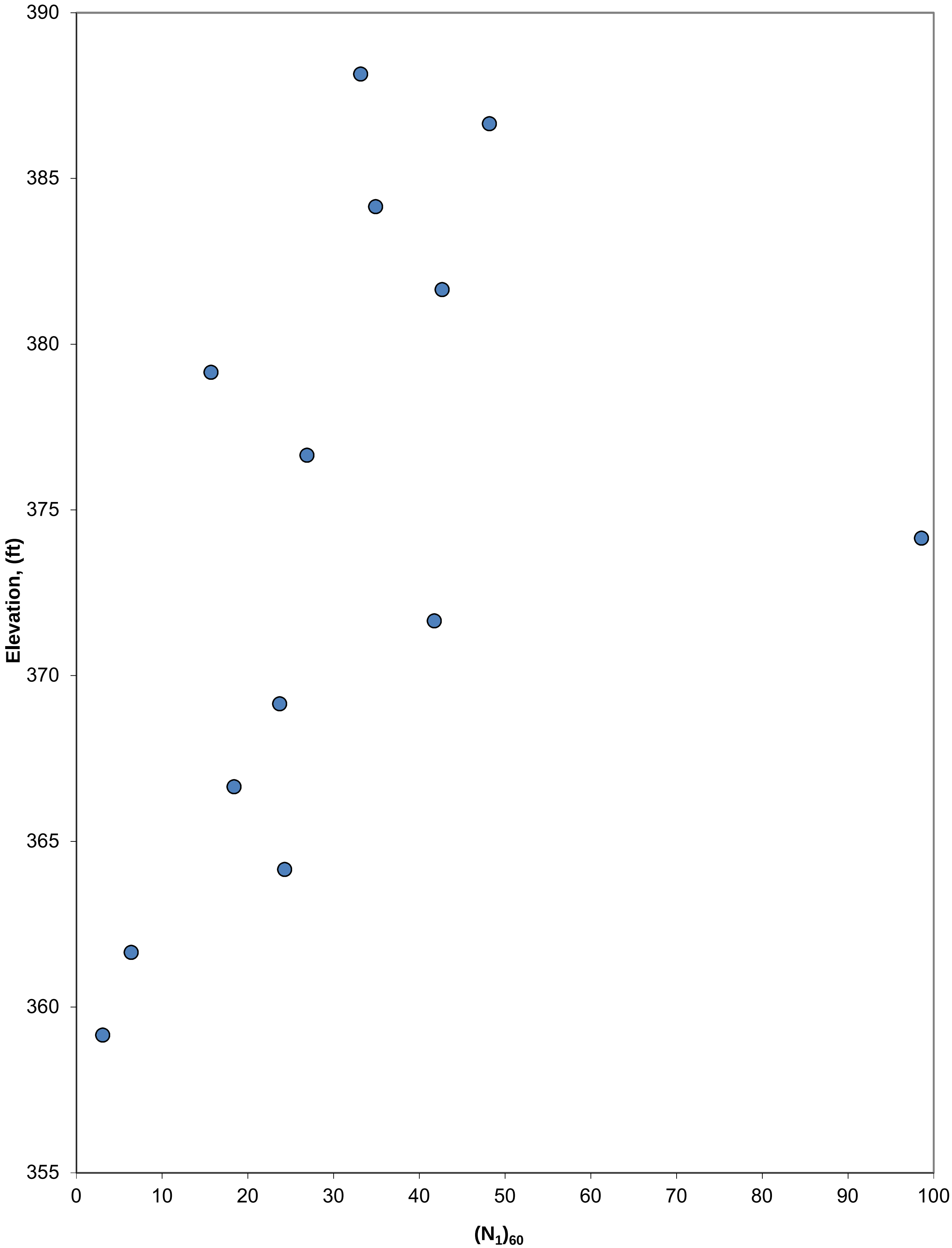
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-5, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



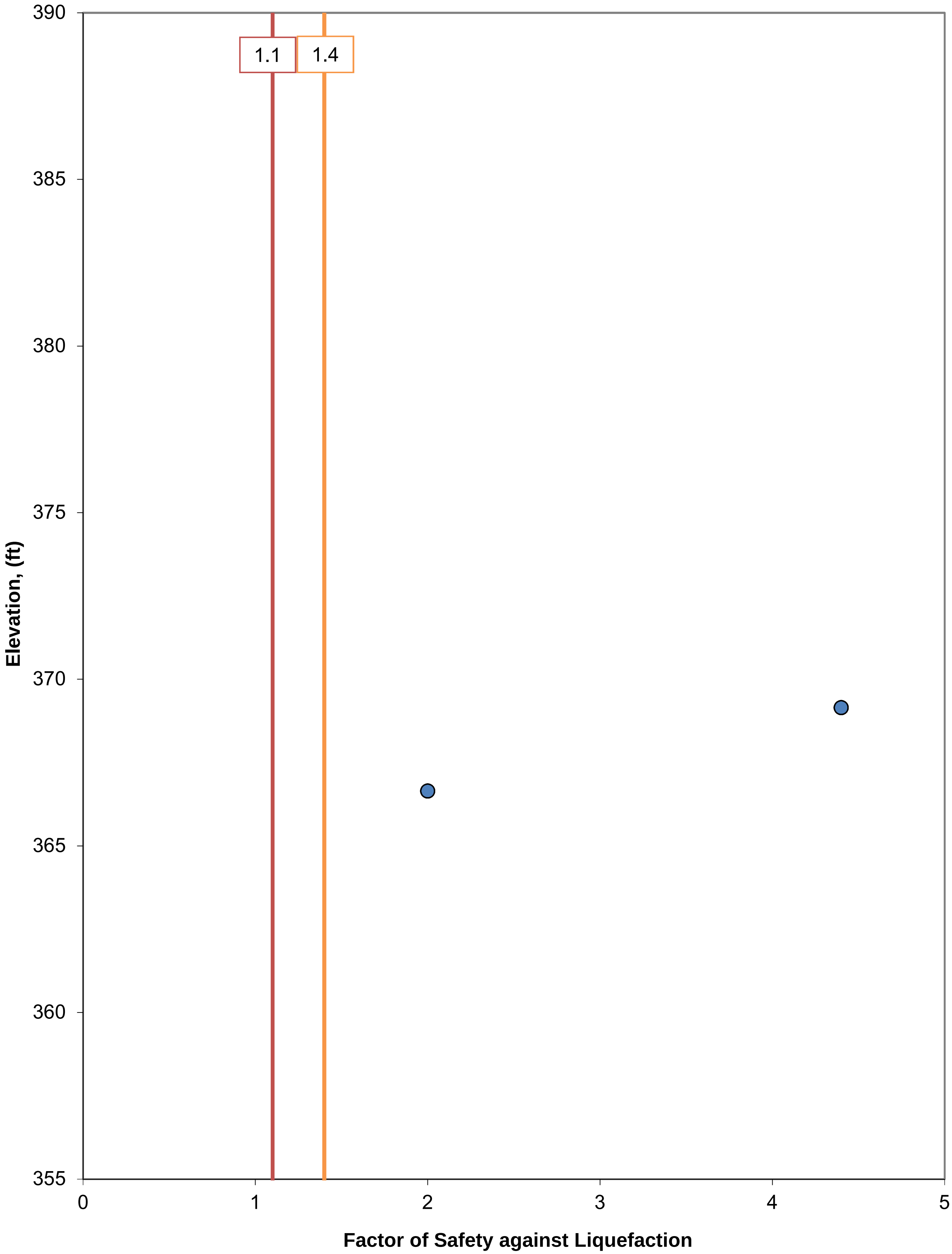

Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-B-6, Liquefaction Triggering Results
Top of Boring
El. 389.9 ft
El. 379.9
El. 369.9
El. 359.9

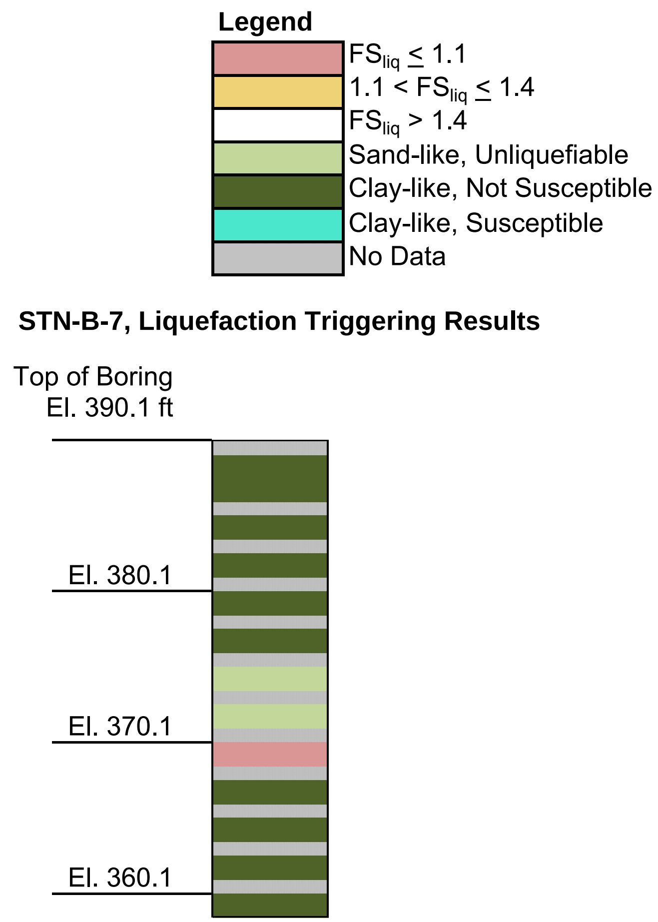

Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Unliquefiable
No Data
STN-B-6, Liquefaction Triggering Results
Top of Boring
El. 389.9 ft
El. 379.9
El. 369.9
El. 359.9

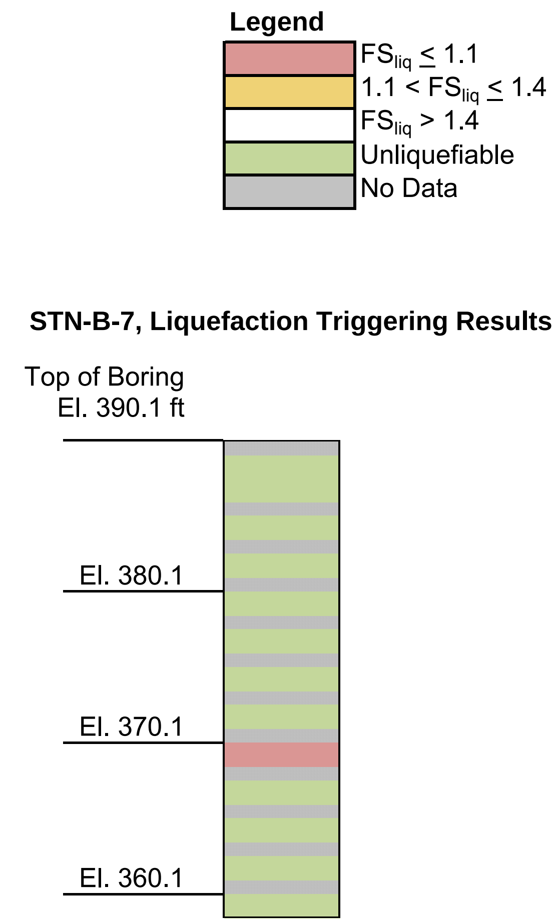
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-6, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



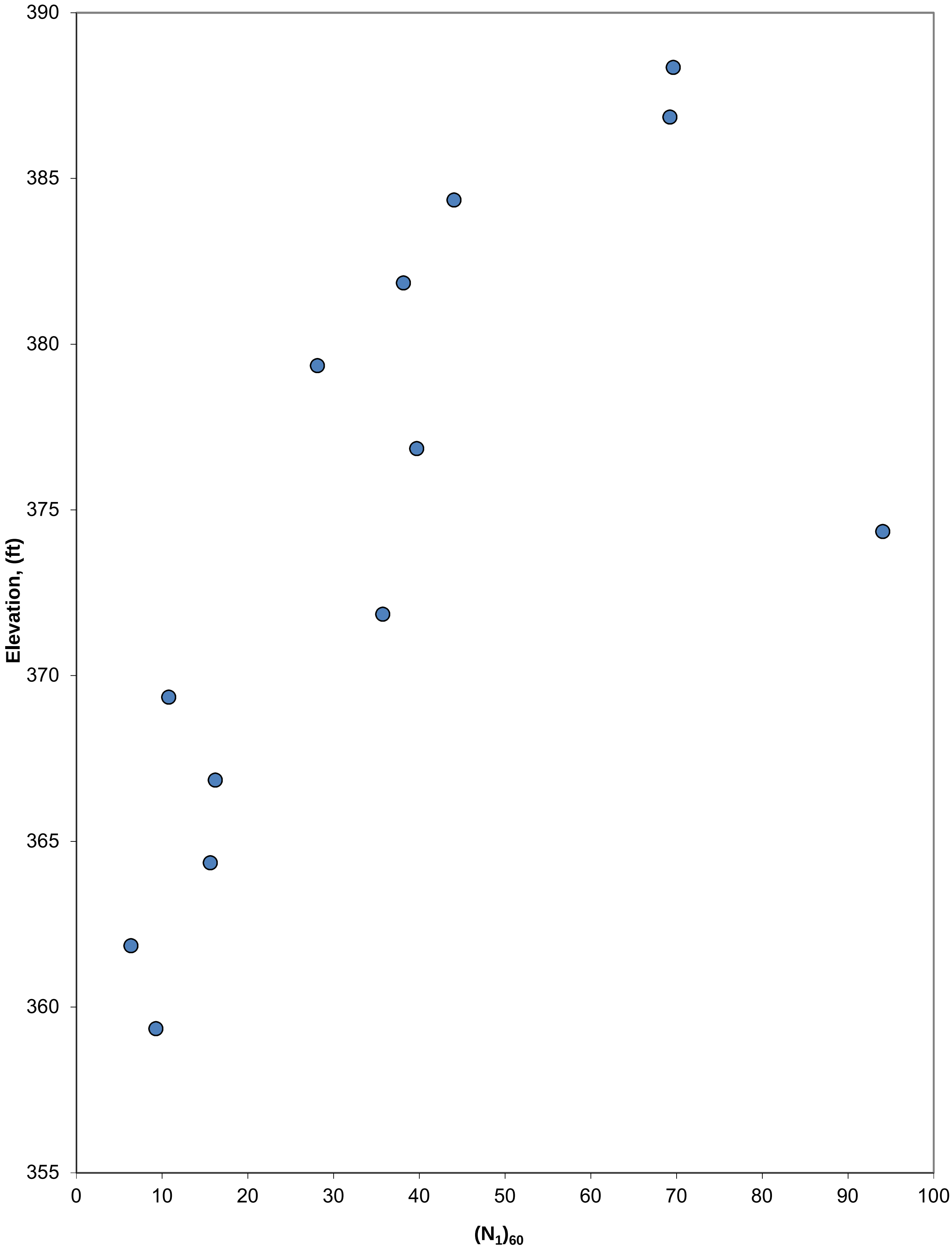
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-6, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



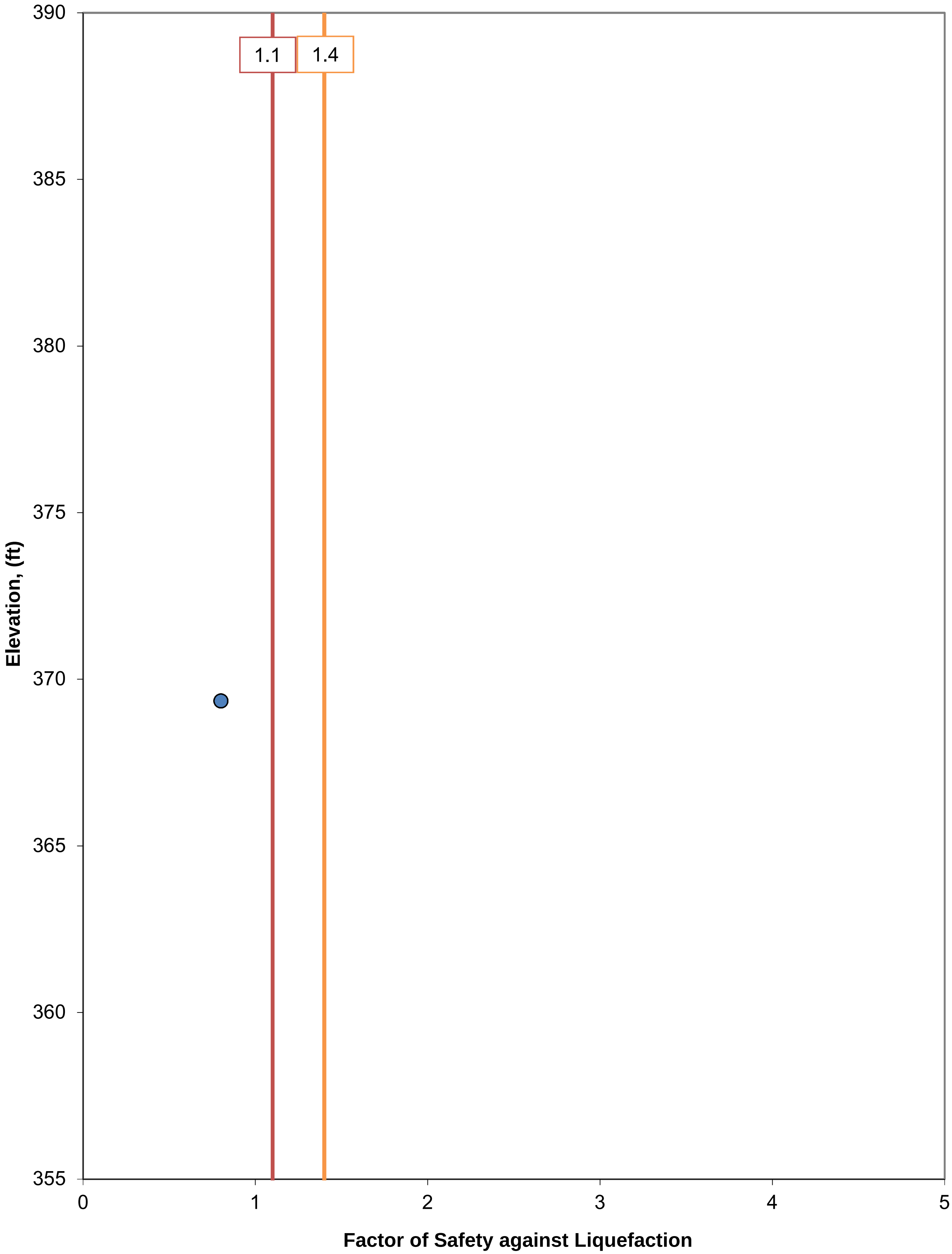

Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-B-7, Liquefaction Triggering Results
Top of Boring
El. 390.1 ft
El. 380.1
El. 370.1
El. 360.1

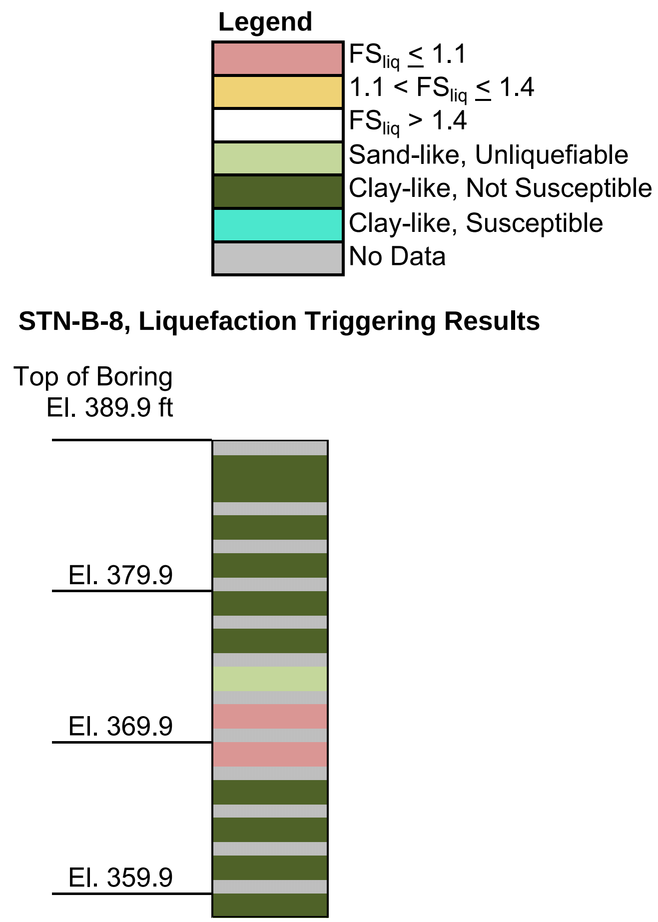

Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Unliquefiable
No Data
STN-B-7, Liquefaction Triggering Results
Top of Boring
El. 390.1 ft
El. 380.1
El. 370.1
El. 360.1

