

PERIODIC SAFETY FACTOR ASSESSMENT

COAL COMBUSTION RESIDUALS IMPOUNDMENT EMBANKMENTS

Charles R. Lowman Power Plant

Three Notch Reference Number: R021225012

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Prepared for:
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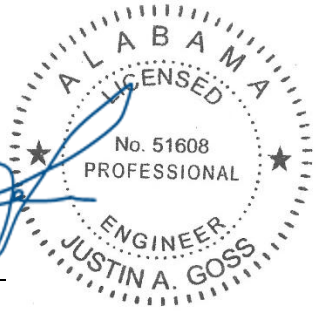
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CERTIFICATION PAGE

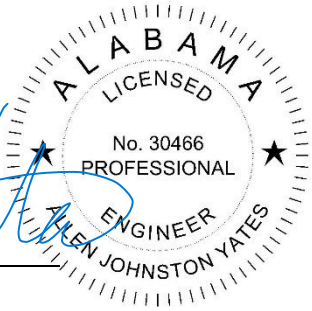
I hereby certify that, in my professional judgement, the components of this document and associated work satisfy the applicable requirements set forth in ADEM Admin. Code r. 335-13-15-.04(4)(e) and 40 CFR 257.73(e) and are consistent with generally accepted professional engineering principles and practices. The information submitted herein to the best of my knowledge and belief is true, accurate and complete.

This document has been prepared based on available site assessment data for the Charles R. Lowman Power Plant in Leroy, Washington County, Alabama. The recommended action should not be construed to apply to any other site.



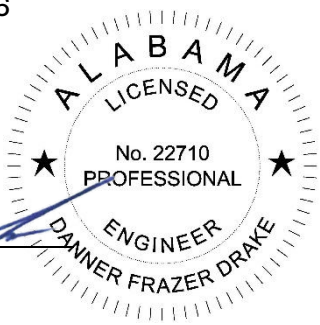
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EXECUTIVE SUMMARY

State [ADEM Admin. Code 335-13-15-.04(4)(e)] and federal (40 CFR 257.73) regulations require that the stability of Coal Combustion Residuals (CCR) impoundment embankments be periodically evaluated. The purpose of this study is to perform the required evaluation for the CCR impoundment embankments at the Charles R. Lowman Power Plant (Lowman). The safety factor assessment was conducted on the exterior embankments associated with the FGD (Flue Gas Desulfurization) Waste, Unit 2/3 Bottom Ash, and Unit 1 Bottom Ash ponds. Regulations state that the evaluation is to be performed under static long-term maximum storage pool, static maximum surcharge pool, and seismic loading scenarios. Additionally, for embankments constructed of soils that have susceptibility to liquefaction, the stability is to be evaluated under liquefaction conditions.

The scope of services performed by Three Notch Group, Inc. (Three Notch) f/k/a CDG Engineers & Associates, Inc. and CDG, Inc. consisted of a subsurface exploration, laboratory testing program, and analyses of embankment stability under the various required loading scenarios. Three Notch identified critical cross-sections along each impoundment embankment by evaluating subsurface information and geometric configurations to define locations where failures are most likely to occur.

The stability analyses indicate that the embankments exhibit factors of safety that equal or exceed the required minimum values under the maximum storage pool, maximum surcharge pool, and seismic loading scenarios. Additionally, the embankment soils are demonstrated not to be susceptible to liquefaction, and further analyses under the liquefaction loading scenario is not required. Therefore, the embankments comply with the periodic safety factor assessment requirements.

1.0 SCOPE OF SERVICES

The geotechnical evaluation included in-situ sampling and testing to determine the subsurface conditions within and underlying the embankments surrounding the FGD (Flue Gas Desulfurization) Waste Pond, the Unit 2/3 Bottom Ash Pond, and the Unit 1 Bottom Ash Pond at the Lowman Power Plant in Leroy, Alabama. Shared interior embankment are located between the FGD Waste and Unit 2/3 Bottom Ash Ponds and the Unit 2/3 Bottom Ash and Unit 1 Bottom Ash Ponds. The interior embankments serve primarily to separate the impoundments. Their stability provides no contribution to the containment of CCR from the surrounding environment. Therefore, the current scope of services consists of evaluating the exterior embankments. Evaluation of the interior embankment has been excluded.

Laboratory tests were performed to define strength parameters of the embankment and foundation materials present at the site. Stability analyses were modeled at various locations throughout the embankments to identify critical cross-sections. The water pool elevations, surcharge loading, and CCR fill were varied to determine maximum loading configurations. Maximum storage pool, maximum surcharge pool, seismic, and liquefaction loading scenarios were considered. Specifically, our scope of services consisted of the following.

- Three Notch previously performed subsurface explorations of the embankments and underlying foundation soils in 2009, 2011, 2016, and 2020. With the exception of a toe buttress constructed along the exterior of the northwest embankment of the Unit 2/3 Bottom Ash and FGD Waste Ponds in 2015, the berms have not been altered following the time of the original explorations. Therefore, additional subsurface exploration was determined to be unnecessary, and the previously obtained subsurface data was employed to perform the current analysis. The previously performed subsurface explorations included the following:
    - Soil test borings along the crest of the embankments surrounding the exterior of the subject impoundments. Borings contained Standard Penetration Tests at regular intervals. Undisturbed samples were obtained at various depths for laboratory testing. The subsurface exploration included thirty (30) soil test borings extending to depths ranging from approximately 40 to 60 feet below the existing ground surface.
    - Cone Penetration Tests (CPT) were performed at the toe of the existing embankments, through interior embankments, and through the impounded ash materials to determine the in-situ characteristics of the foundation soils. CPT data consists of a continuous record of tip resistance, side friction, and pore water pressure. Additionally, shear wave velocity was collected at selected test locations. The investigation included nine (9) CPTs extending to depths ranging from 45 to 70 feet below the ground surface.
    - Piezometers were installed in ten (10) borings to determine delayed groundwater levels.
- Inspection and materials testing during construction of the toe buttress located along the exterior of the northwest embankment of the Unit 2/3 Bottom Ash and FGD Waste Ponds.
- Laboratory tests were performed to determine site-specific soil classification and strength characteristics. Tests included the following: natural moisture content, grain size analysis, Atterberg Limits, and consolidated, undrained (CU) triaxial shear.
- Geotechnical analyses of embankment stability under static loading conditions for both normal pool (maximum storage pool) and flood (maximum surcharge pool) scenarios.
- Seismic and liquefaction analyses consisting of the following four step process:

  - Determination of the shear wave velocity within the subsurface stratigraphy at the site;
  - Establishment of design ground motions for seismic events corresponding to the 2% and 10% probability of exceedance in 50 years. Ground motions were developed based on data available from the United State Geological Survey's latest (2023) national seismic hazard model and subsequently compared to a previously developed 2021 site-specific probabilistic seismic hazard analysis (PSHA).
  - Geotechnical analysis of embankment stability under seismic and liquefaction loading conditions. Rigorous seismic deformation analyses were performed based on the Bray and Travasarou method. The embankment soils were evaluated for liquefaction potential using recommendations by Idriss and Boulanger.

- Evaluation of the information gathered during the subsurface exploration and laboratory testing programs and preparation of this report. The report addresses the following items:
  - Description of the existing embankment configurations and critical cross-sections;
  - Subsurface stratigraphy and material properties used in analyses;
  - Summary of loading conditions and required factors of safety;
  - Slope stability analysis of the exterior embankments forming the FGD Waste, Unit 2/3 Bottom Ash, and Unit 1 Bottom Ash Ponds under the specified loading conditions.

2.0 EMBANKMENT GEOMETRIES AND CRITICAL CROSS-SECTIONS

Embankment geometries were defined by survey data obtained by Three Notch in February 2026 and by design plans provided by PowerSouth. Design plans were entitled *Alabama Electric Cooperative, Inc.; First Unit – Jackson Station* (Stanley Engineering Company, dated January 20, 1970) and *Tombigbee Generating Plan Units 2 & 3* (Burns &McDonnell; dated June 8, 1976). Survey data was unavailable for the interior of the impoundments below the CCR and water levels present at the time of the field work. Therefore, geometries of the interior embankment slopes below the water and CCR levels were based on topographic data obtained from design plans.

Based on the noted information, the crest elevations of the exterior embankments range from ±EL 35.5' (Unit 1 Bottom Ash Pond) to ±EL 43.9' (FGD Waste Pond). Ground surface elevations at the exterior toe of the embankments range from ±EL 18' (FGD Waste Pond) to ±EL 37' (Unit 1 Bottom Ash Pond). The embankments exhibit exterior vertical heights ranging from ±5 feet to a maximum of approximately ±25 feet. The exterior faces of the perimeter embankments exhibit slope inclinations of ±1.6(H):1(V) and flatter.

The CCR impoundments are in the process of closure; however, the configuration of the CCR following closure was undetermined at the time of the current analysis. Surveying data obtained in February 2026 indicates that maximum CCR elevations within the vicinity of the exterior embankments around the Unit 2/3 Bottom Ash and FGD Waste Pond were ±EL 48.6' and ±EL 39.0', respectively. The CCR elevation within these impoundments was modeled based on the noted elevations present at the time of report preparation. CCR was not present in the Unit 1 Bottom Ash Pond at the time of analysis. Additional analysis associated with the final geometry will be required in the future.

A detailed review of the grading and topographic plans was performed to determine the critical cross-sections for the existing embankments. Cross-sections identified for evaluation were selected at numerous locations throughout the embankments to ensure the critical locations were identified. Critical locations for analysis were determined by evaluating subsurface information and geometric configurations of the embankments to define locations exhibiting the greatest driving forces coupled with the smallest resisting forces. A surcharge load of 250 psf was applied atop the Unit 2/3 Bottom Ash and FGD Waste exterior embankments in areas subject to access road traffic. Critical slope sections occur at locations exhibiting combinations of unfavorable subsurface conditions, surcharge loading, maximum slope heights, and steep slope inclinations.

Our evaluation identified 8 cross-sections where the embankments exhibited critical subsurface conditions and geometries. The critical cross-sections are oriented perpendicular to the face of the embankment slopes. Locations of critical cross-sections are presented on the *Test Location Plan* (**Appendix A)**. The following **Table 1** provides a summary of geometric conditions associated with the critical cross-sections identified at the site.

Table 1: Summary of Critical Cross-Sections

Section	Crest Elevation	Toe Elevation	Slope Height	Steepest Inclination
D – D'	±EL 35.5	±EL 24.5'	±11.0 feet	2.4(H):1(V)
E – E'	±EL 37.5	±EL 34.9'	±2.2 feet	4.2(H):1(V)
F – F'	±EL 37.9'	±EL 34.5'	±3.4 feet	6.9(H):1(V)
G – G'	±EL 43.9'	±EL 32.0'	±9.0 feet	5.1(H):1(V)
H – H'	±EL 43.1'	±EL 31.0'	±8.5 feet	7.9(H):1(V)
J – J'	±EL 43.1'	±EL 29.5'	±9.9 feet	4.1(H):1(V)
K – K'	±EL 43.2'	±EL 27.5'	±15.5 feet	4.0(H):1(V)
L – L'	±EL 43.5'	±EL 19.0'	±24.5 feet	3.7(H):1(V)

3.0 SUBSURFACE CONDITIONS AND MATERIAL PROPERTIES

The subsurface exploration included thirty (30) soil test borings and nine (9) cone penetration tests. Details of the conditions encountered at the test locations are contained on the *Boring Logs* (**Appendix B)**, *CPT Logs* (**Appendix C**), and subsurface profiles (**Appendix D**). The stratification lines indicated on the boring logs and profiles represent the approximate boundaries between soil types. The actual transitions may be gradual.

Test locations were estimated in the field by pacing distances and approximating angles from existing features shown on available plans and through the use of a handheld GPS device. Therefore, the test locations indicated on the attached *Test Location Plan* (**Appendix A**) are approximate.

In general, the Charles R. Lowman Power Plant site is underlain by terrace deposits associated with the Tombigbee River overlying Coastal Plain Deposits[1]. The embankments are constructed of previously placed fill and undisturbed deposits. Following is a summary of the subsurface conditions encountered in the borings performed along the FGD Waste, Unit 2/3 Bottom Ash, and Unit 1 Bottom Ash Pond embankments.

3.1 SURFICIAL MATERIAL

Borings B-1 to B-6, S-6 to S-12, T-2 and T-3 were performed along a rail line that has since been removed and encountered approximately 1½' to 2' of crushed aggregate (railroad ballast) at the ground surface. Borings B-12 and B-13 encountered ±2' of blended silty sand and crushed aggregate at the ground surface. Borings B-10, B-11, S-1, and T-1 initially encountered previously placed soil fill as described in the following section. Borings S-2 to S-5, S-13 and S-14 were performed in vegetated areas and encountered topsoil ranging in thickness from approximately 3 to 12 inches.

3.2 PREVIOUSLY PLACED FILL

Underlying the surficial material and at the ground surface at B-1, B-3 to B-9 to B-11, S-1, S-5, S-6 to S-14, and T-1 to T-3, the soil test borings encountered previously placed fill associated with the existing embankments. The fill extended to depths ranging from approximately 6½ to 35 feet below the existing ground surface along the FGD Waste and Unit 2/3 Bottom Ash Pond embankments and approximately 7 to 18 feet below the existing ground surface along the Unit 1 Bottom Ash Pond embankment. Documentation associated with fill placement procedures and applied compactive effort is unavailable.

The existing fill encountered at the boring locations consisted of silty and clayey, fine to coarse-grained sand and fine sandy clay. The fill contained various amounts of rock fragments. The tested fill exhibited Standard Penetration Test (SPT) N-values ranging from 2 to greater than 50 blows per foot (bpf). N-values typically ranged from 10 to 50 bpf and averaged 29 bpf. The unconfined compressive strength (PP_{qu}) of cohesive samples was determined using a hand-held penetrometer. The samples exhibited PP_{qu} values ranging from less than 0.25 to 1.25 tons per square foot (tsf). The fill exhibited a consistency ranging from very loose to very dense and very soft to very stiff.

[1] *Geologic Map of Alabama*; Geologic Survey of Alabama; 1988

Natural moisture contents in samples of the fill ranged from 7% to 27%. Tested samples contained between 7.8% and 83.0% fine-grained (silt and clay size) particles. Atterberg Limits tests indicated the soils were non-plastic (NP) to moderately plastic with Liquid Limits (LL) ranging from NP to 42 and Plasticity Indices (PI) ranging from NP to 25. Based on USCS guidelines, tested samples of the fill are classified as silty sand (SM and SP-SM), sandy clay (CL), clayey sand (SC), , silty sand (SC-SM), and clayey gravel (GC).

3.3 LOW TERRACE AND COASTAL PLAIN DEPOSITS

Low terrace deposits are water deposited soils typically resulting from meanderings of rivers and streams. Coastal Plain Deposits are naturally occurring soils that appear to have formed by the gradual deposition of sediment in an ancient marine environment. Low terrace deposits associated with the Tombigbee River and Coastal Plain Deposits were encountered underlying the previously placed fill. The deposits extended to the boring termination depths ranging from approximately 40 to 60 feet below the existing ground surface.

The deposits encountered in the borings generally consisted of silty and clayey, fine to medium-grained sand, gravel and clay with varying amounts of fine sand. SPT N-values in the deposits ranged from Weight of Hammer (WOH) to greater than 50 bpf. N-values typically ranged from 2 to 25 bpf and averaged 13 bpf. WOH material exhibits a very low consistency and is penetrated under the static weight of the hammer and drilling tools. Cohesive samples of the deposits exhibited PP_{qu} values ranging from less than 0.25 tsf to 2.0 tsf. In general, the deposits exhibited loose to very dense and very soft to hard consistencies.

Natural moisture contents in samples of the deposits ranged from 14% to 38%. Tested samples were non-plastic to highly plastic with LL values ranging from NP to 74 and PI values ranging from NP to 52. The samples contained between 2.3% and 97.7% fine-grained (silt and clay size) particles. Based on the Unified Soil Classification System (USCS), the tested deposits are classified as silty and clayey sand (SM and SM-SC), plastic clay (CH), sandy silt (ML) and silty and fine sandy clay (CL, CL-ML).

3.4 GROUNDWATER

Measurements were made in the open boreholes to determine the depth to groundwater, if present at the time of drilling. Additionally, piezometers were installed in ten (10) borings (B-2, B-5, B-8, B-11, B-13, S-1, S-3, S-7, S-11 and S-13) to determine delayed groundwater levels. Groundwater was encountered at depths ranging from ±9½ feet to ±31 feet below the existing ground surface. The groundwater level was encountered between ±EL10' and ±EL33' at the tested locations. Borings not containing piezometers were backfilled with grout upon completion of drilling operations.

Groundwater depth is highly variable and will often fluctuate due to seasonal variations in precipitation and fluctuations in adjacent bodies of water. Typically, long-term monitoring over several seasons is required to evaluate the stabilized range of depths to groundwater in the upper soils.

4.0 LABORATORY TEST RESULTS

Laboratory tests were performed on representative samples retrieved from the soil test borings. Testing included natural moisture content, Atterberg Limits, and Grain Size Analysis. Results of the laboratory tests and corresponding soil classifications based on USCS guidelines are included on the *Boring Logs* (**Appendix B**).

In addition to the noted laboratory tests, the strength characteristics of representative, undisturbed and remolded soil samples were determined using consolidated, undrained (CU) triaxial shear tests. The laboratory test results were used in evaluation of the strength characteristics of soils modeled in the stability analysis. A summary of the test results is presented in the following **Table 2**.

Table 2: Soil Strength Test Results						
Location	Depth	USCS Classification	Total Stress		Effective Stress	
			Φ	C (psf)	Φ	C (psf)
B-8	21½' – 23½'	CL	12.6°	535	18.7°	449
B-13	11½' – 13½'	SM	31.2°	243	28.3°	651
S-1	10' – 12'	SC	21.8°	1,690	30.1°	398
S-7	26½' – 28½'	CH	0°	1,228	---	---
Dredge Pond, B-1	8' – 10'	SC	21.6°	568	25.0°	484
Dredge Pond, B-1	31.5' – 33.5'	SC	17.7°	384	29.0°	212

5.0 REQUIRED LOADING CONDITIONS AND FACTORS OF SAFETY

State [ADEM Admin. Code 335-13-15-.04(4)(e)(1)] and federal [40 CFR Part 257.73 (e)(1)(i) through (iv)] regulations specify loading conditions and corresponding factor of safety (FS) values for CCR impoundment embankments. The following **Table 3** indicates the required minimum FS values for the various loading conditions specified in the regulations.

Table 3: Minimum Required Factors of Safety	
Static, Long Term Maximum Storage (Normal) Pool	1.50
Static, Maximum Surcharge Pool (Flood)	1.40
Seismic Condition	1.00
Liquefaction	1.20

6.0 EMBANKMENT STABILITY ANALYSIS

Analyses of maximum storage pool, maximum surcharge pool, seismic and liquefaction loading scenarios were performed at critical locations along the perimeter embankments of the FGD Waste, Unit 2/3 Bottom Ash, and Unit 1 Bottom Ash Ponds. The stability analyses of the critical embankment cross-sections were performed using the computer software GeoStudio™2025 *version 2025.2.1*.

The SLOPE/W® module within GeoStudio™ 2025 was used to assess the rotational and translational stability of the existing embankment configurations. The analysis considers circular, block, and composite slip surfaces using the Spencer Method. The Spencer Method is a factor-of-safety, limit-equilibrium procedure that satisfies both force and moment conditions of equilibrium. The geometry of the cross-sections was imported directly into SLOPE/W® from surfaces generated by the topographic survey data.

6.1 ANALYSIS SOIL PROPERTIES

The subsurface conditions used in evaluation of the embankment stability are based on the findings of the subsurface exploration and laboratory testing program. The strength characteristics of the fill, terrace deposits and coastal plain deposits were derived from laboratory data, correlations based on USCS soil classifications provided by NAVFAC[2], and our experience with similar soil types. SPT hammer efficiency was measured as 87.7%.

The embankment and foundation soils have been in place for many years resulting in dissipation of excess pore water pressures. Therefore, static, maximum storage pool and static, maximum surcharge pool analyses of the embankments were based on drained (effective) soil strength parameters. A nominal effective cohesion value was attributed to granular soils to prevent theoretical surficial slides when modeling slope stability.

Seismic loading can result in development of excess pore pressures. Therefore, undrained (total) soil strength parameters were used in modeling short term, seismic loading in cohesive soils. Additionally, the strength parameters of the on-site soils were conservatively reduced to account for cyclic softening associated with seismic loading conditions. Soil strength parameters (C for cohesive soils and tan Ø for cohesionless soils) were reduced by 20% when modeling seismic loading. Soil properties [Mohr-Coulomb C (cohesion) and Ø (internal friction angle)] used in analyses of the embankments are provided in the following **Table 4**.

Table 4: Analysis Soil Properties

Soil Description	γ_{total}	Ø	C (psf)	Ø'	C' (psf)	Reduced Values for Seismic
railroad ballast	105 pcf	n/a	n/a	36°	0 psf	n/a
loose silty sand fill	115 pcf	n/a	n/a	28°	50 psf	Ø' = 23°
medium dense silty sand fill	120 pcf	n/a	n/a	32°	50 psf	Ø' = 26°
dense silty sand fill	125 pcf	n/a	n/a	34°	50 psf	Ø' = 28°
soft sandy clay fill	110 pcf	0°	350 psf	22°	150 psf	C = 280 psf
medium sandy clay fill	115 pcf	0°	500 psf	25°	200 psf	C = 400 psf
stiff sandy clay fill	120 pcf	0°	1,000 psf	27°	400 psf	C = 800 psf
loose clayey sand deposits	115 pcf	n/a	n/a	28°	100 psf	Ø' = 23°
loose silty sand deposits	115 pcf	n/a	n/a	28°	50 psf	Ø' = 23°
medium dense silty sand deposits	120 pcf	n/a	n/a	30°	50 psf	Ø' = 25°
dense silty sand deposits	125 pcf	n/a	n/a	32°	50 psf	Ø' = 26°
soft sandy clay deposits	110 pcf	0°	350 psf	22°	150 psf	C = 280 psf
medium sandy clay deposits	115 pcf	0°	500 psf	25°	200 psf	C = 400 psf
stiff sandy clay deposits	120 pcf	0°	1,000 psf	27°	400 psf	C = 800 psf

[2] Navy Facilities Engineering Command; "*Foundations and Earth Structures, Design Manual 7.02*"; dated September 1, 1986; p. 7.2-39.

Under maximum storage (normal) pool and maximum surcharge (flood) pool conditions, the pool level within the impoundments was modeled based on the findings of the inflow design control plans[3,4,5]. The phreatic surface through the embankments was then assumed to move through horizontally homogenous soil conditions with no toe drain within the embankments. This results in a tailwater depth of 1/3 of the reservoir depth above the embankment toe[6]. Groundwater depth within the embankment was linearly interpolated between the reservoir depth and the tailwater depth.

6.2 STATIC, MAXIMUM STORAGE POOL STABILITY ANALYSIS

Stability analyses were performed at the critical cross-sections under static, maximum storage pool conditions. The static, maximum storage pool represents a long-term condition under typical operation loading parameters. Groundwater levels and pore pressures are assumed to be in equilibrium.

The maximum storage pool elevation for the Unit 1 Bottom Ash Pond of EL 10' was established by the *Inflow Design Control Plan: Unit 1 Bottom Ash Pond*. As part of ongoing closure operations, the Unit 2/3 Bottom Ash and FGD Waste ponds are currently being and will be dewatered through the use of electric and diesel-powered pumps and associated discharge piping. Additionally, rim ditches are present around the interior of the ponds to facilitate removal of interstitial water from the CCR. The combination of the noted dewatering methods will prevent the pool elevation within the FGD Waste and Unit 2/3 Bottom Ash ponds from exceeding EL 31' and 33' under maximum storage pool conditions, respectively. A summary of maximum storage pool elevations is provided in **Table 5.**

Table 5: Maximum Storage Pool Elevations

Impoundment Designation	Cross-Section	Top of Embankment	Normal Pool
Unit 1 Bottom Ash	D - D'	±EL 35.5'	EL 10.0'
Unit 1 Bottom Ash	E – E'	±EL 37.5'	EL 10.0'
Unit 1 Bottom Ash	F – F'	±EL 37.9'	EL 10.0'
Unit 2/3 Bottom Ash	G – G'	±EL 43.9'	EL 33.0'
Unit 2/3 Bottom Ash	H – H'	±EL 43.1'	EL 33.0'
Unit 2/3 Bottom Ash	J – J'	±EL 43.1'	EL 33.0'
FGD Waste	K – K'	±EL 43.2'	EL 31.0'
FGD Waste	L – L'	±EL 43.5'	EL 31.0'

ADEM Admin. Code 335-13-15-.04(4)(e)(1)(i) and 40 CFR Part 257.73(e)(1)(i) state that "The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.5." The analyses indicate minimum FS values ranging from 2.0 to 5.6 under maximum storage pool conditions. The following **Table 6** provides specific results of the analyses. Detailed results of the static, maximum storage pool analyses are included in **Appendix E**.

[3] Three Notch Group, Inc. *Inflow Design Control Plan: Unit 1 Bottom Ash Pond*; dated September 2026.
[4] Three Notch Group, Inc. *Inflow Design Control Plan: FGD Waste Pond*; dated September 2026.
[5] Three Notch Group, Inc. *Inflow Design Control Plan: Unit 2/3 Bottom Ash Pond*; dated September 2026.
[6] US Bureau of Reclamation; *Design of Small Dams*; 1987; p. 191.

Table 6: Analyses Results, Static Maximum Storage Pool		
Impoundment Designation	**Section**	**Factor of Safety**
Unit 1 Bottom Ash	D - D'	2.1
Unit 1 Bottom Ash	E – E'	5.0
Unit 1 Bottom Ash	F – F'	5.6
Unit 2/3 Bottom Ash	G – G'	2.9
Unit 2/3 Bottom Ash	H – H'	3.3
Unit 2/3 Bottom Ash	J – J'	2.6
FGD Waste	K – K'	2.7
FGD Waste	L – L'	2.0

6.3 STATIC, MAXIMUM SURCHARGE POOL STABILITY ANALYSES

The maximum surcharge pool is a temporary condition in which the impoundment floods to a level higher than the normal pool. Although the flooding is temporary, the extreme condition of steady-state seepage at the surcharge pool level is typically modeled. Therefore, tailwater depths are adjusted to a higher elevation from the normal condition to reflect the elevated pool within the impoundment.

Three Notch performed the previously referenced hydraulic analyses as part of the current authorization. The hydraulic analyses were based on the specified storm event resulting from a hazard classification rating of "significant" and various water balance scenarios provided by PowerSouth.

The analyses identified the maximum surcharge pool elevations within the impoundments resulting from a 72-hour duration, 1,000-year storm and abnormal operations (e.g. an extended loss of power or pump failure). The maximum hydraulic grade generated by the analyses was used to evaluate embankment stability for each impoundment.

The analyses indicate that maximum surcharge pool elevations within the impoundments range from approximately 2.6 feet (Unit 2/3 Bottom Ash pond) to 25.1 feet (Unit 1 Bottom Ash pond) below the top of the embankments (including railroad ballast) at the cross-sections. **Table 7** indicates maximum surcharge pool elevations within the specific impoundments.

Table 7: Maximum Surcharge Pool Elevations			
Impoundment Designation	**Cross-Section**	**Top of Embankment**	**Maximum Surcharge Pool Elevation**
Unit 1 Bottom Ash	D - D'	±EL 35.5'	EL 12.8'
Unit 1 Bottom Ash	E – E'	±EL 37.5'	EL 12.8'
Unit 1 Bottom Ash	F – F'	±EL 37.9'	EL 12.8'
Unit 2/3 Bottom Ash	G – G'	±EL 43.9'	EL 40.5'
Unit 2/3 Bottom Ash	H – H'	±EL 43.1'	EL 40.5'
Unit 2/3 Bottom Ash	J – J'	±EL 43.1'	EL 40.5'
FGD Waste	K – K'	±EL 43.2'	EL 39.5'
FGD Waste	L – L'	±EL 43.5'	EL 39.5'

Stability analyses under static, maximum surcharge pool conditions were performed at the critical embankment cross-sections. ADEM Admin. Code 335-13-15-.04(4)(e)(1)(ii) and 40 CFR Part 257.73 (e)(1)(ii) state, "The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.4." Analyses results indicate FS values ranging from 1.8 to 5.5 as provided in **Table 8**. Details of the analyses are included in **Appendix F**.

Table 8: Analyses Results, Maximum Surcharge Pool		
Impoundment Designation	**Section**	**Factor of Safety**
Unit 1 Bottom Ash	D - D'	2.1
Unit 1 Bottom Ash	E – E'	5.0
Unit 1 Bottom Ash	F – F'	5.5
Unit 2/3 Bottom Ash	G – G'	2.4
Unit 2/3 Bottom Ash	H – H'	2.9
Unit 2/3 Bottom Ash	J – J'	2.2
FGD Waste	K – K'	2.4
FGD Waste	L – L'	1.8

6.4 PROBABILISTIC SEISMIC HAZARD ANALYSIS AND SITE RESPONSE ANALYSIS

A seismic evaluation completed for the site [*Development of Site-Specific Design Ground Motions for the Lowman Power Plant* (Lettis Consultants International; dated 7/29/2021)] included performing a probabilistic seismic hazard analysis (PSHA) using seismic source models for the eastern United States. The PSHA integrates many different potential seismic sources based on the earthquake magnitude and distance to the site for each source in order to compute a uniform hazard ground motion for the site. The PSHA results present the bedrock ground motions that have a 2% and 10% probability of being exceeded in 50 years. As rock is relatively deep at the Lowman site, a site response analysis (SRA) was completed. The SRA uses the results of the PSHA and shear wave velocities measured with the CPT to generate site-specific ground motions near the ground surface. Details of the PSHA and SRA can be found in **Appendix G.**

A mean or modal earthquake magnitude is deaggregated from the PSHA results in order to represent the average or controlling seismic hazard. This controlling hazard is used to perform the location-specific seismic embankment displacement and liquefaction calculations. The deaggregation extracts a group of seismic events that are most likely to be impactful to the analysis to be performed. For Lowman, these are magnitudes and sources representing the New Madrid fault zone. As noted previously, the results of the site-specific analyses were compared with ground motions generated by 2023 USGS seismic data and found to be similar.

6.5 SEISMIC STABILITY ANALYSIS FINDINGS

The current industry standard of care requires that the seismic analysis of embankment stability be performed by estimating permanent displacements rather than calculating a FS based on limit equilibrium principles. The estimated permanent deformation is compared to a tolerable displacement value in light of the potential damage to the final embankment configuration. Based on FEMA guidelines, the amount of allowable deformation along critical failure surfaces is limited to 24 inches[7]. The deformation analysis is performed to determine if potential movements resulting from the design earthquake would produce overtopping of the embankments, or if cracks could form in the embankment or foundation soils that could result in failure by internal erosion.

The initial step in the seismic analysis is to assign appropriate dynamic strength parameters (drained or undrained) to the subsurface materials. Reduced soil strengths are applied to soils to model the effect of cyclic loading and undrained shearing that occurs during seismic events. The fine-grained soils were assumed to exhibit 80% of their static cohesion. For non-cohesive soils, the reduced internal friction angle was calculated as $\tan^{-1}(0.8 \times \tan \phi)$.

The reduced soil strength parameters were then used in a pseudo-static (seismic coefficient) analysis. Pseudostatic analysis assumes that the earthquake causes an additional horizontal force in the direction of failure due to the motion of the soil mass. The forces are computed as the product of a seismic coefficient and the weight of the soil mass. A FS is determined based on static analysis of driving and resisting forces.

[7] US Department of Homeland Security – FEMA; *Federal Guidelines for Dam Safety, Earthquake Analyses and Design of Dams*; May 2005; p. 34.

The pseudo-static stability analysis is iteratively performed to evaluate the yield acceleration (k_y) for the individual cross-sections. The yield acceleration is unique to each embankment and represents the smallest horizontal ground acceleration at which a marginally stable condition is produced for a potential slip surface. Therefore, the seismic coefficient that results in a pseudo-static FS of 1.0 represents k_y. Calculations resulted in k_y values ranging from 0.043g to 0.55g at the cross-sections.

The shear wave velocity of the on-site soils was measured using Seismic Cone Penetration Tests (sCPTs) extending to depths of approximately 45 to 70 feet below the existing ground surface (**Appendix C**). Measured shear wave velocities ranged from 37 to 3,327 feet per second (fps), with values within one standard deviation of the mean ranging from approximately 370 to 1,180 fps, and generally increased with depth. A site-specific shear wave velocity profile was developed as a smoothed lognormal average of the measured data, as presented in Figure 9 of the Lettis report referenced in Section 6.4. The profile increases from approximately 650 fps at the ground surface to 800 fps at a depth of 70 feet. A shear wave velocity of 650 fps was used, in conjunction with the embankment height, to estimate the initial fundamental period (Ts) of the sliding masses, as it best represents the near-surface embankment soils.

The design ground motions were developed based on data available from the United State Geological Survey's latest (2023) national seismic hazard model. They were then compared to the motions developed in the site-specific probabilistic seismic hazard analysis (PSHA) and site response analysis presented in the Lettis report and found to be similar. The spectral acceleration at 1.5Ts for each cross-section was obtained from the 5%-damped site-specific response spectrum for the 2,500-year return period (2% probability of exceedance in 50 years). The moment magnitude of the design earthquake was taken as 7.5 based on disaggregation of the computed seismic hazard.

Permanent seismic displacements were then estimated using a procedure developed by Bray and Travasarou (2007). The procedure is a semi-empirical, probabilistic method based on nonlinear, fully coupled sliding block analyses of a large database of recorded earthquake ground motions. It requires four primary inputs: the yield coefficient of the potential sliding mass (ky) described above, the initial fundamental period of the sliding mass (Ts), the spectral acceleration of the design ground motion at a degraded period equal to 1.5Ts, and the moment magnitude of the design earthquake.

Using these inputs, the procedure provides the probability of negligible displacement, defined as displacement less than 1 cm (approximately 0.4 inch), and the median estimated displacement. The probability that displacement will exceed a selected allowable value can then be calculated, accounting for the variability inherent in earthquake ground motions.

ADEM Admin. Code r. 335-13-15-.04(4)(e)1.(iii) and 40 CFR §257.73(e)(1)(iii) require a calculated seismic factor of safety of at least 1.0. In accordance with current engineering practice, seismic performance is best evaluated in terms of permanent displacement relative to the established allowable displacement of 24 inches. The analyses indicate a median estimated displacement of approximately 5 inches at Section D-D'; 1 inch at Sections K-K' and L-L', and negligible displacement at the remaining cross-sections. Therefore, all sections exhibit a FOS of greater than 1.0 when comparing the calculated median displacement to the 24-inch allowable limit. Additionally, the probability of exceeding the allowable 24-inch deformation is 1 percent or less at every section.

Based on guidance from Vick[8], a probability of 1% may be expressed qualitatively as "almost impossible". Therefore, the embankments are expected to perform acceptably under the design earthquake and are judged to conform to the code requirements for seismic loading. Detailed analysis results are presented in **Appendix H** and summarized in **Table 9.**

Table 9: Seismic Slope Deformation Analysis Summary

Impoundment Designation	Cross-Section	k_y[1]	Estimated Median Deformation (in.)	Estimated Probability that Deformation Will Exceed 24 inches
Unit 1 Bottom Ash	D - D'	0.043g	5	1%
Unit 1 Bottom Ash	E – E'	0.550g	0	0%
Unit 1 Bottom Ash	F – F'	0.206g	0	0%
Unit 2/3 Bottom Ash	G – G'	0.260g	0	0%
Unit 2/3 Bottom Ash	H – H'	0.230g	0	0%
Unit 2/3 Bottom Ash	J – J'	0.233g	0	0%
FGD Waste	K – K'	0.099g	1	0%
FGD Waste	L – L'	0.144g	1	0%

Notes: 1 – Minimum yield acceleration of the critical failure surface.

6.6 LIQUEFACTION POTENTIAL

Liquefaction refers to the "quick" condition soils exhibit when rapid motion causes an excessive build-up in pore water pressure. As water pressure increases, soil particles can become suspended within the water and lose particle-to-particle contact. The result is a dramatic drop in shear strength with soils exhibiting a liquid consistency and the potential for significant failures on otherwise stable slopes. Soils most prone to liquefaction are loose sands below the groundwater level.

Liquefaction analysis is performed by first determining potentially liquefiable soil zones within the subsurface profile of the embankment. Secondly, the liquefiable soils are then assigned a residual strength value and the embankment analyzed for stability based on conventional static analysis methods. ADEM Admin. Code 335-13-15-.04(4)(e)(1)(iv) and 40 CFR Part 257.73 (e)(1)(iv) state that, "For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20."

[8] Vick, S. G. (2002) *Degrees of Belief: Subjective Probability and Engineering Judgment*; ASCE Press; New York; pp. 297-299.

The embankment soils were evaluated for liquefaction potential using recommendations by Idriss and Boulanger[9] (2008). That is, the field (or raw) SPT N-values were normalized for overburden pressure, adjusted for a hammer efficiency of 60%, and corrected for fines content and borehole parameters. The design earthquake moment magnitude and resultant peak ground acceleration were determined from the site-specific probabilistic seismic hazard analysis. The average static shear stress on the horizontal plane was approximated based on elastic theory as presented by Poulos and Davis[10]. The FS for liquefaction of soil layers is calculated as the quotient of the cyclic resistance ratio divided by the cyclic stress ratio.