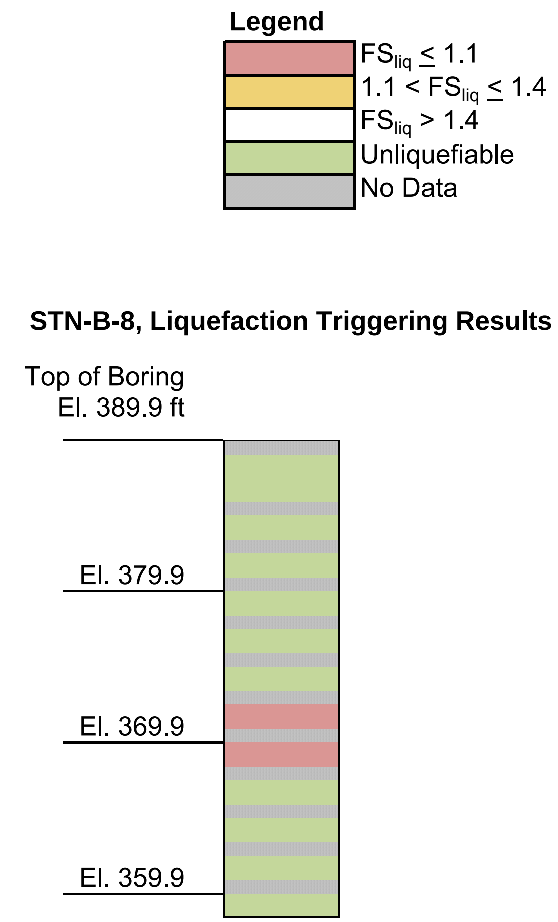
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-7, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



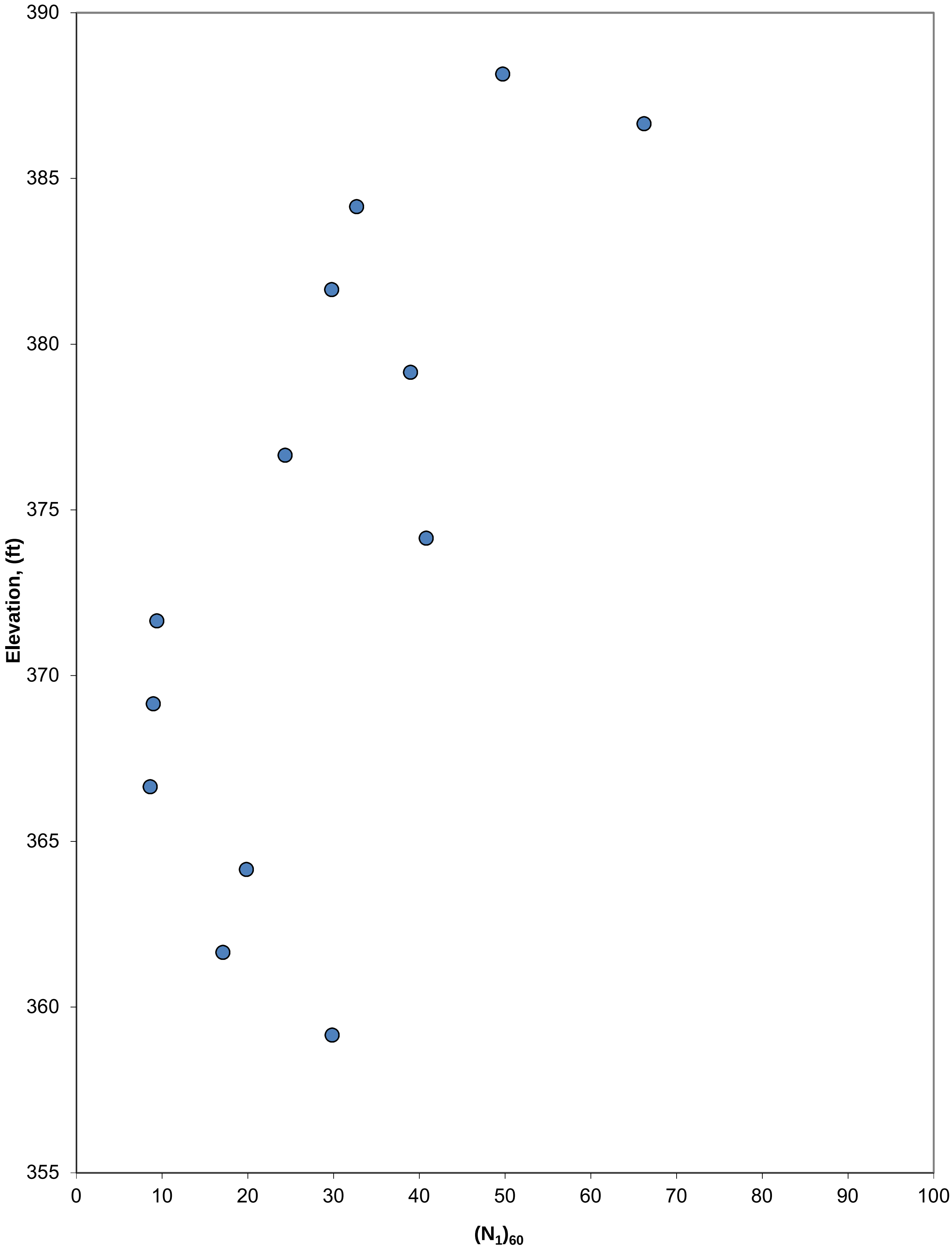
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-7, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



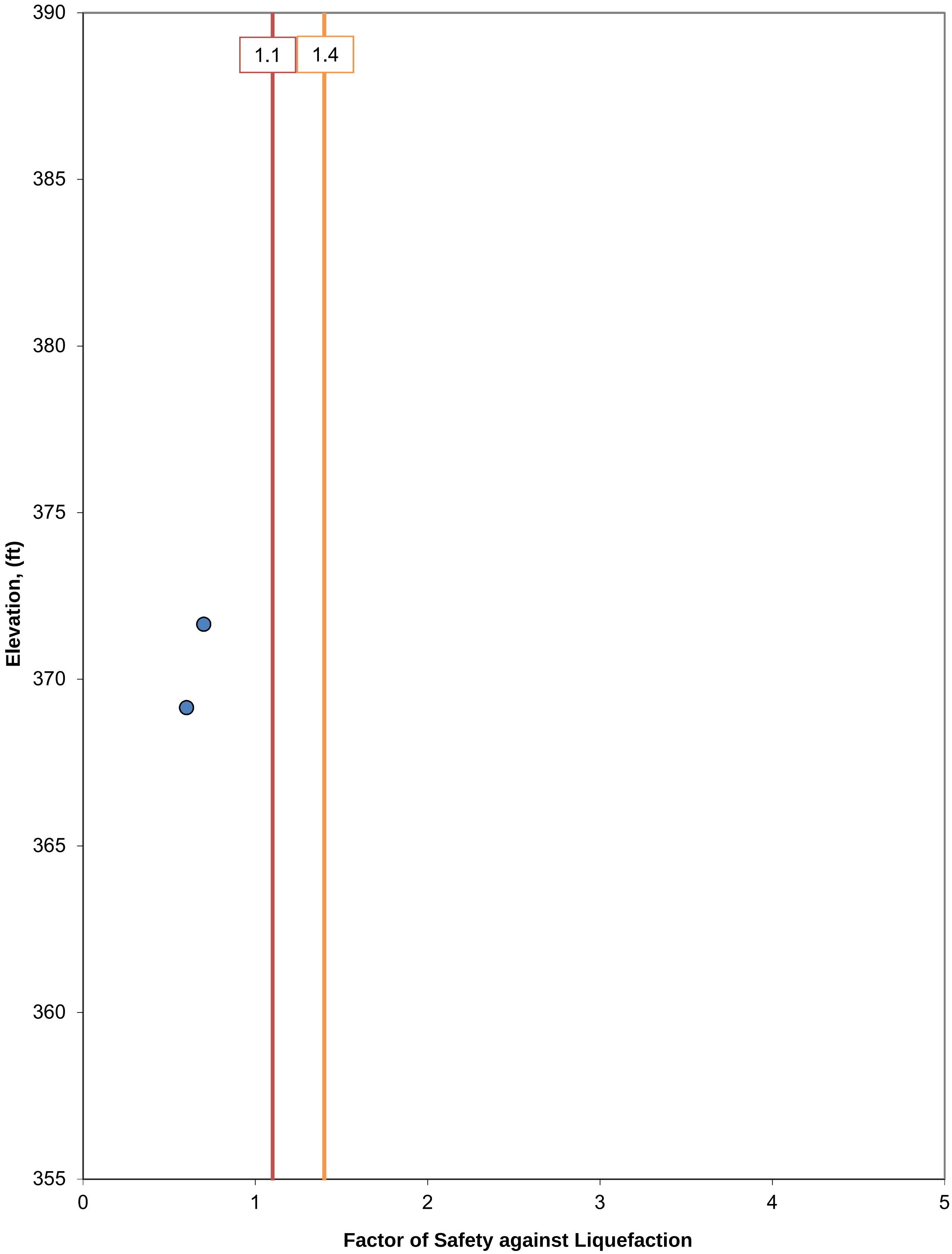

Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-B-8, Liquefaction Triggering Results
Top of Boring
El. 389.9 ft
El. 379.9
El. 369.9
El. 359.9


Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Unliquefiable
No Data
STN-B-8, Liquefaction Triggering Results
Top of Boring
El. 389.9 ft
El. 379.9
El. 369.9
El. 359.9

TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-8, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-8, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

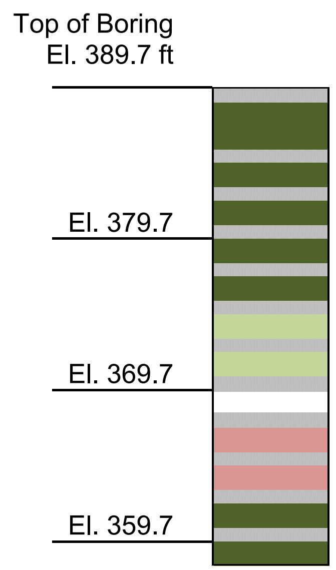


\\Us1243-f01\workgroup\1755\active\175568235\technical_production\analysis\Geotech\Seismic\Liquefaction and Cyclic Softening\Triggering Calculations\SiSALT_SPT\DesignEvent-DesignSource\DesignEvent-DesignSource_SiSALT_SPT_STN-B-8.xlsm, FSliqPlot 8/13/2018, 1 of 1

Legend

- $FS_{liq} \le 1.1$
- $1.1 < FS_{liq} \le 1.4$
- $FS_{liq} > 1.4$
- Sand-like, Unliquefiable
- Clay-like, Not Susceptible
- Clay-like, Susceptible
- No Data

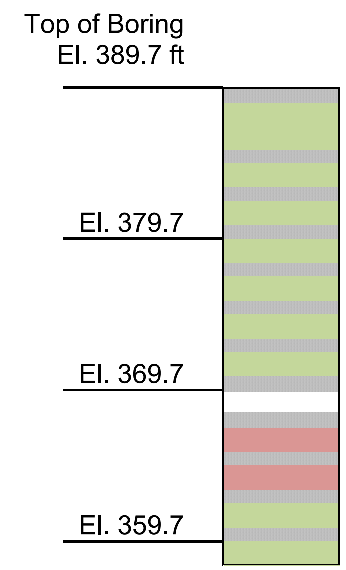
STN-B-9, Liquefaction Triggering Results



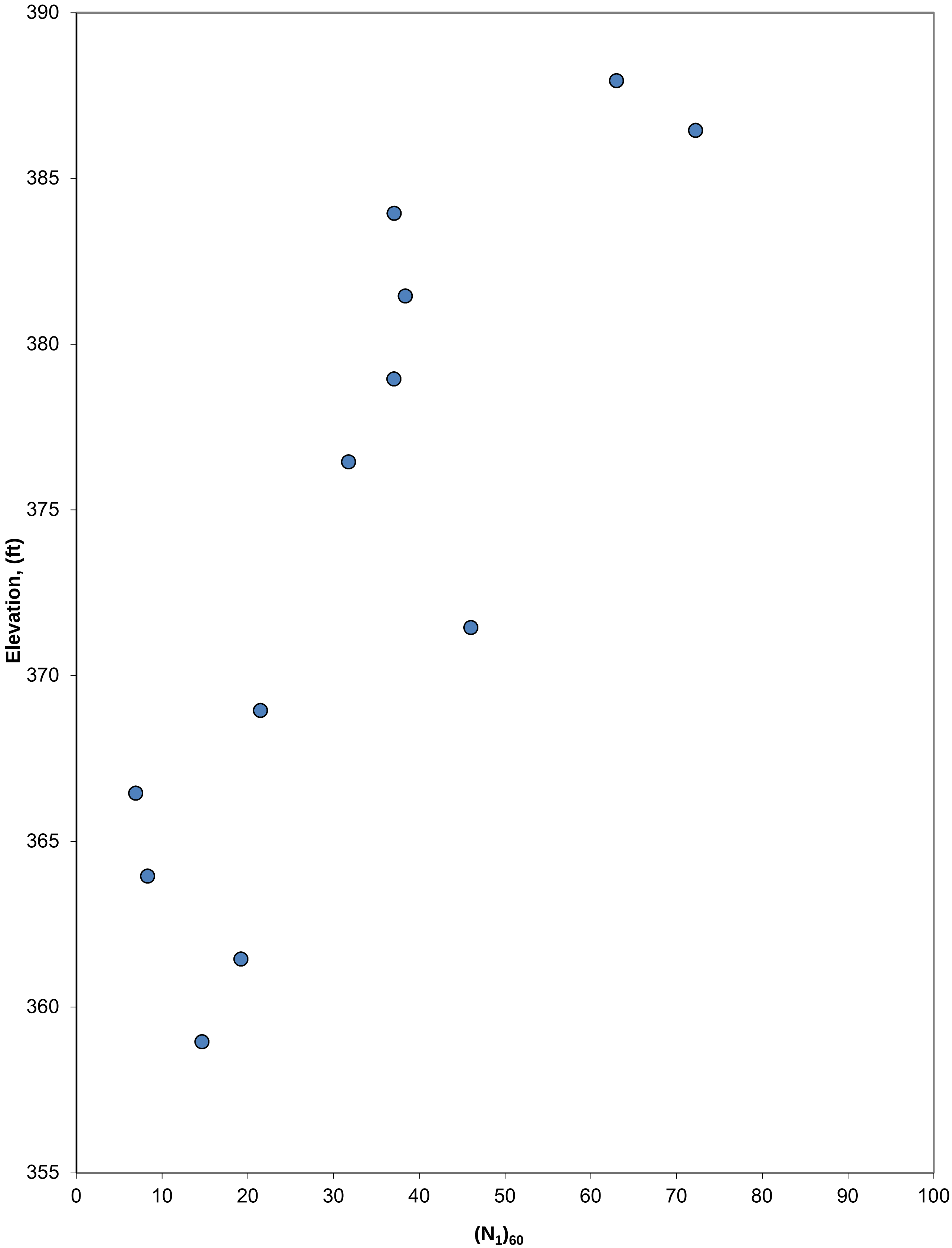
Legend

- $FS_{liq} \le 1.1$
- $1.1 < FS_{liq} \le 1.4$
- $FS_{liq} > 1.4$
- Unliquefiable
- No Data

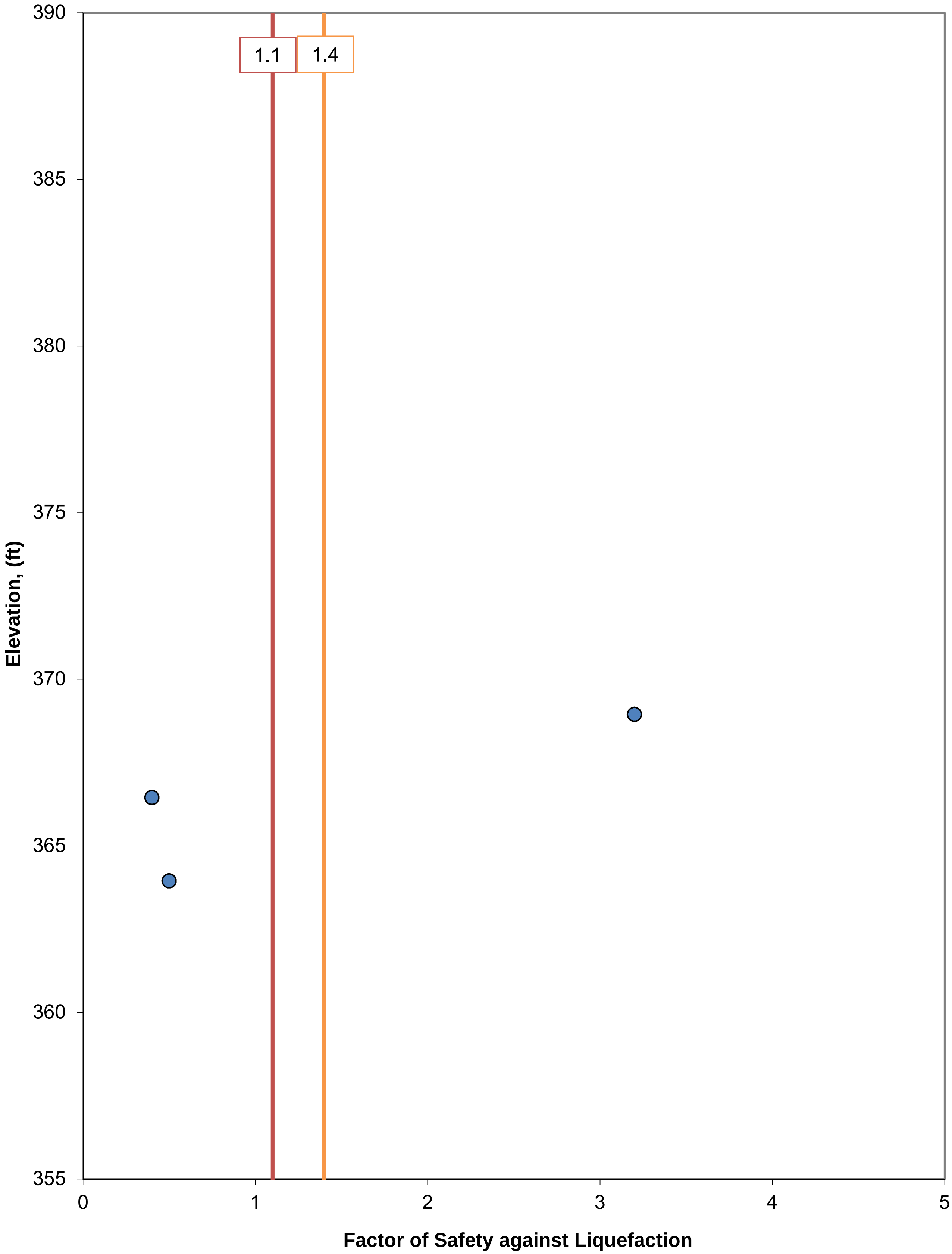
STN-B-9, Liquefaction Triggering Results



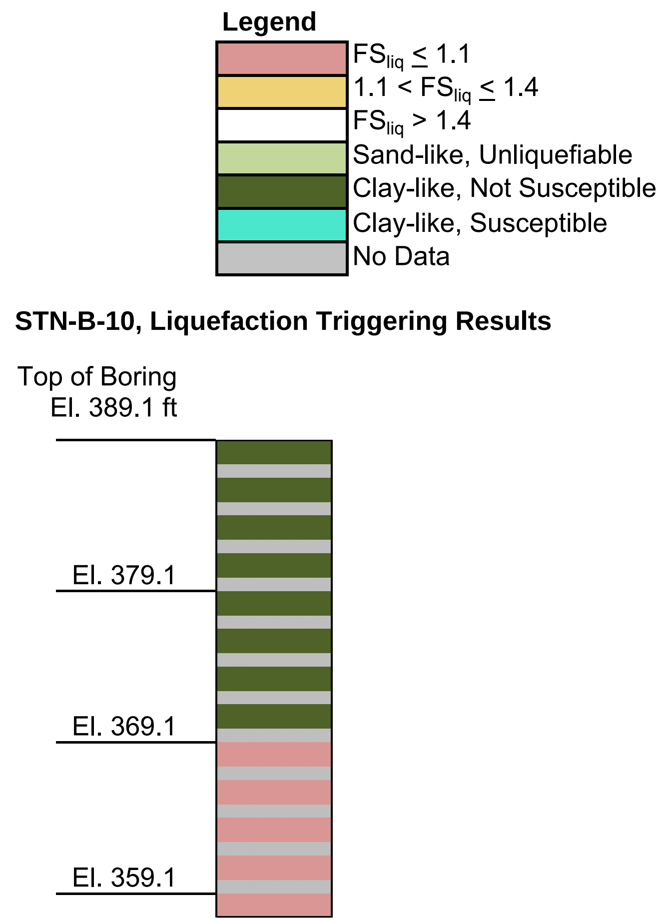
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-9, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