Resultant FS values were a minimum of 2.1. Therefore, the results indicate that the embankments were not constructed of soils that have susceptibility to liquefaction. The following **Table 10** provides the minimum FS at the cross-sections. Details of the liquefaction potential analyses at the cross-sections are included in **Appendix I**.

Table 10: Analyses of Liquefaction Potential

Impoundment Designation	Section	Minimum Factor of Safety
Unit 1 Bottom Ash	D - D'	2.1
Unit 1 Bottom Ash	E – E'	Non-liquefiable[1]
Unit 1 Bottom Ash	F – F'	Non-liquefiable[1]
Unit 2/3 Bottom Ash	G – G'	>>5
Unit 2/3 Bottom Ash	H – H'	7.8
Unit 2/3 Bottom Ash	J – J'	6.4
FGD Waste	K – K'	3.1
FGD Waste	L – L'	>>5

Notes: 1 – Section E and F encountered clay (non-liquefiable) below the water level.

7.0 SUMMARY OF STABILITY FINDINGS

State [ADEM Admin. Code 335-13-15-.04(4)(e)] and federal (40 CFR 257.73) regulations require that the stability of CCR impoundment embankments be periodically evaluated. The purpose of this study was to perform the required evaluations for the embankments at the Lowman Power Plant. The stability evaluation was conducted on the exterior embankments associated with the FGD Waste, Unit 2/3 Bottom Ash, and Unit 1 Bottom Ash ponds.

Regulations state that the evaluation is to be performed under static long-term maximum storage pool, static maximum surcharge pool and seismic loading scenarios. Additionally, for dikes constructed of soils that have susceptibility to liquefaction, the stability is to be evaluated under liquefaction conditions.

[9] Idriss and Boulanger; *Residual Shear Strength of Liquefied Soils*; University of California, Davis – Department of Civil and Environmental Engineering; 2007.

[10] Poulos, H.G. and Davis, E.H., *Elastic Solutions for Soil and Rock Mechanics; 1974.*

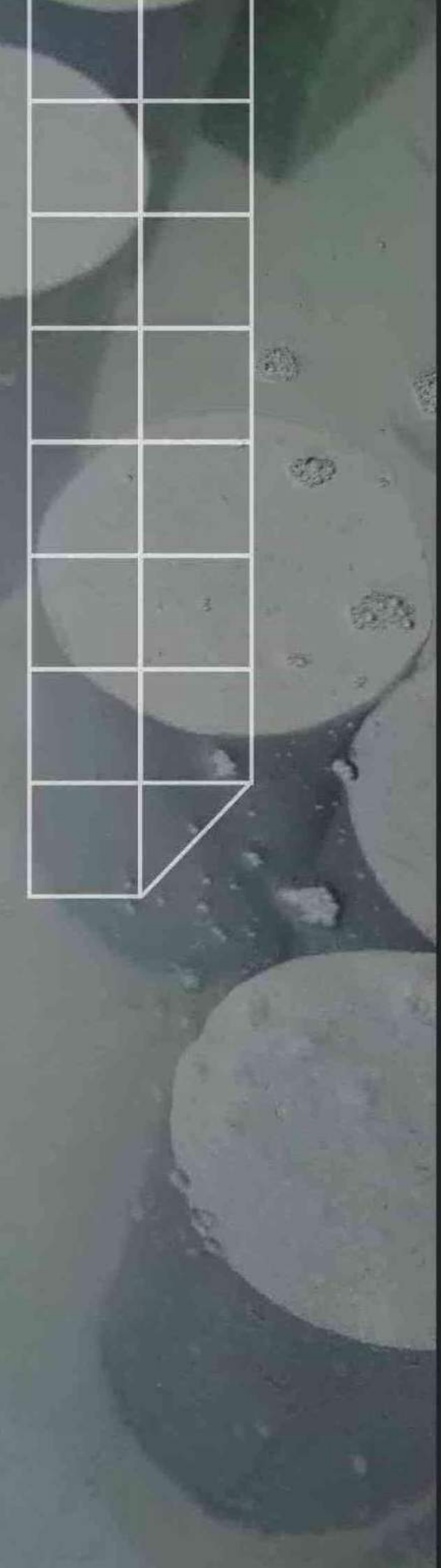
The stability analyses indicate that the embankments exhibit factors of safety that equal or exceed the required minimum values under the maximum storage pool, maximum surcharge pool, and seismic loading scenarios. Additionally, the embankment soils are demonstrated not to be susceptible to liquefaction, and further analyses under the liquefaction loading scenario is not required. Therefore, the embankments comply with the periodic safety factor assessment requirements.

8.0 GENERAL REMARKS AND CLOSING

This report has been prepared for the exclusive use of PowerSouth Energy Cooperative for specific application to the Periodic Safety Factor Assessment (2026) of Coal Combustion Residuals Impoundments Embankments project at Charles R. Lowman Power Plant in Leroy, Alabama and is not transferable to a third party. The recommendations in this report are intended for use on the stated project and should not be used for other purposes.

The analyses and conclusions presented in this report are based upon currently accepted engineering principles, practices, and existing testing standards in the area where the services were provided. No other warranty, expressed or implied, is made.

The findings in this report were developed based on written and verbal information provided by the client and from the limited data obtained from the field and laboratory testing programs. If significant changes are made to the use, capacity or geometry of the embankments and/or impoundments, Three Notch should be allowed to review our findings in light of the changes to determine if additional testing and revised conclusions are needed.



APPENDIX A

Test Location Plan

Legend

- Boring Location (July 2009)
- Boring Location (2011)
- Boring Location (August 2016)
- Boring Location (2020)
- CPT Location (2011)
- CPT Location (2020)
- A - A' Cross Section

Process Waste Pond

FGD Waste Pond

Unit 2/3 Bottom Ash Pond

Unit 1 Bottom Ash Pond

500 0 500

SCALE 1" = 500'

Notes:

Test locations are approximate.

This plan was based on aerial imagery captured on 08/24/26

LOWMAN AUTHORIZATION #7
LEROY, ALABAMA
PROJECT NO.: R021225012

Test Location Plan

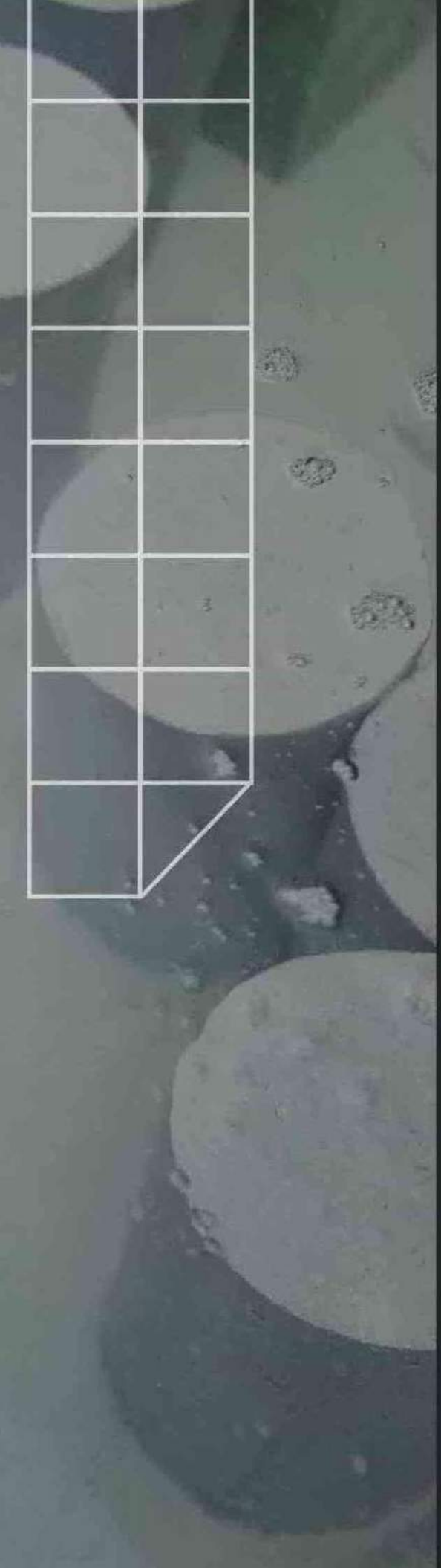
Date: 08/20/2026

Page 1 of 1

Drawn By: AH

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APPENDIX B

Boring Logs

Boring B-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/15/2009 Approx. Ground Elevation: 44.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate											
	40.0		Very dense, red, silty fine to medium SAND, with numerous rock fragments		26-27-28 (55)									
5			...same		20-22-23 (45)									
	35.0		...same		24-24-26 (50)									
10														
	30.0		... with numerous rock fragments		29-37-40 (77)									
15														
	25.0		Very dense, silty, coarse-grained SAND, with trace rock fragments		26-38-43 (81)									▽ GW at +/-EL24 ft. on 7/15/2009.
20														
	20.0		Stiff, brown, fine sandy CLAY with gravel		6-6-7 (13)									
25														

(Continued Next Page)

Boring B-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/15/2009 Approx. Ground Elevation: 44.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Stiff, brown, fine sandy CLAY with gravel *(Continued from previous page)* (Fill)											
30	15.0		Medium, grey and tan, fine sandy CLAY	☒	2-3-4 (7)									
35	10.0		... stiff, grey	☒	3-5-5 (10)								1.25	
40	5.0		(Low Terrace Deposits)	☒	4-4-7 (11)								1.25	
			Boring terminated at 40.0 feet.											
45	0.0													
50	-5.0													

Boring B-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/14/2009 Approx. Ground Elevation: 44.0 ft.

Notes:
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate											
	40.0		Dense, red, silty-clayey fine to medium SAND		19-20-20 (40)			18	14	4	9	33.5		USCS = SC-SM
5			...same		20-21-22 (43)						8			
	35.0		... very dense, with numerous rock fragments		14-23-30 (53)						10			
10														▽ GW at +/-EL33 ft. on 8/4/2009.
	30.0		Dense, tan, silty coarse SAND with numerous rock fragments		21-18-20 (38)						13			▽ GW at +/-EL30.5 ft. on 7/14/2009.
15														
	25.0		Medium dense, reddish tan, silty fine SAND with trace rock fragments		7-10-14 (24)			23	20	3	15	25.5		USCS = SM
20														
	20.0		...dense, with gravel		8-13-20 (33)						11			
25														

(Continued Next Page)

Boring B-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/14/2009 Approx. Ground Elevation: 44.0 ft.

Notes:
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...dense, with gravel *(Continued from previous page)*											
30	15.0		Medium dense, reddish tan, silty fine to coarse-grained SAND		6-12-8 (20)						14			
35	10.0		Loose, grey, silty fine SAND		2-3-4 (7)			NP	NP	NP	28	20.0		USCS = SM
40	5.0		... medium dense, with gravel (Low Terrace Deposits)		6-8-8 (16)						24			
			Boring terminated at 40.0 feet.											
45	0.0													
50	-5.0													

Boring B-3

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant

Project Location: Leroy, Alabama Hammer Type: Manual

CDG Project Number: R021221021 Method: Mud Rotary

Date Drilled: 7/15/2009 Approx. Ground Elevation: 44.0 ft.

Notes:

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate											
	40.0		Medium dense, red, silty fine to medium SAND, with numerous rock fragments	☒	7-10-15 (25)									
5			... dense	☒	10-18-16 (34)									
	35.0		...reddish tan, with gravel	☒	16-18-18 (36)									
10														
	30.0		... very dense, tan, with gravel	☒	24-28-34 (62)									
15														
	25.0		... dense, reddish tan, with trace rock fragments	☒	18-20-29 (49)									▽ GW at +/-EL24 ft. on 7/15/2009.
20														
	20.0		...red, with gravel	☒	10-16-22 (38)									
25														

(Continued Next Page)

Boring B-3

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant

Project Location: Leroy, Alabama Hammer Type: Manual

CDG Project Number: R021221021 Method: Mud Rotary

Date Drilled: 7/15/2009 Approx. Ground Elevation: 44.0 ft.

Notes:

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
30	15.0		Dense, red, silty fine to medium SAND, with gravel (Fill)	☒	11-14-11 (25)									
35	10.0		Stiff, grey and tan, fine sandy CLAY, with gravel	☒	5-6-6 (12)									
40	5.0		Medium dense, grey and tan, silty fine SAND (Low Terrace Deposits)	☒	5-6-7 (13)									
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	0.0													
50	-5.0													

Boring B-4

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/13/2009 Approx. Ground Elevation: 44.0 ft.

Notes:

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate											
	40.0		Dense, red, silty fine to medium SAND	☒	21-24-26 (50)						8			
5			...very dense, with gravel	☒	15-25-27 (52)						9			
	35.0		...medium dense	☒	9-12-15 (27)						14			
10														
	30.0		...very dense, reddish brown	☒	30-35-40 (75)						7			
15														
	25.0		...dense, orange and tan	☒	12-14-19 (33)						15			▽ GW at +/-EL24 ft. on 7/13/2009.
20														
	20.0		...medium dense, red	☒	10-12-15 (27)						21			
25														

(Continued Next Page)

Boring B-4

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant

Project Location: Leroy, Alabama Hammer Type: Manual

CDG Project Number: R021221021 Method: Mud Rotary

Date Drilled: 7/13/2009 Approx. Ground Elevation: 44.0 ft.

Notes:

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...medium dense, red *(Continued from previous page)*											
30	15.0		Medium dense, tan, silty fine SAND, with gravel (Fill)	X	5-7-7 (14)						15			
35	10.0			X	2-3-4 (7)									No recovery
40	5.0		Loose, brown, silty fine SAND (Low Terrace Deposits)	X	4-3-4 (7)						38			
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	0.0													
50	-5.0													

Boring B-5

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/15/2009 Approx. Ground Elevation: 43.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate											
	40.0		Dense, red, silty fine to coarse SAND, with gravel	☒	10-18-20 (38)			17	16	1	7	21.1		USCS = SM
5			...medium dense	☒	8-12-16 (28)						8			
	35.0		...reddish orange	☒	10-11-12 (23)			NP	NP	NP	9	15.3		USCS = SM
10														
	30.0													
			...same	☒	10-12-16 (28)									
15														
	25.0													
			...same	☒	8-10-14 (24)						14			▼ GW at +/-EL24 ft. on 8/4/2009.
20														
	20.0													
			...dense	☒	15-18-23 (41)						15			
25														

(Continued Next Page)

Boring B-5

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/15/2009 Approx. Ground Elevation: 43.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	15.0		... fine to medium grained (Fill)	☒	6-8-8 (16)								1	▽ GW at +/-EL13 ft. on 7/15/2009.
30			Stiff, grey, fine sandy CLAY											
	10.0		Medium dense, grey and tan, silty fine SAND	☒	4-6-6 (12)			23	21	2	30			USCS = SM
35														
	5.0		...same (Low Terrace Deposits)	☒	4-4-7 (11)						29			
40			Boring terminated at 40.0 feet.											
	0.0													
45														
	-5.0													
50														

Boring B-6

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/15/2009 Approx. Ground Elevation: 43.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate											
	40.0		Medium dense, red, silty fine to medium SAND, with trace gravel	☒	10-11-12 (23)									
5			...same	☒	11-14-14 (28)									
	35.0		...red and orange	☒	16-16-14 (30)									
10														▽ GW at +/-EL33 ft. on 7/15/2009.
	30.0		...dense, orange with gravel	☒	19-24-26 (50)									
15														
	25.0		...medium dense	☒	8-9-10 (19)									
20														
	20.0		...same	☒	10-10-12 (22)									
25														

(Continued Next Page)

Boring B-6

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/15/2009 Approx. Ground Elevation: 43.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...same *(Continued from previous page)* (Fill)											
30	15.0		Stiff, grey, fine sandy CLAY	☒	5-5-7 (12)								1.25	
35	10.0		Medium dense, brown, silty fine SAND	☒	6-6-10 (16)									
40	5.0		...tan and brown (Low Terrace Deposits)	☒	6-8-10 (18)									
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	0.0													
50	-5.0													

Boring B-7

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/17/2009 Approx. Ground Elevation: 41.0 ft.

Notes:
Boring backfilled with grout upon completion

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Crushed aggregate with silty SAND											
			Medium dense, brown and orange, silty fine to medium SAND, with gravel	☒	12-13-8 (21)									
5	35.0		...brown and grey, fine - grained	☒	7-12-12 (24)									
			...dense	☒	10-15-16 (31)									
10	30.0													▽ GW at +/-EL31 ft. on 7/17/2009.
			...medium dense, grey	☒	7-9-13 (22)									
15	25.0		(Fill)											
			Loose, brown, silty fine to medium SAND	☒	2-3-3 (6)									
20	20.0													
			...medium dense	☒	5-12-14 (26)									
25														

(Continued Next Page)

Boring B-7

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/17/2009 Approx. Ground Elevation: 41.0 ft.

Notes:
Boring backfilled with grout upon completion

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...medium dense *(Continued from previous page)*											
30	10.0		...same	☒	7-8-6 (14)									
35	5.0		...loose	☒	4-5-5 (10)									
40	0.0		...medium dense (Low Terrace Deposits)	☒	5-7-7 (14)									
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	-5.0													
50	-10.0													

Boring B-8

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/17/2009 Approx. Ground Elevation: 41.0 ft.

Notes:
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Crushed aggregate with silty SAND											
			Dense, greyish brown, silty fine SAND	☒	12-19-20 (39)						15			
5	35.0		...grey and brown	☒	10-16-20 (36)			NP	NP	NP	7	34.5		USCS = SM
10	30.0													▽ GW at +/-EL31 ft. on 7/17/2009.
			...medium dense, brown	☒	5-10-12 (22)						12			
15	25.0		(Fill)											
			Very loose, brown, silty-clayey fine SAND	☒	2-2-2 (4)			24	20	4	30	48.9		USCS = SC-SM
20	20.0			■										Undisturbed Sample
			Stiff, grey, fine sandy CLAY	☒	3-5-5 (10)						33			
25														

(Continued Next Page)

Boring B-8

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/17/2009 Approx. Ground Elevation: 41.0 ft.

Notes:
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
30			...medium, brown	☒	2-3-3 (6)			34	15	19	36	67.9		USCS = CL
	10.0													▼ GW at +/-EL10 ft. on 8/4/2009.
35			Medium dense, brown, silty fine SAND	☒	11-15-15 (30)						19			
	5.0													
40			...same (Low Terrace Deposits)	☒	5-5-7 (12)						32			
	0.0		Boring terminated at 40.0 feet.											
45														
	-5.0													
50														
	-10.0													

Boring B-9

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant

Project Location: Leroy, Alabama Hammer Type: Manual

CDG Project Number: R021221021 Method: Mud Rotary

Date Drilled: 7/16/2009 Approx. Ground Elevation: 41.0 ft.

Notes:

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Crushed aggregate with silty SAND											
			Dense, grey and brown, silty fine SAND	☒	6-13-20 (33)			22	19	3	11	43.9		USCS = SM
5	35.0		...same	☒	14-19-21 (40)						9			
			...same	☒	14-16-16 (32)						11			
10	30.0													▽ GW at +/-EL31 ft. on 7/16/2009.
			Medium dense, brown, fine SAND with silt	☒	5-6-8 (14)			NP	NP	NP	17	7.8		USCS = SM
15	25.0		(Fill)											
			Medium, brown, fine sandy CLAY	☒	3-4-3 (7)						32			
20	20.0													
			...soft	☒	3-3-2 (5)			26	21	5	30	54.4		USCS = CL
25														

(Continued Next Page)

Boring B-9

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant

Project Location: Leroy, Alabama Hammer Type: Manual

CDG Project Number: R021221021 Method: Mud Rotary

Date Drilled: 7/16/2009 Approx. Ground Elevation: 41.0 ft.

Notes:

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
30	10.0		...stiff	☒	5-5-5 (10)						25			
35	5.0		Soft, brown, SILT-CLAY	☒	3-1-3 (4)			25	18	7	30	51.0		USCS = CL-ML
40	0.0		...stiff (Low Terrace Deposits)	☒	2-5-7 (12)						29			
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	-5.0													
50	-10.0													

Boring B-10

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/13/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	N_i60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Very stiff, red, fine sandy CLAY with numerous rock fragments	☒	3-4-10 (14)									
	35.0													
5			...stiff	☒	2-4-5 (9)								1.25	
			(Fill)											
			Very stiff, grey and brown, fine sandy CLAY	☒	5-5-9 (14)									
	30.0													
10			...stiff, grey	☒	4-4-5 (9)									
	25.0		...medium	☒	2-2-4 (6)								0.5	
15														▽ GW at +/-EL24 ft. on 7/13/2009.
	20.0		Very loose, brown, silty fine to medium SAND	☒	2-1-2 (3)									
20														
	15.0		...same	☒	2-1-1 (2)									
25														

(Continued Next Page)

Boring B-10

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/13/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
30	10.0		...brown and grey	☒	3-2-1 (3)									
35	5.0		...loose, grey and orange	☒	3-2-3 (5)									
40	0.0		...grey (Low Terrace Deposits)	☒	4-3-3 (6)									
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	-5.0													
50	-10.0													

Boring B-11

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/13/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Very stiff, orange and tan, fine sandy CLAY	☒	4-7-10 (17)			39	18	21		55.4		USCS = CL
	35.0													
5			...stiff	☒	3-4-4 (8)			36	20	16		56.5		USCS = CL
			Medium dense, brown, clayey fine to medium SAND with gravel	☒	6-9-14 (23)									
	30.0													
10			...with clay	☒	8-8-9 (17)									
			(Fill)											▼ GW at +/-EL26 ft. on 8/4/2009.
	25.0		Soft, grey, silty CLAY with fine sand	☒	2-2-3 (5)			40	17	23		91.6	<0.25	USCS = CL
15														
	20.0		...medium	☒	2-3-3 (6)									
20														
	15.0		Soft, grey, fine sandy CLAY	☒	2-2-3 (5)			28	20	8	35	67.2	<0.25	USCS = CL
25														

(Continued Next Page)

Boring B-11

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/13/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Soft, grey, fine sandy CLAY *(Continued from previous page)*											
30	10.0		Loose, brown, silty fine SAND	☒	3-3-3 (6)						26			
35	5.0		...very loose	☒	1-1-1 (2)			21	20	1	37	19.6		USCS = SM
40	0.0		...loose (Low terrace Deposits)	☒	7-5-4 (9)						27			
			Boring terminated at 40.0 feet.											
45	-5.0													
50	-10.0													

Boring B-12

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/16/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate with silty SAND											
			Medium dense, orange and tan, silty fine to medium SAND	☒	4-5-6 (11)									
	35.0													
5			...same	☒	7-11-11 (22)									
			(Fill)											
			Medium dense, brown, silty fine to medium SAND	☒	5-8-9 (17)									
	30.0													
10														▽ GW at +/-EL29 ft. on 7/16/2009.
			...grey	☒	7-7-10 (17)									
15	25.0													
			Medium, grey, fine sandy CLAY	☒	4-4-4 (8)								0.75	
20	20.0													
			...same	☒	3-4-3 (7)									
	15.0													
25														

(Continued Next Page)

Boring B-12

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/16/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
30	10.0		...very soft	☒	2-1-2 (3)								<0.25	
35	5.0		...soft	☒	2-2-3 (5)								<0.25	
40	0.0		Medium dense, grey, clayey fine to medium SAND (Low terrace Deposits)	☒	7-9-13 (22)									
			Boring terminated at 40.0 feet.											Boring backfilled with grout upon completion
45	-5.0													
50	-10.0													

Boring B-13

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/16/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Crushed aggregate with silty SAND											
			Stiff, red, fine sandy CLAY	☒	5-7-8 (15)			42	19	23	14	51.6		USCS = CL
	35.0													
5			...hard, with gravel	☒	26-28-30 (58)						11			
			(Fill)											
			Medium dense, tan, silty fine to medium SAND	☒	10-10-10 (20)						20			
	30.0													
10														▽ GW at +/-EL29 ft. on 7/16/2009.
				■										Undisturbed Sample
	25.0			☒	3-3-3 (6)									No recovery
15														
			Very soft, grey, fine sandy CLAY	☒	1-1-2 (3)						34		<0.25	▼ GW at +/-EL20 ft. on 8/4/2009.
	20.0													
20														
			Very loose, grey, silty fine to medium SAND	☒	2-1-2 (3)						32			
	15.0													
25														

(Continued Next Page)

Boring B-13

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Manual
CDG Project Number: R021221021 Method: Mud Rotary
Date Drilled: 7/16/2009 Approx. Ground Elevation: 39.0 ft.

Notes:
PPqu = Pocket Penetrometer unconfined compressive strength
Piezometer Installed at the time of boring

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
30	10.0		Very loose, grey, silty fine to medium SAND	☒	2-2-2 (4)						35			
35	5.0		Loose, brown, fine SAND with silt	☒	3-5-5 (10)			NP	NP	NP	26	10.8		USCS = SM
40	0.0		...grey (Low Terrace Deposits)	☒	4-4-4 (8)						33			
			Boring terminated at 40.0 feet.											
45	-5.0													
50	-10.0													

Boring S-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/1/2011 Approx. Ground Elevation: 39.0 ft.

Notes:
+/- 6" of sand/clay at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Loose, reddish brown, silty fine to medium SAND with rock fragments		4-4-3 (7)									
	35.0		Very soft, tan and red, fine SAND and CLAY		0-1-1 (2)			42	17	25		49.4		USCS = SC
5			... brown		1-1-2 (3)									
	30.0		... medium		2-3-4 (7)									
10														
	25.0		brown and grey		2-3-3 (6)									▽ GW at +/-EL25 ft. on 12/1/2011.
15			(Fill)											
	20.0		Very loose, grey, silty fine SAND		0-2-2 (4)									
20														
	15.0		... loose, grey and brown		0-2-3 (5)			NP	NP	NP		42.3		USCS = SM
25														
	10.0		... very loose		1-1-1 (2)									

(Continued Next Page)

Boring S-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/1/2011 Approx. Ground Elevation: 39.0 ft.

Notes:
+/- 6" of sand/clay at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Very loose, grey and brown, silty fine SAND	■										(No Recovery)
35	5.0		...same	☒	0-3-1 (4)									
40	0.0		...loose, grey and tan	☒	1-3-4 (7)									
45	-5.0		... very loose	☒	1-1-3 (4)									
50	-10.0		... loose	☒	1-3-4 (7)									
55	-15.0		... medium dense	☒	4-7-6 (13)			NP	NP	NP		21.4		USCS = SM
	-20.0		...tan and light grey (Coastal Plain Deposits)	☒	6-10-8 (18)									Piezometer Installed

Boring terminated at 60.0 feet.

Boring S-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 38.0 ft.

Notes:
+/- 3" of topsoil at ground surface
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	38.0		Medium dense, brown and tan, silty fine to medium SAND	☒	6-6-6 (12)									
	35.0		... loose	☒	3-4-4 (8)									
5			Stiff, brown CLAY with fine sand	☒	4-4-10 (14)								1.0	
	30.0		... same	☒	3-4-6 (10)								1.0	
10			(Fill)											
	25.0		Medium, grey CLAY with fine sand	☒	3-2-5 (7)									
15														
	20.0		...very soft	☒	0-0-3 (3)								<0.25	
20														
	15.0		Very loose, brown, silty fine to medium SAND	☒	2-1-2 (3)									▽GW at +/-EL14 ft. on 11/30/2001.
25														
	10.0		... loose	☒	2-3-5 (8)							15.4		USCS=SM

(Continued Next Page)

Boring S-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 38.0 ft.

Notes:
+/- 3" of topsoil at ground surface
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Loose, light brown, silty fine to medium SAND											
	5.0													
35			...same	☒	2-4-2 (6)									
	0.0													
40			... medium dense, light brown and grey	☒	2-5-9 (14)									
	-5.0													
45			... light grey and tan	☒	4-8-10 (18)									
	-10.0													Borehole backfilled with grout upon completion.
50			... tan (Coastal Plain Deposit)	☒	5-7-15 (22)									
			Boring terminated at 50.0 feet.											
	-15.0													
55														
	-20.0													

Boring S-3

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 38.0 ft.

Notes:
+/- 4" of topsoil at ground surface
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	35.0		Medium dense, red and tan, silty fine to medium SAND with numerous organics	☒	6-5-8 (13)						14			
5			Medium, brown and tan, fine sandy CLAY	☒	4-6-6 (12)						27		0.75	
	30.0		...light brown and light grey	☒	1-3-3 (6)			41	17	24		83.0	0.75	USCS = CL
10			...brown and grey ... grey	☒	2-2-3 (5)						24			
	25.0			☒	0-2-3 (5)									
15			(Fill)	■										(No Recovery) ▽ GW at +/-EL21 ft. on 11/30/2011.
	20.0		Very soft, grey, fine sandy CLAY	☒	0-0-2 (2)						33		<0.25	
20														
	15.0		Loose, brown and grey, fine to medium SAND with trace silt	☒	2-3-3 (6)						25			▼ GW at +/-EL14 ft. on 5/1/2012.
25														
	10.0		...very loose, light brown and light grey	☒	2-2-2 (4)						25			

(Continued Next Page)

Boring S-3

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 38.0 ft.

Notes:
+/- 4" of topsoil at ground surface
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Very loose, tan, fine to medium SAND with trace silt											
	5.0													
35			...loose	☒	3-4-3 (7)			NP	NP	NP	28	4.1		USCS = SM
	0.0													
40			...very loose	☒	1-1-4 (5)						27			
	-5.0													
45			...same	☒	1-1-2 (3)									
	-10.0													
50			...loose, light grey and tan (Coastal Plain Deposits)	☒	3-4-4 (8)			NP	NP	NP	16	4.5		USCS = SM Piezometer Installed.
			Boring terminated at 50.0 feet.											
	-15.0													
55														
	-20.0													

Boring S-4

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 38.0 ft.

Notes:
+/- 4" of topsoil at ground surface
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	35.0		Medium dense, red and tan, silty fine to medium SAND	☒	5-5-8 (13)									
5			...loose, brown and grey	☒	3-3-4 (7)									
	30.0		... same	☒	2-2-4 (6)									
10			Medium, grey and brown, fine sandy CLAY	☒	2-3-5 (8)								1.25	
	25.0		... same	☒	2-4-5 (9)								1.0	
15			(Fill)											
	20.0		Medium, grey and brown CLAY with fine sand	☒	2-2-3 (5)									
20														
	15.0		...soft	☒	0-1-2 (3)									
25														
	10.0		Loose, brown and grey, clayey fine to medium SAND with trace rock fragments	☒	1-3-2 (5)									▼ GW at +/-EL11 ft. on 11/30/2011.

(Continued Next Page)

Boring S-4

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 38.0 ft.

Notes:
+/- 4" of topsoil at ground surface
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Loose, brown and grey, clayey fine to medium SAND with trace rock fragments *(Continued from previous page)*											
	5.0													
35			... tan	☒	2-4-4 (8)									
	0.0		Very loose, brown and grey, fine to medium SAND with trace silt	☒	3-2-2 (4)							4.1		
40														
	-5.0		...medium dense	☒	12-8-10 (18)									
45														
	-10.0		... loose, light grey and tan (Coastal Plain Deposits)	☒	7-6-4 (10)									Borehole backfilled with grout upon completion.
50			Boring terminated at 50.0 feet.											
	-15.0													
55														
	-20.0													

Boring S-5

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/29/2011 Approx. Ground Elevation: 40.0 ft.

Notes:
+/- 4" of topsoil at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Medium dense, red, brown and tan, silty fine to medium SAND with trace organics		7-8-11 (19)									
5	35.0		... dark grey and light brown		6-5-6 (11)									
			...very loose, brown and grey		1-2-2 (4)									
10	30.0		...grey		1-2-2 (4)									
			... loose, grey and brown		4-3-3 (6)									
15	25.0		(Fill)											
			Medium, grey and brown CLAY with finesand		0-3-2 (5)									
20	20.0													
			Loose, light grey and tan, silty fine SAND		2-2-5 (7)									
25	15.0													
			... very loose, grey and tan		1-2-2 (4)									

(Continued Next Page)

Boring S-5

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/29/2011 Approx. Ground Elevation: 40.0 ft.

Notes:
+/- 4" of topsoil at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Very loose, grey and tan, silty fine SAND											
35	5.0		... loose, tan	☒	3-4-6 (10)									
			... medium dense	☒	5-6-9 (15)									
40	0.0													
			...same	☒	5-6-13 (19)									
45	-5.0													
			... same	☒	13-12-13 (25)									Borehole caved prior to groundwater measurement.Borehole backfilled with grout upon completion.
50	-10.0		(Coastal Plain Deposits)											
			Boring terminated at 50.0 feet.											
55	-15.0													

Boring S-6

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/13/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Loose, red, silty fine to medium SAND	☒	0-4-5 (9)									
5			...medium dense	☒	10-9-10 (19)									
	35.0		...very dense	☒	26-29-30 (59)									
10			... red and tan, with trace rock	☒	24-26-28 (54)									
	30.0													
15			... dense, tan and grey with rock fragments	☒	12-20-28 (48)									▽ GW at +/-EL28 ft. on 12/13/2011.
	25.0													
20			...very dense, tan	☒	14-34-36 (70)									
	20.0													
25			... red	☒	15-31-35 (66)									
	15.0		(Fill)											
			Stiff, red and grey CLAY with fine sand and rock fragments	☒	6-7-7 (14)								1.25	

(Continued Next Page)

Boring S-6

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/13/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Stiff, red and grey CLAY with fine sand and rock fragments *(Continued from previous page)*											
	10.0													
35			... same	☒	5-7-6 (13)								2.0	
	5.0													
40			... soft, light grey and tan	☒	3-3-3 (6)			41	17	24			0.5	USCS=CL
	0.0													
45			... medium, light grey and brown	☒	3-4-4 (8)								0.5	
	-5.0													
50			Medium dense, grey, silty fine SAND	☒	9-14-13 (27)									
	-10.0													
55			... grey and tan	☒	9-10-12 (22)									
	-15.0													
			...same (Coastal Plain Deposits)	☒	12-14-14 (28)									Borehole backfilled with grout upon completion.

Boring terminated at 60.0 feet.

Boring S-7

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Medium dense, silty fine to medium SAND with rock fragments		0-7-10 (17)			NP	NP	NP		27.7		USCS = SM
5			... red, brown and tan, with trace rock fragments		10-13-14 (27)									
	35.0		... very dense, reddish tan with numerous rock fragments		18-27-30 (57)									
10			... dense, reddish brown and tan with trace rock fragments		11-15-16 (31)			NP	NP	NP		24.9		USCS = SM
	30.0													
15			... medium dense, reddish tan with rounded rock fragments		5-6-11 (17)									▽ GW at +/-EL26.5 ft. on 11/30/2011.
	25.0													▼ GW at +/-EL24.5 ft. on 5/1/2012.
20			Medium dense, grey, clayey SAND with trace rock fragments		7-9-12 (21)			30	20	10		28.4		USCS = SC
	20.0			■										(No Recovery)
25			...very loose, grey and tan		2-2-2 (4)									
			(Fill)	■										
	15.0		Stiff, red and grey CLAY with fine sand		3-6-7 (13)								1.5	

(Continued Next Page)

Boring S-7

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/30/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Stiff, red and grey CLAY with fine sand											
	10.0			■										
35			...same	☒	8-11-13 (24)								2.0	
	5.0													
40			... light grey and brown	☒	3-5-7 (12)								1.5	
	0.0													
45			...grey and tan	☒	4-6-8 (14)								1.25	
	-5.0													
50			...same	☒	2-2-3 (5)									
	-10.0													
55			Very dense, silty fine to medium SAND with numerous rock fragments	☒	28-38-40 (78)									
	-15.0													
			... with rounded rock fragments (Coastal Plain Deposits)	☒	30-30-28 (58)									Piezometer Installed.

Boring terminated at 60.0 feet.

Boring S-8

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/12/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Dense, red and brown, silty fine to medium SAND with trace organics	☒	11-20-21 (41)									
5			...red	☒	17-15-17 (32)									
	35.0		... with trace rock fragments	☒	17-20-20 (40)									
10			...very dense, reddish tan with numerous rock fragments	☒	28-30-50 (80)									▽ GW at +/-EL32 ft. on 12/12/2011.
	30.0													▼ GW at +/-EL30 ft. on 12/14/2011.
15			... red and grey	☒	50/5"									
			(Fill)											
	25.0		Loose gravel fragments	☒	5-4-3 (7)							2.3		
20														
	20.0		... with clay	☒	3-4-3 (7)									
25														
	15.0		Stiff, red and grey CLAY with fine sand	☒	3-6-8 (14)								1.25	

(Continued Next Page)

Boring S-8

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/12/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Stiff, red and grey CLAY with fine sand											
	10.0													
35			...same	☒	3-4-5 (9)			71	21	50			1.0	USCS=CH
	5.0													
40			...same	☒	5-6-7 (13)								1.25	
	0.0													
45			Very dense, tan, silty fine SAND	☒	14-22-28 (50)									
	-5.0													
50			Dense, tan, clayey fine SAND	☒	12-20-20 (40)									
	-10.0													
55			Dense gravel fragments	☒	14-16-18 (34)									
	-15.0													Borehole backfilled with grout upon completion.
			...very dense, with fine sand (Coastal Plain Deposits)	☒	18-24-28 (52)									

Boring terminated at 60.0 feet.