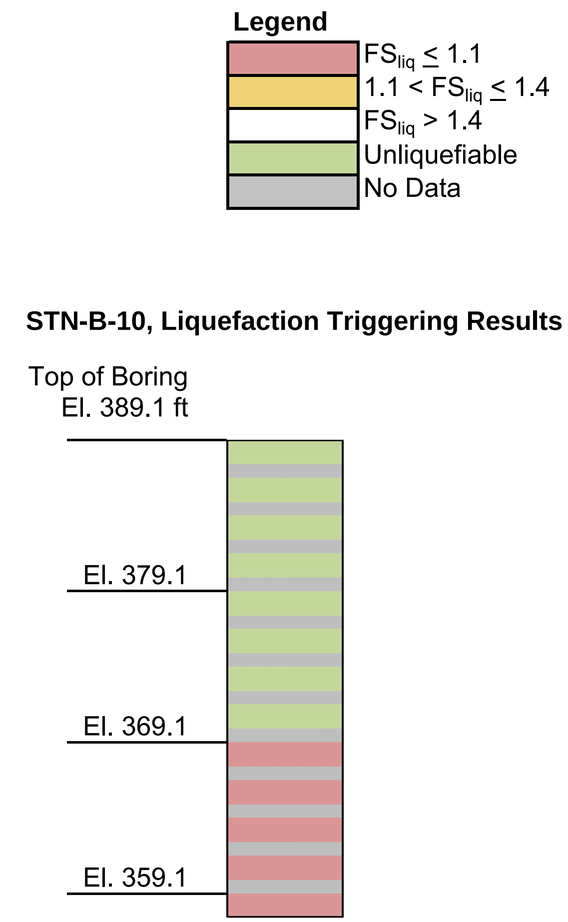


TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-9, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

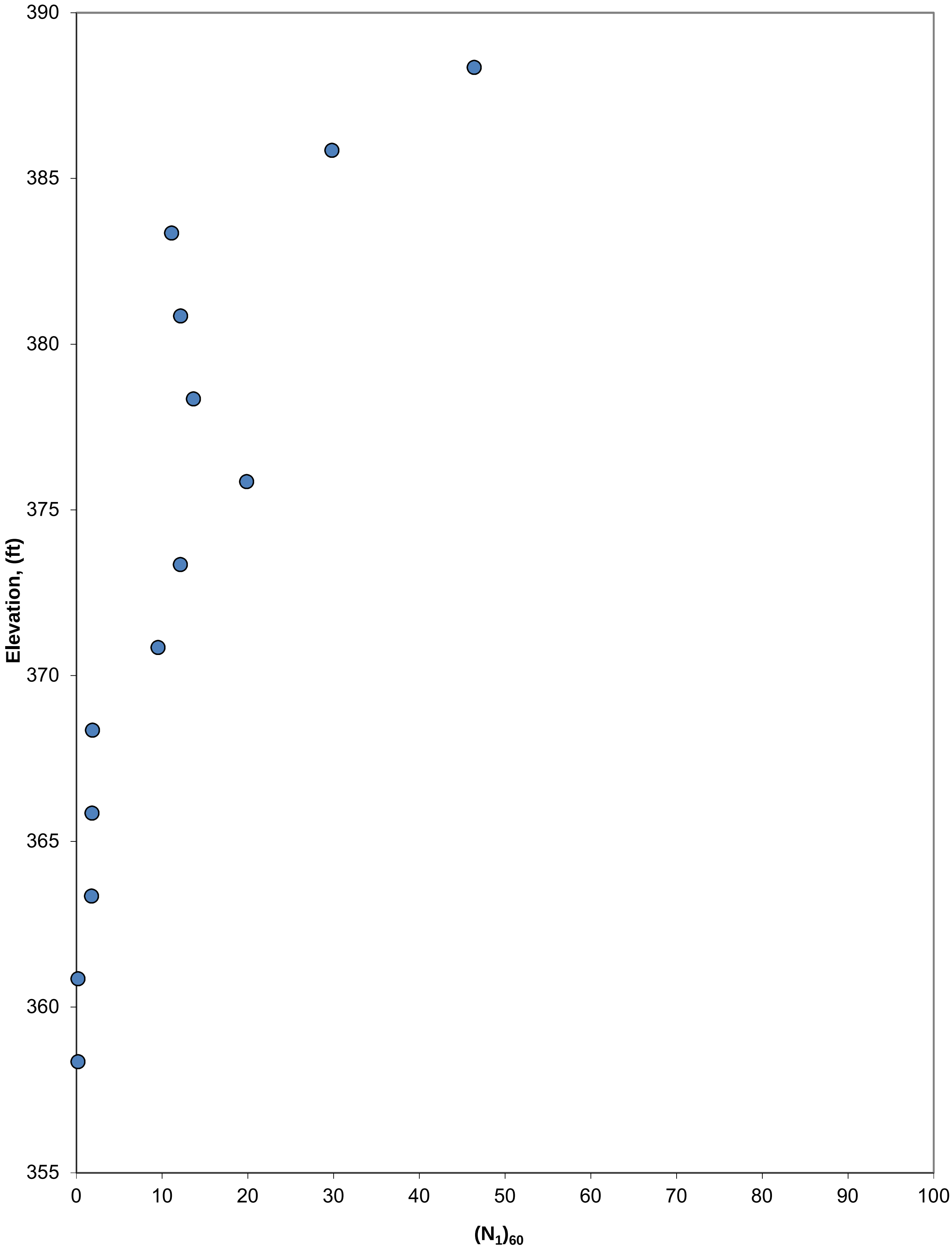


\\Us1243-f01\workgroup\1755\active\175568235\technical_production\analysis\Geotech\Seismic\Liquefaction and Cyclic Softening\Triggering Calculations\SiSALT_SPT\DesignEvent-DesignSource\DesignEvent-DesignSource_SiSALT_SPT_STN-B-9.xlsm, FSliqPlot 8/13/2018, 1 of 1

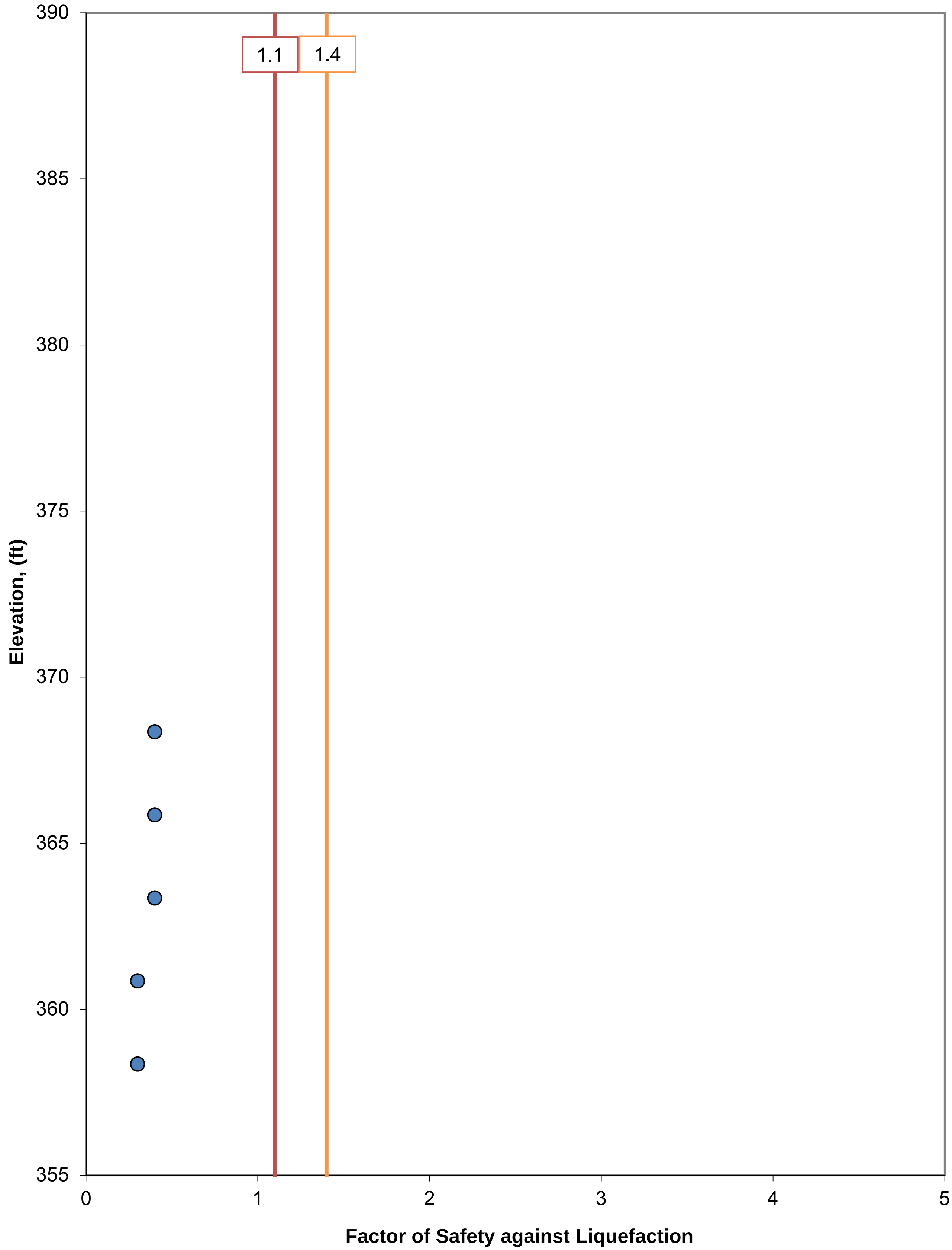

Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-B-10, Liquefaction Triggering Results
Top of Boring
El. 389.1 ft
El. 379.1
El. 369.1
El. 359.1


Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Unliquefiable
No Data
STN-B-10, Liquefaction Triggering Results
Top of Boring
El. 389.1 ft
El. 379.1
El. 369.1
El. 359.1

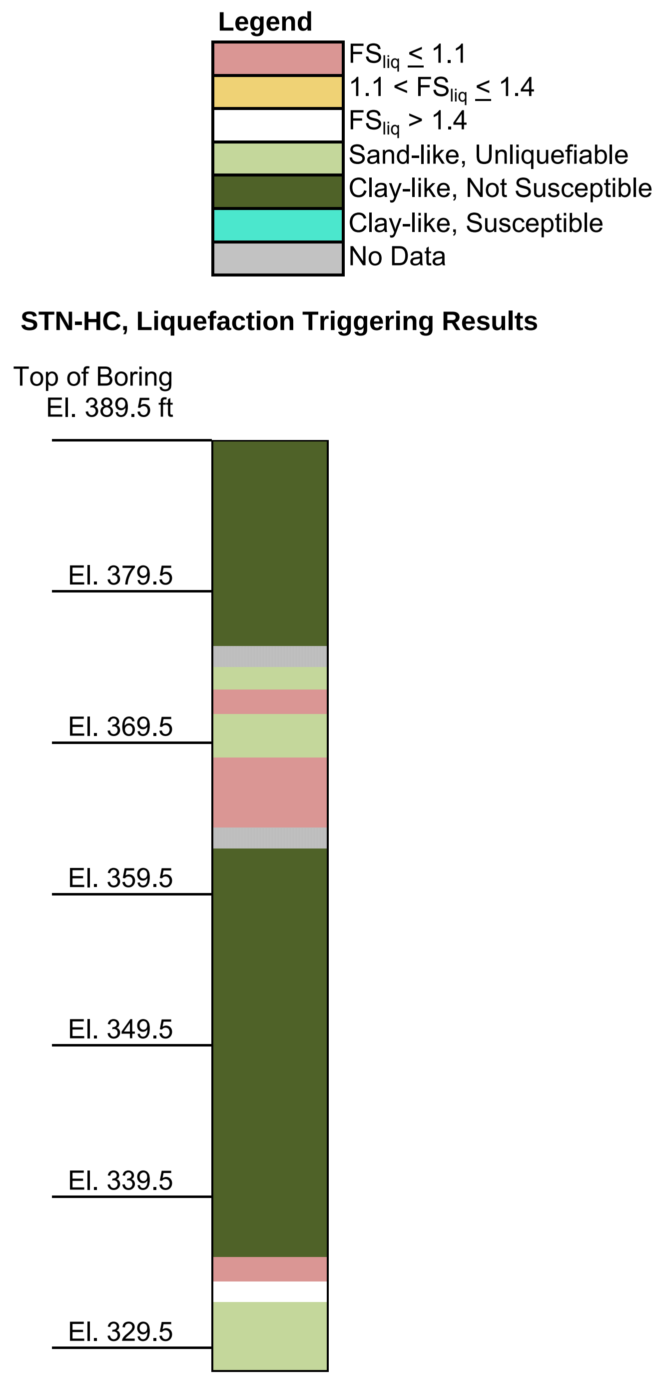
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-10, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

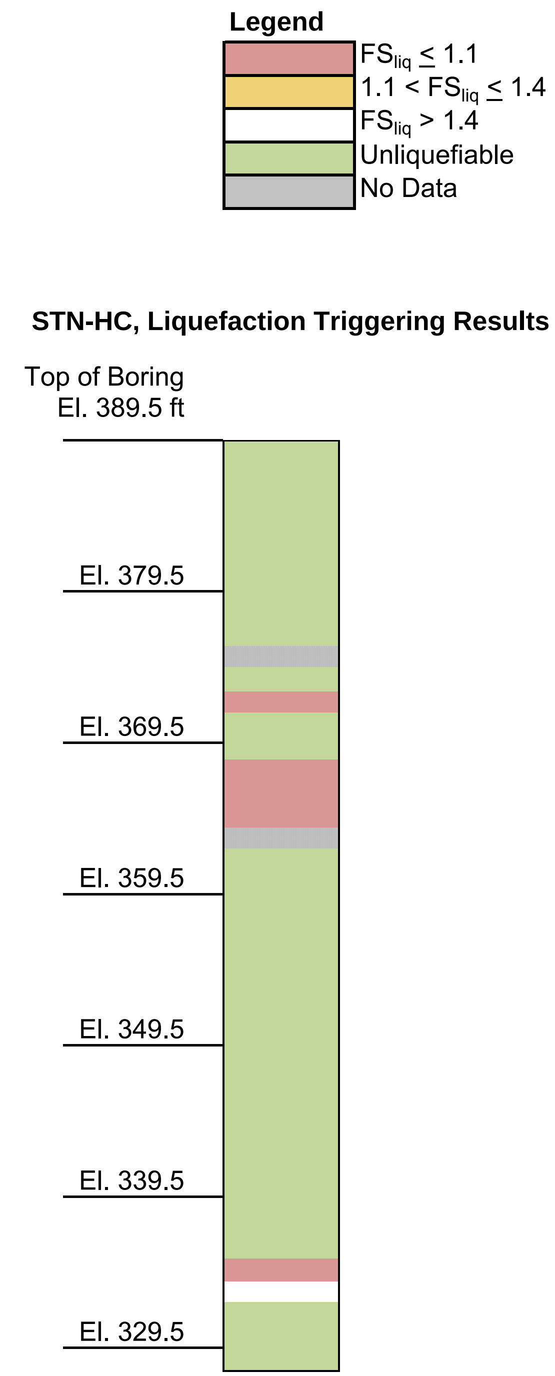


TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-B-10, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

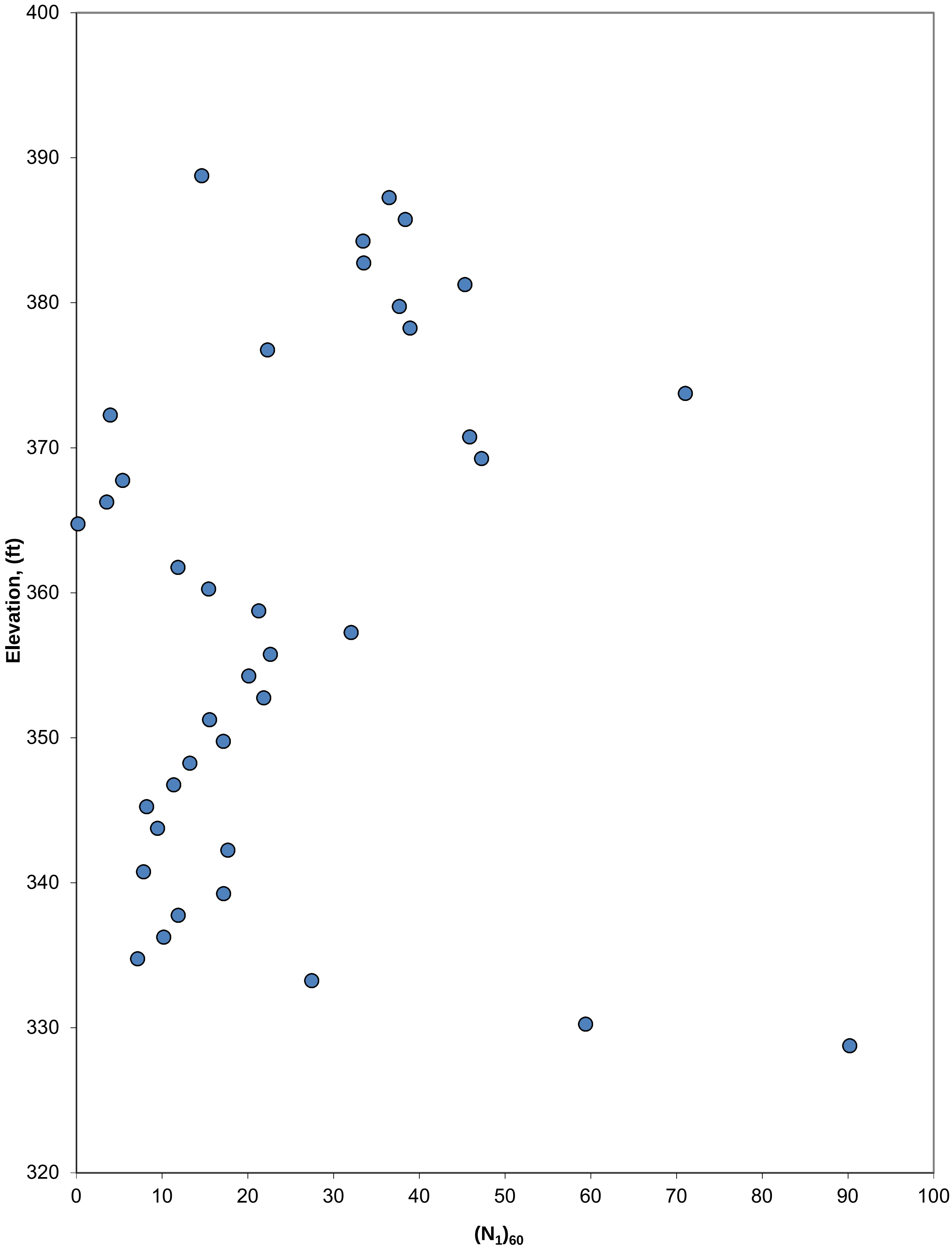


\\Us1243-f01\workgroup\1755\active\175568235\technical_production\analysis\Geotech\Seismic\Liquefaction and Cyclic Softening\Triggering Calculations\SiSALT_SPT\DesignEvent-DesignSource\DesignEvent-DesignSource_SiSALT_SPT_STN-B-10.xlsm, FSliqPlot8/13/2018, 1 of 1


Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-HC, Liquefaction Triggering Results
Top of Boring
El. 389.5 ft
El. 379.5
El. 369.5
El. 359.5
El. 349.5
El. 339.5
El. 329.5