Boring S-9

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/6/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Medium dense, red and brown, silty fine to medium SAND with trace organics											
5			... red	☒	19-14-14 (28)									
	35.0		... dense	☒	10-14-17 (31)									
10			... medium dense, red and tan with trace rock fragments	☒	11-12-13 (25)									▽ GW at +/-EL32.5 ft. on 12/6/2011.
	30.0													
15			... very dense, red (Fill)	☒	14-40-50 (90)									
	25.0		Very stiff, brown and grey, fine sandy CLAY	☒	9-13-20 (33)									
20														
	20.0		Loose, grey, silty fine SAND	☒	2-2-3 (5)									
25														
	15.0		Medium, grey CLAY with fine sand	☒	2-3-4 (7)									

(Continued Next Page)

Boring S-9

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/6/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Medium, grey CLAY with fine sand											
	10.0													(No Recovery)
35			... red and grey		2-3-3 (6)									
	5.0													(No Recovery)
40			... stiff		2-5-6 (11)									
	0.0													
45			...soft, grey		2-2-2 (4)									
	-5.0													
50			... same		2-4-5 (9)									
	-10.0													
55			...hard, grey and tan		40-50-6 (56)									
	-15.0													Borehole backfilled with grout upon completion.
			Very dense, tan, clayey fine to medium SAND with rock fragments (Coastal Plain Deposits)		30-36-40 (76)									

Boring terminated at 60.0 feet.

Boring S-10

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/6/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Dense, red and black, silty fine to medium SAND	☒	0-17-23 (40)									
5			... same	☒	13-23-24 (47)									
	35.0		... red	☒	18-19-20 (39)									
10			...very dense	☒	26-25-30 (55)									▽ GW at +/-EL32.5 ft. on 12/6/2011.
	30.0		... with rock fragments	☒	11-24-28 (52)									
15														
	25.0		... same	☒	18-23-29 (52)									
20			(Fill)											
	20.0		Medium dense, brown, silty fine to medium SAND	☒	9-9-8 (17)									
25														
	15.0			■										(No Recovery)
			Stiff, grey and red CLAY with fine sand	☒	3-4-5 (9)								1.0	

(Continued Next Page)

Boring S-10

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/6/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Stiff, grey and red CLAY with fine sand											
	10.0			■										(No Recovery)
			... grey	☒	2-4-5 (9)			74	22	52			1.5	USCS=CH
35														
	5.0			■										(No Recovery)
			... same	☒	4-5-7 (12)								1.0	
40														
	0.0													
			... same	☒	5-6-7 (13)								1.5	
45														
	-5.0													
			... same	☒	4-5-5 (10)								1.75	
50														
	-10.0													
			... soft	☒	2-2-3 (5)								0.5	
55														
	-15.0													
			Dense, light brown and tan, silty fine to medium SAND (Coastal Plain Deposits)	☒	15-18-31 (49)									Borehole backfilled with grout upon completion.

Boring terminated at 60.0 feet.

Boring S-11

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/8/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Medium dense, red and black, silty fine to medium SAND with rock fragments	☒	0-0-18 (18)									
5			...dense, red	☒	29-21-24 (45)									
	35.0		...medium dense	☒	18-15-13 (28)									
10			...very dense, red and tan, with numerous rock fragments	☒	28-30-31 (61)									▽ GW at +/-EL32.5 ft. on 12/8/2011.
	30.0													
15			...dense	☒	10-23-23 (46)									
	25.0													
20			... very dense, reddish tan	☒	14-28-30 (58)									
	20.0													
25			... dense, red and tan with numerous rock fragments	☒	14-17-18 (35)									▽ GW at +/-EL17 ft. on 5/1/2012.
	15.0													
			... medium dense, red	☒	8-16-14 (30)									

(Continued Next Page)

Boring S-11

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/8/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface.

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Medium dense, red, silty fine to medium SAND with numerous rock fragments (Fill)											
	10.0													
35			Stiff, red and grey CLAY with fine sand	☒	6-9-9 (18)			68	22	46				USCS=CH
	5.0			■										
			...grey and tan	☒	6-8-9 (17)									
40				■										
	0.0													
			Medium dense, grey and tan, silty fine SAND	☒	10-14-15 (29)									
45														
	-5.0													
			... same	☒	6-6-12 (18)							24.7		USCS=SM
50														
	-10.0													
			... tan with rock fragmnets	☒	5-9-10 (19)									
55														
	-15.0													Piezometer Installed.
			... grey and tan (Coastal Plain Deposits)	☒	10-10-11 (21)									

Boring terminated at 60.0 feet.

Boring S-12

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/5/2011 Approx. Ground Elevation: 42.5 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Very dense, red and black, silty fine to medium SAND with rock fragments	☒	34-40-50 (90)			NP	NP	NP		30.2		USCS = SM
5			... red	☒	23-35-35 (70)									
	35.0		... with trace rock fragments	☒	20-31-25 (56)									
10			Very dense, red and tan, fine to medium SAND with trace silt	☒	20-27-30 (57)			NP	NP	NP		8.4		USCS = SP-SM GW at +/-EL33 ft. on 12/6/2011.
	30.0		...medium dense, reddish tan with trace rock fragments	☒	10-16-20 (36)									
15														
	25.0		Dense, red and grey, clayey fine to medium SAND	☒	11-21-22 (43)									▽ GW at +/-EL23 ft. on 12/13/2011.
20														
	20.0		...medium dense, red	☒	5-11-16 (27)									
25			(Fill)											
	15.0		Stiff, grey CLAY with trace fine sand	☒	4-5-6 (11)			67	24	43		97.7	1.25	USCS = CH

(Continued Next Page)

Boring S-12

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 12/5/2011 Approx. Ground Elevation: 42.5 ft.

Notes:
+/- 18" of railroad ballast at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Stiff, grey CLAY with trace fine sand											
	10.0													
35			... medium	☒	3-3-6 (9)								0.5	
	5.0													
40			Loose, grey, silty fine to medium SAND	☒	2-3-3 (6)			NP	NP	NP		29.7		USCS = SM
	0.0													
45			... medium dense, light grey and tan with rock fragments	☒	7-9-10 (19)									
	-5.0													
50			... same	☒	5-6-7 (13)									
	-10.0													
55			Medium dense, light grey and tan, fine to medium SAND with trace silt	☒	9-15-12 (27)			NP	NP	NP		6.8		USCS = SP-SM
	-15.0													Borehole backfilled with grout upon completion.
			... tan (Coastal Plain Deposits)	☒	5-8-6 (14)									

Boring terminated at 60.0 feet.

Boring S-13

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/28/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 12" of topsoil at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Medium dense, red and black, silty fine to medium SAND		6-7-7 (14)									
5			...very loose, tan and red		5-1-3 (4)									
	35.0		... medium dense, reddish brown		5-5-6 (11)									
10			Stiff, red and grey CLAY with fine sand		3-4-8 (12)								1.0	
	30.0		Medium dense, red and grey, silty fine to medium SAND (Fill)		9-6-7 (13)									
15	25.0		Loose, light brown, silty fine to medium SAND		2-2-3 (5)									
20	20.0		... light brown and tan		2-3-4 (7)									
25	15.0		... same		2-3-4 (7)									▼ GW at +/-EL12.5 ft. on 5/1/2012.

(Continued Next Page)

Boring S-13

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/28/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 12" of topsoil at ground surface.
PPqu = Pocket Penetrometer unconfined compressive strength

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Loose, light brown and tan, silty fine to medium SAND											
	10.0													
35			...very loose	☒	3-2-2 (4)									
	5.0													
40			...loose, light brown and tan	☒	3-3-5 (8)									
	0.0													
45			... medium dense	☒	6-7-7 (14)									
	-5.0													
50			...loose	☒	2-3-3 (6)									
			Boring terminated at 50.0 feet.											
	-10.0													
55														
	-15.0													

Piezometer Installed.

Boring S-14

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/29/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 12" of topsoil at ground surface.

PPqu = Pocket Penetrometer unconfined compressive strength"

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0		Medium dense, red, silty fine to medium SAND with rock fragments	☒	5-6-8 (14)									
5			... with trace rock fragments	☒	5-7-8 (15)									
	35.0		... same	☒	5-5-7 (12)									
10			... red and tan	☒	8-8-7 (15)									
	30.0		... same	☒	7-9-14 (23)									
15			(Fill)											
	25.0		Very soft, grey and tan, CLAY with fine sand	☒	1-1-1 (2)									
20														
	20.0		... medium, grey	☒	2-2-4 (6)									
25														
	15.0		Loose, brown, silty fine SAND	☒	2-3-4 (7)									

(Continued Next Page)

Boring S-14

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: 3.25"-ID HSA
Date Drilled: 11/29/2011 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 12" of topsoil at ground surface.

PPqu = Pocket Penetrometer unconfined compressive strength"

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			Loose, brown, silty fine SAND	■										(No Recovery)
	10.0													
35			... brown, tan and grey	☒	2-3-5 (8)									
	5.0													
40			Soft, grey and tan CLAY with fine sand	☒	1-2-4 (6)								0.25	
	0.0													
45			Medium dense, tan, silty fine to medium SAND	☒	5-9-10 (19)									
	-5.0													
50			...loose	☒	2-3-4 (7)									
	-10.0													Borehole caved prior to groundwater measurement.Borehole backfilled with grout upon completion.
55			...same (Coastal Plain Deposits)	☒	3-3-5 (8)									
			Boring terminated at 55.0 feet.											
	-15.0													

Boring T-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/3/2016 Approx. Ground Elevation: 39.0 ft.

Notes:
No topsoil present at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
5	35.0		Loose, reddish brown, silty fine to medium SAND with rock fragment	☒	3-3-2 (5)									
10	30.0		Very soft, brown, fine sandy CLAY with rock fragment	☒	2-2-2 (4)								<0.25	
15	25.0		...soft	☒	2-2-2 (4)			29	18	11		72.3	0.5	USCS=CL
			(Fill)											
20	20.0		Soft, brown, sandy CLAY with trace organics	☒	2-3-4 (7)								0.5	
25	15.0		...very soft	☒	1-0-0 (WOH)			42	21	21		77.8	0.25	USCS=CL

(Continued Next Page)

Boring T-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/3/2016 Approx. Ground Elevation: 39.0 ft.

Notes:
No topsoil present at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...very soft *(Continued from previous page)*											
30	10.0		...with fine sand		0-0-0 (WOH)								0.25	
35	5.0		Very loose, gray and tan, fine sandy SILT		1-1-1 (2)			NP	NP	NP		54.9		USCS=ML
40	0.0		Very soft, gray and brown, fine sandy CLAY		0-0-0 (WOH)			30	21	9		66.8	0.25	USCS=CL
45	-5.0		Loose, gray, silty fine SAND		4-4-4 (8)									
50	-10.0		...medium dense		5-7-11 (18)									

(Continued Next Page)

Boring T-1

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/3/2016 Approx. Ground Elevation: 39.0 ft.

Notes:
No topsoil present at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...medium dense *(Continued from previous page)*											
55	-15.0		...same	☒	5-3-8 (11)									
60	-20.0		...loose, with few pebbles (Coastal Plain Deposits)	☒	3-2-7 (9)									Borehole backfilled with grout upon completion.
			Boring terminated at 60.0 feet.											
65	-25.0													
70	-30.0													
75	-35.0													

Boring T-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/9/2016 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ◧ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0													
5			Dense, red and tan, silty fine to medium SAND with rock fragments	☒	11-27-23 (50)									
	35.0													
10			...medium dense	☒	7-7-8 (15)									
	30.0													
15			...same	☒	1-12-14 (26)			NP	NP	NP		20.1		USCS=SM Small amount of Costal Plain Deposits observed in sample
	25.0		(Fill)											
20			Dense, gray, silty fine to medium SAND	☒	8-17-18 (35)									
	20.0													
25			...loose	☒	3-3-6 (9)			NP	NP	NP		14.7		USCS=SM

(Continued Next Page)

Boring T-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/9/2016 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	15.0		...loose *(Continued from previous page)*											
30			Stiff, light gray and brown, plastic CLAY with fine sand	☒	3-3-4 (7)								1.25	
	10.0													
35			...same	☒	4-4-6 (10)								1.25	
	5.0													
40			...trace sand	☒	4-5-5 (10)			70	25	45		97.6	1.0	USCS=CH
	0.0													
45			...same	☒	5-4-5 (9)								1.25	
	-5.0													
50													0.75	
			...medium, with trace organics	☒	3-3-3 (6)									

(Continued Next Page)

Boring T-2

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/9/2016 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...medium, with trace organics *(Continued from previous page)*											
55			...light gray, with trace organics	☒	4-2-3 (5)								0.5	
	-15.0													
			Dense, tan, silty fine to medium SAND with few rock fragments (Coastal Plain Deposits)	☒	16-20-18 (38)									Borehole backfilled with grout upon completion.
60			Boring terminated at 60.0 feet.											
	-20.0													
65														
	-25.0													
70														
	-30.0													
75														
	-35.0													

Boring T-3

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/10/2016 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	40.0													
5			Medium dense, red, silty fine to medium SAND	☒	3-6-14 (20)									
	35.0													
10			...same	☒	8-12-11 (23)									
	30.0													
15			Dense, red, silty fine to coarse SAND with rock fragments	☒	11-16-15 (31)			NP	NP	NP		10.3		USCS=SP-SM
	25.0													
20			...medium dense	☒	11-15-12 (27)									
	20.0													
25			...dense	☒	15-17-20 (37)									

(Continued Next Page)

Boring T-3

Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/10/2016 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
	15.0		...dense *(Continued from previous page)*											
30			...medium dense	☒	10-9-7 (16)									
	10.0		(Fill)	☒	1-1-1 (2)			66	22	44		84.4	<0.25	USCS=CH
35			Very soft, gray, plastic CLAY with trace of root fragment											
	5.0													
40			...soft	☒	2-3-4 (7)								0.50	
	0.0		Loose, gray, silty fine SAND											
45			...medium dense	☒	5-4-7 (11)									
	-5.0													
50			...same	☒	6-7-8 (15)									

(Continued Next Page)

Boring T-3

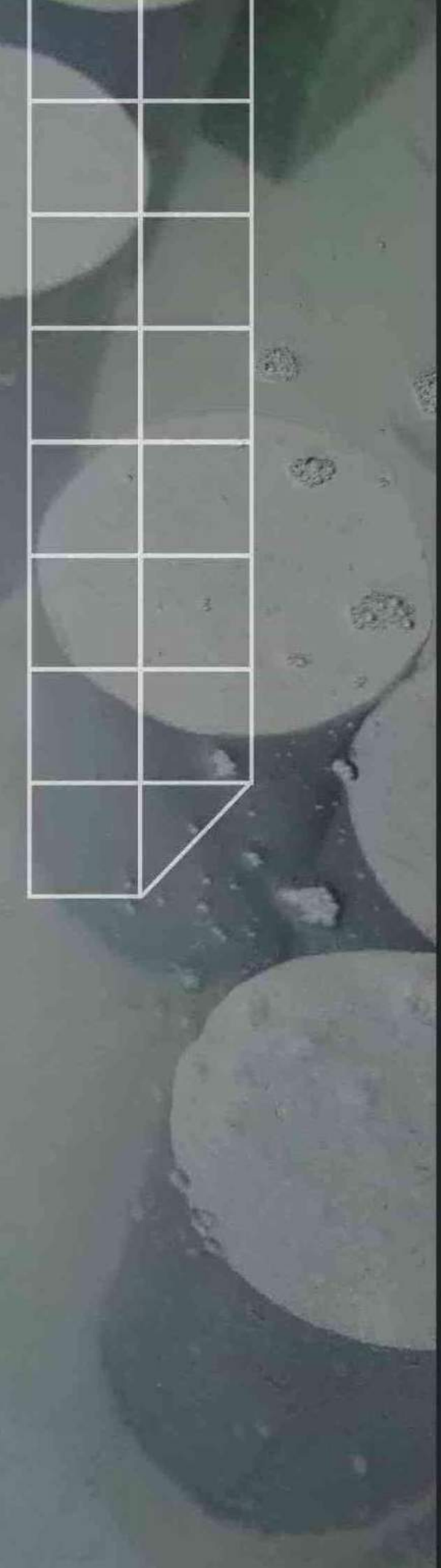
Engineering. Environmental. Answers.

Project Name: Periodic Safety Factor Assessment - Lowman Power Plant
Project Location: Leroy, Alabama Hammer Type: Automatic
CDG Project Number: R021221021 Method: Diedrich D-50 Mud Rotary
Date Drilled: 8/10/2016 Approx. Ground Elevation: 42.0 ft.

Notes:
+/- 18" of railroad ballast at ground surface

☒ - Split Spoon Sample ■ - Undisturbed Sample ▮ - Rock Core

Depth (ft.)	Approx. Elev. (ft.)	Graphic Log	Material Description	Type	Blows/6" (N-Value)	$N_1$60	Rec.% (RQD)	LL	PL	PI	MC	Fines (%)	PPqu (tsf)	Remarks
			...same *(Continued from previous page)*											
55				☒	8-10-12 (22)									
	-15.0													
			...same	☒	8-11-11 (22)									Borehole backfilled with grout upon completion.
60			(Coastal Plain Deposits)											
			Boring terminated at 60.0 feet.											
	-20.0													
65														
	-25.0													
70														
	-30.0													
75														
	-35.0													



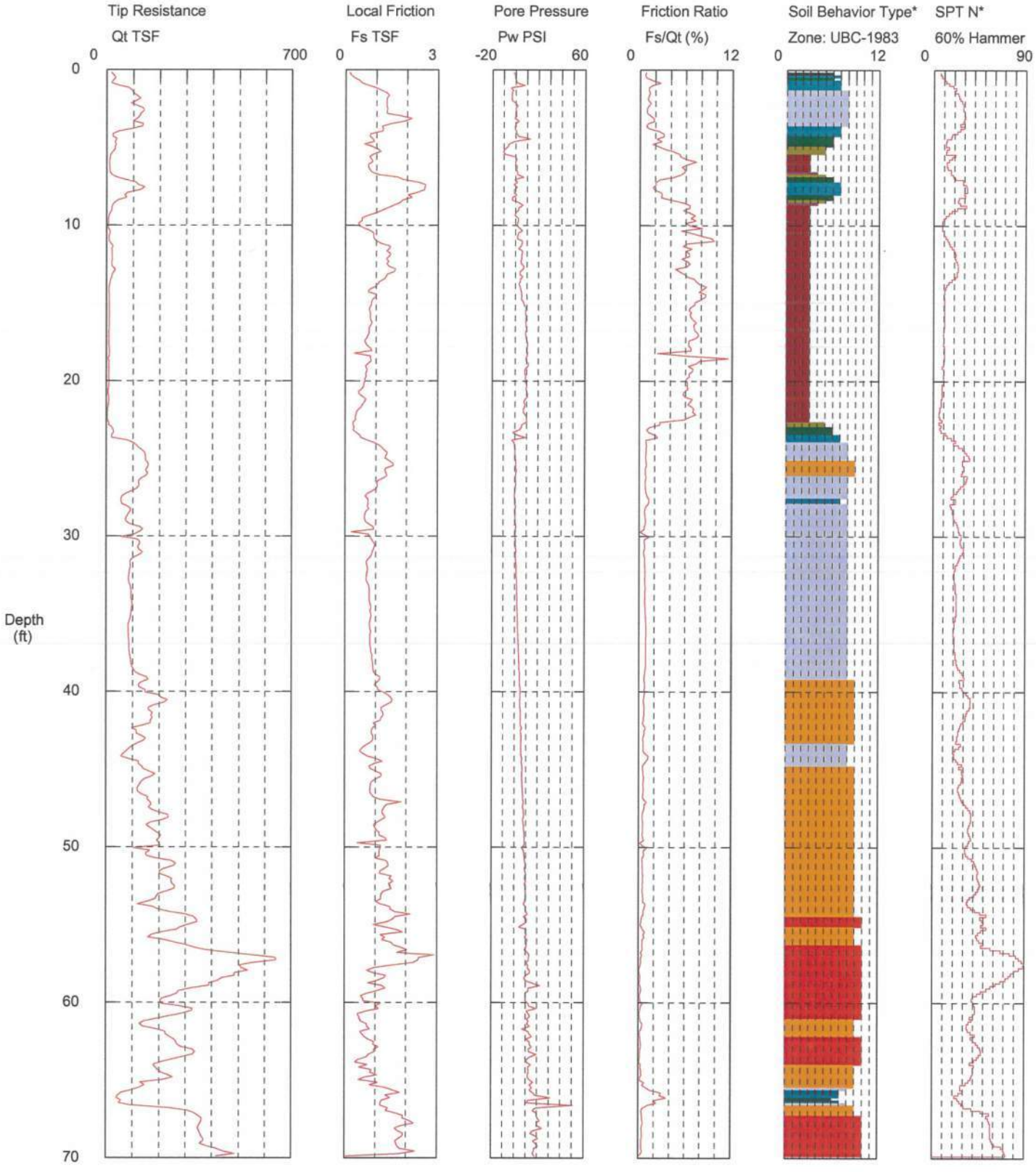
APPENDIX C

Cone Penetration Test Logs

SOUTHERN EARTH SCIENCES, INC.

Operator: DANNY HINES
Sounding: SCPT-1
Cone Used: DDG0892

CPT Date/Time: 11/29/2011 10:34:38 AM
Location: LOWMAN POWER PLANT
Job Number: M11-372



Maximum Depth = 69.88 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

N31.48424 W87.91472
SOUTHERN EARTH SCIENCES

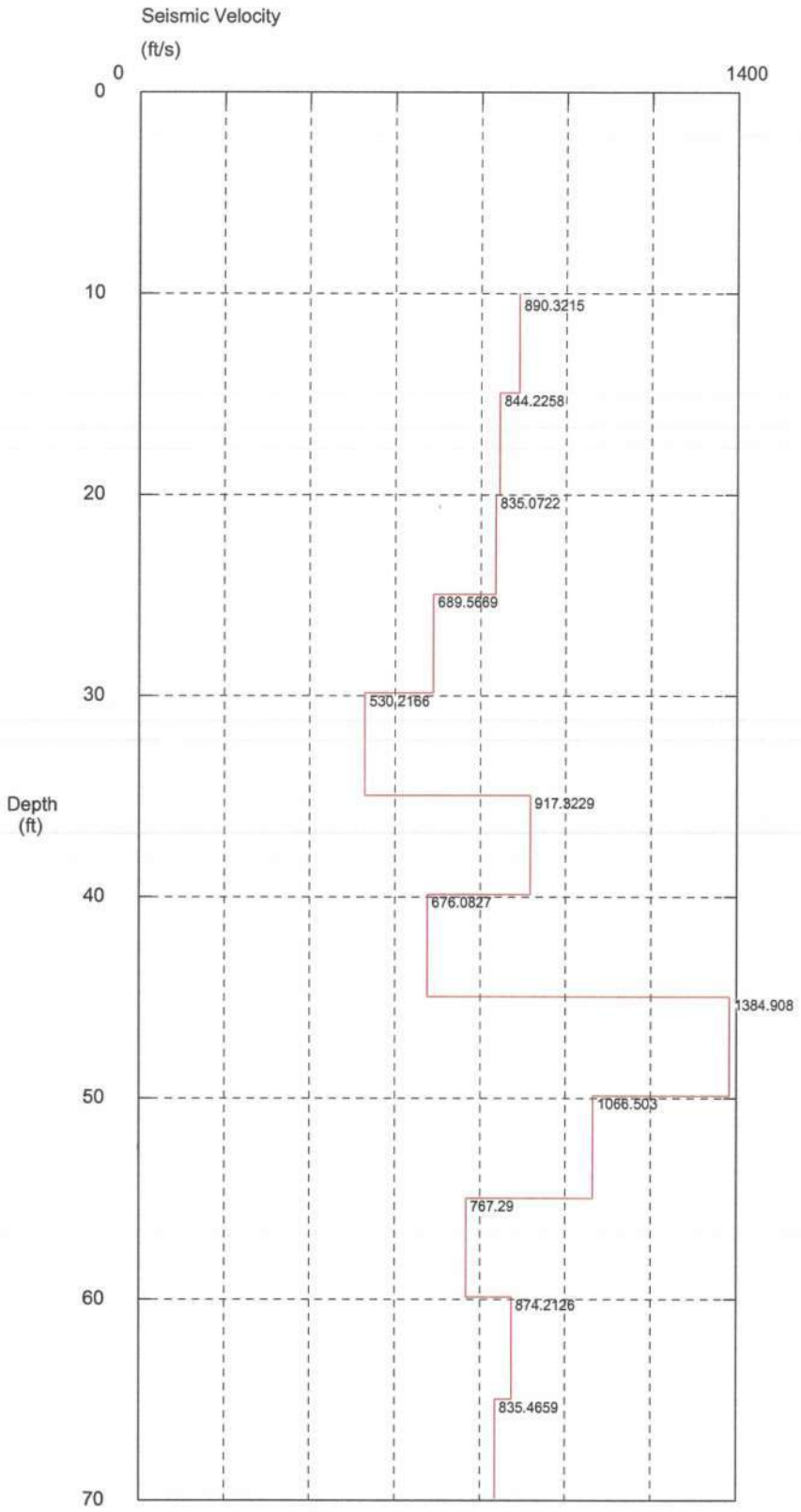
GROUNDWATER: SOUNDING WET AND COLLAPSED AT 0.5 FEET

*Soil behavior type and SPT based on data from UBC-1983

SOUTHERN EARTH SCIENCES, INC.

Operator: DANNY HINES
Sounding: SCPT-1
Cone Used: DDG0892

CPT Date/Time: 11/29/2011 10:34:38 AM
Location: LOWMAN POWER PLANT
Job Number: M11-372



Maximum Depth = 69.88 feet

Depth Increment = 0.164 feet

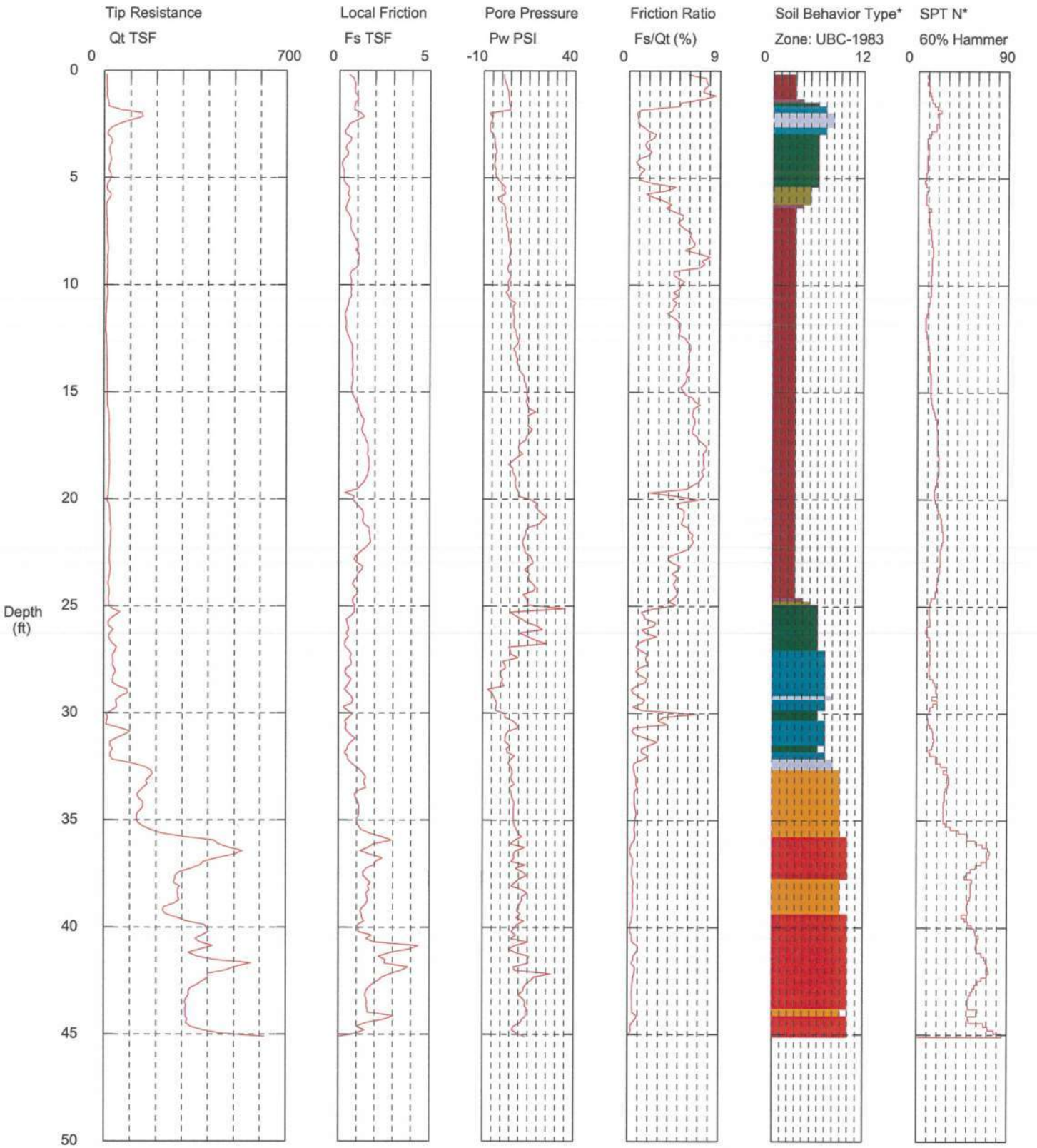
N31.48424 W87.91472
SOUTHERN EARTH SCIENCES

GROUNDWATER: SOUNDING WET AND COLLAPSED AT 0.5 FEET

SOUTHERN EARTH SCIENCES, INC.

Operator: DANNY HINES
Sounding: SCPT-2
Cone Used: DDG0892

CPT Date/Time: 11/29/2011 12:02:16 PM
Location: LOWMAN POWER PLANT
Job Number: M11-372



Maximum Depth = 45.11 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

N31.48615 W87.91843
SOUTHERN EARTH SCIENCES

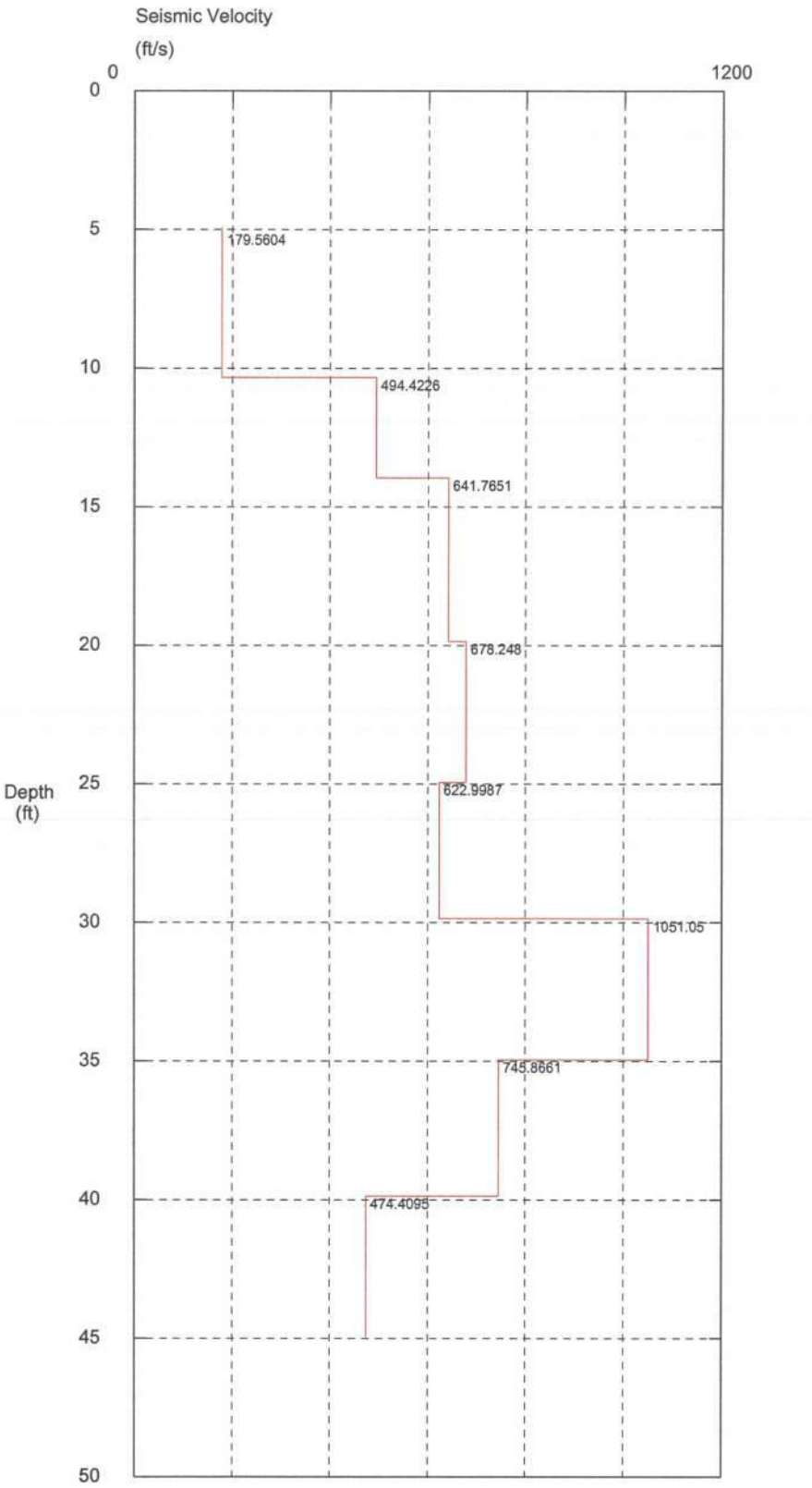
GROUNDWATER:

*Soil behavior type and SPT based on data from UBC-1983

SOUTHERN EARTH SCIENCES, INC.

Operator: DANNY HINES
Sounding: SCPT-2
Cone Used: DDG0892

CPT Date/Time: 11/29/2011 12:02:16 PM
Location: LOWMAN POWER PLANT
Job Number: M11-372



Maximum Depth = 45.11 feet

Depth Increment = 0.164 feet

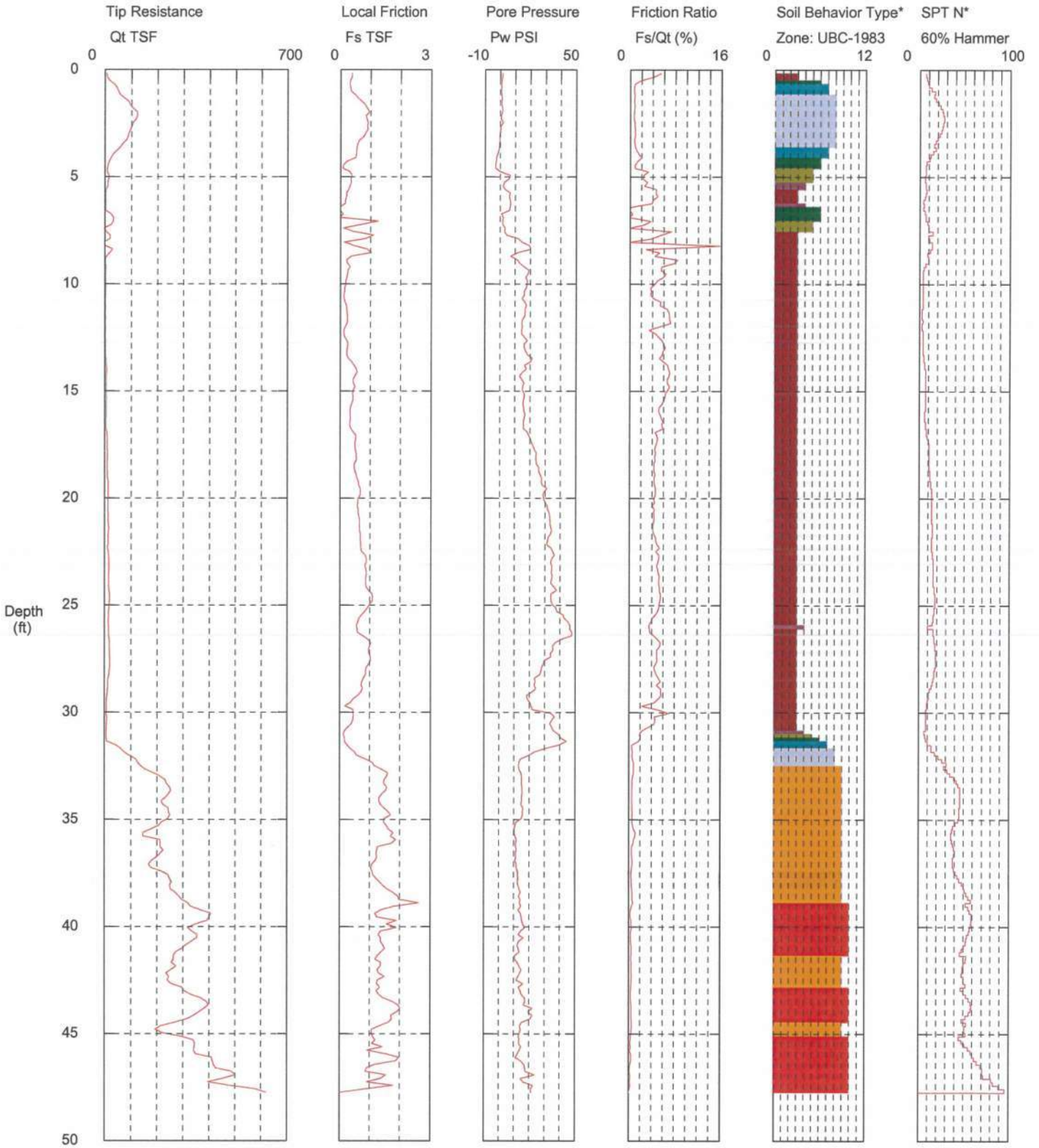
N31.48615 W87.91843
SOUTHERN EARTH SCIENCES

GROUNDWATER:

SOUTHERN EARTH SCIENCES, INC.