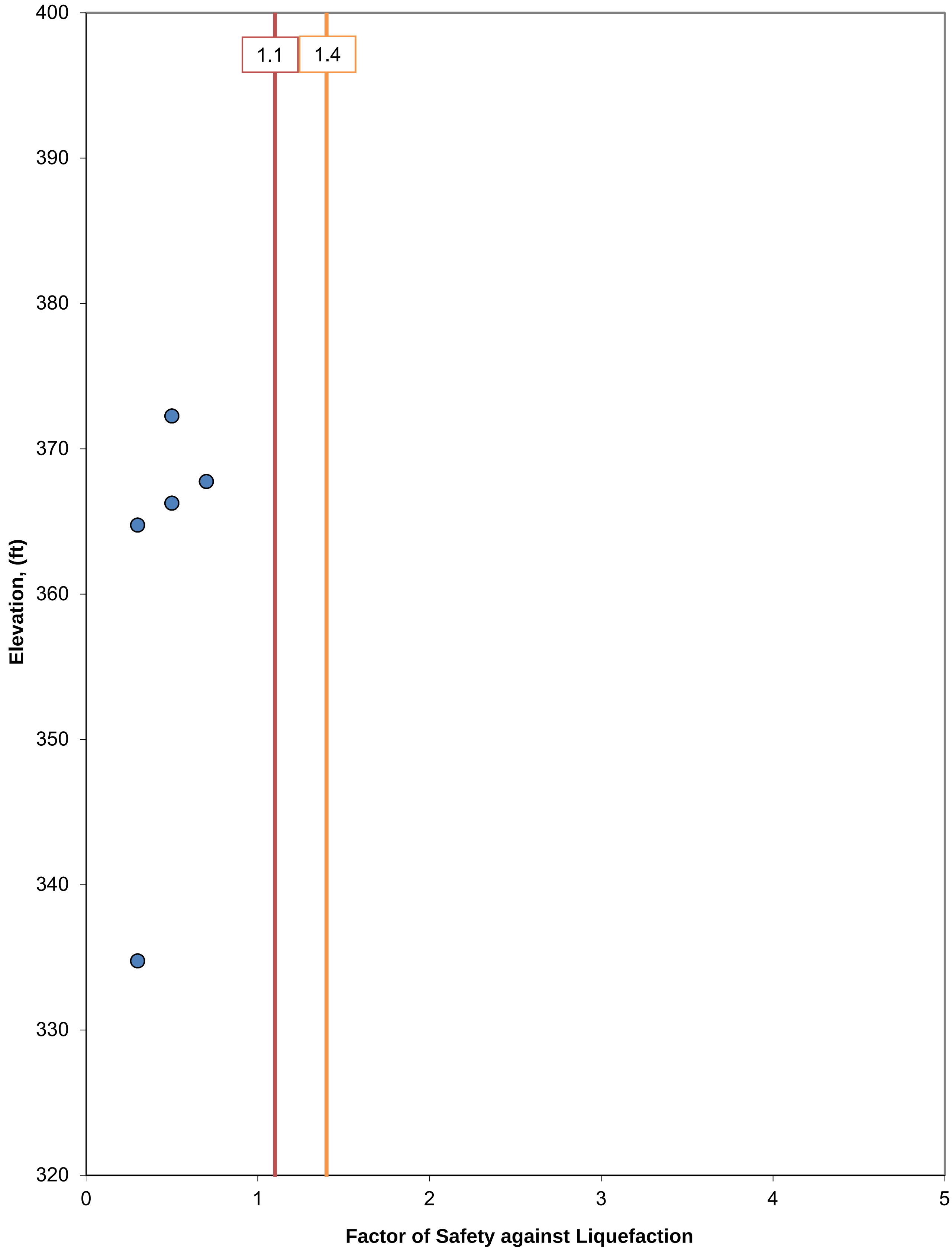

Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Unliquefiable
No Data
STN-HC, Liquefaction Triggering Results
Top of Boring
El. 389.5 ft
El. 379.5
El. 369.5
El. 359.5
El. 349.5
El. 339.5
El. 329.5

TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-HC, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

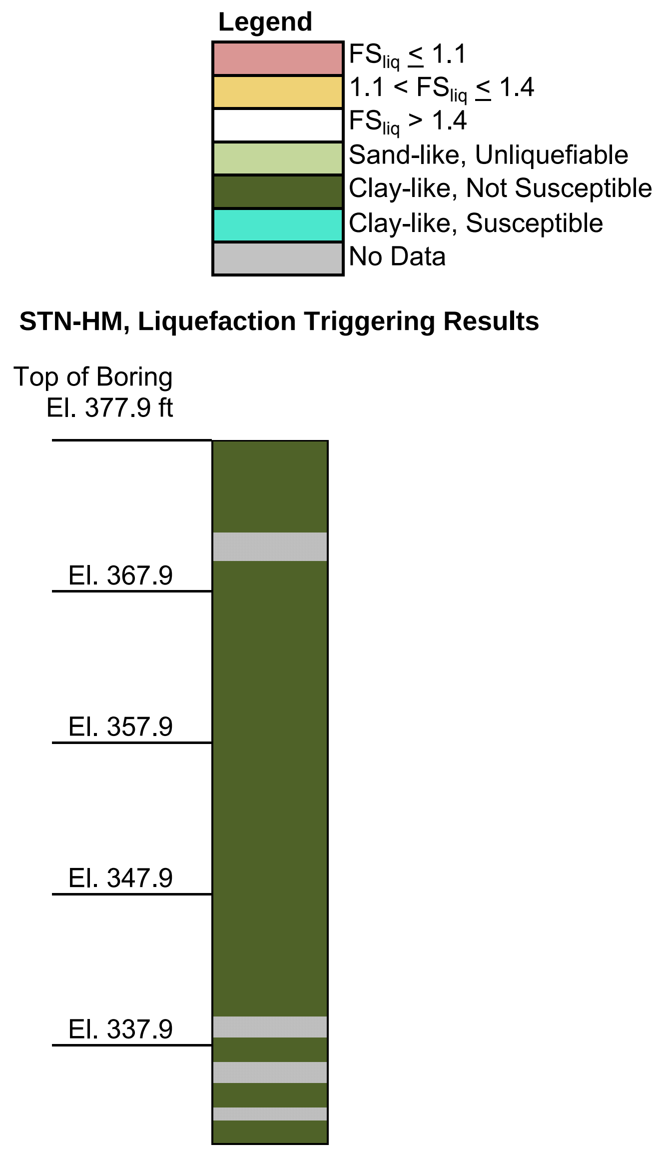


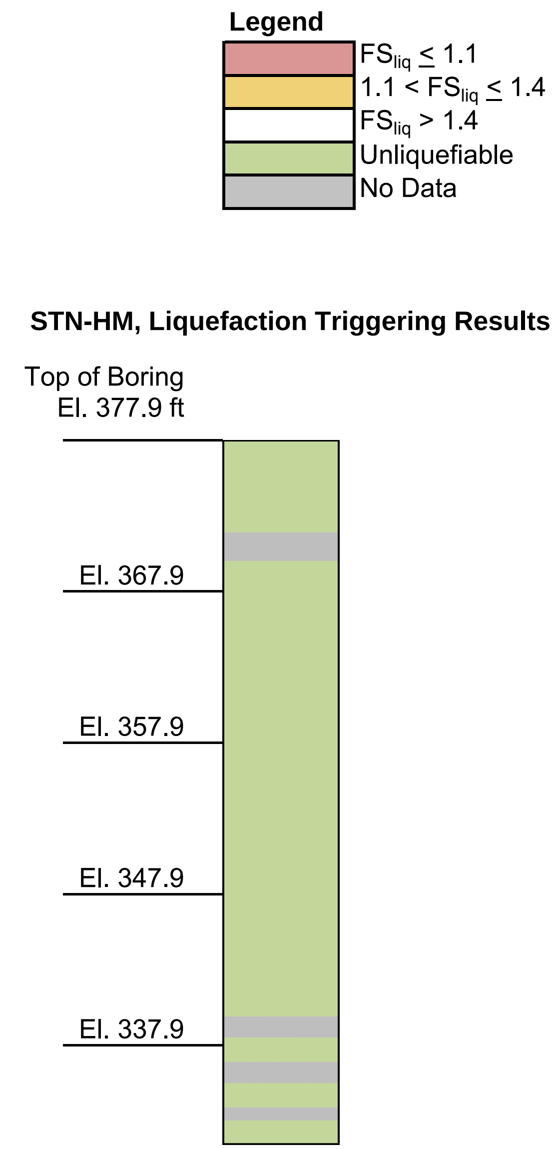
\\us1243-f01\workgroup\1755\active\175568235\technical_production\analysis\Geotech\Seismic\Liquefaction and Cyclic Softening\Triggering Calculations\SiSALT_SPT\DesignEvent-DesignSource\DesignEvent-DesignSource_SiSALT_SPT_STN-HC.xlsm, (N1)60 Plot

TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-HC, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

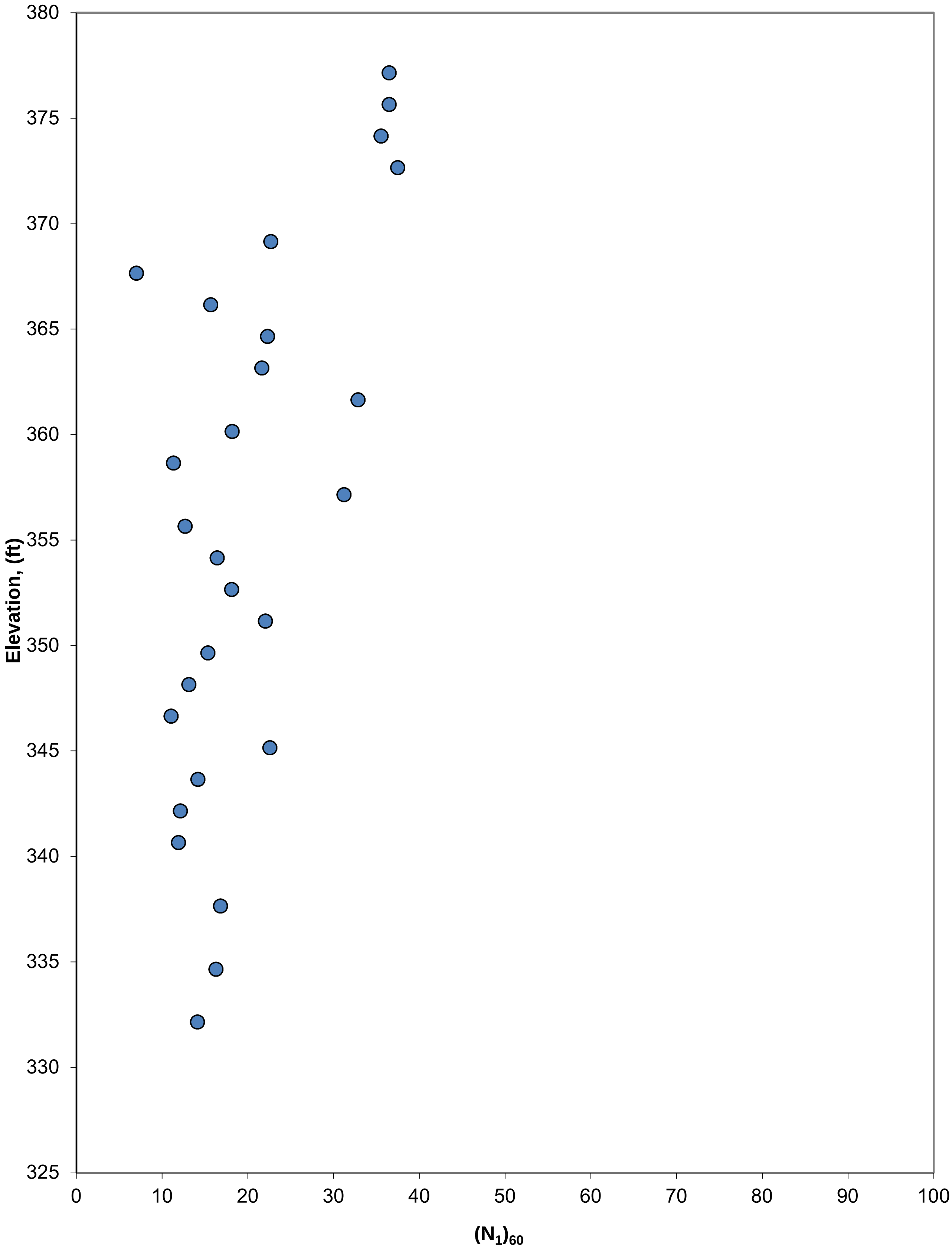


\\Us1243-f01\workgroup\1755\active\175568235\technical_production\analysis\Geotech\Seismic\Liquefaction and Cyclic Softening\Triggering Calculations\SiSALT_SPT\DesignEvent-DesignSource\DesignEvent-DesignSource_SiSALT_SPT_STN-HC.xlsm, FSliqPlot 8/13/2018, 1 of 1

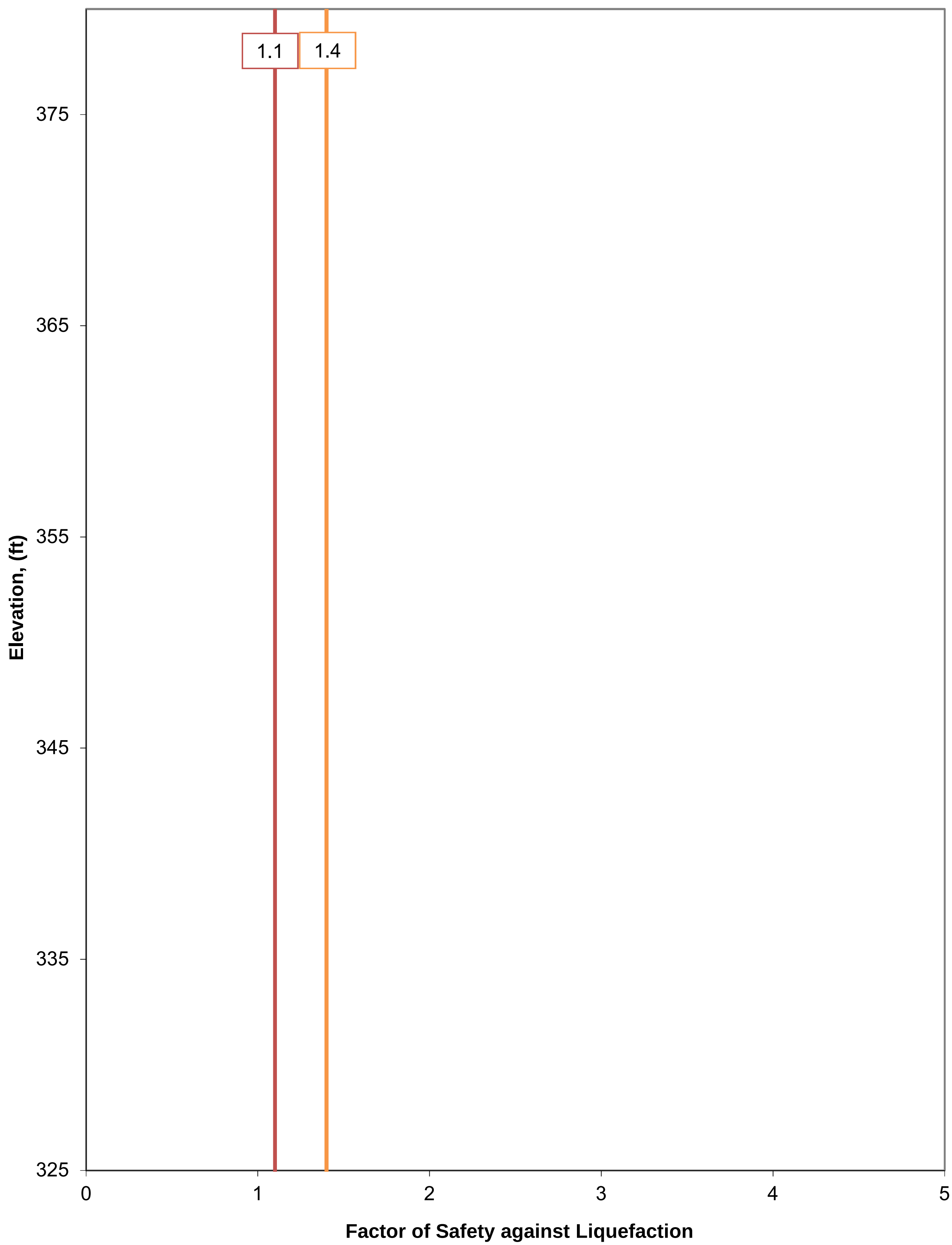

Legend
FS$_{liq}$ ≤ 1.1
1.1 < FS$_{liq}$ ≤ 1.4
FS$_{liq}$ > 1.4
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-HM, Liquefaction Triggering Results
Top of Boring
El. 377.9 ft
El. 367.9
El. 357.9
El. 347.9
El. 337.9

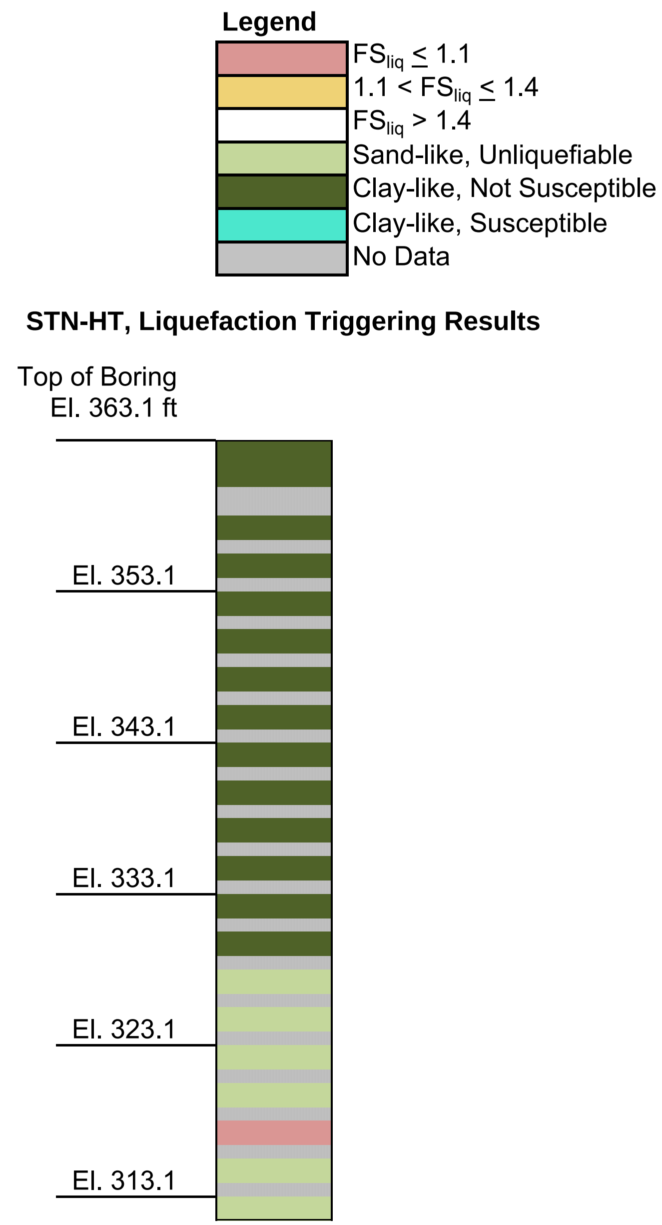

Legend
FS$_{liq}$ ≤ 1.1
1.1 < FS$_{liq}$ ≤ 1.4
FS$_{liq}$ > 1.4
Unliquefiable
No Data
STN-HM, Liquefaction Triggering Results
Top of Boring
El. 377.9 ft
El. 367.9
El. 357.9
El. 347.9
El. 337.9

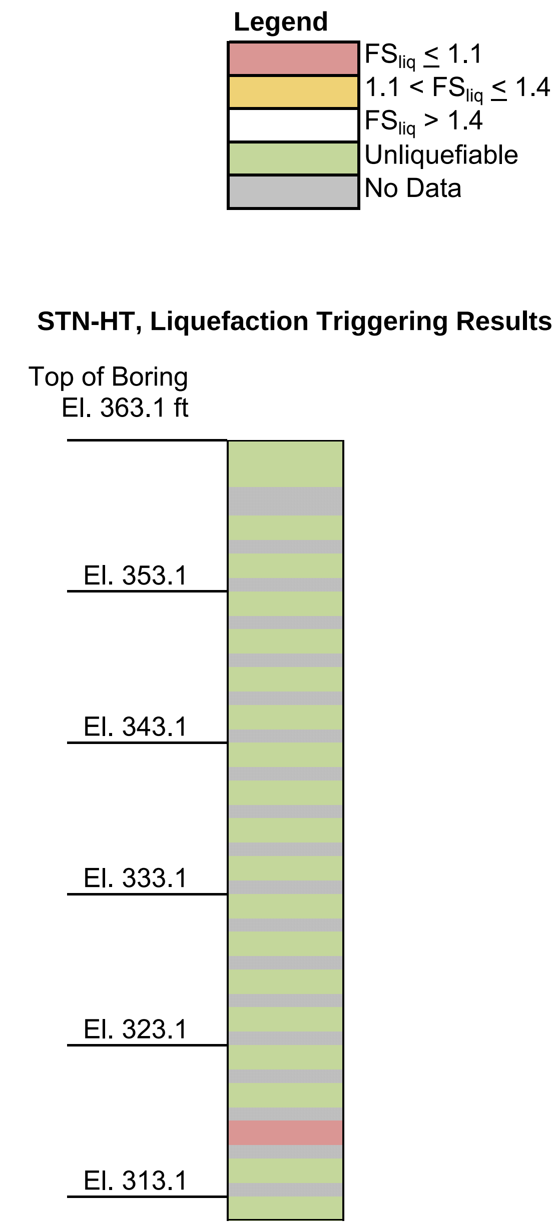
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-HM, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

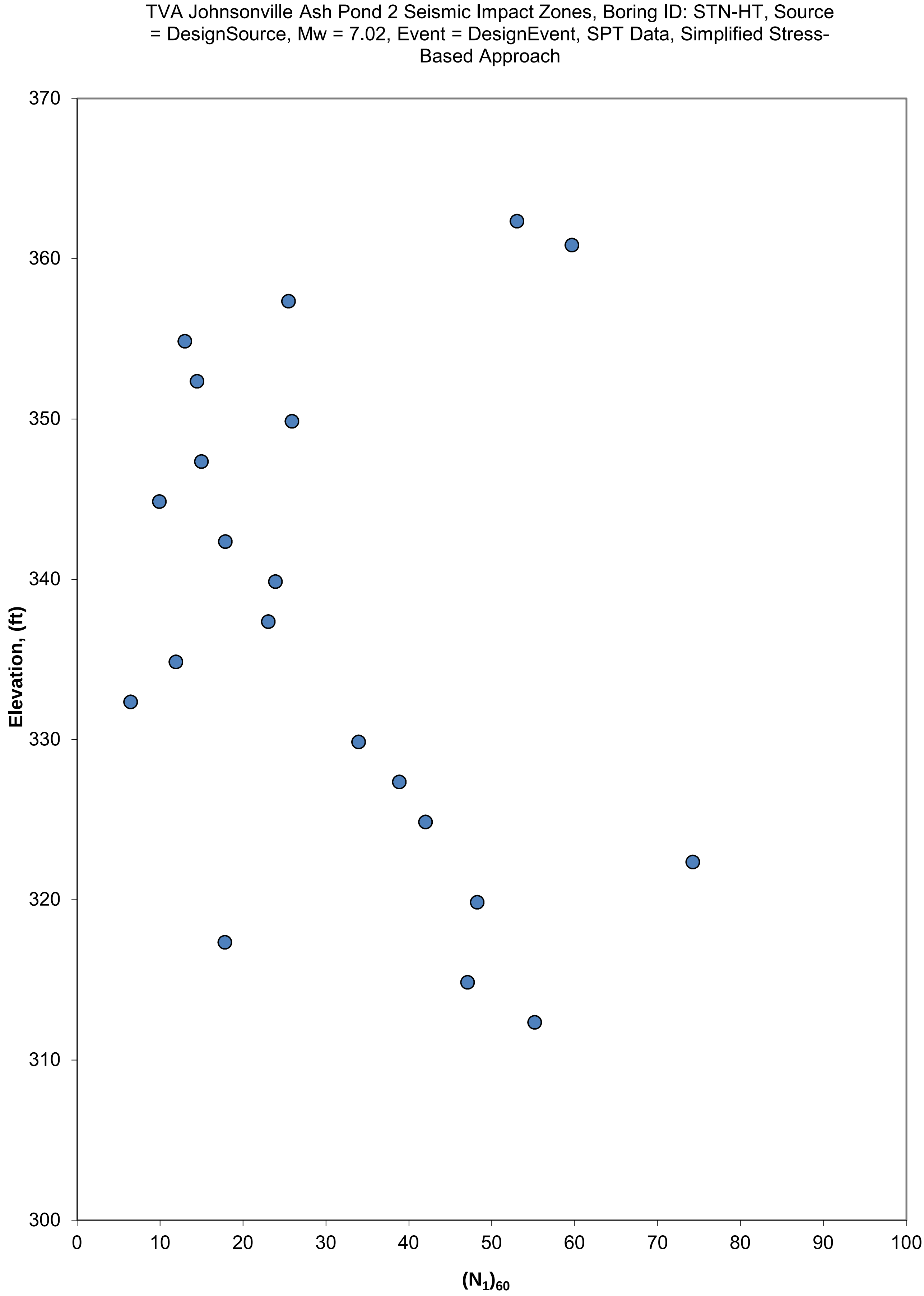


TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-HM, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach

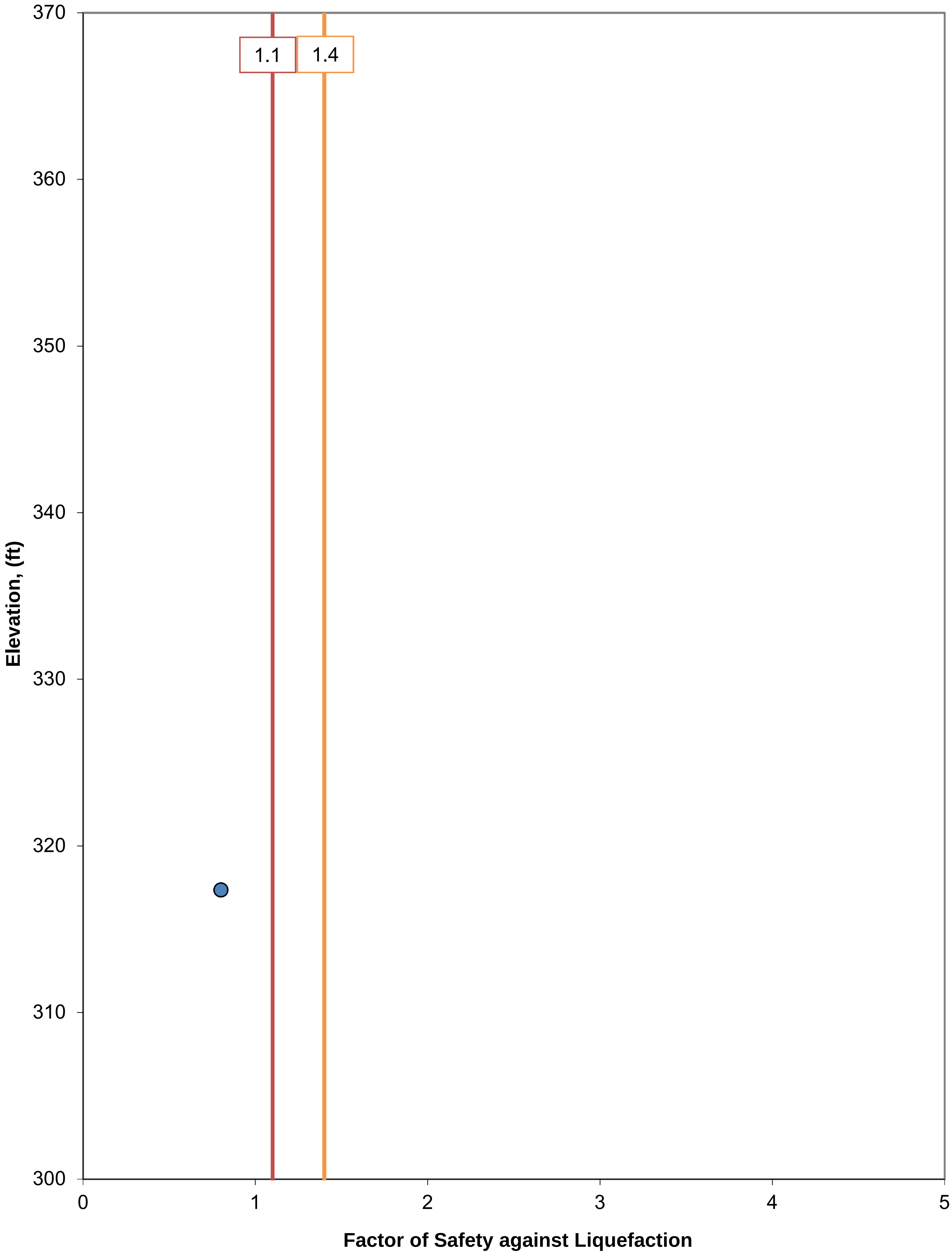



Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-HT, Liquefaction Triggering Results
Top of Boring
El. 363.1 ft
El. 353.1
El. 343.1
El. 333.1
El. 323.1
El. 313.1


Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Unliquefiable
No Data
STN-HT, Liquefaction Triggering Results
Top of Boring
El. 363.1 ft
El. 353.1
El. 343.1
El. 333.1
El. 323.1
El. 313.1


TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-HT, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach
370
360
350
340
330
320
310
300
Elevation, (ft)
0
10
20
30
40
50
60
70
80
90
100
(N1)60

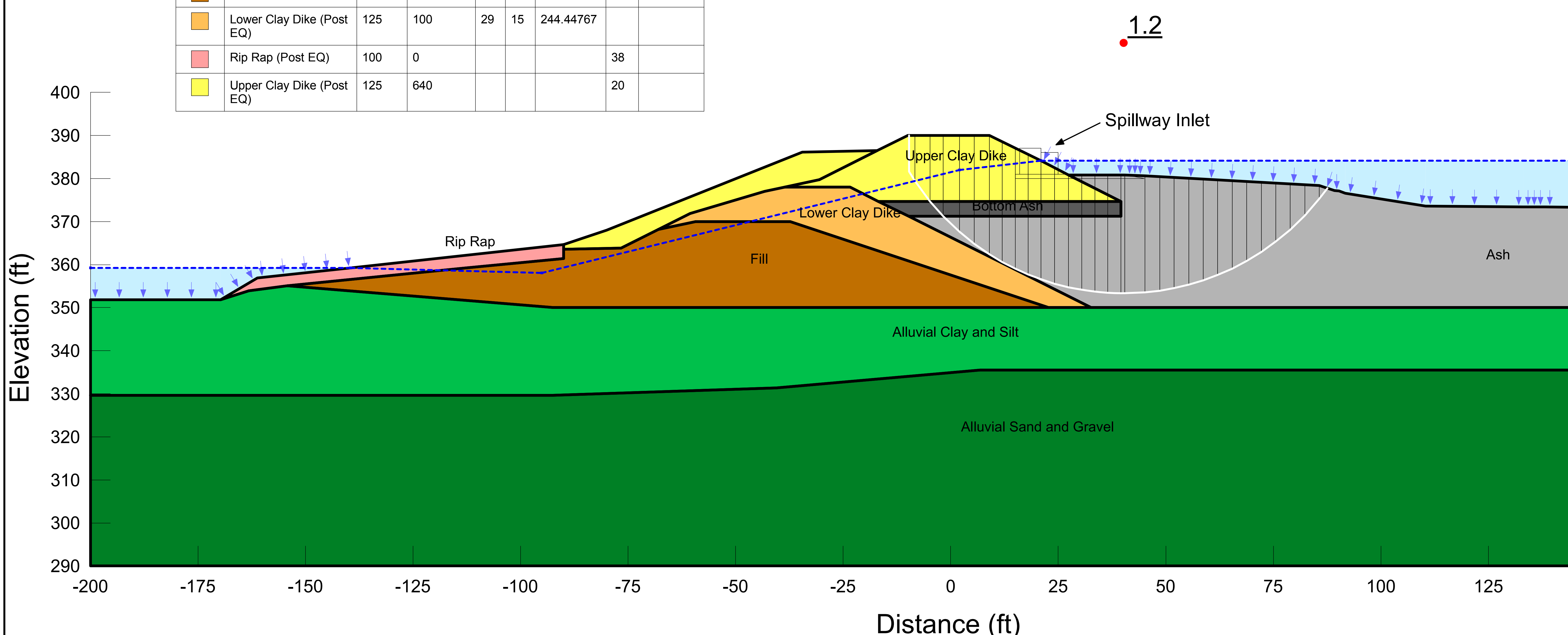
TVA Johnsonville Ash Pond 2 Seismic Impact Zones, Boring ID: STN-HT, Source = DesignSource, Mw = 7.02, Event = DesignEvent, SPT Data, Simplified Stress-Based Approach



ATTACHMENT D

TVA	Johnsonville Fossil Plant	Post-Earthquake Slope Stability Analysis – FS = 1.2	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.
	Humphreys County, Tennessee		
	Ash Pond 2, Profile - Section 1		

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)	Phi' (°)	Cohesion (psf)
	Alluvial Clay and Silt (Post EQ)	124	100	30	14	975.54353		
	Alluvial Sand and Gravel (Post EQ)	120	0				30	
	Ash (Post EQ)	100						150
	Bottom Ash (Post EQ)	125	0				35	
	Fill (Post EQ)	124	50	29	14	1,082.0344		
	Lower Clay Dike (Post EQ)	125	100	29	15	244.44767		
	Rip Rap (Post EQ)	100	0				38	
	Upper Clay Dike (Post EQ)	125	640				20	



Project No. 175568235

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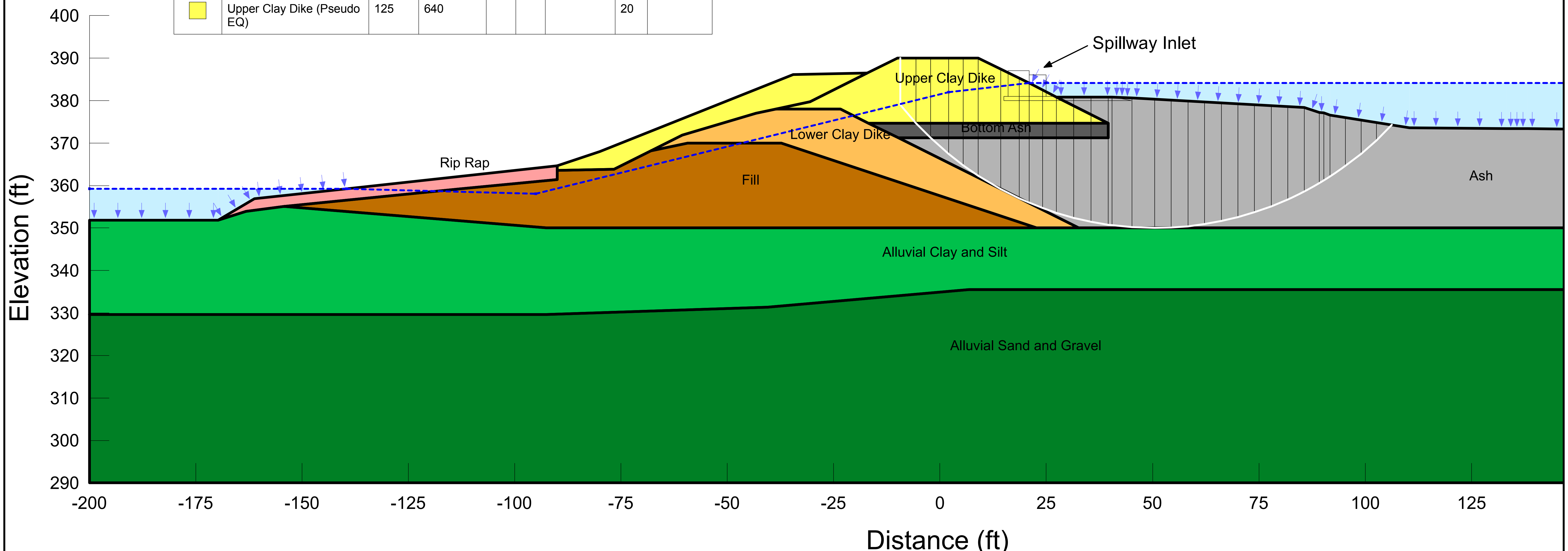
ATTACHMENT E

TVA	Johnsonville Fossil Plant	Pseudostatic Slope Stability Analysis – FS = 1.0	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.
	Humphreys County, Tennessee	Yield Acceleration ky=0.016g	
	Ash Pond 2, Profile - Section 1		

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)	Phi' (°)	Cohesion (psf)
	Alluvial Clay and Silt (Pseudo EQ)	124	100	30	14	975.54353		
	Alluvial Sand and Gravel (Pseudo EQ)	120	0				30	
	Ash (Post EQ)	100						150
	Bottom Ash (Pseudo EQ)	125	0				35	
	Fill (Pseudo EQ)	124	50	29	14	1,082.0344		
	Lower Clay Dike (Pseudo EQ)	125	100	29	15	244.44767		
	Rip Rap (Pseudo EQ)	100	0				38	
	Upper Clay Dike (Pseudo EQ)	125	640				20	



Project No. 175568235

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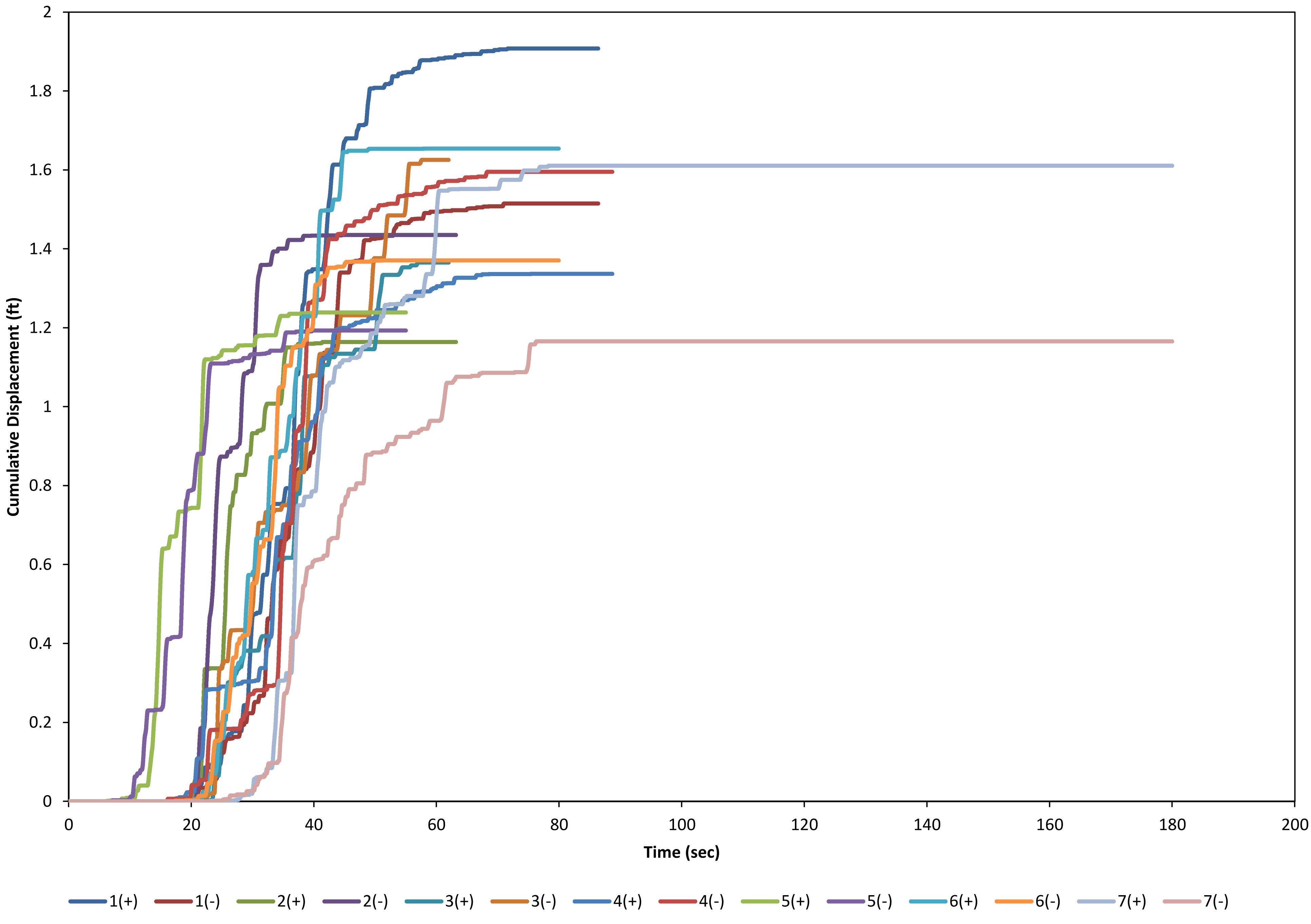
Facility	JOF	
Section/Group	1	
Profile Name	seism_normal	
Top of Profile Elevation	390	ft
Depth to water table during EQ	8	ft
Surcharge Pressure		psf