Operator: DANNY HINES
Sounding: SCPT-3A
Cone Used: DDG0892

CPT Date/Time: 11/29/2011 2:20:34 PM
Location: LOWMAN POWER PLANT
Job Number: M11-372



Maximum Depth = 47.74 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

N31.49008 W87.91789
SOUTHERN EARTH SCIENCES

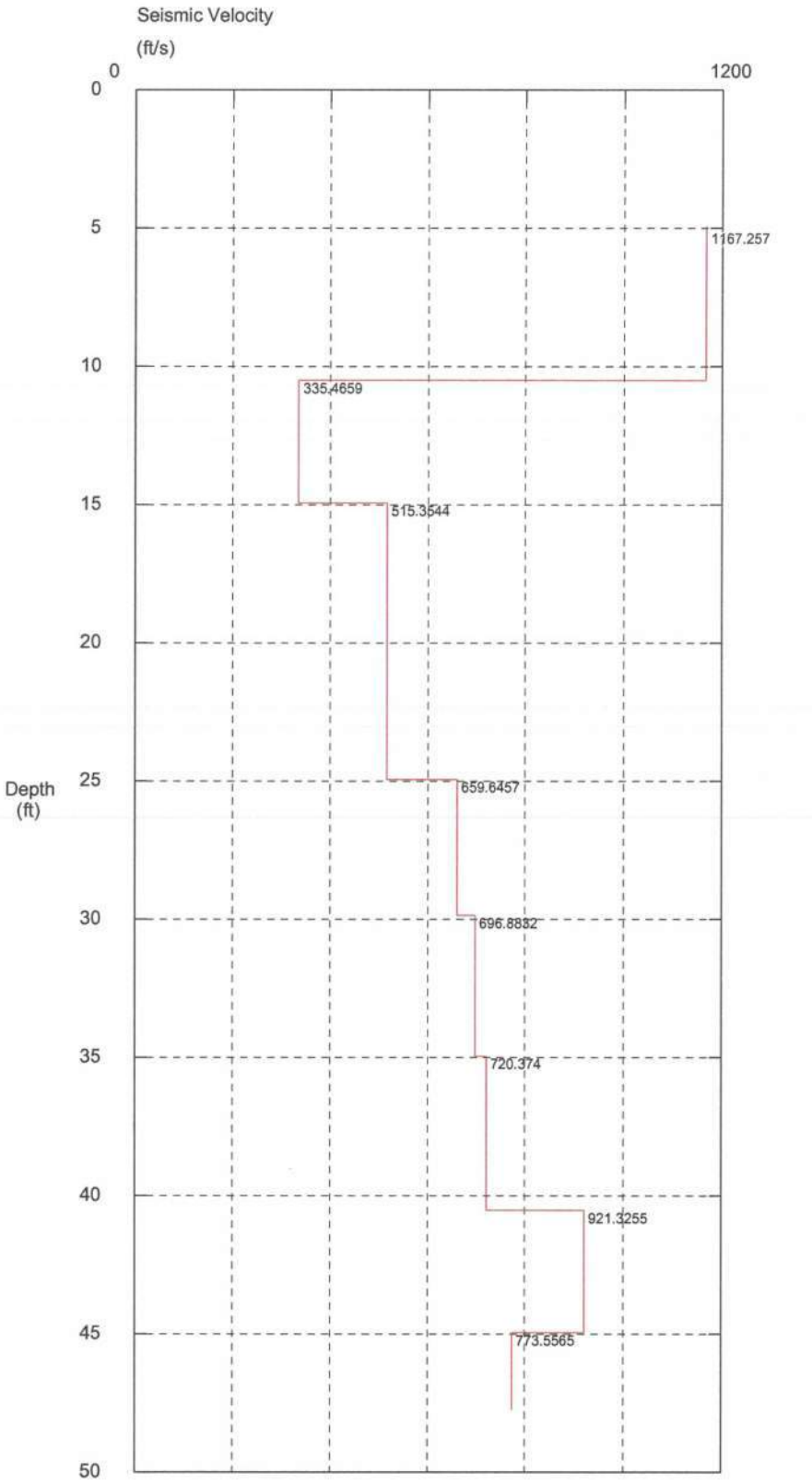
GROUNDWATER:

*Soil behavior type and SPT based on data from UBC-1983

SOUTHERN EARTH SCIENCES, INC.

Operator: DANNY HINES
Sounding: SCPT-3A
Cone Used: DDG0892

CPT Date/Time: 11/29/2011 2:20:34 PM
Location: LOWMAN POWER PLANT
Job Number: M11-372



Maximum Depth = 47.74 feet

Depth Increment = 0.164 feet

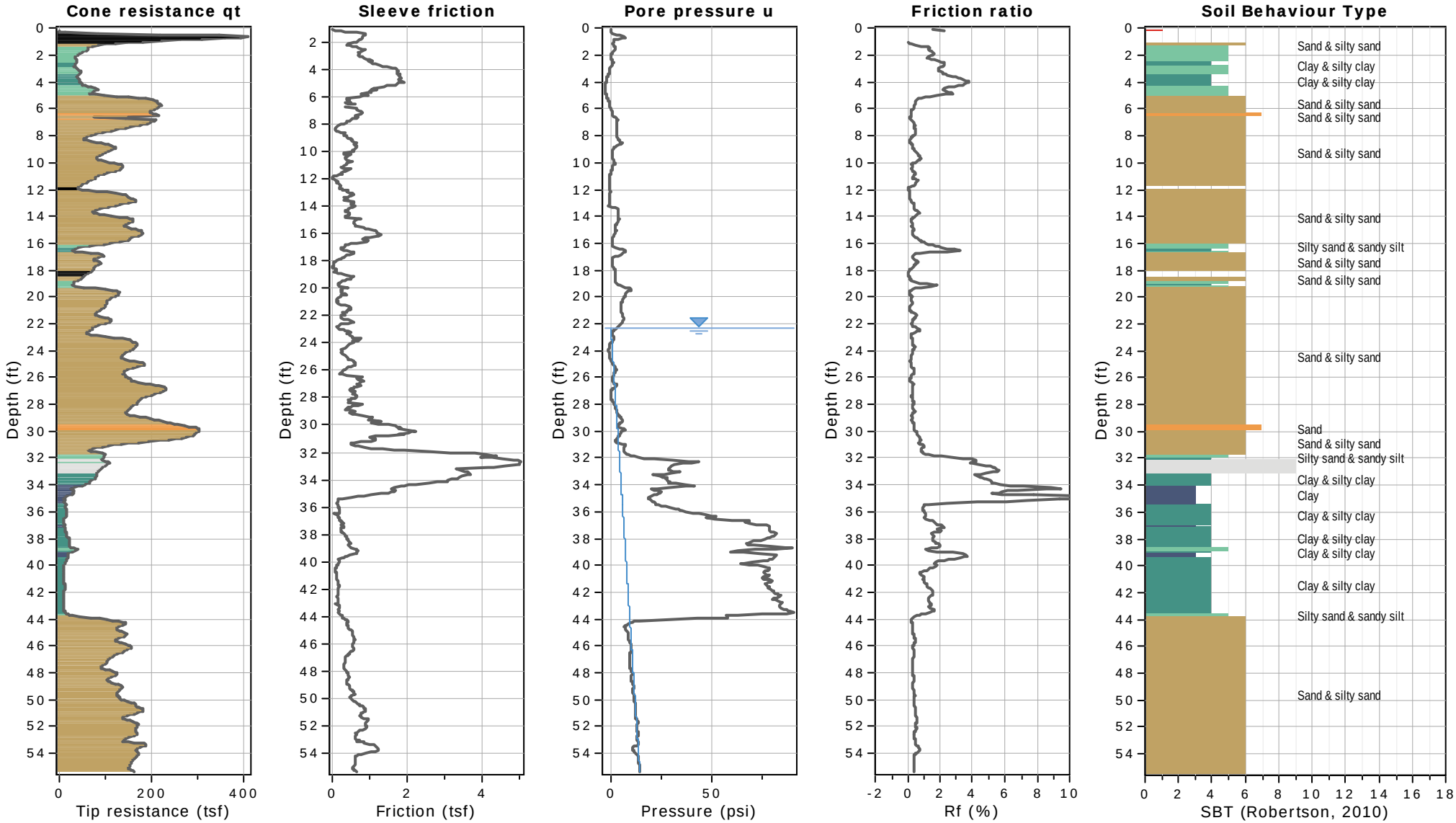
N31.49008 W87.91789
SOUTHERN EARTH SCIENCES

GROUNDWATER:

Walker-Hill Environmental
Chris Hayslip
chris@whenv.com
www.whenv.com

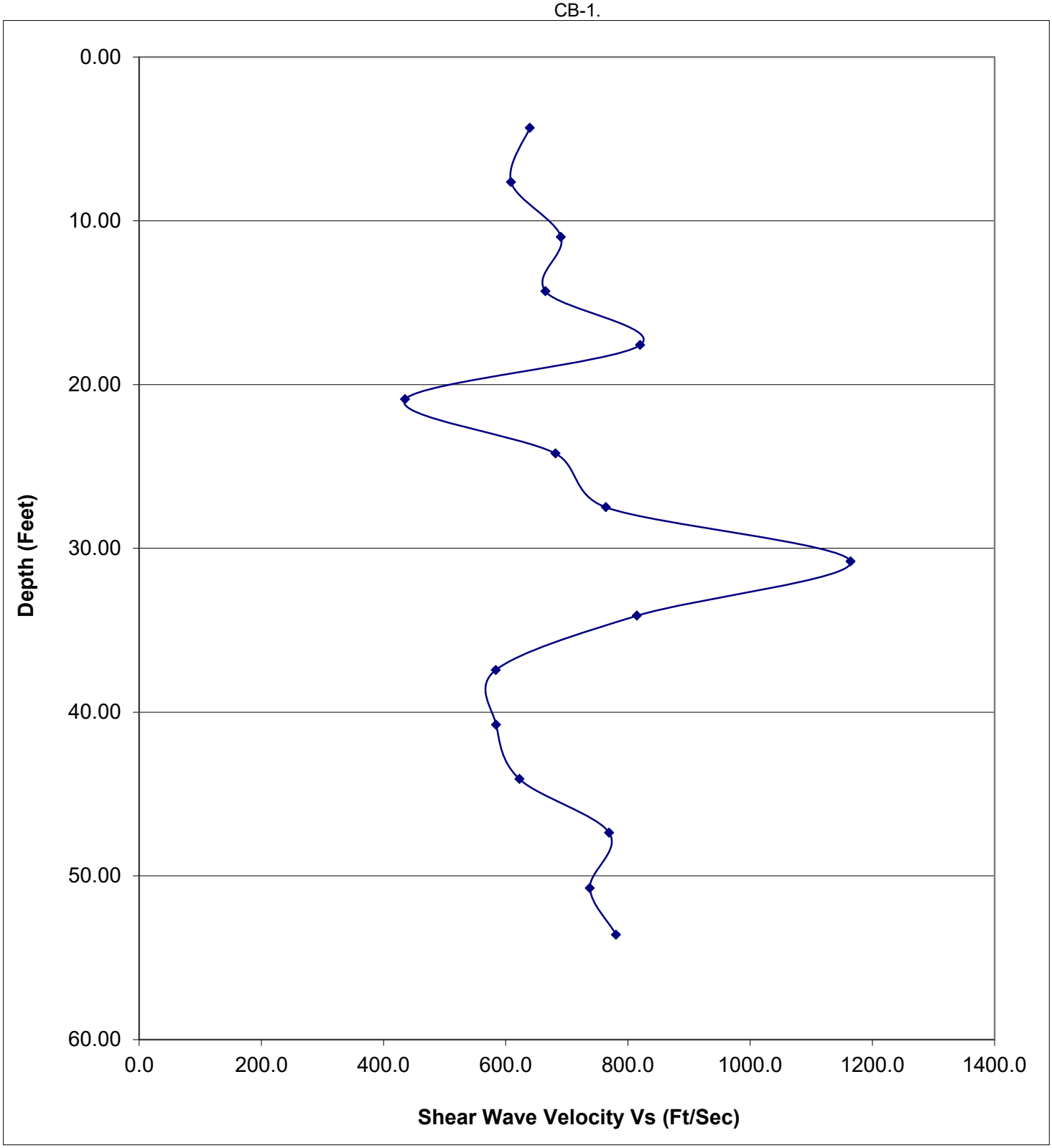
Project: **H20019-37AL**
Location: **Leroy, AL**

CPT: H20019-37LACB-1.
Total depth: 55.38 ft, Date: 6/23/2020
Cone Type: Gregg 15
Cone Operator: Cody Cliburn



Shear Wave Velocity Profile

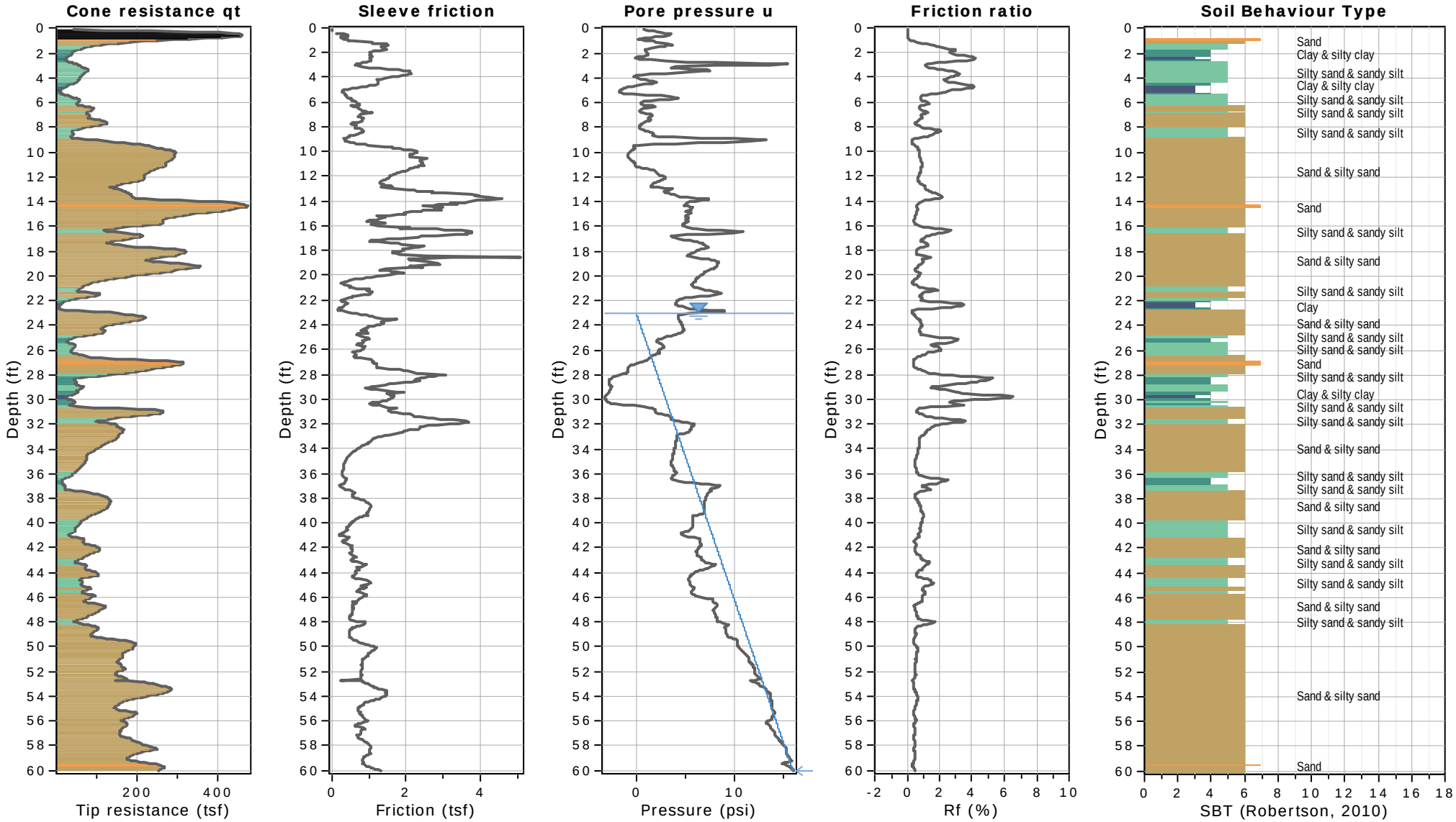
PSEC-LOWMAN POND CLOSURE
H20019-37LA
CB-1.



Walker-Hill Environmental
Chris Hayslip
chris@whenv.com
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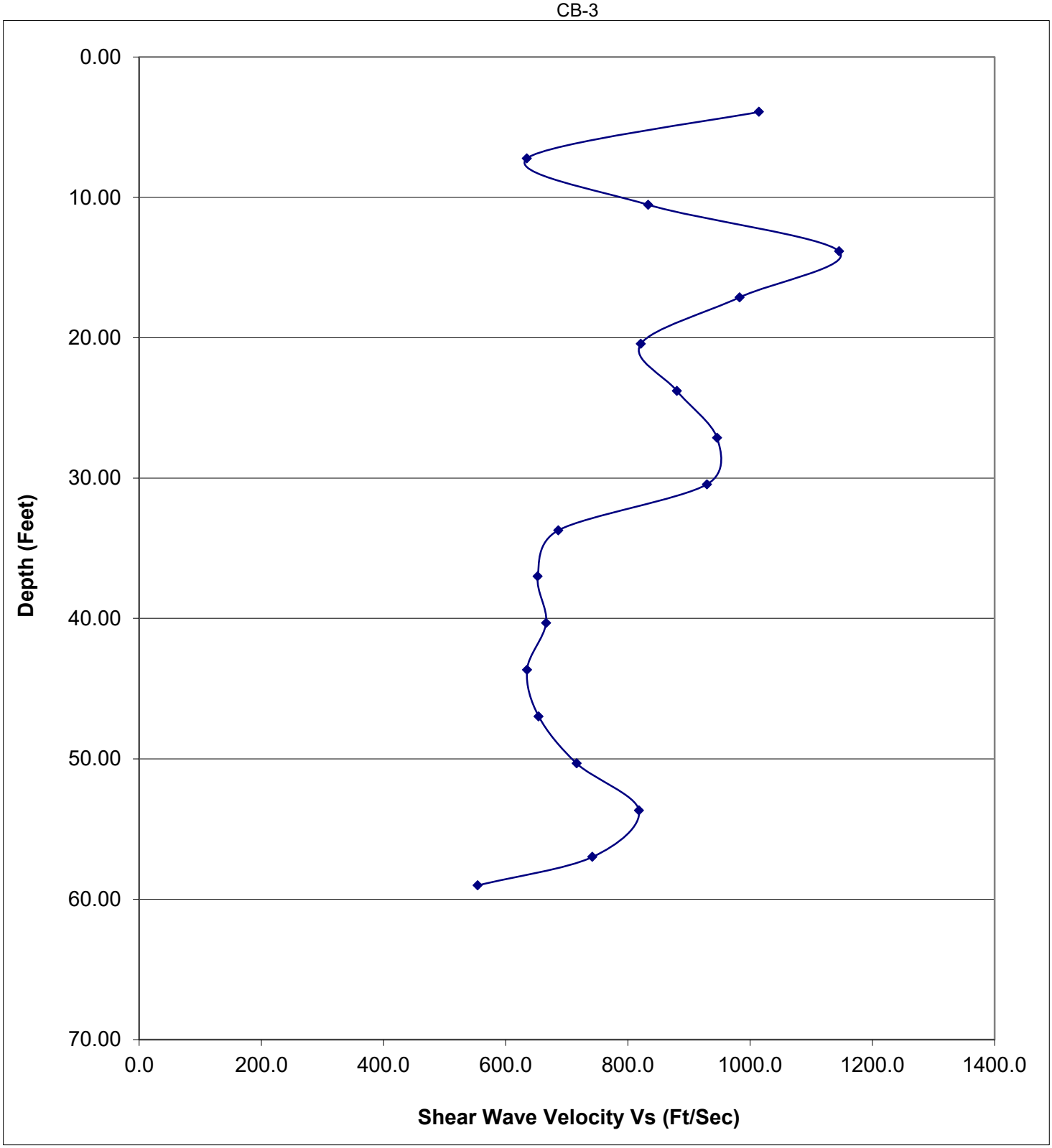
Project: H20019-37AL
Location: Leroy, AL

CPT: H20019-37LACB-3
Total depth: 60.04 ft, Date: 6/23/2020
Cone Type: Gregg 15
Cone Operator: Cody Cliburn



Shear Wave Velocity Profile

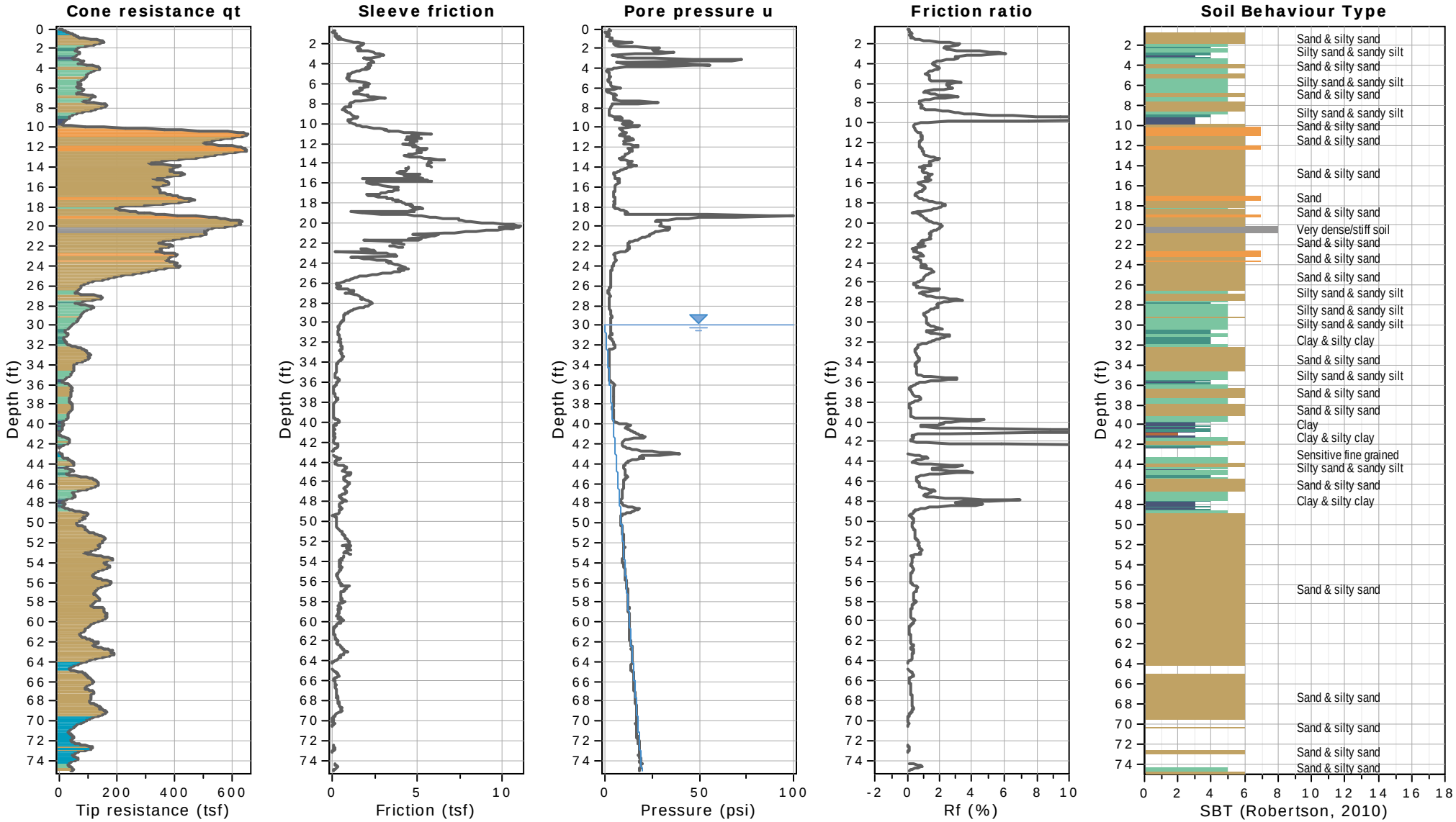
PSEC-LOWMAN POND CLOSURE
H20019-37LA
CB-3



Walker-Hill Environmental
Chris Hayslip
chris@whenv.com
www.whenv.com

Project: **H20019-37AL**
Location: **Leroy, AL**

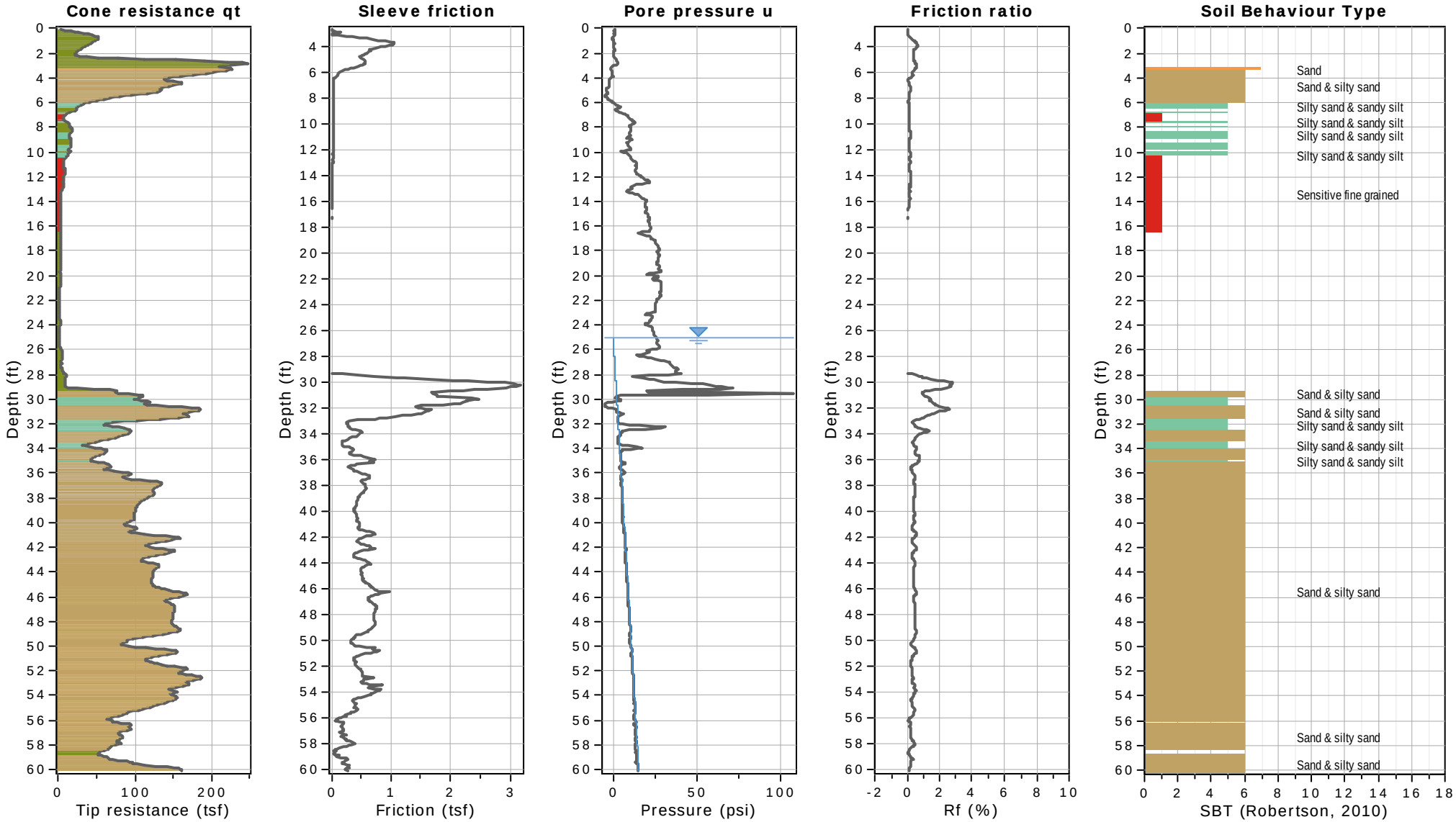
CPT: H20019-37LACB-4
Total depth: 75.07 ft, Date: 6/25/2020
Cone Type: Gregg 15
Cone Operator: Cody Cliburn



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Chris Hayslip
chris@whenv.com
www.whenv.com

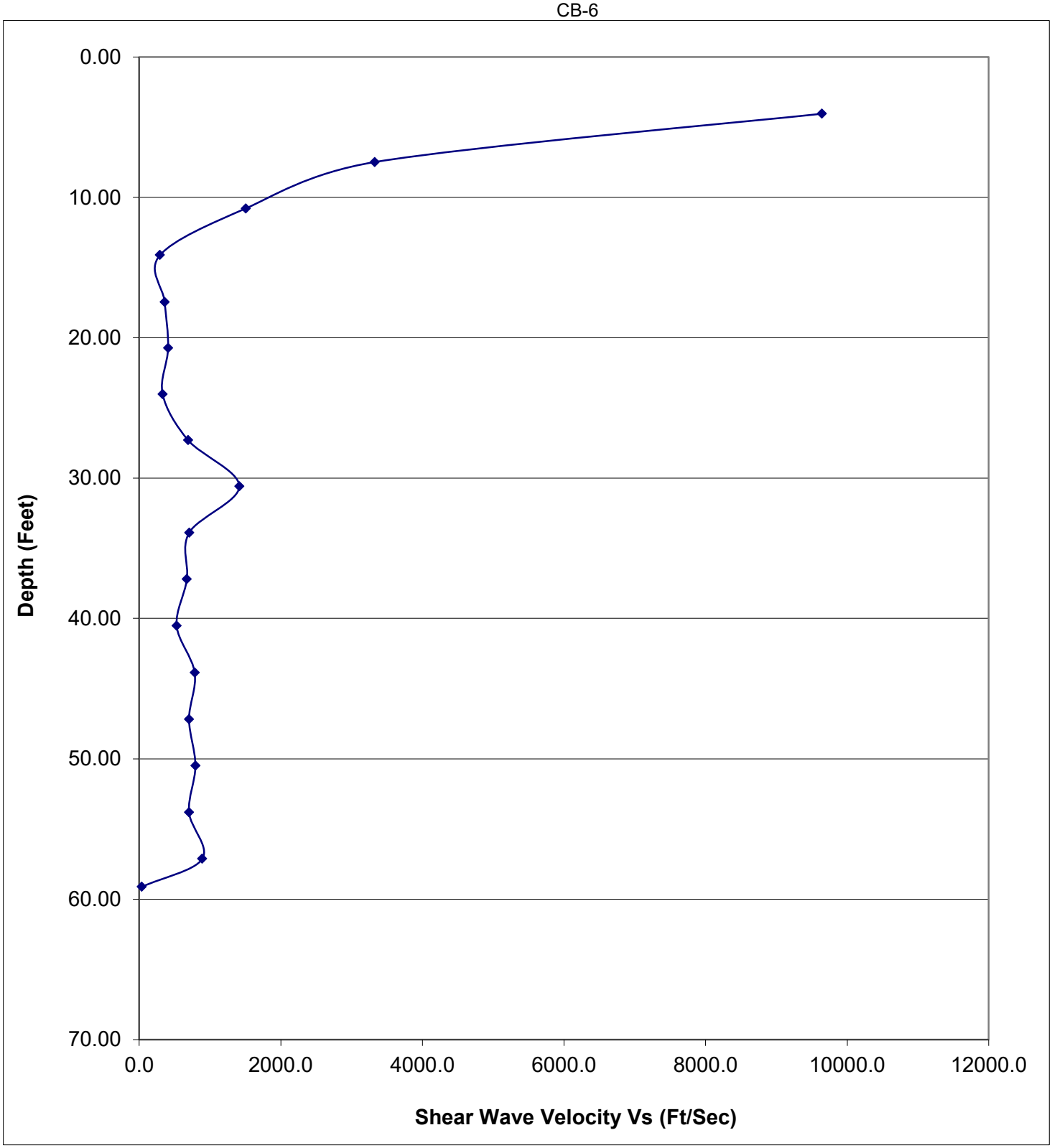
Project: H20019-37AL
Location: Leroy, AL

CPT: H20019-37LACB-6
Total depth: 60.10 ft, Date: 6/24/2020
Cone Type: Gregg 15
Cone Operator: Cody Cliburn



Shear Wave Velocity Profile

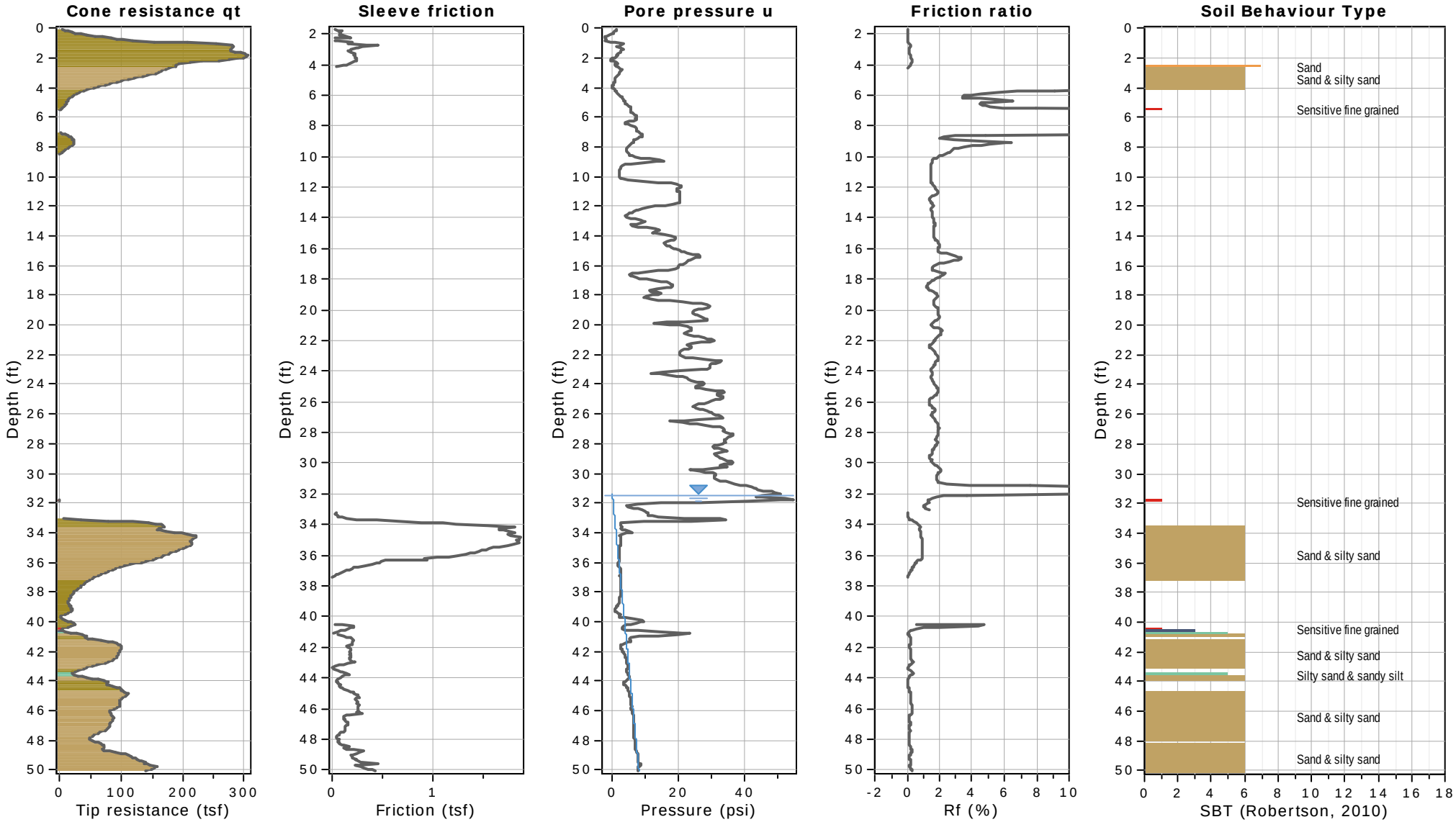
PSEC-LOWMAN POND CLOSURE
H20019-37LA
CB-6