Strata Profile														
Non-input Information								**Strata "Soil Types" Input**				**Strata "Soil Profile" Input**		
Top Depth (ft)	**Bottom Depth (ft)**	**Layer Name**	**Layer Type**	**G_{max} (psf)**	**K_o**	**Mean Effective Stress at midpoint (psf)**	**Plasticity Index**	**Name (for Strata)**	**Unit Weight (pcf)**	**Modulus Reduction (G/Gmax) Model**	**Damping Model**	**Thickness (ft)**	**Soil Type (for Strata)**	**Shear Wave Velocity (fps)**
0	1.45	Upper Clay Dike	Soil	1.32E+06	0.5	60.4	23.88	Upper Clay Dike_1	125	D&S, p'=0-400, PI=20-25	D&S, p'=0-400, PI=20-25	1.45	Upper Clay Dike_1	583
1.45	2.9	Upper Clay Dike	Soil	1.32E+06	0.5	181.3	23.88	Upper Clay Dike_2	125	D&S, p'=0-400, PI=20-25	D&S, p'=0-400, PI=20-25	1.45	Upper Clay Dike_2	583
2.9	4.35	Upper Clay Dike	Soil	1.32E+06	0.5	302.1	23.88	Upper Clay Dike_3	125	D&S, p'=0-400, PI=20-25	D&S, p'=0-400, PI=20-25	1.45	Upper Clay Dike_3	583
4.35	5.8	Upper Clay Dike	Soil	1.32E+06	0.5	422.9	23.88	Upper Clay Dike_4	125	D&S, p'=400-800, PI=20-25	D&S, p'=400-800, PI=20-25	1.45	Upper Clay Dike_4	583
5.8	7.25	Upper Clay Dike	Soil	1.32E+06	0.5	543.8	23.88	Upper Clay Dike_5	125	D&S, p'=400-800, PI=20-25	D&S, p'=400-800, PI=20-25	1.45	Upper Clay Dike_5	583
7.25	8	Upper Clay Dike	Soil	1.32E+06	0.5	635.4	23.88	Upper Clay Dike_6	125	D&S, p'=400-800, PI=20-25	D&S, p'=400-800, PI=20-25	0.75	Upper Clay Dike_6	583
8	9.45	Upper Clay Dike	Soil	1.32E+06	0.5	696.9	23.88	Upper Clay Dike_7	125	D&S, p'=400-800, PI=20-25	D&S, p'=400-800, PI=20-25	1.45	Upper Clay Dike_7	583
9.45	10.9	Upper Clay Dike	Soil	1.32E+06	0.5	757.4	23.88	Upper Clay Dike_8	125	D&S, p'=400-800, PI=20-25	D&S, p'=400-800, PI=20-25	1.45	Upper Clay Dike_8	583
10.9	12.35	Upper Clay Dike	Soil	1.32E+06	0.5	818.0	23.88	Upper Clay Dike_9	125	D&S, p'=800-1600, PI=20-25	D&S, p'=800-1600, PI=20-25	1.45	Upper Clay Dike_9	583
12.35	13.8	Upper Clay Dike	Soil	1.32E+06	0.5	878.5	23.88	Upper Clay Dike_10	125	D&S, p'=800-1600, PI=20-25	D&S, p'=800-1600, PI=20-25	1.45	Upper Clay Dike_10	583
13.8	15.25	Upper Clay Dike	Soil	1.32E+06	0.5	939.0	23.88	Upper Clay Dike_11	125	D&S, p'=800-1600, PI=20-25	D&S, p'=800-1600, PI=20-25	1.45	Upper Clay Dike_11	583
15.25	15.3	Upper Clay Dike	Soil	1.32E+06	0.5	970.3	23.88	Upper Clay Dike_12	125	D&S, p'=800-1600, PI=20-25	D&S, p'=800-1600, PI=20-25	0.05	Upper Clay Dike_12	583
15.3	17.46	Bottom Ash	Soil	2.91E+06	0.5	1016.4	0	Bottom Ash_13	125	D&S, p'=800-1600, PI=0	D&S, p'=800-1600, PI=0	2.16	Bottom Ash_13	866
17.46	19.62	Bottom Ash	Soil	2.91E+06	0.5	1106.5	0	Bottom Ash_14	125	D&S, p'=800-1600, PI=0	D&S, p'=800-1600, PI=0	2.16	Bottom Ash_14	866
19.62	21.78	Bottom Ash	Soil	2.91E+06	0.5	1196.7	0	Bottom Ash_15	125	D&S, p'=800-1600, PI=0	D&S, p'=800-1600, PI=0	2.16	Bottom Ash_15	866
21.78	23.94	Bottom Ash	Soil	2.91E+06	0.5	1286.8	0	Bottom Ash_16	125	D&S, p'=800-1600, PI=0	D&S, p'=800-1600, PI=0	2.16	Bottom Ash_16	866
23.94	26.1	Bottom Ash	Soil	2.91E+06	0.5	1377.0	0	Bottom Ash_17	125	D&S, p'=800-1600, PI=0	D&S, p'=800-1600, PI=0	2.16	Bottom Ash_17	866
26.1	26.3	Bottom Ash	Soil	2.91E+06	0.5	1426.2	0	Bottom Ash_18	125	D&S, p'=800-1600, PI=0	D&S, p'=800-1600, PI=0	0.2	Bottom Ash_18	866
26.3	28.17	Lower Clay Dike	Soil	2.18E+06	0.5	1469.4	27.7	Lower Clay Dike_19	125	D&S, p'=800-1600, PI=25-30	D&S, p'=800-1600, PI=25-30	1.87	Lower Clay Dike_19	750
28.17	30.04	Lower Clay Dike	Soil	2.18E+06	0.5	1547.4	27.7	Lower Clay Dike_20	125	D&S, p'=800-1600, PI=25-30	D&S, p'=800-1600, PI=25-30	1.87	Lower Clay Dike_20	750
30.04	31.91	Lower Clay Dike	Soil	2.18E+06	0.5	1625.5	27.7	Lower Clay Dike_21	125	D&S, p'=1600-2500, PI=25-30	D&S, p'=1600-2500, PI=25-30	1.87	Lower Clay Dike_21	750
31.91	33.78	Lower Clay Dike	Soil	2.18E+06	0.5	1703.5	27.7	Lower Clay Dike_22	125	D&S, p'=1600-2500, PI=25-30	D&S, p'=1600-2500, PI=25-30	1.87	Lower Clay Dike_22	750
33.78	35	Lower Clay Dike	Soil	2.18E+06	0.5	1768.0	27.7	Lower Clay Dike_23	125	D&S, p'=1600-2500, PI=25-30	D&S, p'=1600-2500, PI=25-30	1.22	Lower Clay Dike_23	750
35	36.97	Fill	Soil	2.40E+06	0.5	1833.9	25.4	Fill_24	124	D&S, p'=1600-2500, PI=25-30	D&S, p'=1600-2500, PI=25-30	1.97	Fill_24	789
36.97	38.94	Fill	Soil	2.40E+06	0.5	1914.8	25.4	Fill_25	124	D&S, p'=1600-2500, PI=25-30	D&S, p'=1600-2500, PI=25-30	1.97	Fill_25	789
38.94	40	Fill	Soil	2.40E+06	0.5	1977.0	25.4	Fill_26	124	D&S, p'=1600-2500, PI=25-30	D&S, p'=1600-2500, PI=25-30	1.06	Fill_26	789
40	42.16	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2043.2	19.1	Alluvial Clay and Silt_27	124	D&S, p'=1600-2500, PI=15-20	D&S, p'=1600-2500, PI=15-20	2.16	Alluvial Clay and Silt_27	867
42.16	44.32	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2131.9	19.1	Alluvial Clay and Silt_28	124	D&S, p'=1600-2500, PI=15-20	D&S, p'=1600-2500, PI=15-20	2.16	Alluvial Clay and Silt_28	867
44.32	46.48	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2220.6	19.1	Alluvial Clay and Silt_29	124	D&S, p'=1600-2500, PI=15-20	D&S, p'=1600-2500, PI=15-20	2.16	Alluvial Clay and Silt_29	867
46.48	48.64	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2309.3	19.1	Alluvial Clay and Silt_30	124	D&S, p'=1600-2500, PI=15-20	D&S, p'=1600-2500, PI=15-20	2.16	Alluvial Clay and Silt_30	867
48.64	50.8	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2398.0	19.1	Alluvial Clay and Silt_31	124	D&S, p'=1600-2500, PI=15-20	D&S, p'=1600-2500, PI=15-20	2.16	Alluvial Clay and Silt_31	867
50.8	52.96	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2486.7	19.1	Alluvial Clay and Silt_32	124	D&S, p'=1600-2500, PI=15-20	D&S, p'=1600-2500, PI=15-20	2.16	Alluvial Clay and Silt_32	867
52.96	54	Alluvial Clay and Silt	Soil	2.89E+06	0.5	2552.4	19.1	Alluvial Clay and Silt_33	124	D&S, p'=2500-5000, PI=15-20	D&S, p'=2500-5000, PI=15-20	1.04	Alluvial Clay and Silt_33	867
54	56.61	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	2623.8	0	Alluvial Sand and Gravel_34	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_34	1044
56.61	59.22	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	2724.1	0	Alluvial Sand and Gravel_35	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_35	1044
59.22	61.83	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	2824.3	0	Alluvial Sand and Gravel_36	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_36	1044
61.83	64.44	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	2924.5	0	Alluvial Sand and Gravel_37	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_37	1044
64.44	67.05	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3024.7	0	Alluvial Sand and Gravel_38	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_38	1044
67.05	69.66	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3125.0	0	Alluvial Sand and Gravel_39	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_39	1044
69.66	72.27	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3225.2	0	Alluvial Sand and Gravel_40	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_40	1044
72.27	74.88	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3325.4	0	Alluvial Sand and Gravel_41	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_41	1044
74.88	77.49	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3425.6	0	Alluvial Sand and Gravel_42	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_42	1044
77.49	80.1	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3525.9	0	Alluvial Sand and Gravel_43	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_43	1044
80.1	82.71	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3626.1	0	Alluvial Sand and Gravel_44	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_44	1044
82.71	85.32	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3726.3	0	Alluvial Sand and Gravel_45	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_45	1044
85.32	87.93	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3826.5	0	Alluvial Sand and Gravel_46	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_46	1044
87.93	90.54	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	3926.8	0	Alluvial Sand and Gravel_47	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_47	1044
90.54	93.15	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	4027.0	0	Alluvial Sand and Gravel_48	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_48	1044
93.15	95.76	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	4127.2	0	Alluvial Sand and Gravel_49	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_49	1044
95.76	98.37	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	4227.4	0	Alluvial Sand and Gravel_50	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	2.61	Alluvial Sand and Gravel_50	1044
98.37	100	Alluvial Sand and Gravel	Soil	4.06E+06	0.5	4308.8	0	Alluvial Sand and Gravel_51	120	D&S, p'=2500-5000, PI=0	D&S, p'=2500-5000, PI=0	1.63	Alluvial Sand and Gravel_51	1044
100	Half-Space	Half Space	Half-Space	1.28E+08	0.5	Half-Space	0	Bedrock	165	NA, Half-Space	Damping = 0.5%	Half-Space	Bedrock	5000

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



APPENDIX B
STRUCTURAL ANALYSES

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

JOF - SEISMIC STABILITY

ANALYSIS PROCEDURE

I Stability

a) Stability analysis was performed for the entire inlet structure considering loading in two dimensions.

b) Origin: the origin about which all moments are taken is the bottom of footing at the 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 attachement of pipes, etc. Miscellaneous steel and grating weight was assumed as described below.

2 - *Steel Skimmer Weight* : the skimmer weight was taken from the seismic stability analysis calculations for the inlet structure at the SHF site.

3 - *Hydrostatic Loading* : the elevation of the water in the pool was assumed to be 384.5 ft, which is approximately the top of stoplog crest. The elevation of groundwater was also taken to be 384.5 ft for the purpose of this calculation.

4 - *Static Earth Pressure* : static earth pressure was computed using the Rankine active coefficient, Ka, for soil on the outlet side, assuming tensile crack closure; and the at-rest coefficent, ko, for soil on the inlet side unless otherwise noted.

5 - Dynamic Earth Pressure: dynamic earth pressure was computed using two methods, the General Wedge Method and the Seed-Whitman Method. Dynamic earth pressures were resolved into horizontal and vertical (downward) components as noted in the calculations.

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 was used to simulate the lateral force from the water above the soil on the pond-side of the structure as it will mobilize during the earthquake event.

8 - Pipe Connection Capacity: the capacity of the connecting pipes was used to provide lateral and overturning support to the structure.

e) Stability Assessment: the structure was assessed for sliding stability and overturning. Sliding was assessed along two potential slip planes since construction details for a mechanical connection between the precast inlet units and the slab on grade is not shown in the plans. Therefore, sliding was analyzed along the potenitial slip plane between the precast inlet units and the slab on grade, as well as the potential slip plane between the slab on grade and the underlying soil. The 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 c above). Overturning was assessed based on the location of the resultant force on the base of the precast inlet units.

II Bearing Pressure

Bearing pressure was computed based on a trapezoidal or triangular pressure distribution depending on the degree of eccentricity of the applied vertical load.

III Elongation of HDPE Pipe

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

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

JOF - SEISMIC STABILITY

REFERENCES

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[7] US Army Corps of Engineers (1989). *Retaining and Flood Walls* . Engineer Manual 1110-2-2502. Washington, DC.

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Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

JOF - SEISMIC STABILITY

STRUCTURE LAYOUT

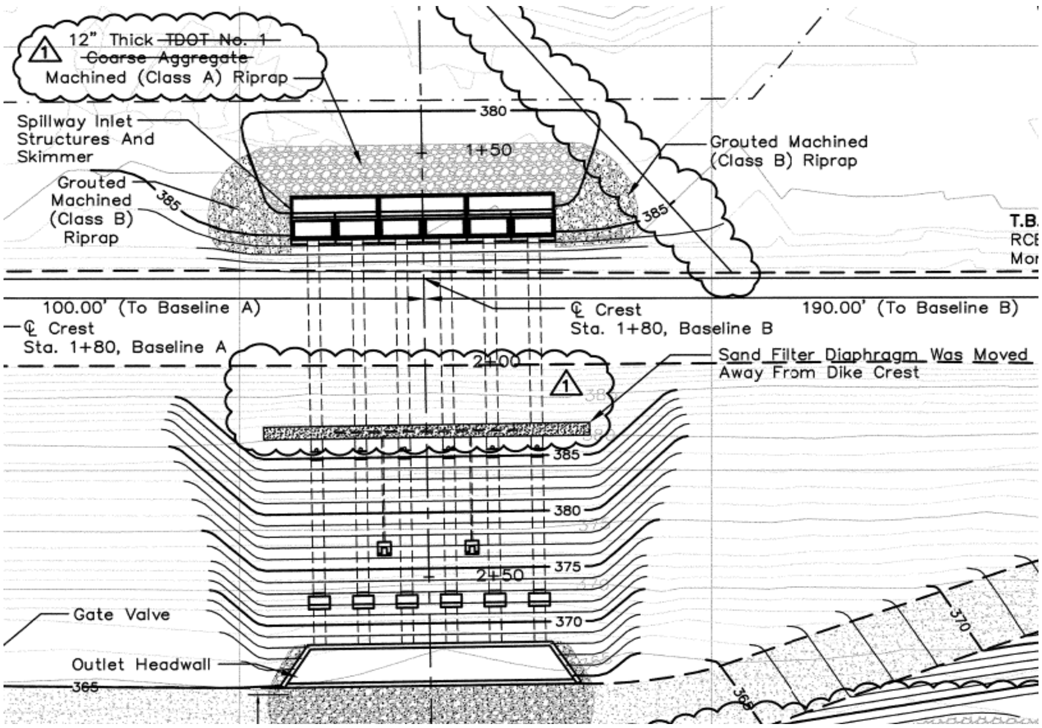

FIGURE 1 - Plan of Spillway (Stantec 2012)

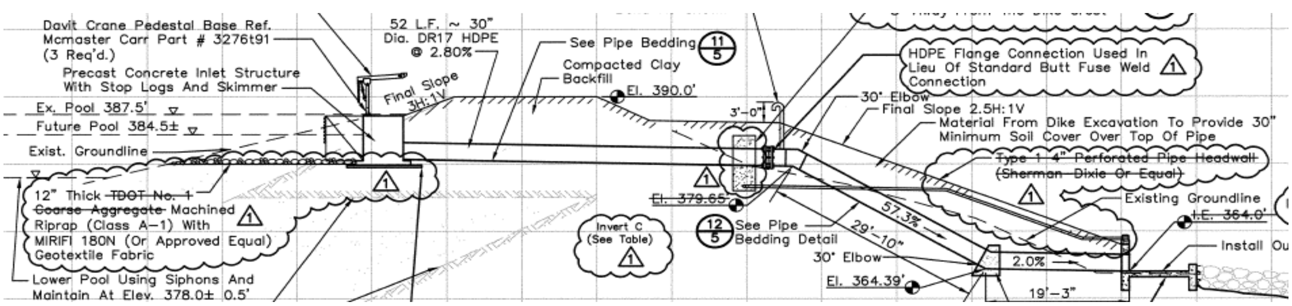

FIGURE 2 - Profile of Spillway (Stantec 2012)

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

JOF - SEISMIC STABILITY

STRUCTURE GEOMETRY AND WEIGHT

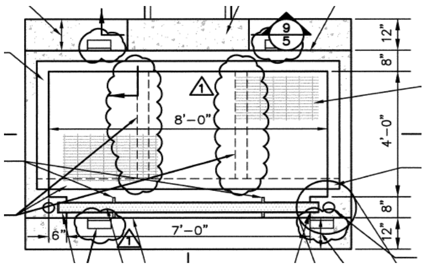

FIGURE 3 - Inlet Plan (Stantec 2012)

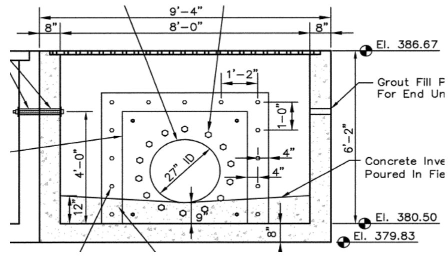

FIGURE 4 - Inlet Elevation (Stantec 2012)

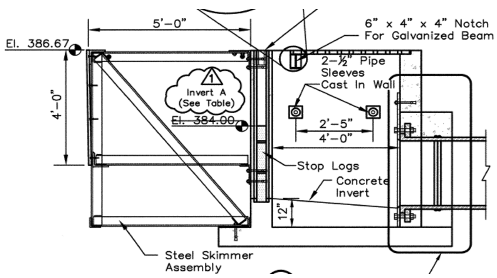

FIGURE 5 - Inlet Cross Section (Stantec 2012)

STEEL GRATING	
assumed weight*:	262.9 lb
number:	6
LESS PIPE VOLUME	
diameter:	2.50 ft
area:	4.91 ft²
backwall thickness:	0.67 ft
number:	6
MISCELLANEOUS STEEL AND STOP LOGS	
assumed weight:	2000.0 lb

*based on 7 lb/ft² assumed and 4'-4" x 8'-8" area.

Concrete Unit Weight (wc):	150 pcf

TOTAL STRUCTURE WEIGHT	
concrete volume:	947.3 cf
concrete weight:	142.1 k
steel weight:	3.58 k
Total Weight (Ws):	**145.67 k**

CENTER OF MASS OF STRUCTURE	
X_c (Relative to "O"):	**4.17 ft**
Y_c (Relative to "O"):	**1.88 ft**

Assume the location of the total structure weight acts coincidentally with the centroid of the concrete portion

FOOTING	
thickness:	0.67 ft
width:	7.33 ft
length:	56.00 ft
INTERIOR WALLS	
thickness:	1.33 ft
width:	4.00 ft
height:	6.17 ft
number:	5
EXTERIOR SIDE WALLS	
thickness:	0.67 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
EXTERIOR FRONT WALL STUBS	
thickness:	0.67 ft
width:	1.17 ft
height:	6.17 ft
number:	2
BACKWALL	
thickness:	0.67 ft
height:	6.17 ft
length:	56.00 ft
FRONT WEIR	
thickness:	0.67 ft
width:	7.00 ft
height:	1.00 ft
number:	6
CONCRETE INVERT	
plan view area:	32.00 ft²
average thickness:	0.94 ft
number:	6

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
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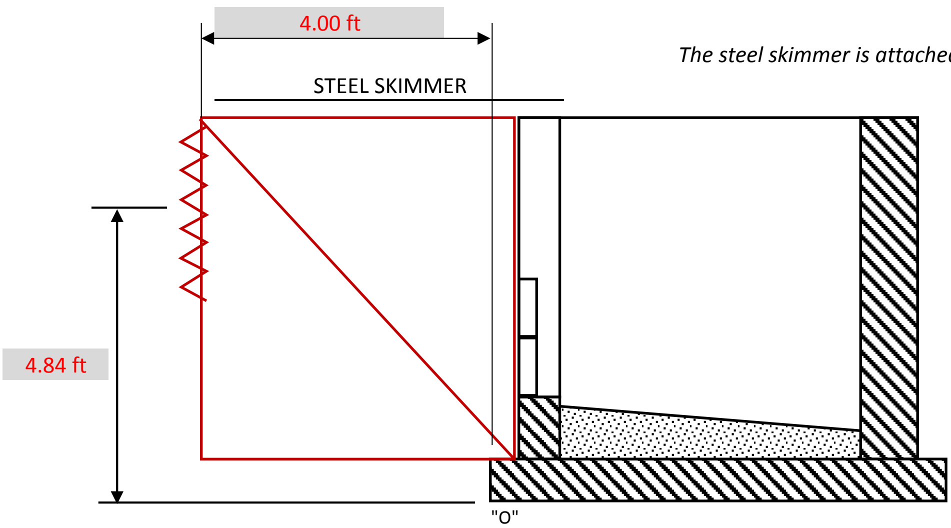
JOF - SEISMIC STABILITY

The steel skimmer is attached as shown below.



Steel Skimmer Weight: 2.50 k *per 2-box unit* → 6 boxes — From SHF

Total Skimmer Weight:	**7.50 k**
X_C (Relative to "O"):	**-4.00 ft**
Y_C (Relative to "O"):	**4.84 ft**

Assume the location of the total skimmer weight acts coincidentally with the centroid of the corrugated steel.

SLAB ON GRADE	
thickness:	0.50 ft
width:	10.00 ft
length:	60.00 ft
Slab Volume	**300.0 cf**
Slab Weight	**45.0 k**

See Sheet 10W502-05 of Plans

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
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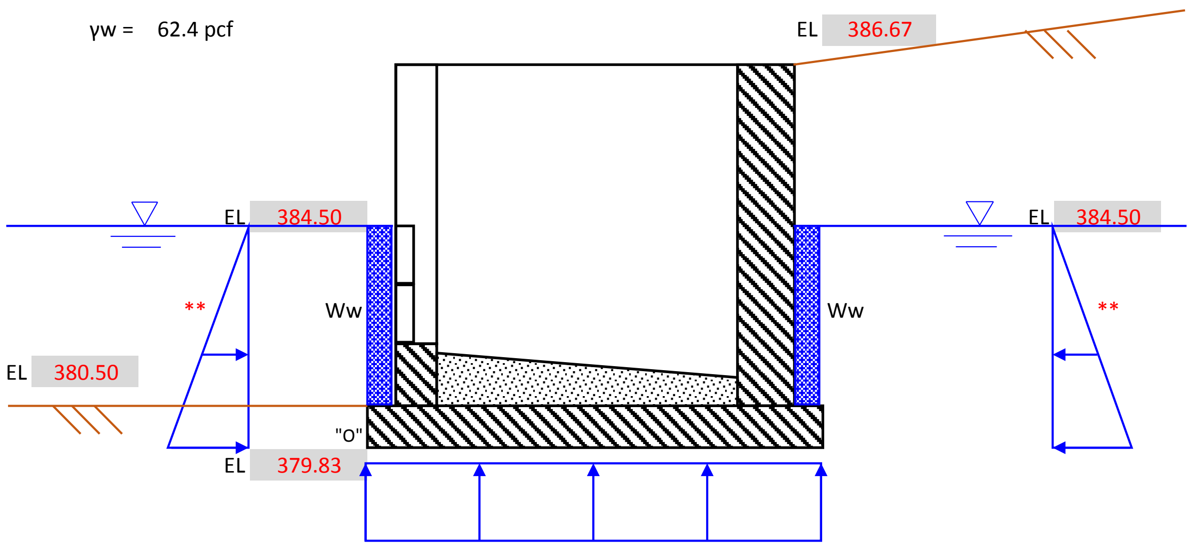
JOF - SEISMIC STABILITY

HYDROSTATIC LOADING



U = γw · (384.5 - 379.83) = **291.4 psf** → **2137.0 plf** @ **3.67 ft** **from "O"** ↑

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

Ww = (384.5 - 380.5) · 1ft ledge · γw =	**249.6 plf**
$X_{C,L}$ (Relative to "O"):	**0.500 ft**
$X_{C,R}$ (Relative to "O"):	**6.833 ft**
Y_C (Relative to "O"):	**2.67 ft**

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
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JOF - SEISMIC STABILITY

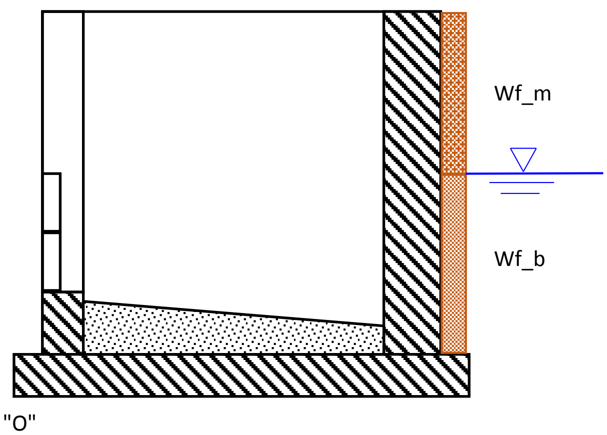
Earth Pressures

Backfill Soil Properties:

ϕ =	20.0°	*internal friction angle*	γm =	125.0 pcf
θ =	0.0°	*wall inclination*	γsat =	125.0 pcf
δ =	23.0°	*wall friction angle*	$\gamma b = \gamma sat - \gamma w$ =	62.6 pcf
β =	18.4°	*slope of backfill*	Cohesion (c) =	640.0 psf

Weight of Soil on Right footing

Wf_m = γm · 1ft · 2.17ft =	**271.3 plf**
Wf_b = γb · 1ft · 4ft=	**250.6 plf**
X_C (Relative to "O"):	**6.83 ft**
$Y_{C,m}$ (Relative to "O"):	**5.76 ft**
$Y_{C,b}$ (Relative to "O"):	**2.67 ft**



Seismic Coefficient:

Maximum Horizontal Acceleration (kh): 0.362g See Maximum Horizontal Acceleration calc. kv = neglect

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
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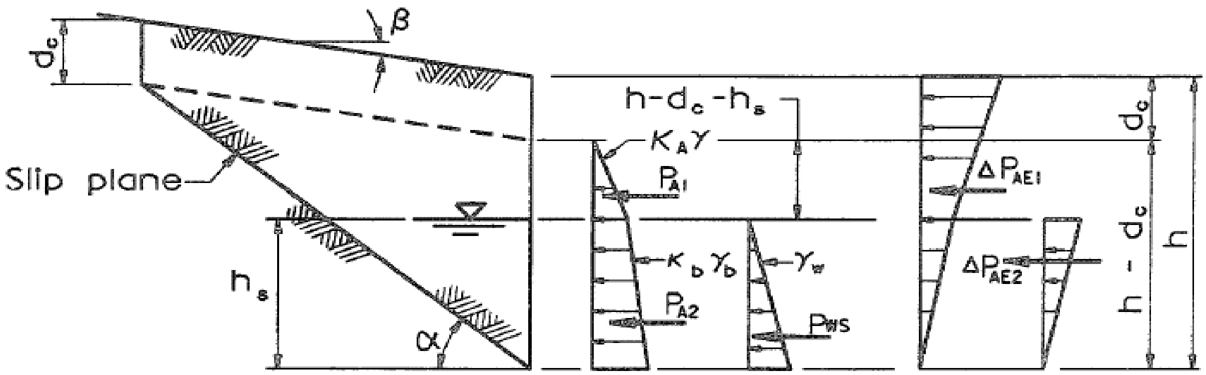
JOF - SEISMIC STABILITY

Method 1: Following EM 1110-2-2502, Chapter 3, General Wedge Earthquake Analysis:



[7] Figure 3-38a

Determination of failure plane angle (α) and depth of vertical crack (dc):

alpha_guess = 30.00°

$$c_1 = \frac{2\tan\phi\,(\tan\phi - k_h) + \dfrac{4c(\tan\phi + \tan\beta)}{\gamma_m(h+d_c)}}{A} = \quad 0.573 \qquad \text{[7] Eq 3-86}$$

$$c_2 = \frac{\tan\phi\,(1-\tan\phi\tan\beta) - (\tan\beta + k_h) + \dfrac{2c(1-\tan\phi\tan\beta)}{\gamma_m(h+d_c)}}{A} = \quad -0.222 \qquad \text{[7] Eq 3-87}$$

$$A = (1 + k_h\tan\phi)\tan\phi + \frac{2c(1-\tan\phi\tan\beta)}{\gamma_m(h+d_c)} = \quad 0.644 \qquad \text{[7] Eq 3-88}$$

$$d_c = \frac{c/\gamma_m}{\cos\alpha\,(\sin\alpha - \tan\phi\cos\alpha)} = \quad 31.99 \text{ ft} \qquad \text{[7] Eq 3-89}$$

$$\alpha = \tan^{-1}\left(\frac{c_1 + \sqrt{c_1^2 + 4c_2}}{2}\right) =$$

Does not converge

Check Guess [7] Eq 3-85

JOF - SEISMIC STABILITY

Static Earth Pressure Coefficients:

$$K_A = \left(\frac{1-\tan\phi\cot\alpha}{1+\tan\phi\tan\alpha}\right)\left(\frac{\tan\alpha}{\tan\alpha-\tan\beta}\right) =$$ 0.721 [7] Eq 3-83

$$K_b = \left(\frac{1-\tan\phi\cot\alpha}{1+\tan\phi\tan\alpha}\right)\left[1+\left(\frac{\tan\alpha}{\tan\alpha-\tan\beta}-1\right)\frac{\gamma_m}{\gamma_b}\right] =$$ 1.134 [7] Eq 3-84

Static Earth Pressures:

$P_{A1} = \frac{1}{2}K_A\gamma_m(h-d_c-h_s)^2 =$	0.0 plf	@	0.00 ft	from "O"	←	
$P_{A2.1} = K_A\gamma_m(h-d_c-h_s)h_s =$	0.0 plf	@	0.00 ft	from "O"	←	[7] Eq 3-79
$P_{A2.2} = \frac{1}{2}K_b\gamma_b h_s^2 =$	0.0 plf	@	0.00 ft	from "O"	←	
Total Static Lateral Earth Force:	**0 plf**	**@**	**0.000 ft**	**from "O"**	←	

Note: *1. The USACE equations assume dc does not extend beyond the depth of ground water table. The excel formulas were appropriately modified to capture the case of a deep tensile crack.*
2. The total inlet side static earth pressure is calculated below as part of Method 2.

Dynamic Earth Pressures:

$\Delta P_{AE1} = \frac{k_h\gamma_m(h-d_c)^2}{2(\tan\alpha-\tan\beta)} =$	0 psf	@	0.00 ft	from "O"	←	[7] Eq 3-81
$\Delta P_{AE2} = \frac{k_h(\gamma_{sat}-\gamma_m)h_s^2}{2\tan\alpha} =$	0 psf	@	0.00 ft	from "O"	←	
Total Dynamic Lateral Earth Force:	**0 plf**	**@**	**0.000 ft**	**from "O"**	←	

Note: *1. The USACE equations assume dc does not extend beyond the depth of ground water table. The excel formulas were appropriately modified to capture the case of a deep tensile crack.*
2. The exponents included in the USACE equations are printed erroneously. The equations were modified as shown above to correctly calculate the weight of the soil wedge, which is the intent of the General Wedge method equations according to Article 3-26 c of Reference [7].

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
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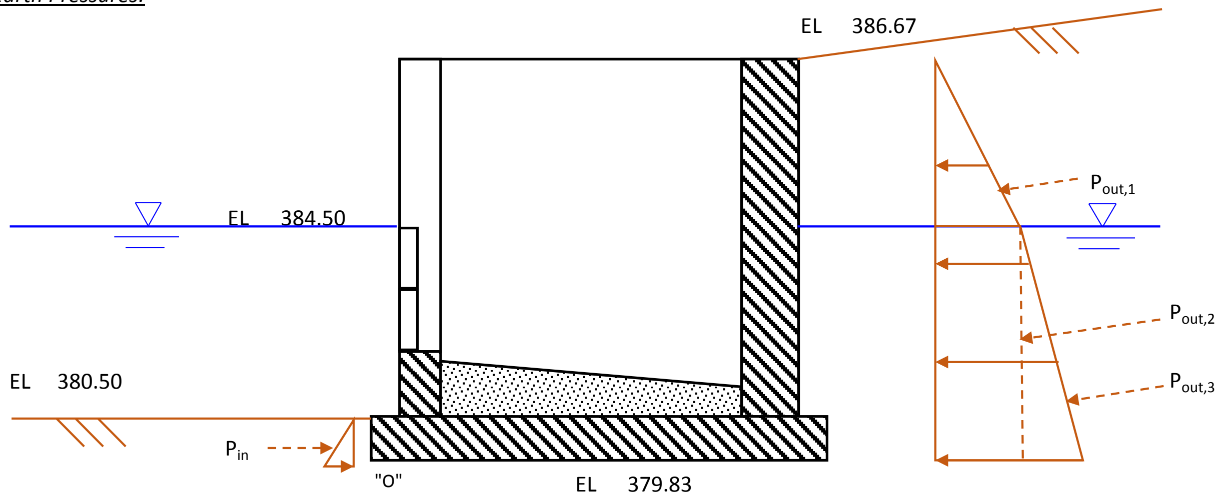
JOF - SEISMIC STABILITY

Method 2: Following Seed-Whitman (1970) Publication:

Static Earth Pressures:



$K_{0,in} = (1 - \sin\phi) =$ 0.658 [4] Eq 11.5

$K_{a,out} = \cos\beta \dfrac{\cos\beta - \sqrt{(\cos\beta)^2 - (\cos\phi)^2}}{\cos\beta + \sqrt{(\cos\beta)^2 - (\cos\phi)^2}} =$ 0.718 [4] Eq 11.24

P_{in} =	10.08 plf	@	0.22 ft	from "O"	→
$P_{out,1}$ =	211.2 plf	@	5.39 ft	from "O"	↙
$P_{out,2}$ =	909.0 plf	@	2.34 ft	from "O"	↙
$P_{out,3}$ =	489.8 plf	@	1.56 ft	from "O"	↙
Total Inlet Side Lateral Soil Load:	**10.08 plf**	**@**	**0.223 ft**	**from "O"**	→
Total Outlet Side Lateral Soil Load:	**1527.7 plf**	**@**	**2.50 ft**	**from "O"**	←
	508.2 plf	**@**	**7.333 ft**	**from "O"**	↓

Assume tensile crack closure on the outlet side and the force acts at an angle β from the horizontal

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
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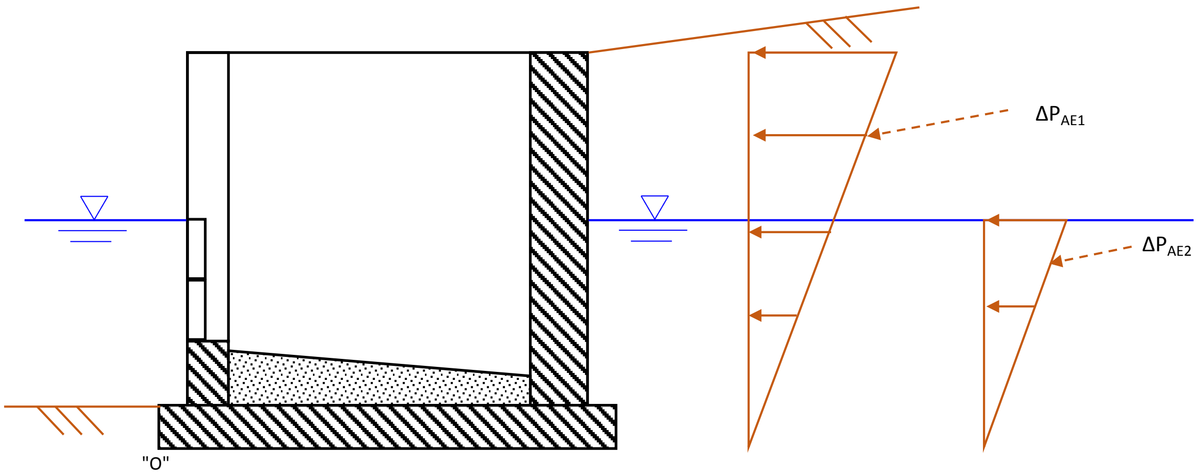
JOF - SEISMIC STABILITY

Dynamic Earth Pressures:



$\Delta K_{AE} = \frac{3}{4} k_h =$ 0.2715 [5]

$\Delta P_{AE1} = 0.5 \Delta K_{AE} \gamma_m h^2 =$	794 plf	@	4.56 ft	from "O"	↙
$\Delta P_{AE2} = 0.5 \Delta K_{AE} (\gamma_{sat} - \gamma_m) h_s^2 =$	0.0 plf	@	3.11 ft	from "O"	↙
Total Horizontal Dynamic Earth Force:	**731 plf**	**@**	**4.954 ft**	**from "O"**	←
Total Vertical Dynamic Earth Force:	**310 plf**	**@**	**7.333 ft**	**from "O"**	↓

Assume the force acts at an angle δ from the horizontal

Stantec

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

JOF - SEISMIC STABILITY

Method Selection and Earth Loads Summary

SELECT METHOD:

2: SEED-WHITMAN

Horizontal Static Earth Force:	**1528 plf**	**@**	**2.499 ft**	**from "O"**	←
Vertical Static Earth Force:	**508 plf**	**@**	**7.333 ft**	**from "O"**	↓
Horizontal Dynamic Earth Force:	**731 plf**	**@**	**4.954 ft**	**from "O"**	←
Vertical Dynamic Earth Force:	**310 plf**	**@**	**7.333 ft**	**from "O"**	↓

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

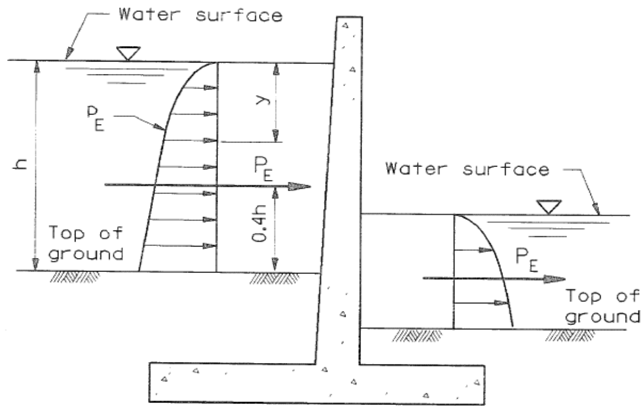
JOF - SEISMIC STABILITY

INERTIAL FORCE

Wt = Ws + Skimmer + (2 · Ww + Wf_m + Wf_b) · base length = 210.34 k

Inertial Force of Structural Wedge = Wt · kh = **76.14 k** **@** **2.425 ft** **from "O"** ← [7] Eq 3-101

HYDRODYNAMIC FORCE DUE TO WATER ABOVE GROUND LEVEL



[7] 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.

$P_E = \left(\frac{2}{3}\right) C_E k_h h^2 =$ **0.197 klf** **@** **2.268 ft** **from "O"** ← [7] Eq 3-102

CONNECTING PIPE CAPACITY

Tensile Strength:	3.60 ksi
Area of Pipe:	166 in²

30" HDPE DR17 Pipe [3]

Area calculated based on specs for linear weight of pipe and material density [3]

For 6 Pipe Connections:

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

→ 1793 k

Slip Plane Parameters

Slip Plane 1 - Interface between Precast Inlet Units and Slab on Grade:

Concrete Friction Coefficient: **0.4**

Taken conservatively as 0.4 based on values in Reference [1] Section 11.7.4.3

Slipe Plane 2 - Interface between Slab on Grade and Soil:

Friction Angle (δ): **23.0°**

Cohesion (c): **0 psf**

Project: JOF Seismic Demonstration	Calc. by: JEH
Job No.: 175568235	Date: 8/3/2018
Location: Ash Disposal Area No. 2	Check by: PRS
Contents: Inlet Stability Analysis	Date: 8/7/2018

JOF - SEISMIC STABILITY

SUMMATION OF FORCES AND MOMENTS

Select Load Condition: Normal Pool w/ MDE

Select Slip Plane: Inlet-Slab Interface

Description	Force	Direction	Arm	Moment
Vertical				
Structure Weight	145.67 k	↓	4.17 ft	-607 k-ft
Weight of Skimmer	7.50 k	↓	-4.00 ft	30 k-ft
Water Weight (Inlet Side)	13.98 k	↓	0.50 ft	-7 k-ft
Water Weight (Outlet Side)	13.98 k	↓	6.83 ft	-96 k-ft
Uplift	119.67 k	↑	3.67 ft	439 k-ft
Weight of Moist Soil Above Footing	15.19 k	↓	6.83 ft	-104 k-ft
Weight of Bouyant Soil Above Footing	14.03 k	↓	6.83 ft	-96 k-ft
Static Earth Pressure (Outlet Side)	28.46 k	↓	7.33 ft	-209 k-ft
Dynamic Earth Pressure (Outlet Side)	17.37 k	↓	7.33 ft	-127 k-ft
TOTAL VERTICAL (P)	136.50 k	↓	--	--
Horizontal				
Static Earth Pressure (Outlet Side)	85.55 k	←	2.50 ft	214 k-ft
Dynamic Earth Pressure (Outlet Side)	40.92 k	←	4.95 ft	203 k-ft
Inertial Force of Structural Wedge	76.14 k	←	2.43 ft	185 k-ft
Hydrodynamic Force Due to Water (Inlet Side)	11.046 k	←	2.27 ft	25.1 k-ft
TOTAL HORIZONTAL (V)	213.67 k	←	--	--
Resistance				
Static Earth Pressure (Inlet Side)	0.56 k	→	0.223 ft	-0.1 k-ft
Base Resistance (P · concrete friction coefficient)	54.60 k	→		
Resistance from Pipe (Req)	158.50 k	→	2.54	-402.6 k-ft
Resistance from Pipe (Max)	1792.80 k	→		
TOTAL RESISTANCE (F)	1847.97 k	→	--	--
TOTAL MOMENT (M)	--	--	--	-553 k-ft

Required Sliding Factor of Safety: 1.1

Sliding Factor of Safety (F/V): 8.649 OK

Eccentricity (e = M/P - b/2): 0.386 ft ≤ b/2 = 3.667 ft OK *measured from center of base*