Walker-Hill Environmental
Chris Hayslip
chris@whenv.com
www.whenv.com

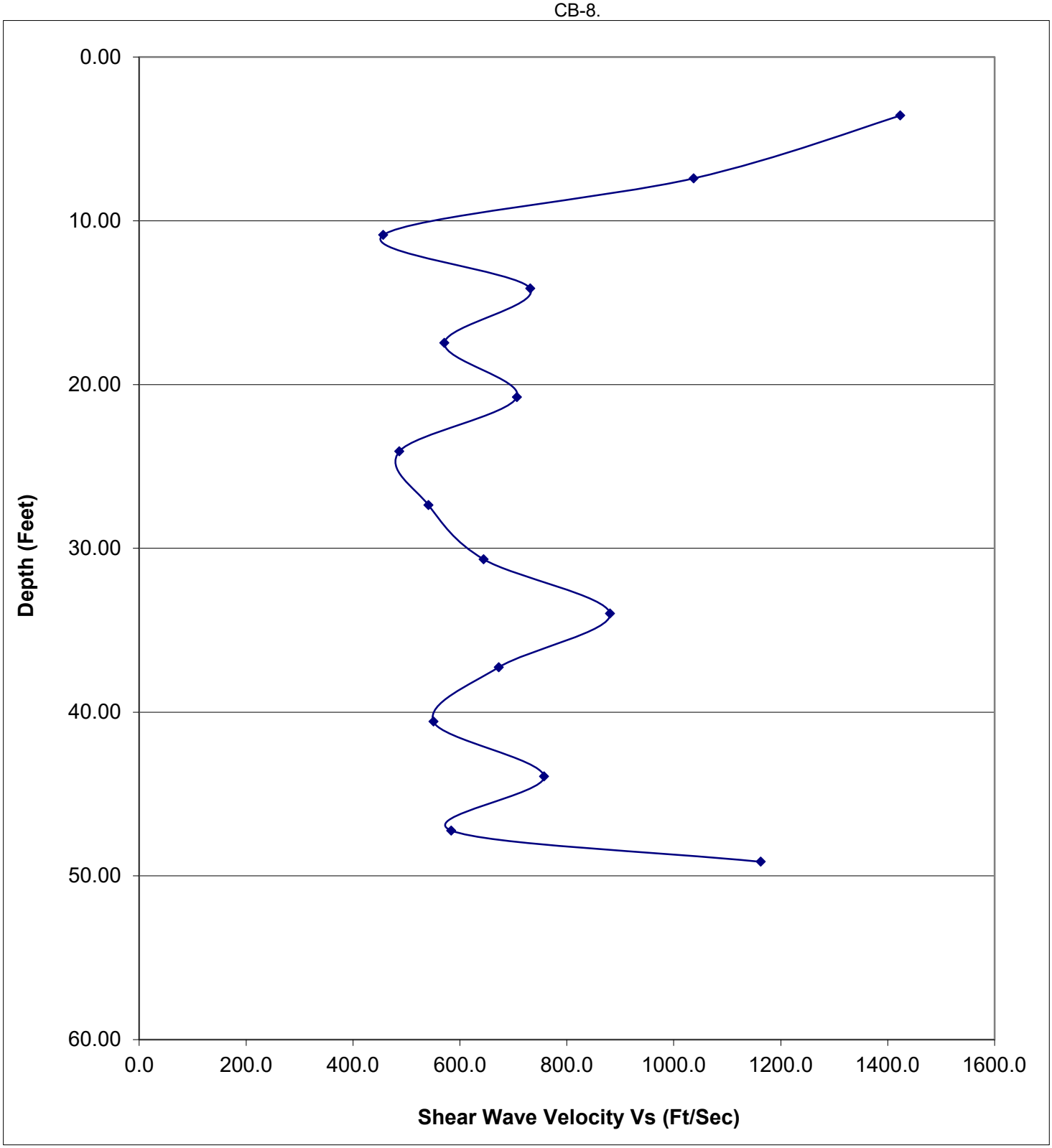
Project: H20019-37AL
Location: Leroy, AL

CPT: H20019-37LACB-8.
Total depth: 50.07 ft, Date: 6/24/2020
Cone Type: Gregg 15
Cone Operator: Cody Cliburn



Shear Wave Velocity Profile

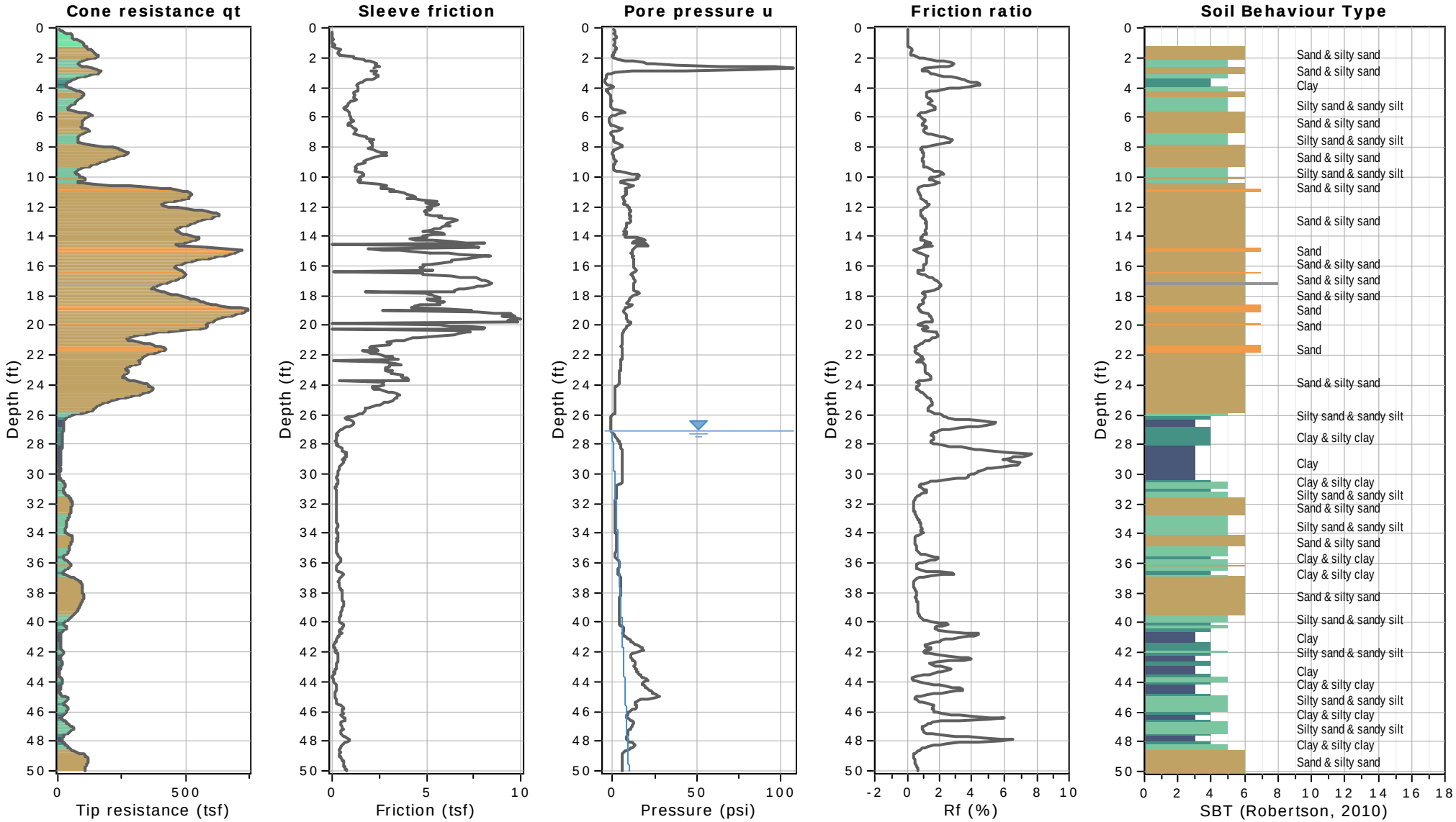
PSEC-LOWMAN POND CLOSURE
H20019-37LA
CB-8.

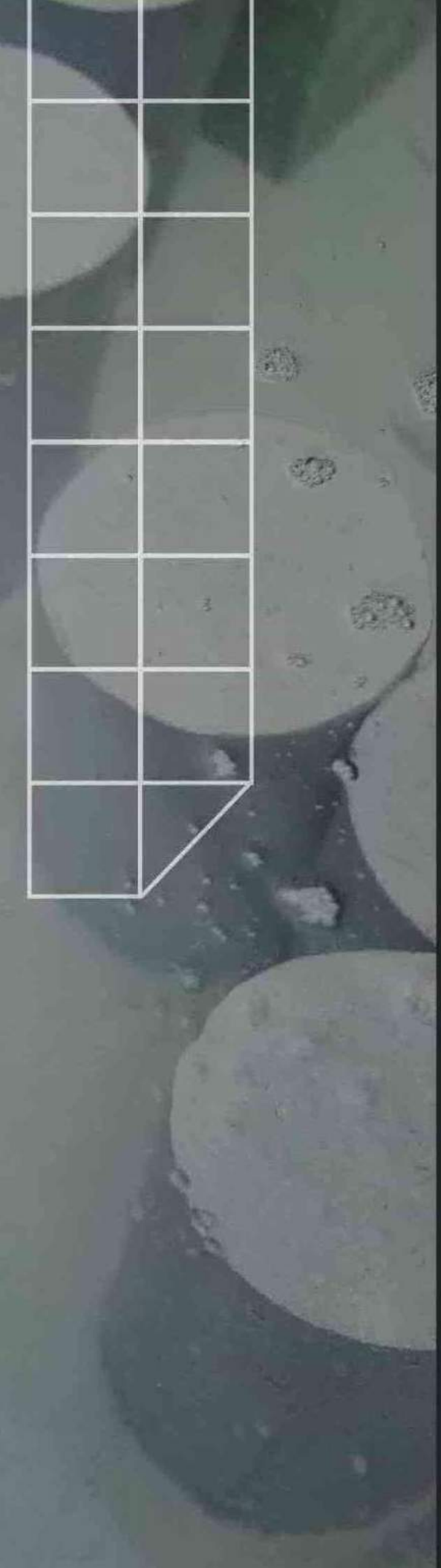


Walker-Hill Environmental
Chris Hayslip
chris@whenv.com
www.whenv.com

Project: **H20019-37AL**
Location: **Leroy, AL**

CPT: H20019-37LACB-9.
Total depth: 50.00 ft, Date: 6/25/2020
Cone Type: Gregg 15
Cone Operator: Cody Cliburn





APPENDIX D

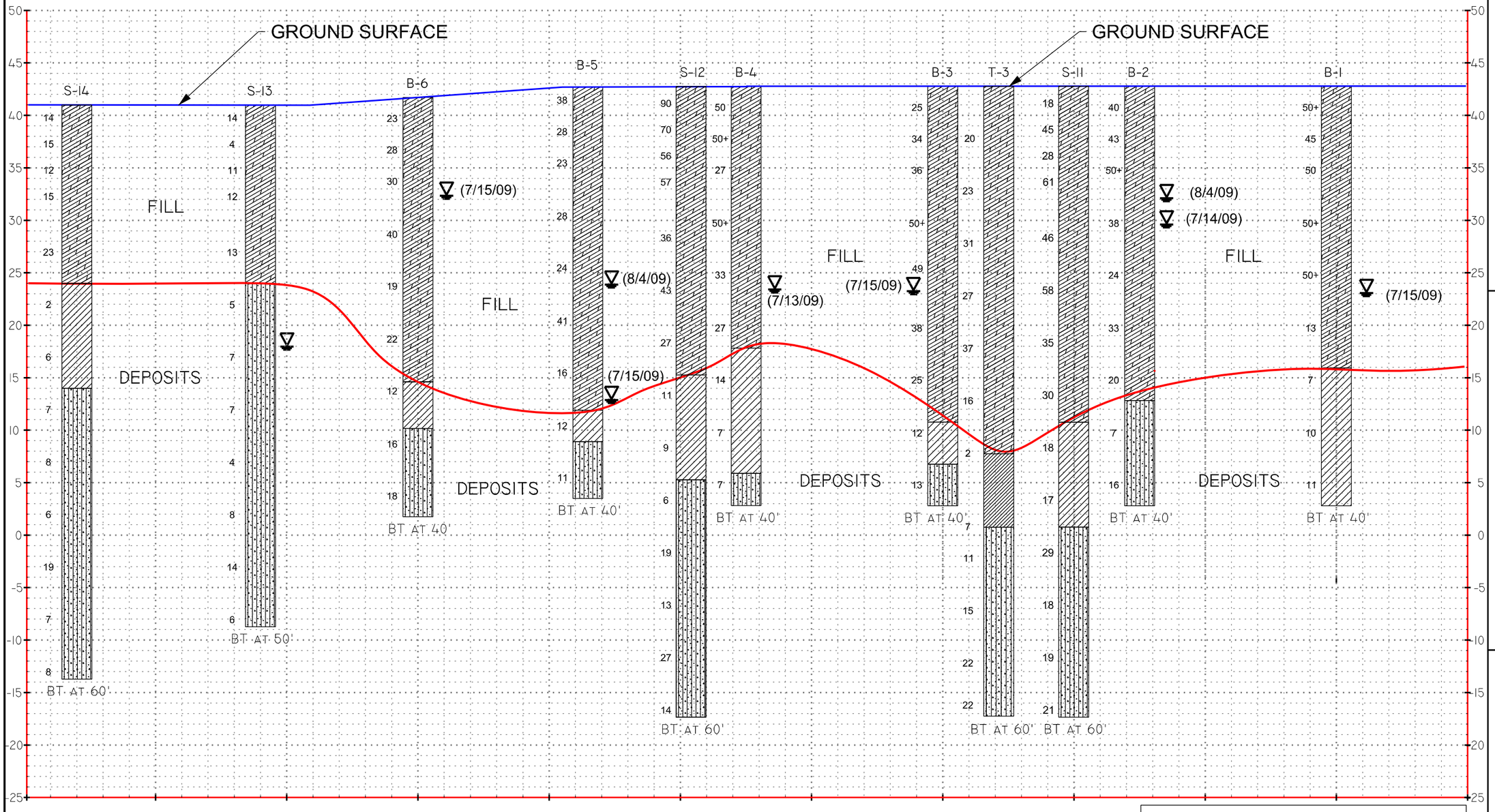
Subsurface Profiles

A

APPROXIMATE EL (MSL)

A'

APPROXIMATE EL (MSL)



Periodic Safety
Factor Assesment
Leroy, Alabama
CDG Reference No. : R021221021

Subsurface Profile A - A'

Date: 09/14/2021

Scale:
(H):1"=+/-160'
(V): 1"=+/-10'

Drawn By: WBB

CDG

Engineering. Environmental. Answers.

B

APPROXIMATE EL (MSL)

B'

APPROXIMATE EL (MSL)

GROUND SURFACE

GROUND SURFACE

B-1 S-10 S-9 T-2 S-8 S-7 S-6

FILL

FILL

FILL

FILL

DEPOSITS

DEPOSITS

DEPOSITS

(7/15/09)

BT AT 40'

BT AT 60'

BT AT 60'

BT AT 60'

BT AT 60'

BT AT 60'

BT AT 60'

Poorly-Graded SANDS (SP)

Silty SANDS (SM)

Clayey SANDS (SC)

Sandy SILTS and SILTS (ML)

Sandy CLAYS and CLAYS (CL)

Elastic SILTS (MH)

Plastic CLAYS (CH)

FILL Material (All soil types)

ALLUVIUM (All soil types)

DECOMPOSED ROCK/BOULDERS

= Groundwater Encountered (date measured)

BT = Boring Terminated

Periodic Safety Factor Assesment
Leroy, Alabama
CDG Reference No. : R021221021

Subsurface Profile B - B'

Date: 09/14/2021

Scale: (H):1"=+/-160' (V): 1"=+/-10'

Drawn By: WBB

CDG

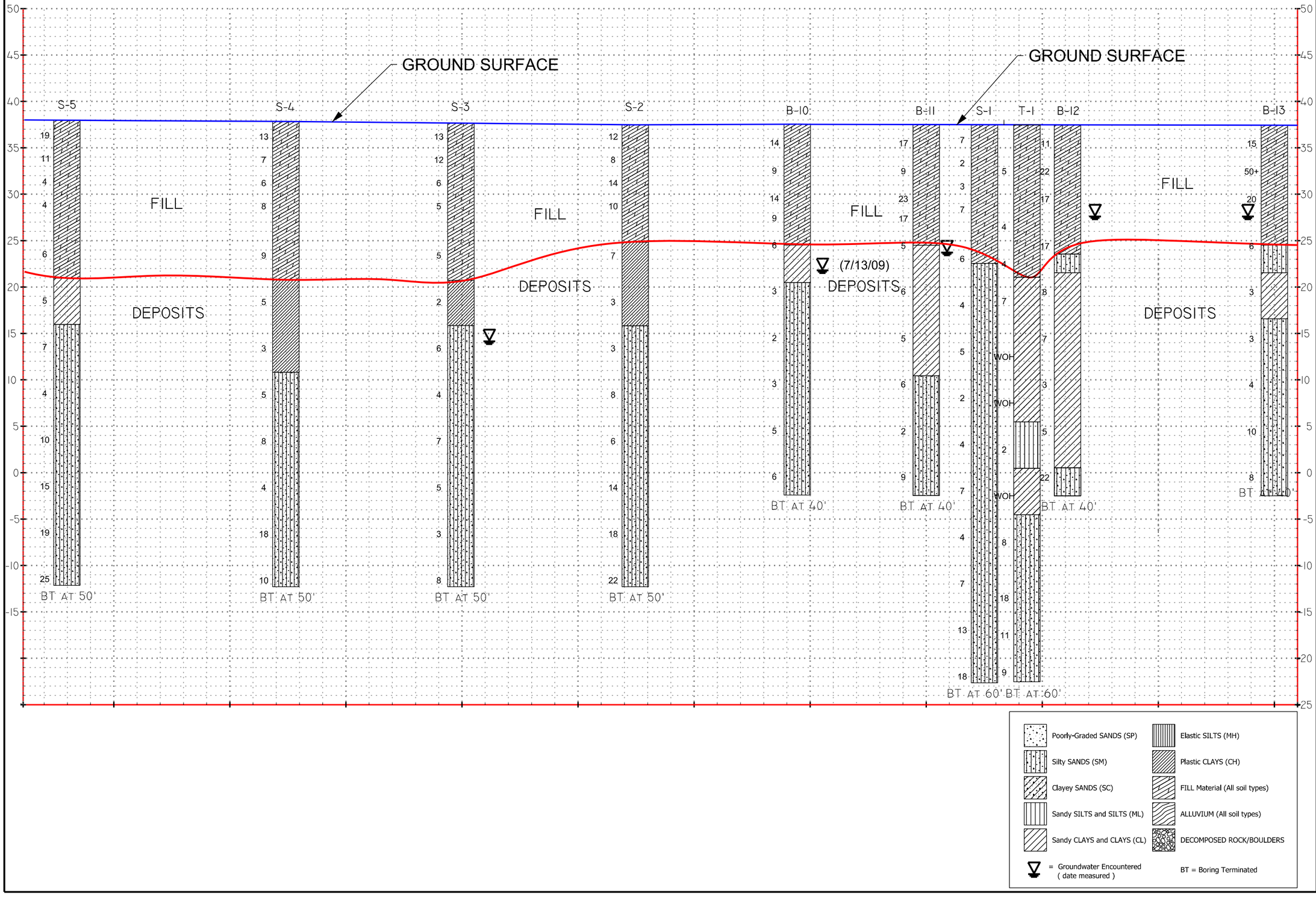
Engineering. Environmental. Answers.

C

APPROXIMATE EL (MSL)

C'

APPROXIMATE EL (MSL)



Periodic Safety
Factor Assesment
Leroy, Alabama
CDG Reference No. : R021221021

Subsurface Profile C - C'

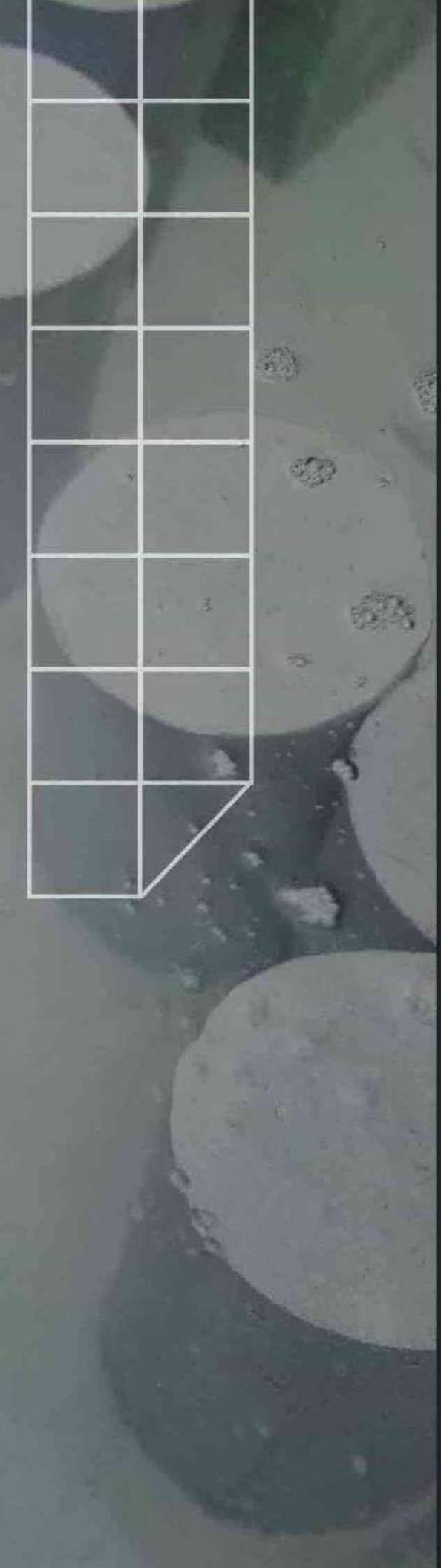
Scale: (H):1"=+/-160' (V): 1"=+/-10'

Date: 09/14/2021

Drawn By: WBB

CDG

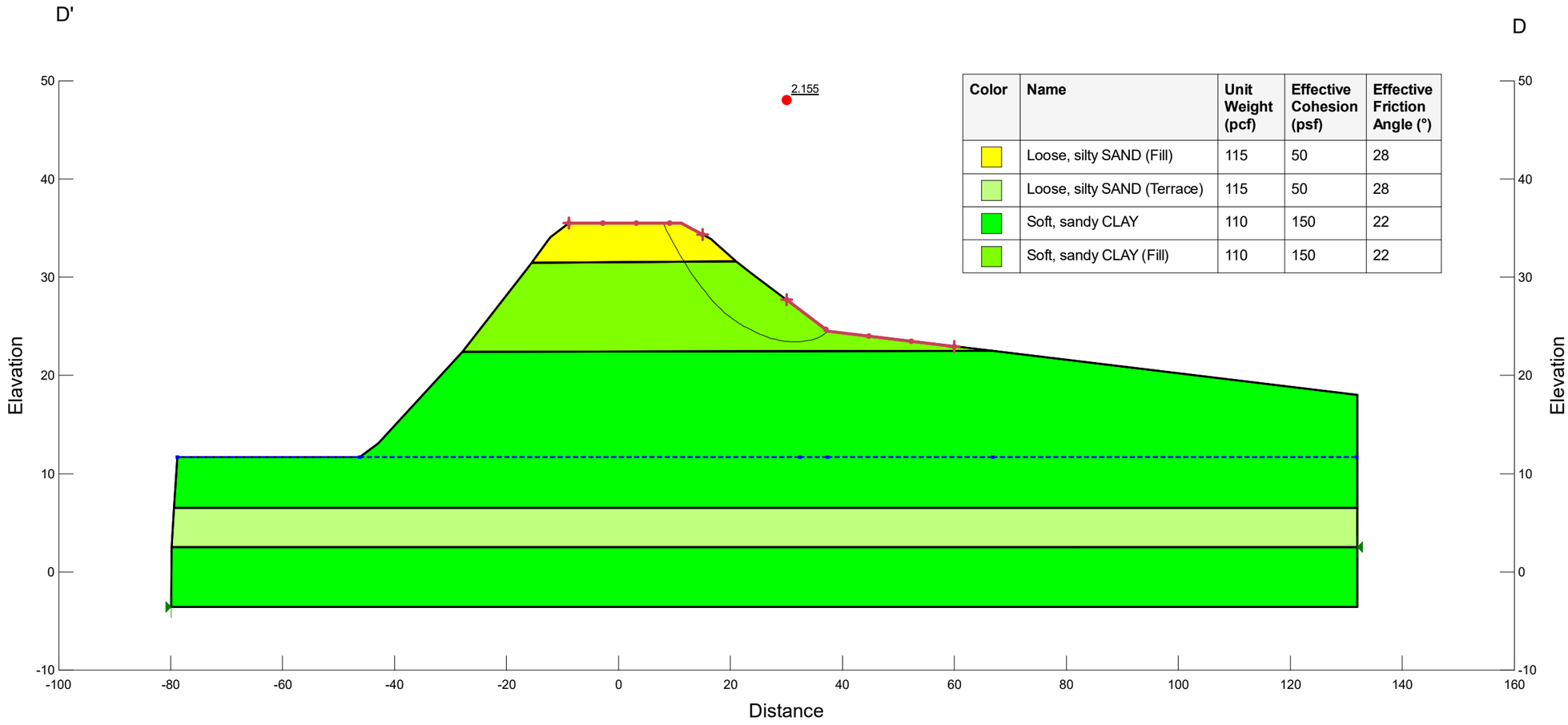
Engineering. Environmental. Answers.



APPENDIX E

Static, Maximum Storage Pool Stability Analysis

Method: Spencer
Calculated Factor of Safety: 2.155
Required Minimum Factor of Safety: 1.5



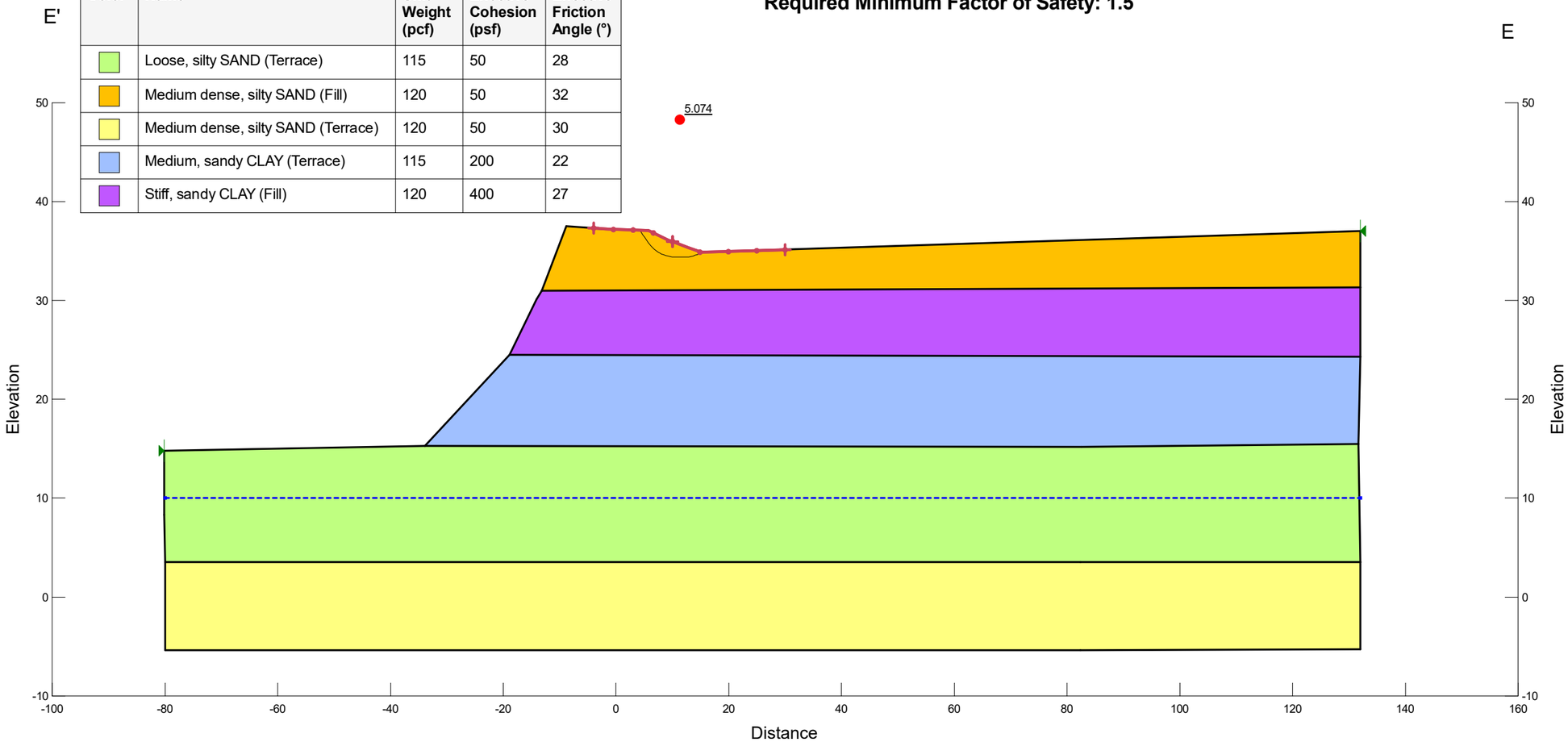
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, silty SAND (Fill)	115	50	28
	Loose, silty SAND (Terrace)	115	50	28
	Soft, sandy CLAY	110	150	22
	Soft, sandy CLAY (Fill)	110	150	22

Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section D-D' - Maximum Storage Pool		
Section D-D' - Maximum Storage Pool.gsz		
09/02/2026	1:260	Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 5.074
Required Minimum Factor of Safety: 1.5

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, silty SAND (Terrace)	115	50	28
	Medium dense, silty SAND (Fill)	120	50	32
	Medium dense, silty SAND (Terrace)	120	50	30
	Medium, sandy CLAY (Terrace)	115	200	22
	Stiff, sandy CLAY (Fill)	120	400	27

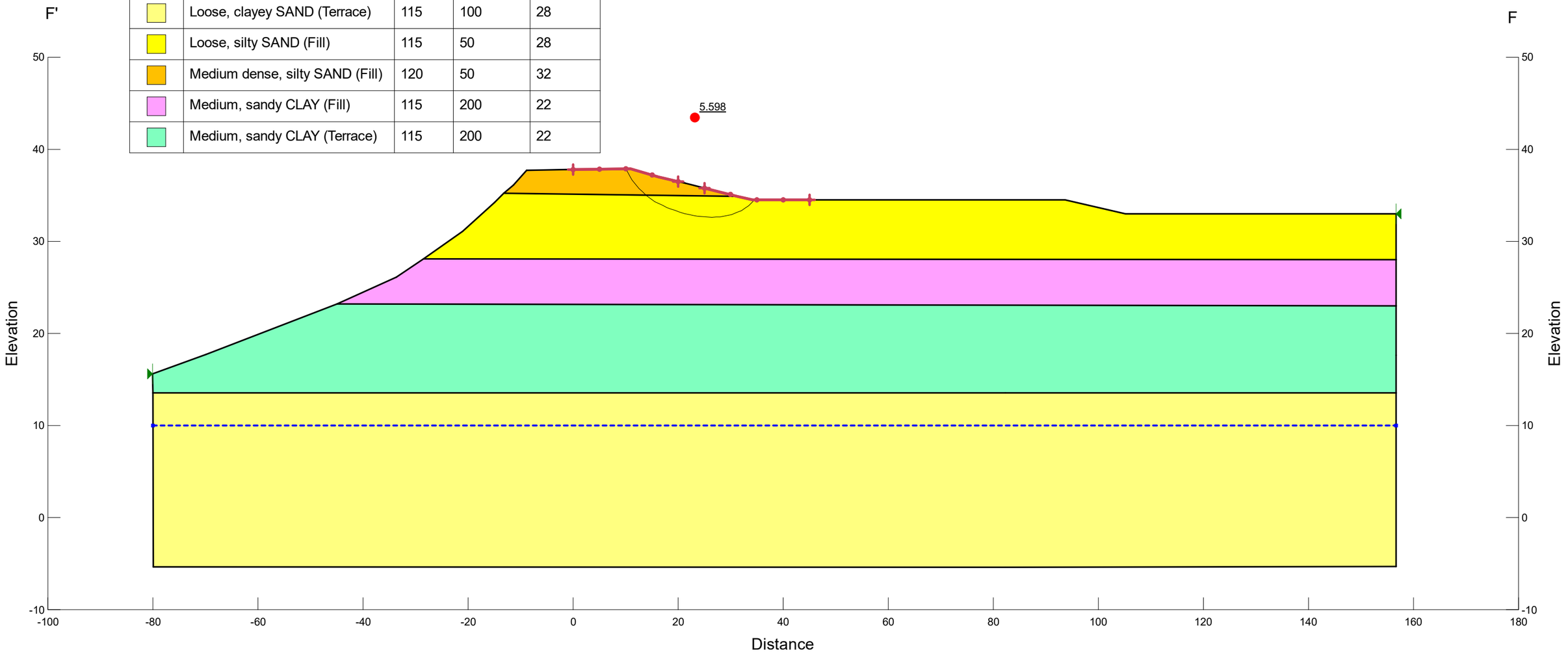


Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section E-E' - Maximum Storage Pool
Section E-E' - Maximum Storage Pool.gsz
08/27/2026 1:260 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 5.598
Required Minimum Factor of Safety: 1.5

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, clayey SAND (Terrace)	115	100	28
	Loose, silty SAND (Fill)	115	50	28
	Medium dense, silty SAND (Fill)	120	50	32
	Medium, sandy CLAY (Fill)	115	200	22
	Medium, sandy CLAY (Terrace)	115	200	22

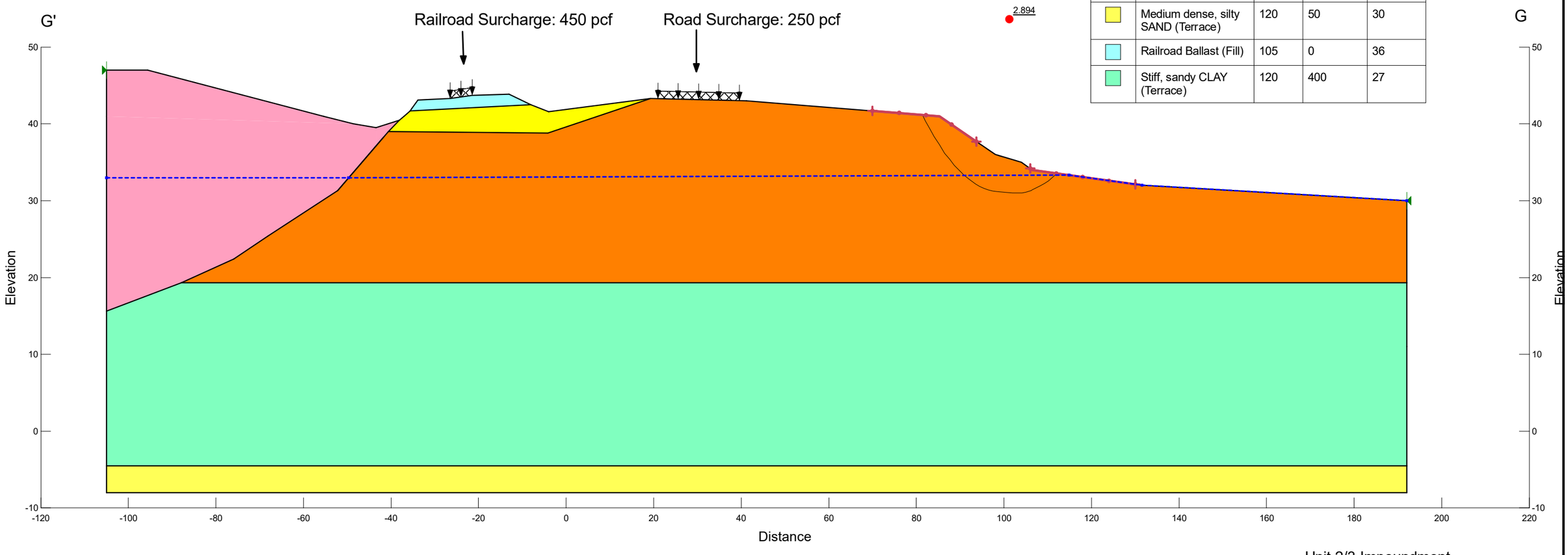


Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section F-F' - Maximum Storage Pool
Section F-F' - Maximum Storage Pool.gsz
08/27/2026 1:225 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.894
Required Minimum Factor of Safety: 1.5

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	50	34
	Loose, silty SAND (Fill)	115	50	28
	Medium dense, silty SAND (Terrace)	120	50	30
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	400	27

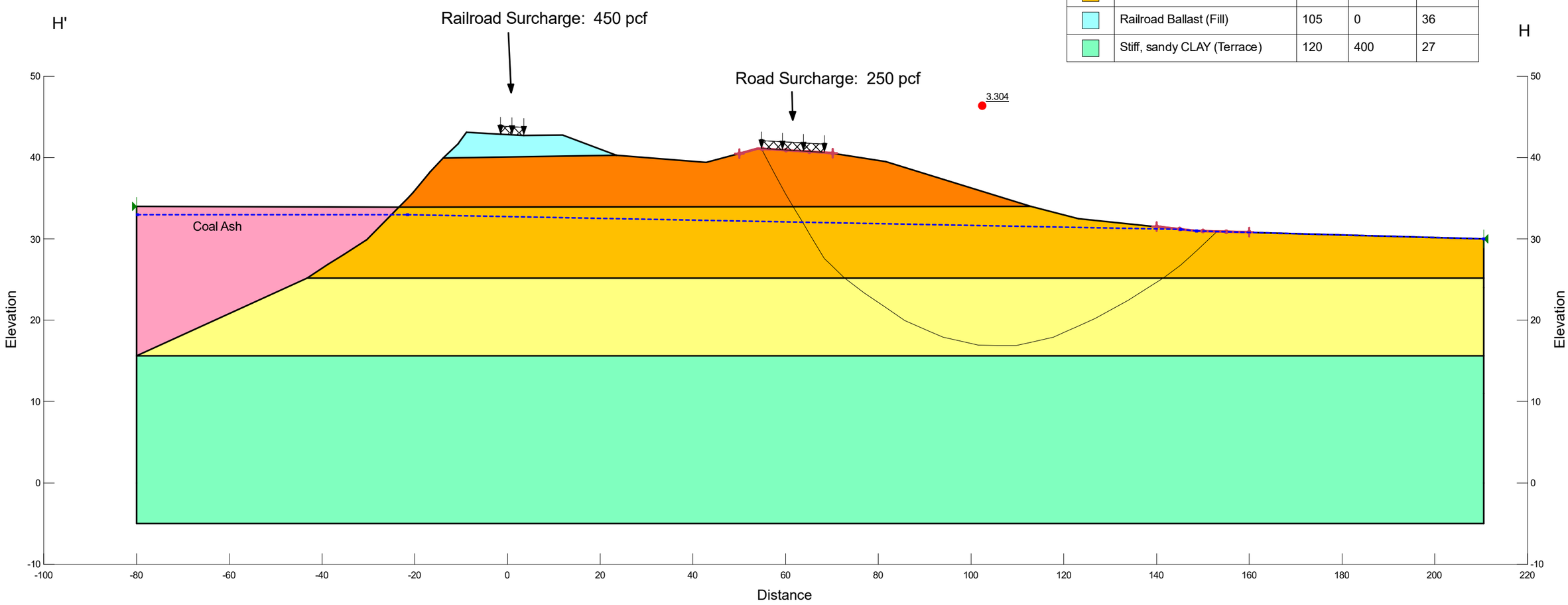


Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section G-G' - Maximum Storage Pool		
Section G-G' - Maximum Storage Pool.gsz		
09/02/2026	1:270	Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 3.304
Required Minimum Factor of Safety: 1.5

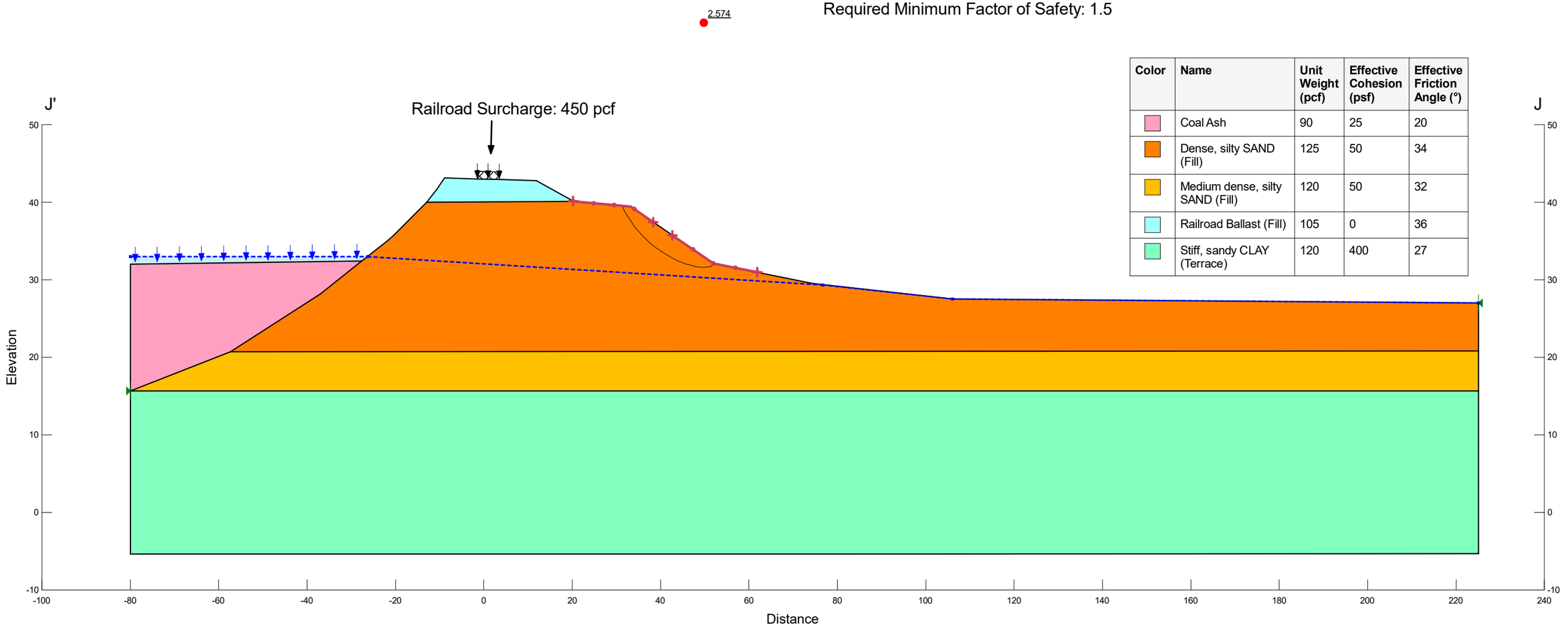
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	50	34
	Loose, silty SAND (Terrace)	115	50	28
	Medium dense, silty SAND (fill)	120	50	32
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	400	27



Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section H-H' - Maximum Storage Pool
Section H-H' - Maximum Storage Pool.gsz
09/02/2026 1:260 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.574
Required Minimum Factor of Safety: 1.5



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	50	34
	Medium dense, silty SAND (Fill)	120	50	32
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	400	27

Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

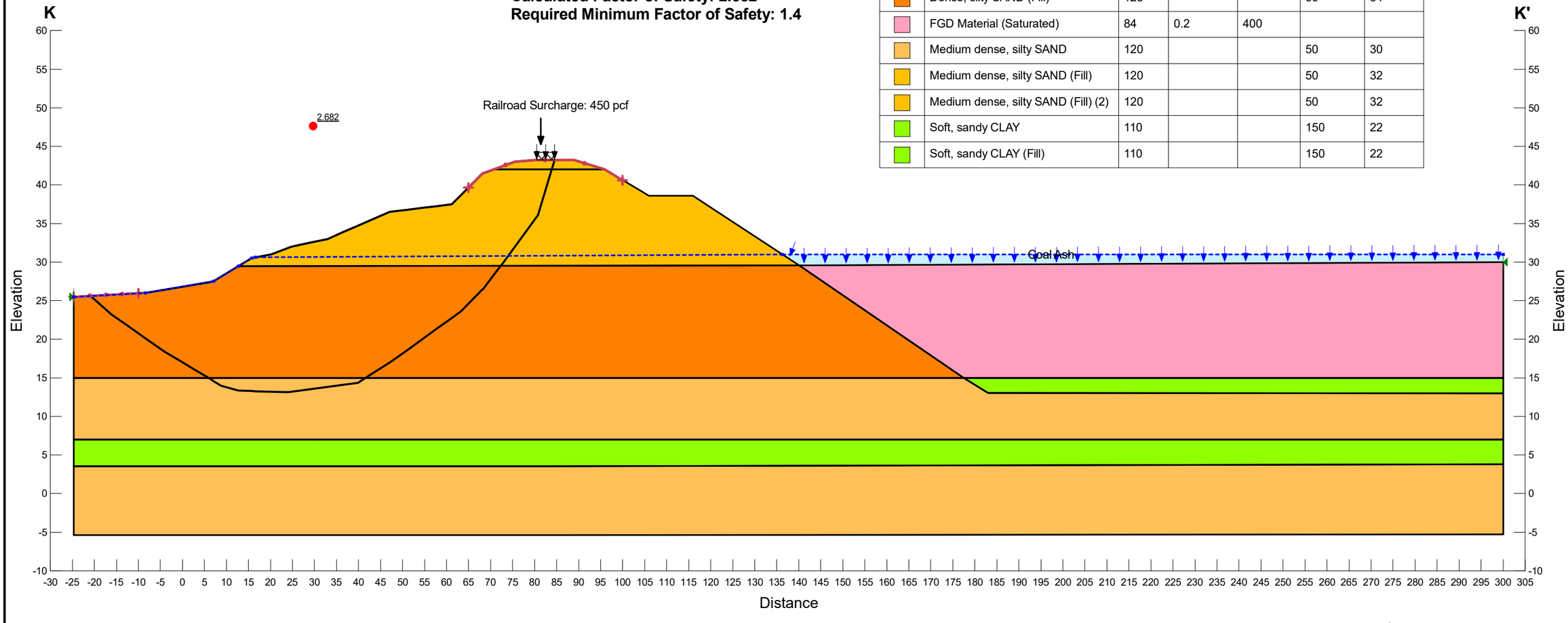
Section J-J' - Maximum Storage Pool

Section J-J' - Maximum Storage Pool.gsz

09/02/2026 1:275 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.682
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Tau/Sigma	Minimum Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense, silty SAND (Fill)	125			50	34
	FGD Material (Saturated)	84	0.2	400		
	Medium dense, silty SAND	120			50	30
	Medium dense, silty SAND (Fill)	120			50	32
	Medium dense, silty SAND (Fill) (2)	120			50	32
	Soft, sandy CLAY	110			150	22
	Soft, sandy CLAY (Fill)	110			150	22

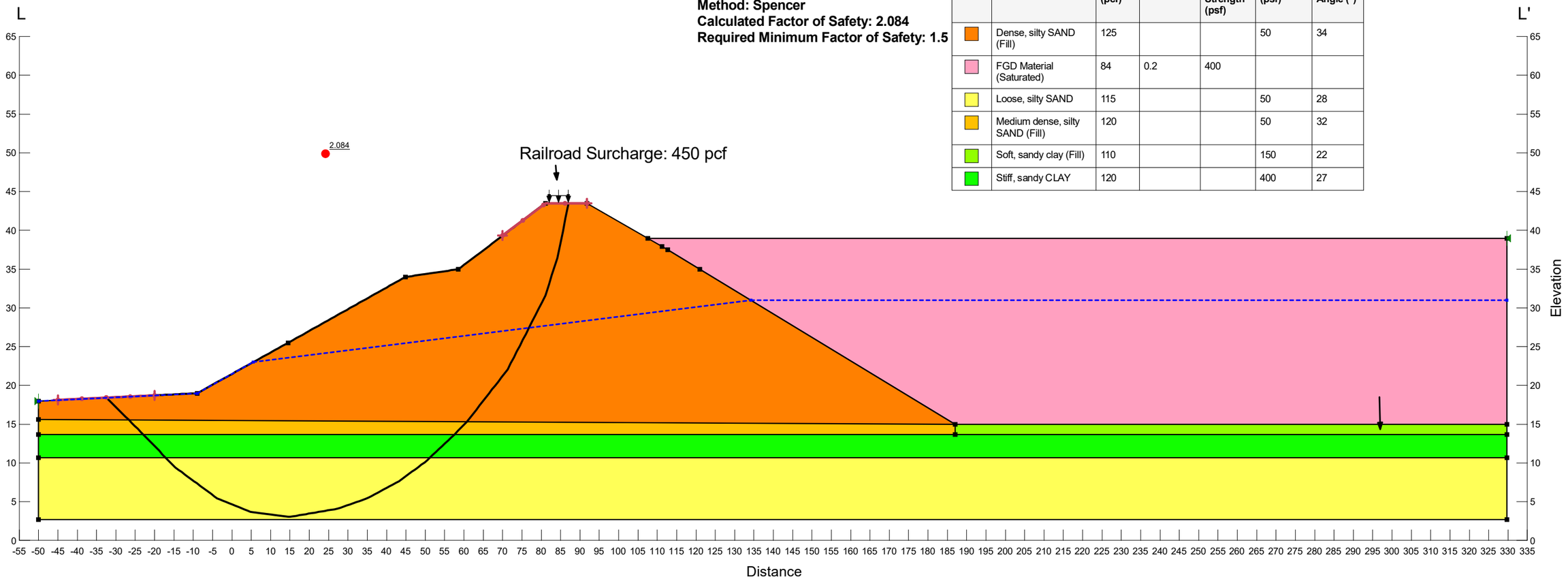


FDG Waste Pond
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section K-K' - Maximum Storage Pool		
Section K-K' - Maximum Storage Pool.gsz		
09/03/2026	1:280	Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.084
Required Minimum Factor of Safety: 1.5

Color	Name	Unit Weight (pcf)	Tau/Sigma	Minimum Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense, silty SAND (Fill)	125			50	34
	FGD Material (Saturated)	84	0.2	400		
	Loose, silty SAND	115			50	28
	Medium dense, silty SAND (Fill)	120			50	32
	Soft, sandy clay (Fill)	110			150	22
	Stiff, sandy CLAY	120			400	27

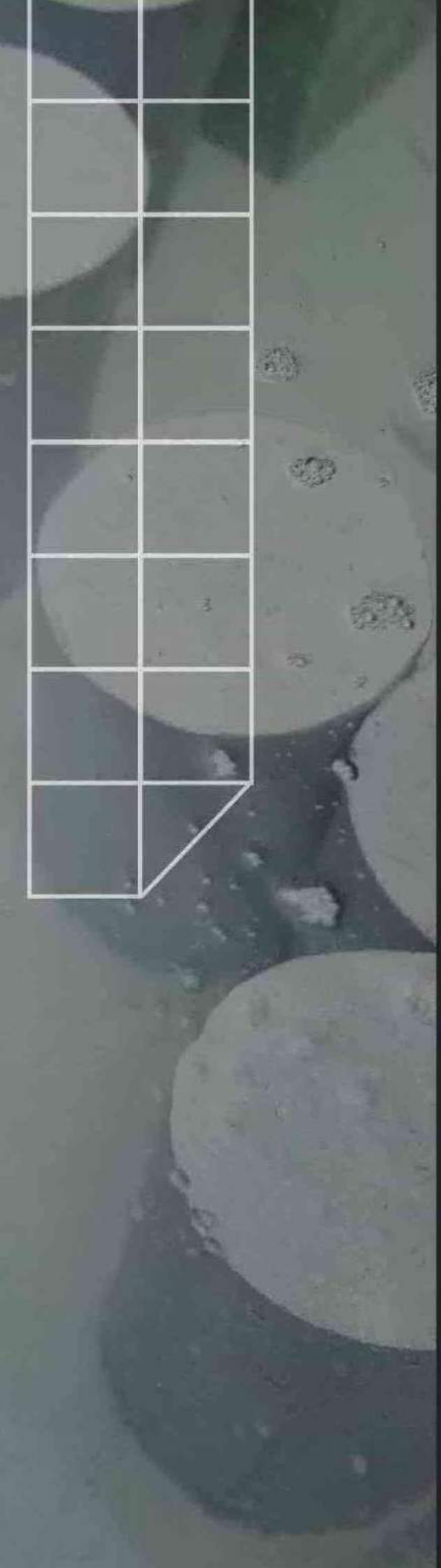


FDG Waste Pond
Charles R. Lowman Power Plant
Three Notch Project Number: R021225012

Section L-L' - Maximum Storage Pool

Section L-L' - Maximum Storage Pool.gsz

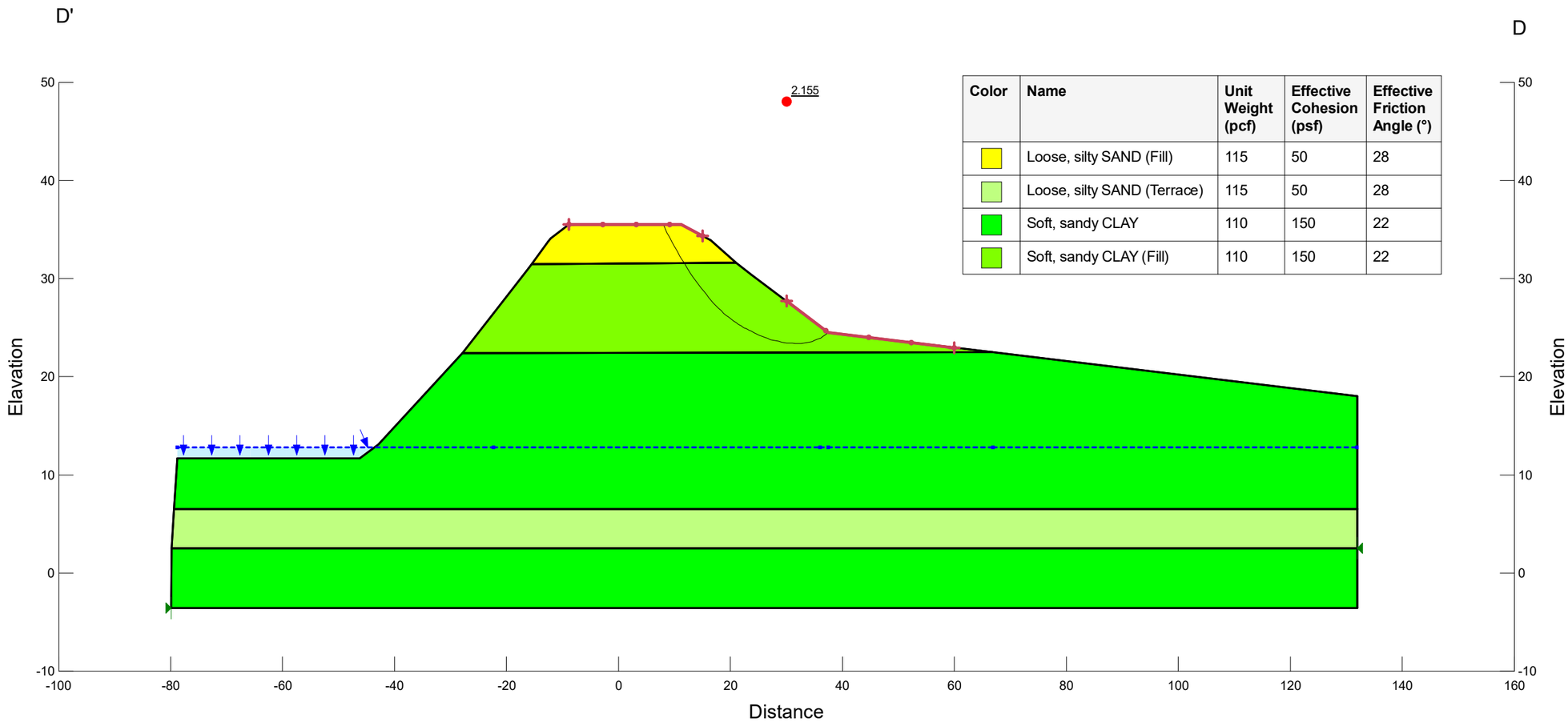
09/03/2026 1:320 Vertical Exaggeration: 2



APPENDIX F

Static, Maximum Surcharge Pool Stability Analyses

Method: Spencer
Calculated Factor of Safety: 2.155
Required Minimum Factor of Safety: 1.4



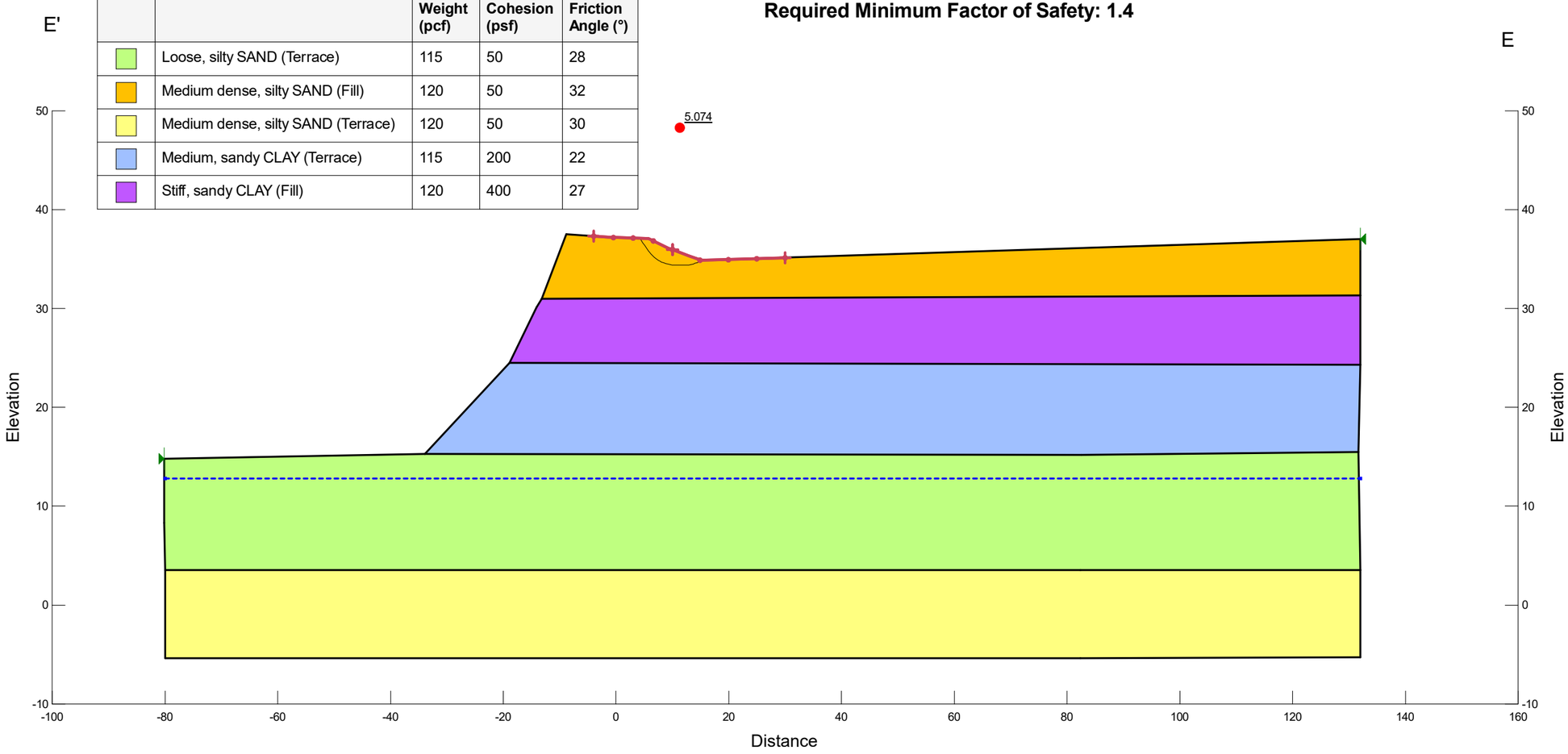
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, silty SAND (Fill)	115	50	28
	Loose, silty SAND (Terrace)	115	50	28
	Soft, sandy CLAY	110	150	22
	Soft, sandy CLAY (Fill)	110	150	22

Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section D-D' - Maximum Surcharge Pool		
Section D-D' - Maximum Surcharge Pool.gsz		
09/03/2026	1:260	Vertical Exaggeration: 1.75

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, silty SAND (Terrace)	115	50	28
	Medium dense, silty SAND (Fill)	120	50	32
	Medium dense, silty SAND (Terrace)	120	50	30
	Medium, sandy CLAY (Terrace)	115	200	22
	Stiff, sandy CLAY (Fill)	120	400	27

Method: Spencer
Calculated Factor of Safety: 5.074
Required Minimum Factor of Safety: 1.4



Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

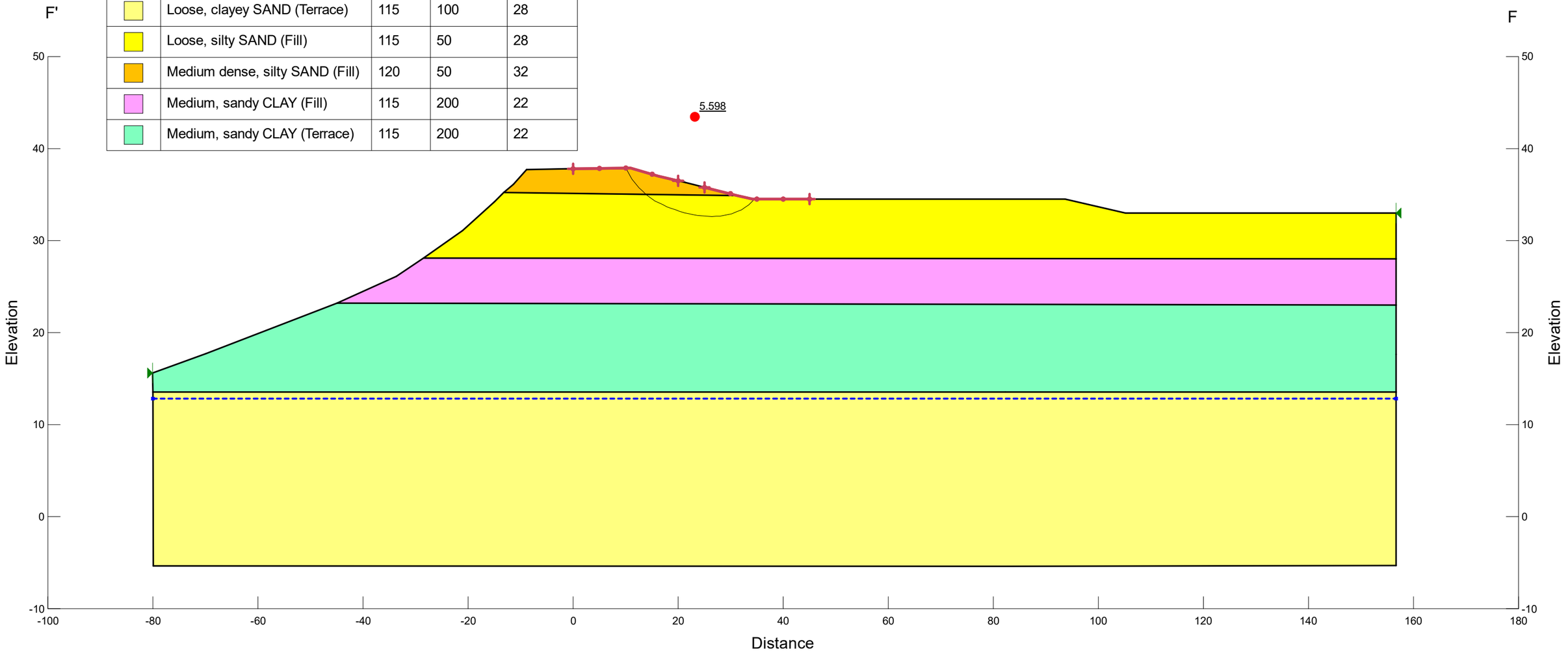
Section E-E' - Maximum Surcharge Pool

Section E-E' - Maximum Surcharge Pool.gsz

09/03/2026 1:260 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 5.598
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, clayey SAND (Terrace)	115	100	28
	Loose, silty SAND (Fill)	115	50	28
	Medium dense, silty SAND (Fill)	120	50	32
	Medium, sandy CLAY (Fill)	115	200	22
	Medium, sandy CLAY (Terrace)	115	200	22

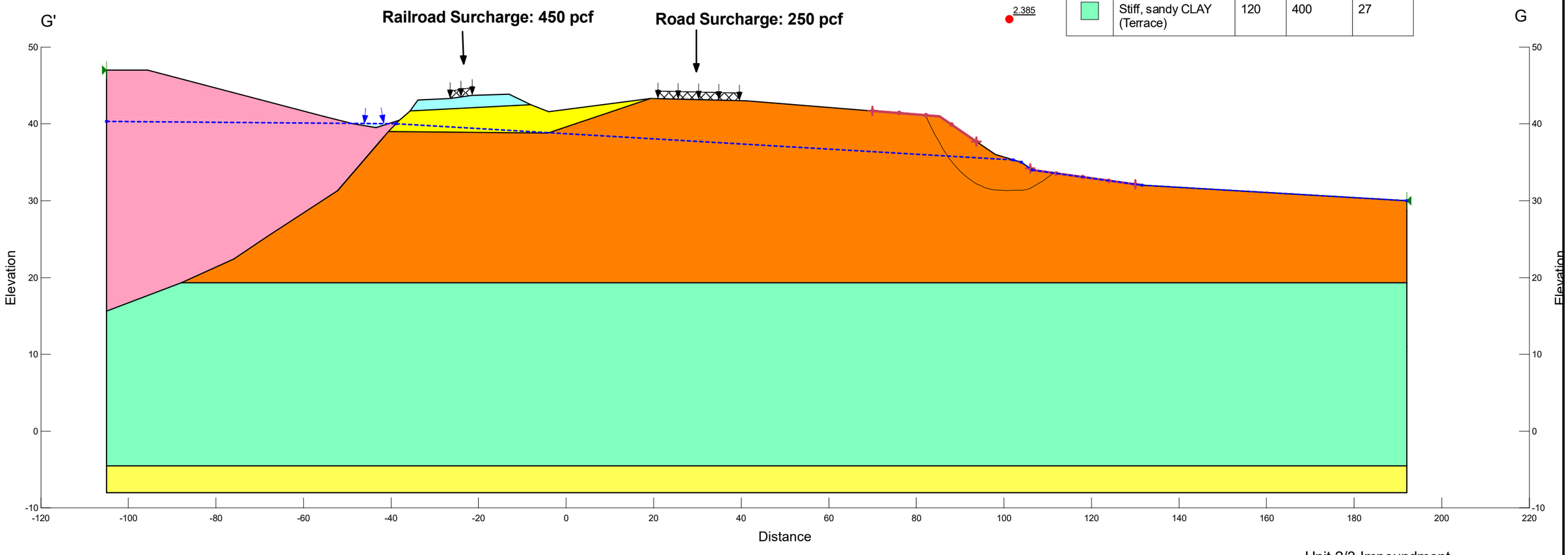


Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section F-F' - Maximum Surcharge Pool
Section F-F' - Maximum Surcharge Pool.gsz
09/03/2026 1:225 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.385
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	50	34
	Loose, silty SAND (Fill)	115	50	28
	Medium dense, silty SAND (Terrace)	120	50	30
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	400	27

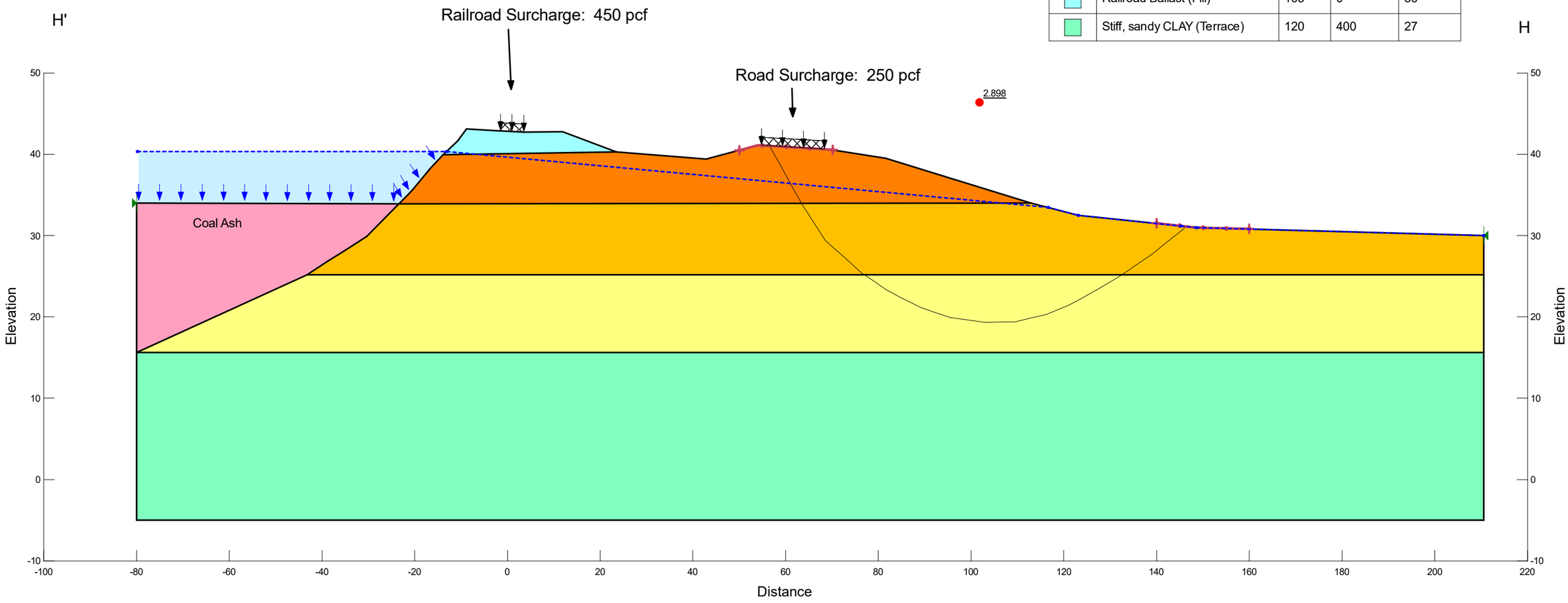


Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section G-G' - Maximum Surcharge Pool		
Section G-G' - Maximum Surcharge Pool.gsz		
09/02/2026	1:270	Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.898
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	50	34
	Loose, silty SAND (Terrace)	115	50	28
	Medium dense, silty SAND (fill)	120	50	32
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	400	27

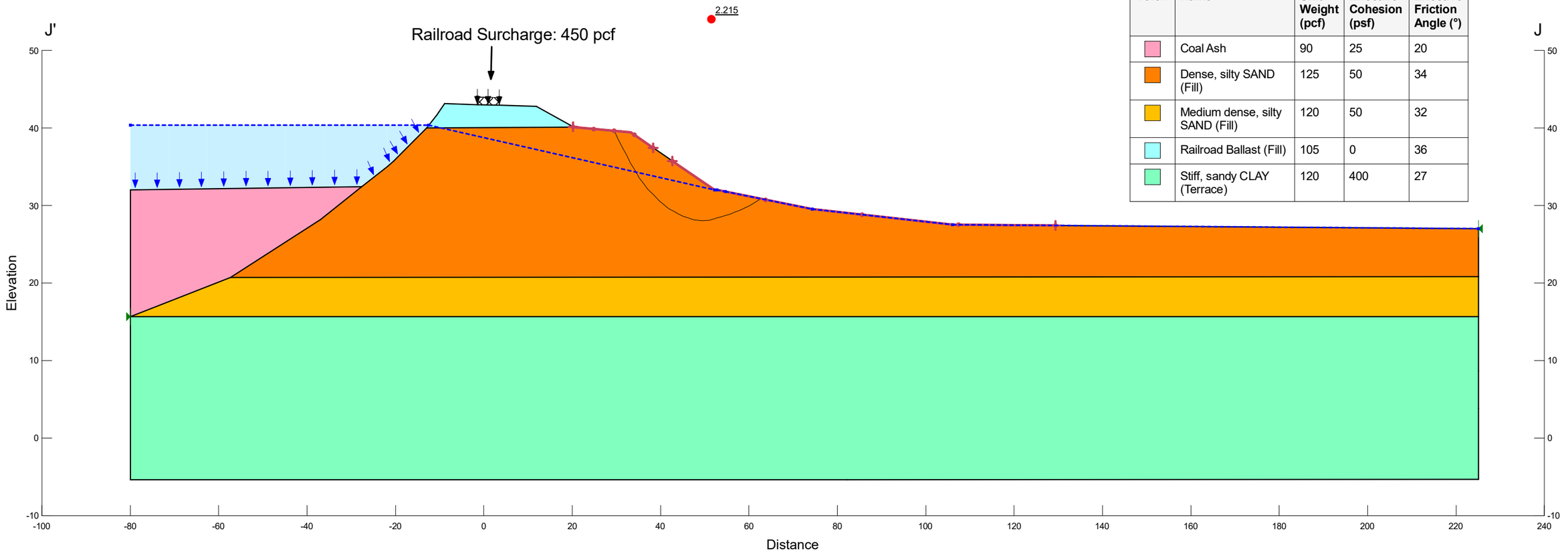


Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section H-H' - Maximum Surcharge Pool		
Section H-H' - Maximum Surcharge Pool.gsz		
09/02/2026	1:260	Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.215
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	50	34
	Medium dense, silty SAND (Fill)	120	50	32
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	400	27



Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

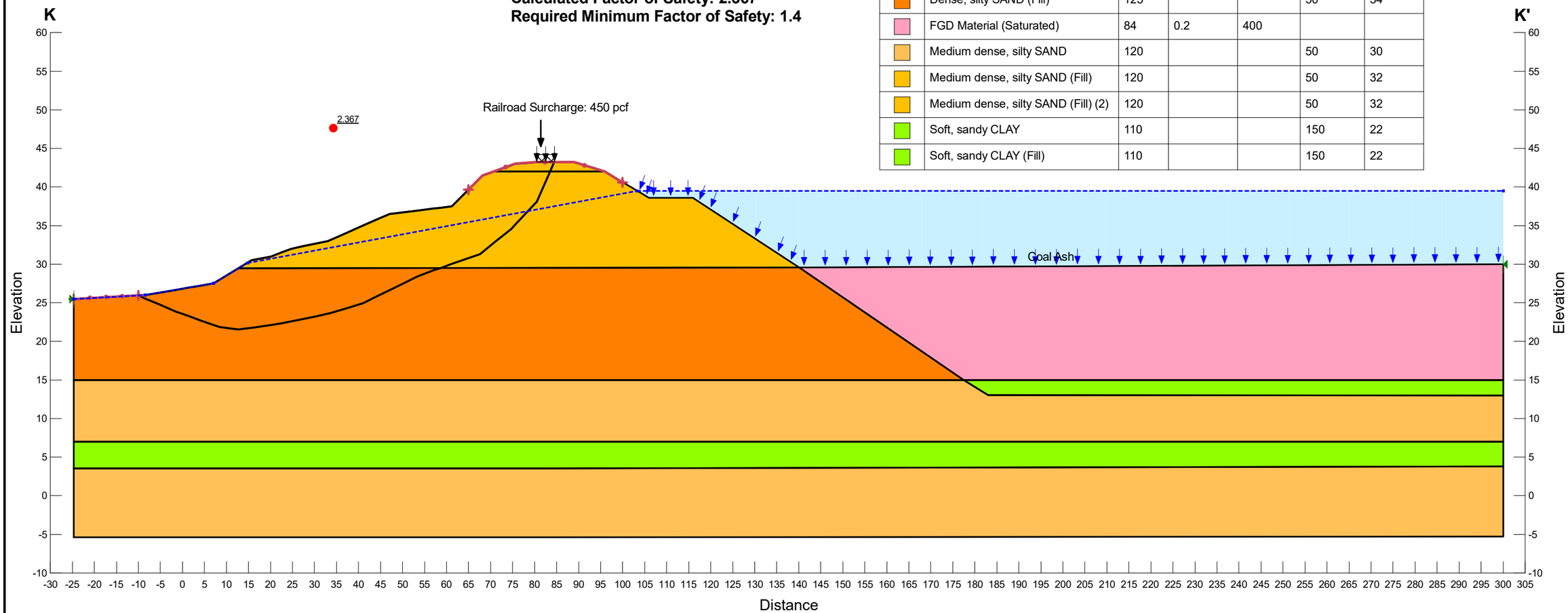
Section J-J' - Maximum Surcharge Pool

Section J-J' - Maximum Surcharge Pool.gsz

09/02/2026 1:275 Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 2.367
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Tau/Sigma	Minimum Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense, silty SAND (Fill)	125			50	34
	FGD Material (Saturated)	84	0.2	400		
	Medium dense, silty SAND	120			50	30
	Medium dense, silty SAND (Fill)	120			50	32
	Medium dense, silty SAND (Fill) (2)	120			50	32
	Soft, sandy CLAY	110			150	22
	Soft, sandy CLAY (Fill)	110			150	22

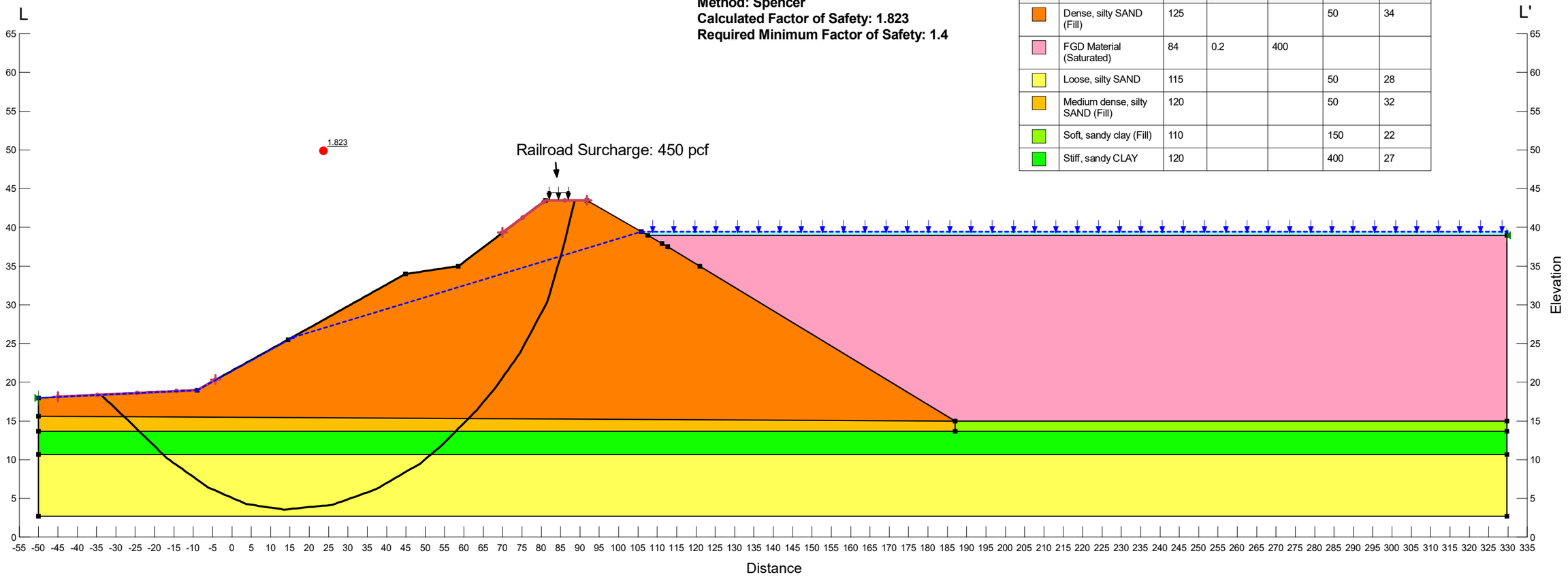


FDG Waste Pond
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section K-K' - Maximum Surcharge Pool		
Section K-K' - Maximum Surcharge Pool.gsz		
09/03/2026	1:280	Vertical Exaggeration: 1.75

Method: Spencer
Calculated Factor of Safety: 1.823
Required Minimum Factor of Safety: 1.4

Color	Name	Unit Weight (pcf)	Tau/Sigma	Minimum Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense, silty SAND (Fill)	125			50	34
	FGD Material (Saturated)	84	0.2	400		
	Loose, silty SAND	115			50	28
	Medium dense, silty SAND (Fill)	120			50	32
	Soft, sandy clay (Fill)	110			150	22
	Stiff, sandy CLAY	120			400	27

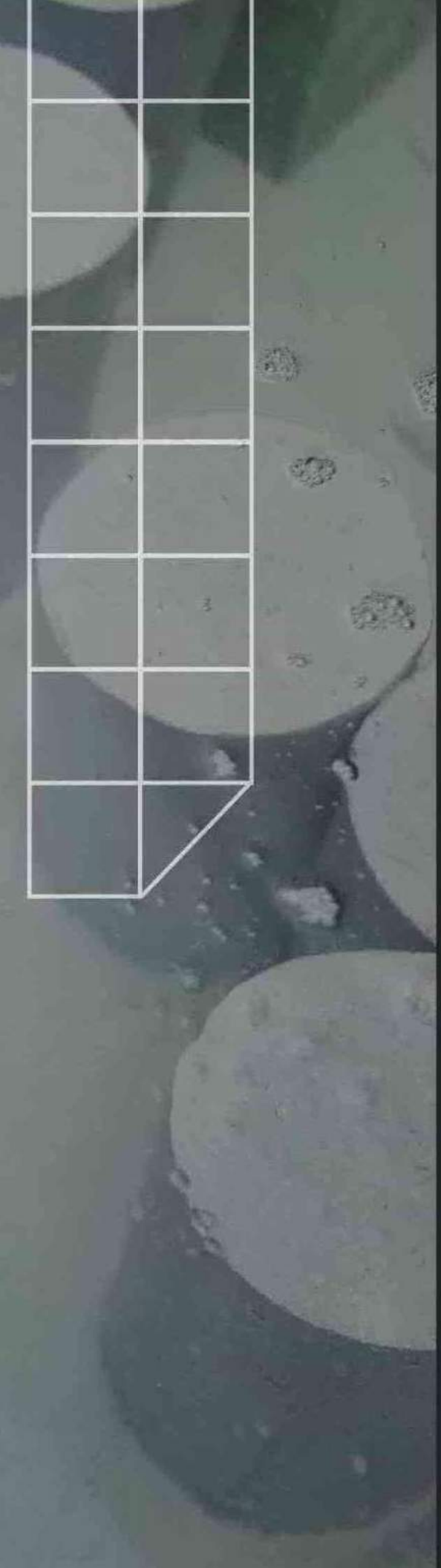


Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section L-L' - Maximum Surcharge Pool

Section L-L' - Maximum Surcharge Pool.gsz

09/03/2026 1:320 Vertical Exaggeration: 2



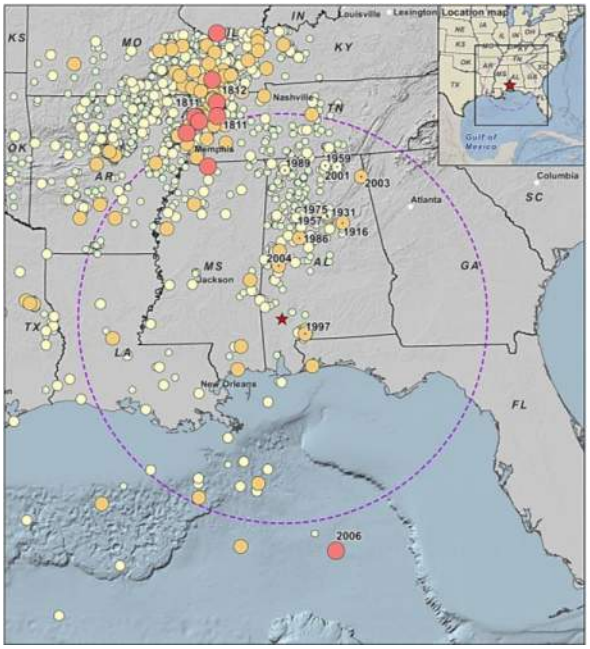
APPENDIX G

Site-Specific Probabilistic Seismic Hazard Analysis

Final Report

Development of Site-Specific Seismic Design Ground Motions for the Charles Lowman Power Plant,

Leroy, Alabama



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29 July 2021

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1.0 INTRODUCTION

At the request of CDG Engineers and Associates, Inc., Lettis Consultants International (LCI) has performed a site-specific probabilistic seismic hazard analysis (PSHA) and site response analysis of the Charles Lowman Power Plant near Leroy, Alabama (Figures 1 and 2). The purpose of the study is to develop site-specific seismic design ground motions for the site. The site is located in southernmost Alabama 130 km north of the Gulf of Mexico. Although southernmost Alabama has a low level of seismicity, the western part of the state is seismically active because of its location within the Southern Appalachian Seismic Zone (Figure 3). The seismic hazard in the region is considered to be low (Petersen *et al*., 2020). There are no known Quaternary active faults within 400 km of the site (Figure 4).

In this study, a seismic source characterization (SSC) model and ground motion models used in evaluating the seismic hazard at nuclear power plants in the Central and Eastern U.S. (CEUS), were used to evaluate and characterize potential seismic sources, the likelihood of earthquakes of various magnitudes occurring on those sources, and the likelihood of the earthquakes producing ground motions over a specified level. The CEUS-SSC model used was the EPRI/DOE/NRC (2012) model (referred to as the CEUS-SSC model from hereon). The Next Generation of Attenuation (NGA)-East ground motion models (GMMS) were used in the PSHA. Our evaluation relied solely on available data and information.

The PSHA methodology used in this study allows for the explicit inclusion of the range of possible interpretations in components of the model, including seismic source characterization and ground motion estimation. Uncertainties in models and parameters are incorporated into the PSHA through the use of logic trees. Because the site is located on deep soil, a site-specific site response analysis was also performed to adjust the probabilistic hard rock hazard results for site effects. This report describes the seismic source characterization, the ground motion models used in the PSHA, the hazard results, the site response analysis results, and the development of seismic design parameters. No deterministic seismic hazard analysis was performed in this study because there are no known Quaternary-active faults within 400 km of the site.

1.1 SCOPE OF WORK

As described in our proposal dated 19 April 2021, the following tasks were performed.

Task 1 – Development of Site-Specific Shear-Wave Profiles

We reviewed the site-specific shear wave velocity (V_S) profiles used in the 2012 Pacific Engineering & Analysis (PE&A) analysis (Darragh and Silva, 2012) and updated them based on more recent Vs data for the site.

Task 2 –Probabilistic Seismic Hazard Analysis (PSHA)

A Cornell-type PSHA using logic trees was performed to evaluate ground motions for specified probabilities or return periods of not being exceeded. The PSHA was performed for hard rock site conditions. Inputs into the PSHA consisted of a seismic source model and ground motion models (GMMs). The seismic source model consisted of the active faults and regional seismic source zones. We used the EPRI/DOE/NRC (2012) seismic source model developed for nuclear power plants in the U.S. and the Next Generation of Attenuation (NGA)-East GMMs for hard rock.

The seismic hazard expressed in terms of the probabilities of exceeding peak and spectral accelerations was computed by combining the following three probability distributions for all seismic sources: (1) probability distribution of earthquake magnitude in time (earthquake recurrence); (2) probability distribution of distance from the earthquake rupture area to the site given magnitude (geometry); and (3) probability distribution of peak and spectral accelerations given magnitude and distance (attenuation). Logic trees to address epistemic uncertainty in the seismic source characterization and GMMs were developed. Products consisted of hard rock hazard curves and deaggregation results.

Task 3 – Site Response Analysis

Site response analyses was performed to adjust the hard rock hazard curves to site-specific geologic conditions. The inputs into the analyses were V_S profiles representative of the site (Task 1) and non-linear dynamic properties. The V_S profiles were randomized using a correlation model to capture the variability in V_S across the site. Site response analyses was performed to calculate a suite of amplification factors at a suite of spectral frequencies i.e., PGA to 10 sec spectral acceleration and input motions. A state-of-the-art random-vibration-theory methodology based on an equivalent-linear approach was used.

Task 4 – Final Report and Meetings

A draft final report that summarizes the approaches and results was produced and submitted for review. All review comments will be addressed and a final report prepared and transmitted.

1.2 ACKNOWLEDGEMENTS

Our thanks to Ben Turner, Allen Yates, Danner Drake, and Tim Siegel for their project management support and for providing data and information, to Claire Unruh, Zoya Farajpour, and Javier Chalini for their assistance in the preparation of this report.

2.0 PSHA METHODOLOGY

The PSHA approach used in this study is based on the model developed principally by Cornell (1968). The occurrence of earthquakes on a fault is assumed to be a Poisson process. The Poisson model is widely used and is a reasonable assumption in regions where data are sufficient to provide only an estimate of average recurrence rate (Cornell, 1968). The occurrence of ground motions at the site in excess of a specified level is also a Poisson process, if (1) the occurrence of earthquakes is a Poisson process, and (2) the probability that any one event will result in ground motions at the site in excess of a specified level is independent of the occurrence of other events.

The probability that a ground motion parameter "*Z*" exceeds a specified value "*z*" in a time period "t" is given by:

$$P(Z > z) = 1\text{-}e^{-\nu(z)\cdot t} \quad (1)$$

where $\nu(z)$ is the annual mean number (or rate) of events in which *Z* exceeds *z*. It should be noted that the assumption of a Poisson process for the number of events is not critical. This is because the mean number of events in time t, $\nu(z)\cdot t$, can be shown to be a close upper bound on the probability $P(Z > z)$ for small probabilities (less than 0.10) that generally are of interest for engineering applications. The annual mean number of events is obtained by summing the contributions from all sources, that is:

$$\nu(z) = \sum_{n} \nu_n(z) \quad (2)$$

where $\nu_n(z)$ is the annual mean number (or rate) of events on source n for which *Z* exceeds *z* at the site. The parameter $\nu_n(z)$ is given by the expression:

$$\nu_n(z) = \sum_{i}\sum_{j} ß_n(m_i)\cdot p(R=r_j|m_i)\cdot P(Z>z|m_i,r_j) \quad (3)$$

where:

$ß_n(m_i)$ = annual mean rate of recurrence of earthquakes of magnitude increment m_i on source n;

$p(R=r_j|m_i)$ = probability that given the occurrence of an earthquake of magnitude m_i on source n, r_j is the closest distance increment from the rupture surface to the site;

$p(Z>z|m_i,r_j)$ = probability that given an earthquake of magnitude m_i at a distance of r_j, the ground motion exceeds the specified level *z*.

The calculations were made using LCI's proprietary PSHA computer program APEX. The program has been validated and verified under a NQA-1 program.

2.1 SEISMIC SOURCE CHARACTERIZATION

Two types of earthquake sources are characterized in the CEUS-SSC model: (1) known fault sources; and (2) background earthquakes. Uncertainties in the CEUS-SSC source parameters are modeled using logic trees. In this procedure, values of the source parameters are represented by the branches of logic trees with weights that define the distribution of values (Figure 5). In general, three or five values for each parameter were weighted and used in the analysis. Note that the weights associated with the percentiles are not equivalent to probabilities for these values, but rather are weights assigned to define the distribution.

2.1.1 Fault Sources

Fault sources are modeled as three-dimensional fault surfaces and details of their behavior are incorporated into the source characterization. The geometric source parameters for faults include fault location, segmentation model, dip, and thickness of the seismogenic zone. The recurrence parameters include recurrence model, recurrence rate (slip rate or average recurrence interval for the maximum event), slope of the recurrence curve (*b*-value), and maximum magnitude. Clearly, the geometry and recurrence are not totally independent.

Source Geometry

In the PSHA, it is assumed that earthquakes of a certain magnitude may occur randomly along the length of a given fault or segment. The distance from an earthquake to the site is dependent on the source geometry, the size and shape of the rupture on the fault plane, and the likelihood of the earthquake occurring at different points along the fault length. The distance to the fault is defined to be consistent with the specific GMM used to calculate the ground motions. The distance, therefore, is dependent on both the dip and depth of the fault plane, and a separate distance function is calculated for each geometry and each ground motion prediction model. The size and shape of the rupture on the fault plane are dependent on the magnitude of the earthquake, with larger events rupturing longer and wider portions of the fault plane.

For a given magnitude, the associated rupture surface is uniformly distributed along the fault length and width. Ruptures are constrained to occur entirely on the defined fault plane. The rupture dimensions can be modeled using magnitude-rupture area and rupture-width relationships.

Fault Recurrence

The recurrence relationships for faults are generally modeled using the exponentially truncated Gutenberg-Richter, characteristic earthquake, and the maximum moment (magnitude) recurrence

models. These models are weighted to represent judgment on their applicability to the sources. The CEUS-SSC models all faults using the maximum moment model.

The general approach of Molnar (1979) and Anderson (1979) is often used to arrive at the recurrence for the exponentially truncated model. The number of events exceeding a given magnitude, *N(m)*, for the truncated exponential relationship is

$$N(m) = \alpha(m^o)\frac{10^{-b(m-m^o)} - 10^{-b(m^u-m^o)}}{1 - 10^{-b(m^u-m^o)}} \quad (4)$$

where $\alpha(m^o)$ is the annual frequency of occurrence of earthquake greater than the minimum magnitude, m^o; *b* is the Gutenberg-Richter parameter defining the slope of the recurrence curve; and m^u is the upper-bound magnitude event that can occur on the source. A m^o of **M** 5.0 was used for the hazard calculations.

A popular model often used in PSHA is where faults rupture with a “characteristic” magnitude on specific segments; this model is described by Aki (1983) and Schwartz and Coppersmith (1984). For the characteristic model, the numerical model of Youngs and Coppersmith (1985) is often used. In the characteristic model, the number of events exceeding a given magnitude is the sum of the characteristic events and the non-characteristic events. The characteristic events are distributed uniformly over a $\pm$ 0.25 magnitude unit around the characteristic magnitude and the remainder of the moment rate is distributed exponentially using equation 4 with a maximum magnitude 0.25 unit lower than the characteristic magnitude (Youngs and Coppersmith, 1985).

The maximum moment model can be regarded as an extreme version of the characteristic model. The model proposed by Wesnousky (1986) is often used when there is no exponential portion of the recurrence curve, i.e., no events can occur between the minimum magnitude of **M** 5.0 and the distribution about the maximum magnitude.

The recurrence rates for the fault sources are defined by either the slip rate or the recurrence interval for the maximum or characteristic event and the recurrence *b*-value. The slip rate is used to calculate the moment rate on the fault using the following equation defining the seismic moment:

$$M_o = \mu\ A\ D \quad (5)$$

where M_o is the seismic moment, μ is the shear modulus, A is the area of the rupture plane, and D is the slip on the plane. Dividing both sides of the equation by time results in the moment rate as a function of slip rate:

$$\dot{M}_o = \mu \, A \, S \quad (6)$$

where $\dot{M}_o$ is the moment rate and S is the slip rate. M_o has been related to moment magnitude, **M**, by Hanks and Kanamori (1979):

$$\mathbf{M} = 2/3 \log M_o - 10.7 \quad (7)$$

Using this relationship and the relative frequency of different magnitude events from the recurrence model, the slip rate can be used to estimate the absolute frequency of different magnitude events.

The average return time for the characteristic or maximum magnitude event defines the high magnitude (low likelihood) end of the recurrence curve. When combined with the relative frequency of different magnitude events from the recurrence model, the recurrence curve is established.

2.1.2 Background Earthquakes

The inventory of fault sources in the CEUS is small and undoubtedly incomplete. Given this shortcoming, the historical seismicity is used as a proxy to address the hazard from buried or unknown faults in the CEUS. This background seismicity was modeled such that the spatial density of historical seismicity was assumed to be stationary; in the CEUS-SSC model, recurrence rates for small areas were computed and smoothed using a penalty-likelihood approach.

For areal source zones, only the area, seismogenic thickness, maximum magnitude, and recurrence parameters (based on the historical earthquake record) need to be defined. Also, only a truncated exponential recurrence relationship is assumed appropriate.

2.2 Ground Motion Characterization

To characterize the ground motions at a specified site as a result of the seismic sources considered in the PSHA, we used the state-of-the-art GMMs for spectral accelerations. The uncertainty in ground motion models was included in the PSHA by using the log-normal distribution about the median values as defined by the standard deviation associated with each model. Five standard deviations about the median value were included in the PSHA.

3.0 SEISMOTECTONIC SETTING AND HISTORICAL SEISMICITY

The historical seismicity record used in this study was adopted from the CEUS-SSC model updated with data from the USGS Advanced National Seismic System (ANSS) catalog to August 2018. This updated catalog was used solely to illustrate the most up-to-date seismicity in the site region. It was not used in any update to recurrence in the CEUS-SSC model. Induced earthquakes such as those in Texas and Oklahoma (Figure 3) contained in the ANSS catalog were not removed. The hazard from induced earthquakes is not accounted for in the CEUS-SSC model.

3.1 SEISMOTECTONIC SETTING

The site is located in the Gulf Coastal Plain physiographic province (Figure 2). The Gulf Coastal Plain physiographic province extends from the Gulf of Mexico about 200 km into southeastern Texas. It includes the subprovinces of the Coastal Prairies, Interior Coastal Plains, and Blackland Prairies, which parallel the coast and extend progressively farther inland (Bureau of Economic Geology, 1996). It is underlain by crust formed during the breakup of Pangaea and the formation of the Gulf of Mexico. Within this area, the crust is composed of three large domains, including, from north to south, thick transitional crust, thin transitional crust, and oceanic crust (Sawyer *et al*., 1991).

3.2 REGIONAL AND LOCAL SEISMICITY

The historical and instrumental records indicate a relatively low level of seismicity in the Gulf region (Figure 3). The site is located at the very southern tip of the Southern Appalachian Seismic Zone (SASZ). Damaging earthquakes have been infrequent in Alabama although since about 2009, there has been an increase in number of small-to-moderate size earthquakes in specific regions of the eastern United States such as Alabama due to anthropogenic causes. Some if not most of the earthquakes are induced due to coal mining.

In past few decades, the regional seismic network coverage in the southeastern U.S. has been sparse. In 1980, the only operating seismic station in Alabama was located in the northwestern corner of the state. Because of the insufficient number of seismic stations in the state, there are large uncertainties associated with the earthquake locations (e.g., latitude, longitude, and depth) (Frankel, 1995). However, the more recent earthquakes are better located due to better network coverage.

The SASZ extends from southern Virginia to the Alabama-Mississippi border (Figure 3). Crustal uplift and compression in the SASZ have resulted in the reactivation of ancient faults (Thomas, 2006). Steigert (1982) considered the SASZ as one of the most active intraplate seismic zones in the eastern United States. The Bahamas fracture runs from the Florida Panhandle northwest across lower Alabama, into Mississippi is another primary seismic zone in this area. Additionally, large earthquakes within the New Madrid seismic zone have the potential to generate ground

shaking in northern Alabama as illustrated by the 1811-1812 earthquake sequence with **M** > 7. The USGS estimates the probabilities of occurrence of major earthquakes in the NMSZ in 50 years to be 25%-40% and the probability of the occurrence of a similar earthquake of 1811-1812 in the same time period as 7%- 10% (Petersen *et al*., 2015).

The historical seismicity record for Alabama extends back to the first recorded earthquake in 1886. The USGS maintains a comprehensive catalog for earthquakes with **M** ≥ 3 dating back to 1568 (https://github.com/usgs/nshmp-haz-catalogs). This catalog was used for the 2018 National Seismic Hazard Map (Petersen *et al*., 2020).

The first significant earthquake in Alabama was the 1916 **M** 5.0 Irondale, Jefferson County, event which is the largest earthquake in the state to date. The event caused five water wells to dry out immediately after the mainshock and decreased the water level in many wells. The 1931 **M** 3.9 earthquake near Birmingham, Jefferson County, earthquake was also felt in Georgia and South Carolina. The 1957 **M** 4.0 earthquake near Birmingham, Jefferson County, earthquake was felt in Georgia and Tennessee. The 1959 **M** 3.8 Hazel Green, Madison County, event was the strongest earthquake in Madison County in northern Alabama. The event was felt in the state and Tennessee. The 1975 **M** 3.9 earthquake near Palmerdale, Jefferson County, caused slight damage in the town of Watson and was also felt in Tennessee. The 1989 **M** 3.9 earthquake between Littleville and Russellville in Colbert County was felt in Florence, Lauderdale, Lawrence, and Morgan (Stover, 1993).

More recently, Alabama has also experienced some notable earthquakes such as the: 1) 1986 local Richter magnitude (M_L) 3.6 Tuscaloosa earthquake which was associated with a rockburst and roof collapse in an active long-wall coal mine (Long and Copeland, 1989); 2) the 1997 **M** 4.9 Escambia earthquake possibly due to hydrocarbon recovery (Gomberg and Wolf, 1999) (Figure 6); 3) the 2001 **M** 3.9 Pleasant Grove earthquake east of Huntsville; 4) the 2003 **M** 4.6 Fort Payne earthquake in northeastern Alabama; and 5) an event in 2004 **M** 4.2 southwest of Tuscaloosa. Although most of the earthquakes occurring in the state are shallow, a 15-km depth was estimated for the 2003 Fort Payne event. Both the Escambia and Fort Payne earthquakes caused some minor ground failure and structural damage. The Fort Payne earthquake also caused a shutdown of the underground water supply for the town of Valley Head. Prior to the onset of the recent Greene County cluster, the historical record contains 10 earthquakes recorded in the Greene County area (Chen, 2017). This earthquake cluster may have been induced due to wastewater disposal, oil and gas extraction, or mining (Chen, 2017). Chen and Wolf (2018) suggested that the swarm more likely represents a continuation or southwesterly migration of the SASZ.

In terms of seismicity in the Gulf Coast, an earthquake of **M** 5.8 occurred more than 500 km km east-southeast of the site on 10 September 2006 (Figure 3). This is the largest event ever observed in the Gulf of Mexico and so it is an important event in the context of the seismotectonic setting of the Gulf of Mexico. Although significantly offshore in the Gulf, the earthquake was felt widely along the coast of the southeastern U.S. Items were knocked off shelves in Florida and

water was sloshed in swimming pools. The earthquake occurred at a depth of about 10 km according to the NEIC and resulted from thrust faulting with a smaller strike-slip component.

The 10 September 2006 earthquake was preceded by another large event of **M** 4.9 on 10 February 2006 also in the Gulf of Mexico (Figure 3). This event was located about 550 km from the site. Gangopadhyay and Sen (2008) proposed that the 10 February Green Canyon earthquake may be due to a stress concentration in mechanical properties between the salt and surrounding sediments. Similarly, Nettles (2007) suggests the 10 February earthquake is the result of a gravity-driven displacement on a low-angle detachment surface within the sedimentary wedge.

The largest and closest earthquake within 100 km of the site based on the historical record is the 1997 **M** 4.9 Escambia (Figures 3 and 8).

3.2.1 New Madrid Earthquakes

Although just more than 500 km away from the site, the most significant earthquakes to have occurred in the central U.S. in terms of hazard are the 1811-1812 New Madrid earthquake sequence which occurred in the New Madrid fault system (NMFS) (Section 4.1.4; Figures 3 and 4). The New Madrid earthquake sequence occurred over the winter of 1811-1812 in southeastern Missouri/northeastern Arkansas and was felt as far away as Boston. It consisted of three principal events on 16 December 1811, 23 January 1812, and 7 February 1812 (referred to as NM1, NM2, and NM3, respectively in Hough *et al*., 2000). Despite considerable analysis, there remains significant uncertainty regarding the sizes of these three earthquakes although all three events appeared to be larger than **M** 7. Because the New Madrid region was sparsely populated at the time of the events, little structural damage occurred, and the maximum Modified Mercalli (MM) intensity was VIII (NM1) as reinterpreted by Hough *et al.* (2000). Based on the isoseismal map of NMI, the site may have been subjected to mild ground shaking of MM III.

4.0 PSHA INPUTS

The inputs into the PSHA are described below.

4.1 SEISMIC SOURCES

Seismic source characterization is concerned with three fundamental elements: (1) the location, geometry, and characteristics of significant sources of future earthquakes; (2) the maximum size of these earthquakes; and (3) the rate at which different size earthquakes occur. Two types of seismic sources were considered in this PSHA: discrete fault or fault zone sources and regional seismic source zones (Figure 6). The following sections describe the CEUS-SSC model, which was used in this PSHA. The model was used as originally developed by EPRI/DOE/NRC (2012) and not updated in this study. An update was not deemed necessary based on LCI studies performed elsewhere in the CEUS. However only the portion of the model was used in the PSHA. For the seismic zones and fault sources, the model out to 640 and 1,000 km were used, respectively (Figures 7 and 8).

In the CEUS-SSC model, there are two zonation models that account for earthquakes associated with buried or generally unknown faults (background) that were characterized and included in the PSHA; these models include multiple zones, many having alternative geometries (Figures 7 and 8). Background sources were simplified by modeling them as point sources. This includes collapsing background source logic tree branches to simplify the alternatives shown in Tables H-3-1, H-3-2, H-4-2, and H-4-3 of EPRI/DOE/NRC (2012) regarding seismogenic crustal thickness, rupture characteristics, sense of slip, orientation, and dip. Correction factors are applied to convert epicentral distance to an equivalent rupture distance and to incorporate additional aleatory variability in ground motion estimates to account for random rupture orientation with respect to the site (EPRI, 2013a).

A major challenge in understanding the earthquake potential in the CEUS has been associating the observed seismicity with specific geologic structures. Few active faults are known east of the Rocky Mountains. Thus, the traditional approach in addressing the seismic hazard in the CEUS has been to rely on the historical earthquake record in conjunction with seismic source zones that separate regions of different seismotectonic characteristics and hence possibly different earthquake potential. Each seismic source zone is defined and characterized according to geologic, tectonic, and seismicity data. The zones comprise regions having a common geologic history that distinguishes them from neighboring areas. They may have a similar structure (e.g., faults or fractures of similar age, type, orientation), a similar pattern of seismicity, and/or a homogeneous stress regime. The CEUS-SSC model retains this methodology by dividing the CEUS into numerous "seismotectonic zones," defined by differences in various seismic source assessment criteria such as style of faulting, earthquake recurrence, maximum magnitude, seismogenic thickness, etc. (Figure 7). The model includes an alternative approach to dividing the CEUS into source zones, which is based solely on the expected maximum magnitude (Mmax)

in the zone. This alternative zonation approach divides the study area into “Mmax zones” (Figure 8). The seismotectonic zone approach receives slightly higher weight, 0.6, than the Mmax zone approach. There are two Mmax zones and 12 seismotectonic zones in the CEUS-SSC model. The Mmax zones and some seismotectonic zones have one or more alternate geometries.

Table 2 summarizes the source zone parameters used in the analysis. The seismic source zones include the Extended Continental Crust–Gulf Coast (ECC-GC) and the Gulf Highly Extended Crust (GHEX) zones, the former of which includes the site (Figure 7).

The source parameters for several fault sources or repeated large magnitude earthquakes (RLME) as they are referred to in the CEUS-SSC model (Figure 5) were characterized for input into the hazard analysis. There are no known seismogenic faults within 400 km of the site (Figure 4). The CEUS-SSC model includes sources defined based on RLMEs rather than only fault sources. Many of the RLMEs correlate with identified geologic faults, but some are defined solely by geographically clustered paleoliquefaction events that suggest a localized source even if the responsible fault has not been identified and characterized. The NMFS and its associated elements, although quite distant (Figure 6), was included in the PSHA because its maximum earthquakes and relatively high activity rate often dominate the hazard in the CEUS, particularly at long-period ground motions. The Reelfoot Rift-Eastern Rift Margin (ERM) fault, the Reelfoot Rift-Marianna fault, the Reelfoot-Commerce fault zone, and the Wabash Valley fault zone were also included in the PSHA. Tables 2 and 3 summarize the RMLE source parameters used in the analysis.

The following section describes the most significant seismic sources that contribute to the hazard at the site and that were characterized in the PSHA.

4.1.1 Seismotectonic Zones

Gulf Coast Highly Extended Crust Zone (GHEX)

The Gulf Highly Extended Crust (GHEX) seismotectonic zone, which contains the site (Figure 7), corresponds to Sawyer *et al*.’s (1991) “thin transitional crust”, which underwent extension in the middle Jurassic prior to emplacement of oceanic crust and is basinward of the early Cretaceous carbonate platform. It is more extended and has subsided more than “thick transitional crust” to the north; it reaches thicknesses of 8 to 15 km and is buried to depths of 10 to 16 km below sea level by post-rift marine and terrigenous sediments (Sawyer *et al*., 1991). The source zone includes most of the belt of margin-parallel growth faults that accommodate the gravitational failure and extension of the post-rift sedimentary sequence. Earthquakes in this zone are likely to occur on pre-existing basement structures associated with rifting and development of the Gulf of Mexico (EPRI/DOE/NRC, 2012).

As discussed in Section 3.2, the Gulf of Mexico is a region of low seismicity (Figure 3). The largest observed earthquakes have been less than **M** 6.0. Maximum magnitudes for the GHEX zone

range from **M** 6.0 to 8.1, with **M** 7.2 preferred (Table 1). The extended seismogenic crustal thickness is assumed to be limited by the crustal thickness. Therefore, the model includes values of 8 and 15 km, weighted equally (Table 1).

Extended Continental Crust - Gulf Coast (ECC-GC)

The Extended Continental Crust–Gulf Coast (ECC-GC) seismotectonic zone forms an approximately 200 to 300-km-wide band paralleling the coast just inland of the GHEX zone (Figure 7); to the east it broadens and extends offshore where it is approximately defined by the northwest-trending Bahamas fracture zone that extends from the tip of the Florida Peninsula to the northwestern Gulf of Mexico, albeit with some modifications to include all Mesozoic extended crust in the zone (EPRI/DOE/NRC, 2012; Sawyer *et al*., 1991; Thomas, 2006). It is underlain by Sawyer *et al*.'s (1991) "thick transitional crust", which is inland of the early Cretaceous carbonate margin and is characterized by moderate thickness (about 20 to 25 km) and subsidence (2 to 12 km below sea level). The thickness of the crust is variable, however, with high regions of normal continental thickness interspersed with thinner, more extended low regions and can range between 20 and 40 km (Sawyer *et al*., 1991). This thick transitional crust was also thinned during opening of the Gulf of Mexico but not as drastically as that occupying the GHEX zone. The ECC-GC zone is distinguished from the zones to the north and south by its different crustal thickness and history of Mesozoic extension, which is likely to yield a different Mmax for the zone.