Project:	JOF Seismic Demonstration	Calc. by:	JEH
Job No.:	175568235	Date:	8/3/2018
Location:	Ash Disposal Area No. 2	Check by:	PRS
Contents:	Inlet Stability Analysis	Date:	8/7/2018

JOF - SEISMIC STABILITY

BEARING PRESSURES

Max Allowable Soil Pressure:	6.000 ksf	50% increase per [7] Section 3.10
Area of Base:	410.7 ft²	

q_{max}	**0.437 ksf**	**OK**	
q_{min}	**0.227 ksf**	**OK**	*positive denotes compression*

ELONGATION OF HDPE PIPE

Net tension on 1 pipe:	26.42 k	
Tensile stress on 1 pipe (σ):	0.159 ksi	
Elastic Modulus of HDPE (E_H):	120 ksi	[3]

Strain in HDPE ($\varepsilon = \sigma/E_H$):	0.0013261
Length of Horizontal Pipe to apply strain to (L):	44.0 ft

Total Elongation (ε · L · 12):	0.700 in

DISTRIBUTED PRESSURES ON WALL

Project: Johnsonville Fossil Plant - SIZ

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

Section Location: Back Wall

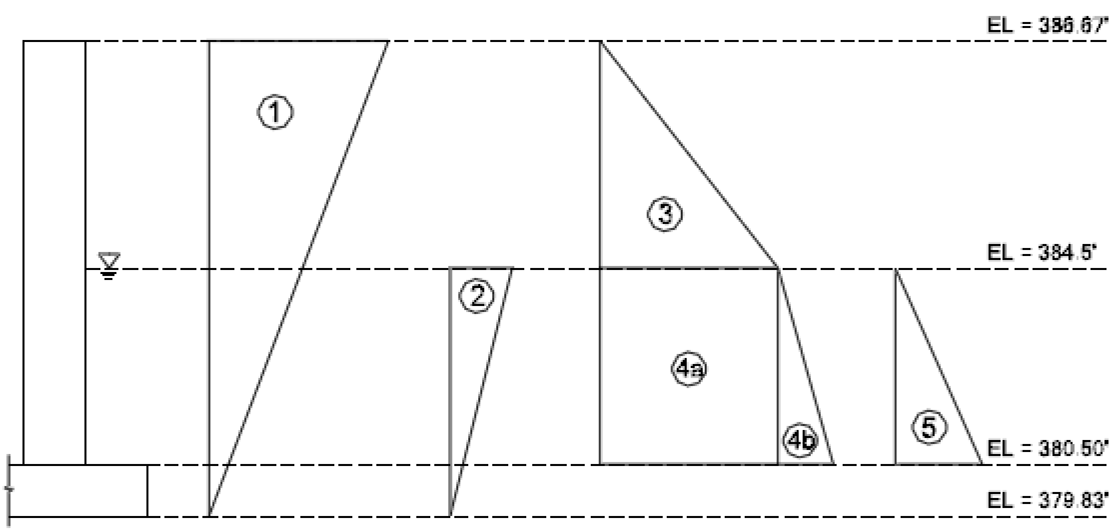
Input By : PRS

Input Checker: JEH

Date : 08/13/2018

Date: 8/29/2018

Load Diagram (acting towards wall)



Refer to file named "JOF Seismc Stability" for resultant forces

1. Dynamic Soil Pressure for Moist Soil

Resultant Force: $P_1 := 731\text{plf}$

Distributed Height: $h_1 := 386.67\text{ft} - 379.83\text{ft} = 6.84\cdot\text{ft}$

Distributed Pressure: $w_1 := \frac{2\cdot P_1}{h_1^{\,2}} = 31.25\cdot\frac{\text{psf}}{\text{ft}}$

SAP2000 Input:

Joint Pattern: Seismic_Soil

Load Pattern: Seismic

2. Additional Dynamic Soil Pressure for Saturated Soil

Resultant Force: $P_2 := 0\text{plf}$

Distributed Height: $h_2 := 384.5\text{ft} - 379.83\text{ft} = 4.67\cdot\text{ft}$

Distributed Pressure: $w_2 := \frac{2\cdot P_2}{h_2^{\,2}} = 0\cdot\frac{\text{psf}}{\text{ft}}$

SAP2000 Input:

Joint Pattern: Seismic_Soil

Load Pattern: Seismic

3. Lateral Earth Pressure: Moist Soil

Moist Soil Unit Weight: $\gamma_m := 125pcf$

Active Earth Pressure Coefficient: $K_a := 0.718$

Distributed Height: $h_4 := 386.67ft - 384.5ft = 2.17 \cdot ft$

Distributed Pressure: $w_4 := \gamma_m \cdot K_a = 89.75 \cdot \frac{psf}{ft}$

SAP2000 Input:

Joint Pattern: Soil_Moist

Load Pattern: Soil

4a. Lateral Earth Pressure: Saturated Soil - Pressure due to Surcharge

Distributed Height: $h_5 := 384.5ft - 380.5ft = 4 \cdot ft$

Distributed Pressure: $w_{5a} := w_4 \cdot h_4 = 194.76 \cdot psf$

SAP2000 Input:

Applied as uniform pressure

Load Pattern: Soil

4b. Lateral Earth Pressure: Saturated Soil - Triangular Pressure Distribution

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

Unit Weight of Water: $\gamma_w := 62.4pcf$

Distributed Pressure: $w_{5b} := (\gamma_{sat} - \gamma_w) \cdot K_a = 44.95 \cdot \frac{psf}{ft}$

SAP2000 Input:

Joint Pattern: Soil_Effective

Load Pattern: Soil

5. Hydrostatic Pressure

Distributed Height: $h_6 := 384.5ft - 380.5ft = 4 \cdot ft$

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

SAP2000 Input:

Joint Pattern: Hydro

Load Pattern: Hydro

6. Pipe Resultant Force:

Required Tension on Pipe: $P_{pipe} := \frac{158.5kip}{6} = 26.42 \cdot kip$

Plate Bearing Width: $B_{plate} := 8in$

Plate Bearing Length: $L_{plate} := 3 \cdot 48in + 2 \cdot 8in = 13.33\ ft$

Plate Bearing Area: $A_{plate} := B_{plate} \cdot L_{plate} = 8.89\ ft^2$

Bearing Pressure: $w_{plate} := \frac{P_{pipe}}{A_{plate}} = 2.97 \cdot ksf$

SAP2000 Input:

Load Pattern: HDPE

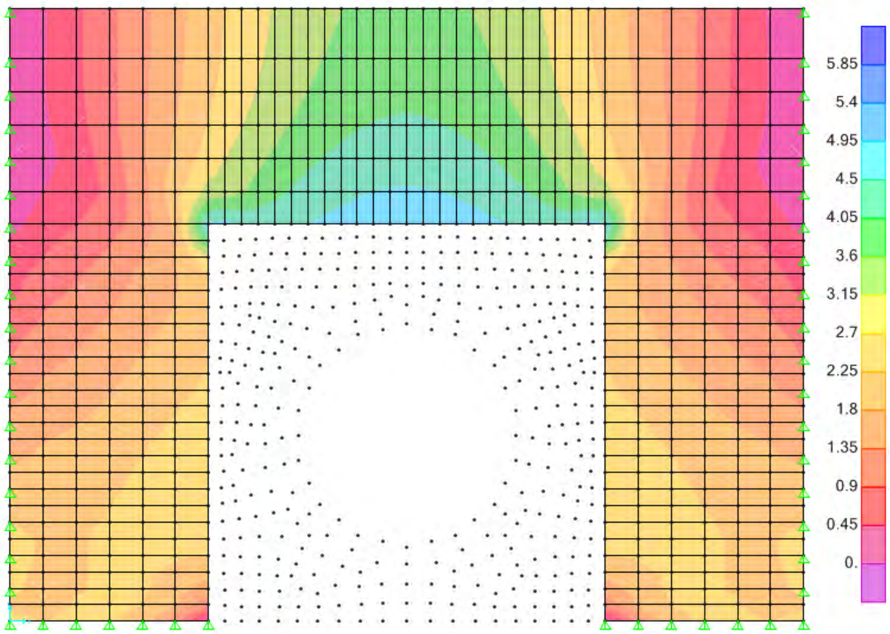
SAP2000 MODEL OUTPUTS

PROJECT: JOHNSONVILLE FOSSIL PLANT - SIZ

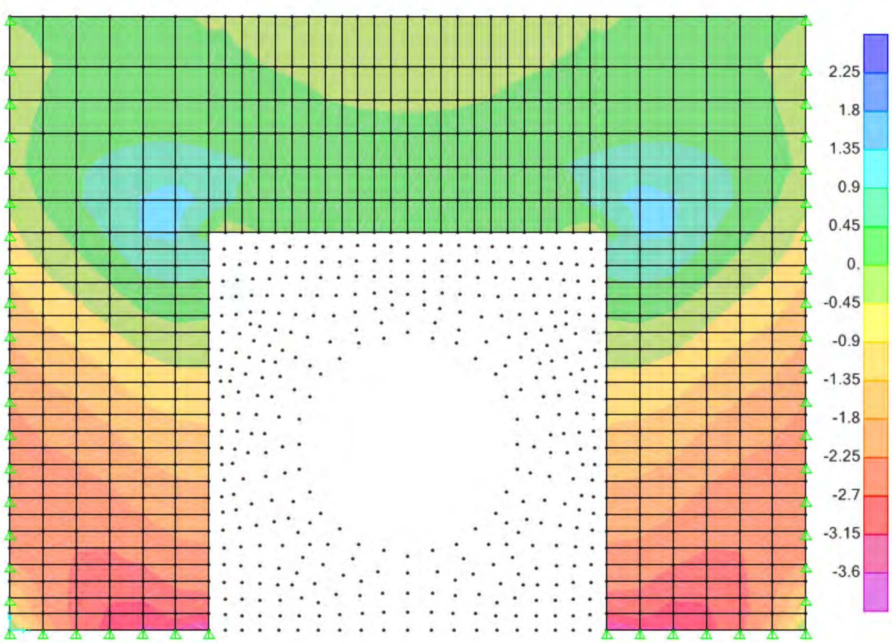
Input By: PRS   Checker: JEH

Date: 08/27/2018   Date: 8/29/2018

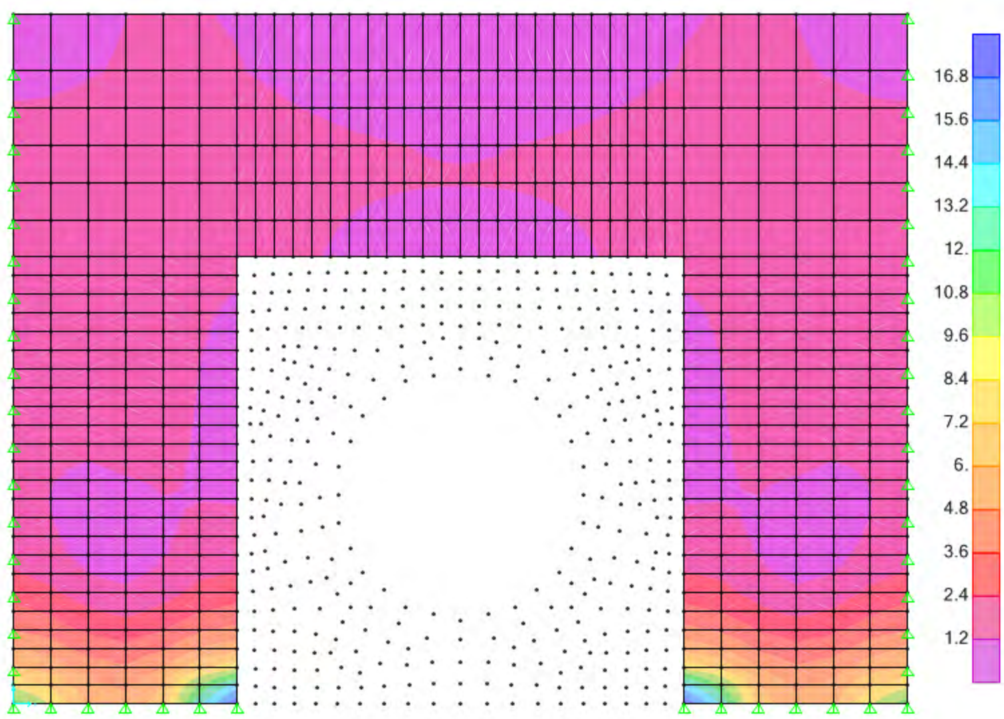
M_{max} = 4.76 ft-kip (Service M_{max} = 4.05 ft-kip)



M_{min} = -3.76 ft-kip (Service M_{min} = -3.25 ft-kip)



V_{max} = 16.75 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**
Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
S1	0.776	-2.946E-16	2.662	-0.00009689	-1.6911	-1.416E-15
LC1	0.931	-3.535E-16	2.885	-0.00009689	-1.8786	-1.699E-15
LC2	0.698	-2.651E-16	1.608	-0.00009689	-1.0956	-1.274E-15

REINFORCEMENT DESIGN FOR WALL SECTIONS

Project: Johnsonville Fossil Plant - SIZ

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

Section Location: Back Wall - Positive Moment

Input By : PRS

Input Checker: JEH

Date : 08/27/2018

Date: 8/29/2018

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: $b_s := 8ft$
- Member Thickness: $h := 8in$
- Width of Analysis Section: $b_w := 1ft$

Concrete and Reinforcement Properties

- Concrete Strength: $f'_c := 5000psi$
- Steel Yield Strength: $f_y := 60 \cdot ksi$
- 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

Flexural Reinforcement Details

- Flexural Bar Diameter: $d_b := 0.625in$ #5 bar
- Flexural Bar Area: $A_b = 0.31 \cdot in^2$
- Reinforcement Spacing: $s := 8in$
- Concrete Cover : $cov := 0.5(h - d_b) = 3.69 \cdot in$ (Conservatively Assume reinforcement centered in wall)

Shrinkage & Temparature Reinforcement Details

- S&T Bar Area: $A_{st} := 0.31in^2$ #5 bar
- S&T Reinforcement Spacing: $s_{st} := 8in$

2.0 DETERMINE CRITICAL MOMENT AND SHEAR (use 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: $M_{ser} := 4.05 ft \cdot kip$
- Controlling Factored Shear: $V_f := 2.89 kip$ Tributary to Wall Width
- Controlling Factored Moment: $M_u := 4.76 ft \cdot kip$
- 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)

- Dist. from Comp. Face to CL of Tension Reinforcement: $d := h - \left(cov + \frac{d_b}{2}\right) = 4 \cdot in$
- Area of Moment Reinforcement: $A_s := \frac{A_b \cdot b_w}{s} = 0.46 \cdot in^2$
- Reinforcement Ratio: $\rho_w := \frac{A_s}{b_w \cdot d} = 9.59 \times 10^{-3}$

Factored Shear Strength (ACI 350 - 11.3.2, 11.5.5):

- Design Factored Shear: $V_u := V_f = 3 \cdot kip$
- Concrete Shear Strength:

  ACI 350 - 11.3.2.1

$$\phi V_c := \phi_v \cdot \left[\left[1.9 \cdot \sqrt{\left(\frac{f'_c}{psi}\right)} + 2500 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u \cdot \left(\frac{b_s}{b_w}\right)}\right] \cdot (psi) \cdot d \cdot b_s\right] = 38.87 \cdot kip$$

- Shear Capacity Demand Ratio: $PR_V := \frac{\phi V_c}{V_u} = 13.45$

$\text{if}\left(PR_V \geq 2.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Factored Moment Strength: ACI 350 - 10.2

- Stress Block Factor:

  ACI 350 - 10.2.7.3

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ \left[0.85 - 0.05\left(\frac{f'_c - 4000psi}{1000psi}\right)\right] & \text{if } 4000psi < f'_c < 8000psi \\ 0.65 & \text{if } f'_c \ge 8000psi \end{cases} = 0.8$$

- Dist. from Comp. Face to N.A.:

$$c := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b_w} = 0.68 \cdot in$$

ACI 350 - 10.2.7.1, 10.2.7.2

- Strain in Steel for Concrete Crushing:

$$\varepsilon_s := 0.003 \cdot \left(\frac{d - c}{c}\right) = 0.015$$

> 0.005, Tension controlled, **OK**

ACI 350 - 10.3.4

- Available Moment Strength:

$$\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d - \frac{\beta_1 \cdot c}{2}\right) = 7.72 \cdot kip \cdot ft$$

- Controlling Factored Moment:

$$M_u = 4.76 \cdot kip \cdot ft$$

- Moment Capacity Demand Ratio:

$$PR_M := \frac{\phi M_n}{M_u} = 1.62$$

$$\text{if}\left(PR_M \ge 1.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

REINFORCEMENT DESIGN FOR WALL SECTIONS

Project: Johnsonville Fossil Plant - SIZ

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

Section Location: Back Wall - Negative Moment

Input By : PRS

Input Checker: JEH

Date : 08/27/2018

Date: 08/29/2018

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: $b_s := 8ft$
- Member Thickness: $h := 8in$
- Width of Analysis Section: $b_w := 1ft$

Concrete and Reinforcement Properties

- Concrete Strength: $f'_c := 5000psi$
- Steel Yield Strength: $f_y := 60 \cdot ksi$
- 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

Flexural Reinforcement Details

- Flexural Bar Diameter: $d_b := 0.625in$ #5 bar
- Flexural Bar Area: $A_b = 0.31 \cdot in^2$
- Reinforcement Spacing: $s := 8in$
- Concrete Cover : $cov := h - 2in - d_b = 5.37 \cdot in$ (Assume 2in cover over positive reinf.)

Shrinkage & Temparature Reinforcement Details

- S&T Bar Area: $A_{st} := 0.31in^2$ #5 bar
- S&T Reinforcement Spacing: $s_{st} := 8in$

2.0 DETERMINE CRITICAL MOMENT AND SHEAR (use 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: $M_{ser} := 3.25 ft \cdot kip$
- Controlling Factored Shear: $V_f := 2.89 kip$ Tributary to Wall Width
- Controlling Factored Moment: $M_u := 3.76 ft \cdot kip$
- 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)

- Dist. from Comp. Face to CL of Tension Reinforcement: $d := h - \left(cov + \frac{d_b}{2}\right) = 2.31 \cdot in$
- Area of Moment Reinforcement: $A_s := \frac{A_b \cdot b_w}{s} = 0.46 \cdot in^2$
- Reinforcement Ratio: $\rho_w := \frac{A_s}{b_w \cdot d} = 0.02$

Factored Shear Strength (ACI 350 - 11.3.2, 11.5.5):

- Design Factored Shear: $V_u := V_f = 3 \cdot kip$
- Concrete Shear Strength: ACI 350 - 11.3.2.1

$$\phi V_c := \phi_v \cdot \left[\left[1.9 \cdot \sqrt{\left(\frac{f'_c}{psi}\right)} + 2500 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u \cdot \left(\frac{b_s}{b_w}\right)}\right] \cdot (psi) \cdot d \cdot b_s\right] = 22.5 \cdot kip$$

- Shear Capacity Demand Ratio: $PR_V := \frac{\phi V_c}{V_u} = 7.78$

  $if\left(PR_V \geq 2.0, "OK", "NG"\right) = "OK"$

Factored Moment Strength: ACI 350 - 10.2

- Stress Block Factor:
  ACI 350 - 10.2.7.3

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000psi \\ \left[0.85 - 0.05\left(\frac{f'_c - 4000psi}{1000psi}\right)\right] & \text{if } 4000psi < f'_c < 8000psi \\ 0.65 & \text{if } f'_c \ge 8000psi \end{cases} = 0.8$$

- Dist. from Comp. Face to N.A.:

$$c := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b_w} = 0.68 \cdot in$$

ACI 350 - 10.2.7.1, 10.2.7.2

- Strain in Steel for Concrete Crushing:

$$\varepsilon_s := 0.003 \cdot \left(\frac{d - c}{c}\right) = 0.007$$

> 0.005, Tension controlled, **OK**
ACI 350 - 10.3.4

- Available Moment Strength:

$$\phi M_n := \phi_m \cdot A_s \cdot f_y \cdot \left(d - \frac{\beta_1 \cdot c}{2}\right) = 4.23 \cdot kip \cdot ft$$

- Controlling Factored Moment:

$$M_u = 3.76 \cdot kip \cdot ft$$

- Moment Capacity Demand Ratio:

$$PR_M := \frac{\phi M_n}{M_u} = 1.12$$

$\text{if}\left(PR_M \ge 1.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Stantec

Project: TVA Seismic Demonstration (JOF)
Job No.: 175568235

Compiled by: JEH 6/15/2018
Checked by: PRS 8/13/2018

Maximum Horizontal Acceleration

Longitude: -87.9983333 deg
Latitude: 36.02888889 deg

Return Period (Years)	PHGA (g)
2500	0.3024

Source: USGS, 2014 Seismic Hazard Data

Site Class: D *(Default Assumption)*

F_{PGA} = 1.197632 *(ASCE 7-10, Table 11.8-1)*

K_h = PHGA x F_{PGA} = 0.362g