Similar to the GHEX zone, the ECC-GC zone has exhibited a low level of historical seismicity (Figures 3 and 7). Mmax for the zone is uncertain, and the CEUS-SSC model utilizes three methods for estimating it based on three different magnitude earthquakes: the largest pre-instrumental observed earthquake ([**M**] 5.6), the largest instrumentally observed earthquake (**M** 4.9), and the largest paleo-earthquake capable of producing liquefaction features observed near the junction of Arkansas, Louisiana and Mississippi (**M** 5 to 6.5) (e.g., Cox *et al*., 2007; Cox, 2002; 2009). Maximum magnitudes for the ECC-GC zone range from **M** 6.0 to 8.1, with **M** 7.2 preferred (Table 1). Recurrence for the ECC-GC is discussed in Section 4.1.3. The seismogenic crustal thickness of the ECC-GC zone is somewhat greater than that of the GHEX zone and is represented by a distribution of 13 km (0.4), 17 km (0.4), and 22 km (0.2) (Table 1).

4.1.2 Mmax Zones

The foregoing seismotectonic source zone discussion describes a model for addressing areal sources and background seismicity in which the region is divided into seismic source zones based on distinctive geologic and/or seismic characteristics. In addition, we include a model for source zonation from EPRI/DOE/NRC (2012) in which the region is divided into only two zones: the extended margin, comprising Mesozoic and younger extended crust (MESE), and the non-extended craton (NMESE) (Figure 8). The model is based on the observation that within the global catalogue of earthquakes within stable continental regions, there is little to distinguish any of them in a statistically significant way except those larger earthquakes seem to occur more commonly

within those parts of the stable continental regions that have undergone extension, especially Mesozoic or younger extension (Johnston *et al*., 1994). Consequently, the model is based on using global analogues to characterize the maximum magnitudes, with regions divided into extended and cratonic categories, each with a different distribution of maximum magnitudes. We adopt the zone boundaries and maximum magnitude distribution of the CEUS-SSC model.

The CEUS-SSC statistical analysis of the global database of earthquakes in stable continental regions (SCR) showed that the distinction between Mesozoic extended crust and non-extended crust noted by Johnston *et al*. (1994), while present, is only marginally significant. Therefore, within the Mmax zonation approach, two models are included: 1) the CEUS is divided into two Mmax zones (MESE and NMESE), each with its own Mmax distribution, based on the presence or absence of Mesozoic-extended crust, and 2) the CEUS can be described by a single Mmax zone with a single Mmax distribution (Study R). The former model has slightly higher weight because of the marginally significant difference observed in the statistical analyses.

Mesozoic and Younger Extended Crust (MESE)

The Mesozoic extended zone (MESE) includes areas that underwent Paleozoic and Mesozoic or younger extension and includes the Atlantic and Gulf coastal regions as well as the failed rifts in the central U.S. (including the Reelfoot Rift and southern Oklahoma aulocogen) (Figure 8).

Non-Mesozoic and Younger Extended Crust (NMESE)

The Non-Mesozoic and Younger extended crust (NMESE) include that part of the CEUS stable continental region that has not undergone Mesozoic or younger extension (Figure 8). This includes primarily interior cratonic regions and overlaps significantly with the MidC seismotectonic zone.

The boundaries between the extended and non-extended Mmax zones have two alternatives, reflecting uncertainty in the geographic extent of extended crust. The MESE-N (N = “narrow”) includes regions that have definitively experienced Mesozoic extension as determined based on the presence of certain distinguishing characteristics. These may include Mesozoic grabens and rift basin, Mesozoic and younger plutons, Mesozoic and younger uplift and unroofing associated with normal faulting (EPRI/DOE/NRC, 2012). Generally, regions that meet most of these criteria are considered to be extended and are assigned to the MESE-N zone. Regions with less compelling evidence, such as localized Mesozoic and younger reactivation of older structures or the presence of structures favorably oriented for reactivation, are less certainly extended and are assigned to the MESE-W (W = “wide”) zone. The NMESE-N and NMESE-W zones include the rest of the CEUS region outside the MESE-N and MESE-W zones, respectively. The narrow boundary, dividing definitively extended crust from the rest of the craton receives most of the weight (0.8) due to the lack of clear evidence for extension in the MESE-W zone.

The narrow and wide geometry for each zone has its own maximum magnitude distribution for this region, based on the largest historical earthquake known in each zone. These appear in Table 1 (Table 6.3.2-1 in EPRI/DOE/NRC, 2012).

Study Region (Study R)

The single-zone alternative of the Mmax zone model is the Study Region source zone, which encompasses the entire CEUS. The Study R source zone is represented by a single Mmax distribution (Table 1).

4.1.3 Recurrence for Seismic Zonation

The CEUS-SSC model is based on the spatial stationarity of seismicity, which is defined from small- to moderate-magnitude earthquakes that have occurred during a relatively short historical and instrumental record (EPRI/DOE/NRC, 2012). For the seismotectonic and Mmax source zones, the seismicity rates are determined from the historical seismicity catalog. All dependent earthquakes were removed from the catalog, and earthquakes associated with the RLME sources were also removed to avoid double-counting. The cell size for all seismotectonic source zones except MidC was 0.25 degrees; the cell size for MidC was set to 0.5 degrees. The spatial smoothing operation, a penalized-likelihood function, is based on calculations of earthquake recurrence within each cell. Both *a*- and *b*- values are allowed to vary, but the degree of variation has been optimized such that *b*-values vary little across the study region, and the *a*-values are neither too smooth nor spikey. Also, the recurrence calculations consider weighting of magnitudes in the recurrence rate calculations, with moderate events assigned more weight than smaller events.

Five alternative cases were considered for weights, which affect the degree of smoothing, for various magnitude bins; Cases A, B, C, D, and E (EPRI/DOE/NRC, 2012). Case C was dropped as it is very similar to Case B, and Case D was considered too extreme. Thus for each source zone three magnitude weighted cases were used: A, B, and E, with weights of 0.3, 0.3, and 0.4, respectively.

Furthermore, more than point estimates of the recurrence parameters are needed as modern PSHA requires an assessment of the epistemic uncertainty associated with these estimates, including correlations between the recurrence parameters of cells in the same geographical region, which may jointly affect the hazard at one site. The approach used to generate alternative maps of the recurrence parameters uses a technique known as Markov Chain Monte Carlo (MCMC) (EPRI/DOE/NRC, 2012).

This resulted in eight alternative maps representing the epistemic uncertainty in recurrence parameters that result from the limited duration of the catalog. The eight realizations are equally weighted in the CEUS-SSC model.

4.1.4 RLME

The NMFS RLME is the only RLME that is significant to the site (Figures 4 and 6).

NMFS RLME

The New Madrid Seismic Zone (NMSZ) is the source of the 1811-1812 New Madrid earthquake sequence, which includes three of the largest earthquakes to have occurred within the North American plate in historical times (Johnston and Shedlock, 1992) (Figure 3). The pattern of seismicity and surface uplift is generally interpreted as delineating a left-stepping, right-lateral, strike-slip fault system (Cox *et al*., 2001; Johnston and Schweig, 1996). Johnston and Schweig (1996) developed faulting models for the 1811-1812 sequence based on geological, geophysical, seismological, and historical data. They concur with the commonly held assumption that the current seismicity is illuminating the most active faults, i.e., those that ruptured in 1811–1812 and also prior to 1811.

Schweig and Ellis (1994) and Johnston and Schweig (1996) provide summaries of the seismological, geodetic, and paleoseismologic data that have been used to assess the repeat times of large-magnitude events in the New Madrid region. In addition, Wheeler and Perkins (2000) provide additional information from the 2002 USGS National Hazard Maps for the CEUS. Correlation of dated liquefaction features suggest that widespread liquefaction occurred within the zone in A.D. 1811-1812, 1450, 900, 300 as well as about 2350 B.C. (Tuttle *et al*., 2005). Liquefaction deposits can constrain the ages of prehistoric events but not the causative faults. However, several of the prehistoric liquefaction deposits are composite, indicating they were formed in multiple episodes within a short period and thus may have occurred in a rapid sequence of large earthquakes similar to the 1811-1812 sequence.

The occurrence of two large events in A.D. ~900 and 2500-1400 B.C. is supported by studies of Mississippi River channel morphology that suggest that the Mississippi River changed its course in response to a sudden localized change in base level at those times (Holbrook *et al*., 2006). That change in base level is attributed to uplift of the downstream side of the channel across the Reelfoot reverse fault (described below).

These paleoseismic results indicate a recurrence interval of about 500 years for large earthquakes or earthquake sequences in the NMSZ over the past 2000 years. The absence of paleoseismic evidence for earthquakes between 300 A.D. and ca 2200-2350 B.C. has been cited as indicative of temporal clustering of earthquakes in the NMSZ, with large earthquakes or earthquake sequences happening every few hundred years over a period of time followed by a long hiatus in activity (Holbrook *et al*., 2006). However, at this point it remains uncertain if the lack of events documented between A.D. 300 and 2200 B.C. in New Madrid is due to clustering or an incomplete paleoseismic record.

The possibly clustered behavior in the NMSZ, coupled with the discovery of paleoliquefaction features in the Reelfoot Rift southwest of the New Madrid zone (indicative of large earthquakes between about 5,000 and 7,000 years ago but not during the New Madrid cycles), has led to the suggestion that the locus of earthquake activity moves around the Reelfoot Rift, on time scales of 5 to 15 kyr. In this model, the New Madrid region is the current, or most recent, locus of activity, but other areas have been so in the past, and the locus may shift again.

In the CEUS-SSC model, the elevated seismicity in the NMSZ is included in the RR seismotectonic zone, whereas large historical and paleoseismic events that likely occurred on the structures that ruptured in 1811-1812 are modeled as part of the NMFS RLME, in keeping with the CEUS-SSC model. The source zone accommodates the hazard from background seismicity; the NMFS contributes an additional hazard (Tables 2 and 3). In the seismic source model, the NMFS comprises three distinct fault zones, located within the NMSZ source zone. The three NMFS faults, defined after the models of Van Arsdale (2000) and Johnston and Schwieg (1996), include: 1) the southern section (NMS), comprising the Blytheville arch (BA), extending into the Blytheville fault zone (BFZ) and Bootheel lineament (BL) area, 2) the central section, comprising the Reelfoot reverse fault (RFT), and 3) the northern section, comprising the New Madrid North fault and the Northwestern Seismicity Arm (NMN) (Table 2). Each of these sections ruptured to produce the 1811 and 1812 earthquakes.

There is significant uncertainty regarding the exact identification and geometry of the faults that ruptured in the 1811-1812 and earlier earthquakes, and some models of rupture (e.g., EPRI/DOE/NRC, 2012; STNOC 2011; USNRC, 2006) include weighted alternative geometries for each of the three faults. We adopt the characterization of EPRI/DOE/NRC (2012; Table 2). We include two alternative geometries for the northern extent of the southern section, the Blytheville fault zone (NMS: BA-BL), weighted 0.4, and the Bootheel Lineament (NMS: BA-BFZ), weighted 0.6. For the central and northern sections, we include two alternatives: short and long (RFT-S, RFT-L, NMN-S, MNM-L). The short central section (RFT-S) includes only that part of the Reelfoot reverse fault that is defined by the Reelfoot scarp and extends from the Blytheville fault to the New Madrid North fault; the long alternative (RFT-L) extends both east and west, based on continued seismicity. The short alternative for the New Madrid north fault (NMN-S) is the fault as defined by Johnston and Schweig (1996); the long alternative (NMN-L) extends the source along northward continuations of seismicity identified by Wheeler (1997). Because the causative faults are not well understood, the dips are not well constrained. The northern and southern sections of the system are modeled as vertical. The Reelfoot thrust fault is modeled with a 40-degree southwest dip.

The EPRI/DOE/NRC (2012) characterization also addresses the apparent clustering of activity along the NMFS faults. The rate of earthquakes and geomorphic expression of faulting on the RFT in the late Holocene suggests that the system is or has recently been in a cluster. However, geodetic data gathered over the last decade or so suggest that little or no interseismic deformation

is occurring across the NMSZ, which some researchers have interpreted as evidence that the system is shutting down and entering an inter-cluster period of quiescence (e.g., Calais *et al*., 2005; Calais and Stein, 2009). The CEUS-SSC model strongly favors the interpretation that the system is currently in a cluster (0.9), based on the recent history of activity and the unlikelihood that we have just happened upon the exact moment the system is shutting down. However, they, and we, give some weight to two alternative models: 1) only the RFT is currently in a cluster, and the other faults are quiescent (0.05), and 2) the entire system is out of a cluster (0.05). In the former case, the RFT is active, but at a lower rate than the in-cluster case; in the latter case, no faults are active and the system defaults to the RR background zone characterization. The CEUS-SSC model includes distributions for the NMS, RFT, and NMN sections of the NMFS of **M** 6.7 to 7.9, **M** 7.1 to 7.8, and **M** 6.8 to 7.6, respectively.

4.2 GROUND MOTION MODELS

Crustal GMMs for tectonically active regions like the western U.S. are empirical in nature and derived from strong motion data from such areas as California, Taiwan, Japan, and Italy. In contrast, few useable strong motion records exist for earthquakes in the Central and Eastern North America (CENA). Thus, GMMs for the CENA have been developed, in large part, using seismological-based numerical models. During the past decade, ground motion models for the CENA have been derived using three different approaches: the stochastic method, the Green's function method, and the complex/empirical source method.

The Pacific Earthquake Engineering Research Center (PEER) developed a new ground-motion characterization (GMC) model applicable for use in central and eastern North America (CENA, east of approximately 105 degrees west) through the Next Generation Attenuation for Central and Eastern North America project (NGA-East) (Goulet *et al*., 2018) (Figure 5). The GMC model consisted of two parts for use in a PSHA:

- A set of new GMMs for the median ground motions and their associated weights in the associated logic-trees.
- A set of new models for the aleatory standard deviations of ground motion, and their associated logic trees.

The final results and models of NGA-East were developed under the framework of the Seismic Senior Hazard Analysis Committee (SSHAC) (Budnitz *et al.*, 1997; NRC 2012) Level 3.

The NGA-East GMC is to be used in conjunction with the CEUS SSC products (EPRI/DOE/NRC, 2012). The CEUS-SSC Project was also conducted under the SSHAC Level 3 framework; thus, having the GMC component of the NGA-East under SSHAC Level 3 framework will ensure that both projects are of the same high-quality level for the end-uses.

The NGA-East ground-motion database (Goulet *et al.*, 2014) included the two- and three-component ground-motion recordings from numerous selected events (**M** > 2.5, distances up to 1500 km) recorded in the CENA region since 1988, up to December 2011. The database contains

over 29,000 records from 81 earthquake events and 1379 recording stations. The events included "mainshocks," "aftershocks," and tectonic events, as well as potentially induced events (PIEs). The NGA-East database constitutes the largest database of processed recorded ground motions in stable continental regions (SCRs).

The NGA-East Project provided the best estimate of the distribution (median and standard deviation) of RotD50 (Boore, 2010) horizontal ground motions for horizontal peak ground acceleration (PGA), peak ground velocity (PGV), and 5%-damped pseudo-acceleration response spectra (PSA) for oscillator periods ranging from 0.01 to 10 sec on reference "hard-rock" site condition. The reference site conditions were defined by the NGA-East Geotechnical Working Group as corresponding to a V_S of 3000 m/sec and a kappa (κ) of 0.006 sec (Hashash *et al*., 2014; Campbell *et al*., 2014). The significance of the reference rock definition was that it represents the site condition for which ground motions will be predicted using the GMMs, and it represents the site condition to which site amplification factors are referenced (i.e., site amplification is unity for reference rock).

Final models in NGA-East are applicable to **M** 4.0 to 8.2 range, and for distances up to 1500 km from earthquakes source, using rupture distance as a distance metric. For the median GMMs, a total of 30 median GMMs were initially considered. Ten of the models were from the recent EPRI Review Project (EPRI, 2013). Ultimately, none of the EPRI (2013) models were used in the development of the final GMMs; this was primarily because they have been updated and replaced by more recent models. Following the evaluation of the candidate GMMs, the Technical Integrator (TI) team selected 19 GMMs for further investigation. A certain level of smoothing of the extrapolated GMMs was also carried out. Some GMMS were selected for a limited range of frequencies. This set of 19 selected, extrapolated, and smoothed GMMs are denoted as "seed GMMs." This GMM set was used to generate a continuous distribution of GMMs for each GMM. A set of 10,000 GMMs was sampled from the continuous GMM distribution developed above. These GMMs are referred to as sampled GMMs. At this stage, Sammon's (1969) maps were used to visualize the ground-motion space spanned by the seed and sampled GMMs. The 10,000 "sampled GMMs" were "grouped' into 17 cells in the Sammon's map space. For each of these 17 cells, a process to define a representative model in each cell was defined. The 17 resulting GMMs are the final NGA-East GMMs, which are also referred to as the "representative sampled GMMs". These 17 GMMs were then assigned weights by the TI team for logic tree and PSHA applications. Therefore, the final median ground-motion logic tree, consists of 17 branches, each corresponding to a GMM.

The NGA-East Project evaluated regions of different geologic and tectonic settings in the CENA. The regions are the Central North America (CNA), the Appalachian Province (APP), the Atlantic Coastal Plain (ACP), and the Gulf Coast/ Mississippi Embayment Region (GCR). Each region is described by a statistically representative crustal seismic velocity-depth structure and Q-factor model. The crustal structural models are for very hard rock conditions and do not include any

sediments. The ground motion analyses for different regions demonstrates that there are two distinct attenuation groups:

- Mid-continent region (MCR): Central North America, Appalachians, and the Atlantic Coastal Plain.
- Gulf Coast Region (GCR): Gulf Coast including the Mississippi Embayment.

The GCR was found to clearly belong to a separate attenuation group. This result is in agreement with previous analyses that have found that the GCR has unique attenuation characteristics. The NGA-East base median GMMs are defined for the MCR. Adjustment models for the GCR are developed in the NGA-East Project. The site is situated within the GCR (Figure 1).

In addition to regionalization, the NGA-East Project developed adjustment models for source depth and hanging-wall effects. The source-depth adjustment accounts for depth dependence of the stress parameter. The hanging-wall adjustment accounts for the increase in ground motions at sites located above dipping earthquake ruptures compared to what would be expected at sites located at the same rupture distance but on the footwall side of the rupture.

In NGA-East GMC, standard deviation models are provided for site-specific analysis (single-station standard deviation) and for general PSHA applications (ergodic standard deviation). PSHA studies typically aim to evaluate the hazard at a single site. This requires characterizing the ground motion (median and variability) at the particular site due to the future occurrence of different earthquakes. Since repeated recordings at the site of interest are usually not available, the ergodic assumption is adopted, whereby the ground-motion variability over different sites and source regions is assumed applicable to an individual site. In such cases, the use of the ergodic assumption is valid as the PSHA is typically concerned with rare events with the likelihood of repeated occurrences being very low. More recently, the availability of repeated recordings at individual stations in ground-motion databases allowed for the estimation of the site-to-site variability and for removing it from the ground-motion aleatory variability. This led to a reduced aleatory variability as observed in previous studies (e.g., Rodriguez-Marek *et al*., 2011; 2013). The application of the single-station approach, however, requires adjusting the median GMMs from representing average site conditions in the dataset used to develop the models to become site-specific. Site-specific adjustments of the median GMMs can be done using empirical ground-motion recordings when available at or near the site of interest or as part of the more commonly used analytical site response studies. The epistemic uncertainty resulting from the limited knowledge of the site-specific parameters and adjustment factors needs to be properly accounted for as part of the single-station approach.

In addition to the reduction in aleatory variability, the use of single-station sigma has multiple advantages. It allows for proper identification of the various components of ground-motion variability where some of the apparent randomness can be transformed to epistemic uncertainty. With the acquisition of additional data, this epistemic uncertainty can, in theory, be removed or decreased. Moreover, the use of single-station sigma has the advantage of avoiding double

counting of the uncertainty in cases where the ergodic sigma is used and a site-specific response study performed. In such cases, the double-counting of the uncertainty is the result of the site-to-site variability being part of the total ergodic sigma, and the uncertainty in the site amplification factors are also captured as part of the site response study.

Nine single-station and ergodic sigma models were developed which represent the discrete approximations of the three continuous distributions of sigma models. To reduce the computational burden in hazard analyses resulting from the use of nine alternative sigma values, the three continuous distributions of the sigma models along with their weights are represented by a composite sigma model. The composite sigma model is then discretized into a three-point distribution. Three discrete points selected at the 5^{th}, 50^{th}, and 95^{th} percentiles. The distributions for sigma show only a small degree of skewness, and the nominal weights of 0.185, 0.63, and 0.185 on the 5^{th}, 50^{th}, and 95^{th} percentiles, respectively, produce values for the mean and standard deviation of σ that adequately represent those computed from the continuous distribution.

The NGA-East Project adopted two alternatives to represent the shape of ground-motion residuals: traditional lognormal distribution with a weight of 0.2 and a mixture of two normal distributions with weight of 0.8. The mixture model consists of an equally weighted mixture of two normal distributions, with one component having 0.8 times the normal sigma and the other component having 1.2 times the normal sigma.

The TI team considered the CENA single-station sigma and total ergodic sigma models are applicable to both MCR and GCR. For this study, the full NGA-East median GMMs logic (17 models with associated weights) are used along with the logic tree given for single-station aleatory standard deviation.

4.3 SITE CHARACTERIZATION

The Lowman Power Plant site is located in the Coastal Plain geomorphic province and is underlain by silty and clayey sand and gravel, and silty clay, associated with recent alluvial, coastal plain, and terrace deposits related to the Tombighee River (CDG Engineers & Associates, 2016). The depth to basement rock is estimated to be at approximately 20,000 ft (6,100 m) (Darragh and Silva, 2012).

A site-specific V_S profile was developed from a total of seven SCPT and CPT logs collected in 2011 (Southern Earth Sciences, Inc.) and 2020 (Walker Hill Environmental), respectively. V_S measurements extended to depths from 45 to 70 ft and ranged from 180 to 1,385 ft/sec. CPT data were normalized to the top of the original ground surface to account for up to 32 ft of fill, and a lognormal average profile was computed and smoothed. The final profile developed is similar to the previous profile from PE&A (2012) at shallow depths, which only used the SCPT data (Figure 9).

At depths greater than 70 ft, two base case profiles were used in the site response analysis to reflect uncertainty in the velocities below the measured profiles. The deeper profile was assumed

to be similar to the profile at the Joseph M. Farley Nuclear Plant, on the west side of the Chattahoochee River in Houston County, Alabama. Both measured profiles extended to 79.2 ft, to match the thickness of the 2012 shallow profile, and then velocity steps were added to join up with the Farley Nuclear Plant profile at greater depths as was done in Darragh and Silva (2012). Profiles P1 and P2 reflect epistemic uncertainty in the step size (200 vs 100 ft). Figure 10 shows the two base case profiles P1 and P2 down to 3 km depth.

5.0 PSHA RESULTS

The hard rock hazard results and a comparison with the 2018 USGS National Seismic Hazard Maps are presented below.

5.1 HARD ROCK RESULTS

The results of the PSHA are presented in terms of ground motion for hard rock site conditions as a function of annual exceedance frequency (AEF). AEF is the reciprocal of the average return period. The hard rock hazard was computed for the 22 periods for which the NGA-East GMMs are defined. The ground motion parameters chosen to illustrate the hazard at the site in this report are PGA and 1.0 sec SA.

Figure 11 shows the mean, median (50th percentile), 5th, 16th, 84th, and 95th percentile hazard curves for PGA. These fractiles indicate the range of epistemic uncertainties about the mean hazard. The uncertainties are large due to the large uncertainties in the GMMs. The difference between the 5th and 95th percentile values at 2,500 years is about a factor of five but at very low ground motions (Figure 11). The 1.0 sec horizontal SA hazard is shown on Figure 12. The hard rock values at the return periods of 500 and 2,500 years are listed in Table 4.

The contributions of the various seismic sources to the mean PGA and 1.0 sec SA hazard are shown on Figures 13 to 16. NMFS controls the PGA hazard at return periods up to 10,000 years and up to 100,000 years for 1.0 sec SA. The NMFS controls the hazard because there is very little hazard from any other seismic sources around the site. This is typical of many sites in the CEUS.

The hazard can also be deaggregated in terms of the joint magnitude-distance-epsilon probability conditional on the ground motion parameter (PGA or SA exceeding a specific value). By deaggregating the PGA and 1.0 sec SA hazard by magnitude, distance and epsilon bins, we can illustrate the contributions by events at 500 and 2,500-year return periods (Figures 17 and 18). The hazard at PGA at both 500 and 2,500-year return periods is from earthquakes centered at **M** 7.25 at distance of 480 km.

Deaggregation by magnitude and distance provides the modal magnitude M^{mode} and modal distance D^{mode}, which aid in defining the controlling earthquakes. The M^{mode} and D^{mode} for the 500 and 2,500-year return periods for PGA and 1.0 sec horizontal SA are provided in Table 5.

Horizontal Uniform Hazard Spectra (UHS) on hard rock computed at seven periods (frequencies) for the 500 and 2,500-year return periods are shown on Figure 19. A UHS shows the hazard across all periods or frequencies for the same annual exceedance frequency or return period.

5.2 COMPARISON WITH USGS NATIONAL SEISMIC HAZARD MAPS

In 1996, the USGS released a “landmark” set of National Seismic Hazard Maps (NSHM) for earthquake ground shaking, which was a significant improvement from previous maps they had developed (Frankel *et al.*, 1996). These maps were the result of the most comprehensive analyses of seismic sources and ground motion prediction ever undertaken on a national scale. The maps are the basis for the NEHRP Maximum Considered Earthquake (MCE) maps, which are used in the International Building Code.

For a 2,500-year return period (2% exceedance probability in 50 years), the latest 2018 NSHMs indicate hard rock PGA and 1.0 sec SA of 0.063 and 0.060 g, respectively, for the site (Petersen *et al.*, 2020). The site-specific hard rock PGA and 1.0 sec SA computed in this study are 0.054 and 0.056 g for the 2,500-year return period (Table 5). The differences are likely due to differences in GMMs although differences in the seismic source characterization may also be a factor.

6.0 SITE RESPONSE ANALYSIS

The site is located on deep soil (Figure 1). The CEUS ground motion models are for hard rock which is at least 7 km deep at the site and so we performed a site-specific site response analysis. Traditionally in the estimation of site-specific probabilistic ground motions for a soil site, a rock ground motion is calculated and modified by deterministic site response analyses derived for the soil column to arrive at the ground motions at the soil surface. In doing so, the annual exceedance frequency of that soil motion is generally unknown, varies with period, and may be of a higher frequency than the control (rock) motion. If a risk analysis is desired, the surface motions must be hazard consistent, i.e., the annual exceedance frequency of the soil ground motion should be the same as the rock ground motion. In NUREG/CR-6728 (McGuire *et al.*, 2001), several site response approaches are recommended to produce soil motions consistent with the rock outcrop hazard. The approaches also incorporate the aleatory variabilities in the soil properties into the soil motions. McGuire *et al.* (2001) identified four basic approaches for determining the ground motions at a soil site. The approaches range from a PSHA using ground motion prediction models for the specific site (or location) of interest (Approach 4) to scaling the rock motion on the basis of a site response analysis using a broadband input motion (Approach 1). Conceptually, Approach 4 is the ideal approach and other approaches are approximations to it.

To compute the ground motions for the site, we implemented Approach 3 as it is called (McGuire *et al.*, 2001; Bazzurro and Cornell, 2004). Approach 3 is a fully probabilistic analysis procedure which moves the site response, in an approximate way, into the hazard integral. The approach is described by Bazzurro and Cornell (2004) and NUREG/CR-6769 (McGuire *et al.*, 2002). In this approach, the hazard at the soil surface is computed by integrating the site-specific hazard curve at generic rock or soil level with the probability distribution of the amplification factors (Lee *et al.*, 1998; 1999). The site-specific amplification, relative to hard rock, is characterized by a suite of frequency-dependent amplification factors that can account for nonlinearity in soil response. Approach 3 involves approximations to the hazard integration using suites of transfer functions, which result in complete hazard curves at the ground surface for specific ground motion parameters (e.g., spectral accelerations) and a range of frequencies.

The basis for Approach 3 is a modification of the standard PSHA integration:

$$P[A_S>z] = \int\int\int P\left[AF > \frac{z}{a}\middle|m,r,a\right] f_{M,R|A}\,(m,r;a) f_A(a) dm dr da \qquad (8)$$

where A_S is the random ground-motion amplitude on soil at a certain natural frequency; z is a specific level of A_S; m is earthquake magnitude; r is distance; a is an amplitude level of the random rock ground motion, A, at the same frequency as A_S; $f_A(a)$ is derived from the rock hazard curve for this same frequency (namely it is the absolute value of its derivative); and $f_{M,R|A}$ is the

deaggregated hazard (i.e., the joint distribution of M and R, given that the rock amplitude is level a). AF is an amplification factor defined as:

$$AF = A_S/a \tag{9}$$

where AF is a random variable with a distribution that can be a function of m, r, and a. To accommodate epistemic uncertainties in site dynamic material properties, multiple suites of AF may be used, and the resulting hazard curves combined with weights to properly reflect mean hazard and fractiles.

Soil response is controlled primarily by the level of rock motion and m, so Equation 8 can be approximated by:

$$P[A_S>z] = \int\int P[AF > \frac{z}{a}(m,a)]f_{M|A}(m;a)f_A(a)dmda \tag{10}$$

where r is dropped because it has an insignificant effect in most applications (McGuire *et al.*, 2001). To implement Equation 10, only the conditional magnitude distribution for relevant amplitudes of a is needed. $f_{M|A}(m;a)$ can be represented (with successively less accuracy) by a continuous function, with three discrete values or with a single point, (e.g., $m^1(a)$, the mean magnitude given a). With the latter, Equation 10 can be simplified to:

$$P[A>z] = \int\int P[AF > \frac{z}{a}|a,m^1(a)]f_A(a)da \tag{11}$$

where, $f_{M|A}(m;a)$ has been replaced with m^1 derived from deaggregation. With this equation, one can integrate over the rock acceleration, a, to calculate $P[A_S>z]$ for a range of soil amplitudes, z.

6.1 Implementation of Approach 3

In Approach 3, the following steps were performed:

- Randomization of base case site-dynamic material properties to produce a suite of velocity profiles as well as G/Gmax and hysteretic damping curves that incorporate site randomness.
- Computation of transfer functions (hereafter termed amplification factors) as characterized by a mean and distribution for each set of base case site properties using the RVT-based equivalent-linear site response model.
- Full integration of the mean hazard curves for the generic site condition in this case hard rock and amplification factors to arrive at a distribution of site-specific hazard curves.

Approach 3 is implemented through a number of computer programs, which are described below. The computation of the amplification factors is the first phase of the calculations and is similar to what is done in other site-response approaches.

6.1.1 RVT-Based Equivalent-Linear Site Response Approach

To compute the ground motions at the ground surface, the results of the PSHA are modified using a site-response model. The conventional site response approach in quantifying the effects of soil and other unconsolidated sediments on strong ground motions involves the use of time histories compatible with the specified outcrop response spectra to serve as control (input) motions. The control motions are then used to drive a nonlinear computational formulation to transmit the motions through the profile.

The computational formulation that has been most widely employed to evaluate 1D site response assumes verticallypropagating plane S-waves. Departures of soil response from a linear constitutive relation are treated in an approximate manner through the use of the equivalent-linear formulation. The equivalent-linear formulation, in its present form, was introduced by Idriss and Seed (1968). A stepwise analysis approach was formalized into a 1D, vertically propagating S-wave code called SHAKE (Schnabel *et al*., 1972). Subsequently, this code has become the most widely used and validated analysis package for 1D site response calculations.

The computational scheme employed to compute the amplification factors in this study uses an alternative approach employing random vibration theory (RVT) (Silva and Lee, 1987). In this approach, as embodied in the computer program RASCALS, the control motion power spectrum is propagated through the 1D soil profile using the plane-wave propagators of Silva (1976). The power spectrum is derived from the uniform hazard spectrum by spectral matching assuming the controlling earthquake. In this formulation only SH waves are considered. Arbitrary angles of incidence may be specified. In this case, vertical incidence was assumed.

Inputs to RASCALS are as follows:

- Location of input and output motions within the site profile.
- Input (control) motions characterized by earthquake power spectra.
- Incidence angles of input motion.
- A representation of the rock and soil at the site, consisting of homogeneous layers with specified thickness, seismic velocity, and density.
- A representation of the dynamic material properties of the rock and soil at the site, consisting of strain-dependent shear modulus and damping curves for each layer.

Control motions (power spectral density) must be calculated for input into the site response analysis that are representative of the earthquake magnitude and distance dominating the hazard at the desired rate of exceedance. The basis for the control motions are the magnitude and distances specified by the hazard deaggregation. Control motions may be specified by a response spectrum, which is then followed by an RVT spectral match to generate a power spectral density. This is then input to the site column as an outcrop motion at the control point. Evaluation

of site-response using the equivalent-linear site response model is based on convolution of appropriate control motions through randomized velocity profiles combined with randomized G/Gmax and hysteretic damping curves. The randomized profiles and curves are generated from base case velocity and nonlinear dynamic properties. The convolutions yield transfer functions for 5%-damped response spectra and peak particle velocity.

A large number of simulations can be required to achieve stable statistics on the response. To achieve statistical stability, 30 randomizations were produced using the velocity correlation models for the base case velocity profile and each base case nonlinear dynamic property curve. In order to randomly vary the V_S profile, a profile randomization scheme has been developed which varies both layer velocity and thickness. The randomization is based on a correlation model developed from an analysis of variance on about 500 measured V_S velocity profiles (EPRI, 1993; Silva *et al.*, 1996). Profile depth (depth to competent material) is also varied on a site-specific basis using a uniform distribution. The depth range is generally selected to reflect expected variability over the structural foundation as well as uncertainty in the estimation of depth to competent material.

6.1.2 Inputs and Analysis

To perform the site response analysis, representative V_S profiles of the site and shear modulus (G/Gmax) reduction and damping curves are required. Thirty randomized V_S profiles were generated from the single base case V_S profile (Section 4.3) using a soil correlation model developed by Silva *et al.* (1996) described in EPRI (2013).

Associated with each of the 30 randomized profiles was also a set of randomized dynamic material property curves. To accommodate the large uncertainty in nonlinear dynamic material properties, the EPRI (1993) (M1) and Peninsular Range (M2) curves for cohesionless soils (Silva *et al.*, 1996) were used to approximate a nonlinear response over the top 500 ft, with linear response below (Silva *et al*., 1996). The second set, termed Peninsular Range curves, which are a subset of the EPRI (1993) curves use the EPRI (1993) 51 to 120 ft curves for 0 to 50 ft and the 501 to 1,000 ft curves for deeper materials and reflect much more linear response than the EPRI curves. The two sets of curves were given equal weights and are considered to cover the range in nonlinear dynamic material properties.

Based on the RASCALS runs for the 30 V_S profiles for the base case profile and material properties, a probability distribution of amplification factors was calculated. Input control motions are computed using RASCALS for each set of 30 V_S profiles and dynamic property curves. RASCALS is used for horizontal spectra using normally-incident and inclined SH-waves. For each control motion, mean and standard deviation are computed from the 30 response spectra (from 30 randomized profiles). Thirty realizations result in stable estimates. The mean response spectrum from the 30 convolutions is divided by the mean (log) spectrum for hard rock spectrum to produce the amplification factors. The amplification factors include the effects of the inherent

aleatory variability (randomness) of the site properties about each base case and any possible effects of magnitude of the control motions.

RASCALS was used to generate control motions and acceleration power response spectra for three earthquakes, **M** 5.5, **M** 6.5 and 7.5 in order to cover fully cover the range of events contributing to the hazard. The events were placed at a suite of distances to produce expected median rock peak accelerations of 0.01, 0.05, 0.10, 0.20, 0.30, 0.40, 0.50, 0.75, 1.00, 1.25 and 1.50 g. This standard suite of input ground motions covers the range of events contributing to the hazard. The amplification factors (the ratios of the response spectra at the top of the site profiles to the hard rock profiles) are a function of the reference (hard rock) peak acceleration (or SA), spectral frequency, and nonlinear dynamic material properties. The linear range of the soil response is from 0.01 to 0.10 g and amplification factors for 0.10 g are used for ground motions less than 0.1 g.

6.2 SITE-SPECIFIC HORIZONTAL RESULTS

The hard rock hazard curves derived from the PSHA and the amplification factors relative to hard rock were integrated to arrive at site-specific amplified hazard curves. The hazard curves calculated using the amplification factors from the **M** 5.5, 6.5 and 7.5 earthquakes were weighted based on their contributions to the hazard at each spectral frequency using the deagregation results (Figures 17 and 18). The uncertainty or epistemic variability in seismic hazard is typically represented by a set of weighted hazard curves. Using these sets of curves as discrete probability distributions, they can be sorted by the frequency of exceedance at each ground-motion level and summed into a cumulative probability mass function. When the cumulative probability mass function for a particular exceedance frequency equals or exceeds fractile y, then the exceedance frequency represents the y^{th} fractile. The weighted-mean hazard curve is the weighted average of the exceedance frequency values. This approach is a standard practice in PSHA.

Figure 19 also shows the ground surface 5%-damped UHS for the return periods of 500 and 2,500 years resulting from the site response analysis (Table 6). The ground surface UHS are deamplified from PGA to about 0.3 sec and amplified at longer periods relative to the hard rock UHS (Figure 19). This effect is due to the damping effects of the soil column on short-period ground motions and amplification of the long-period ground motions due to the thick low velocity material.

In Figures 20 and 21, we compare this study's spectra with the 2012 spectra from Darragh and Silva (2012) for the return periods of 500 and 2,500 years. At the latter return period, the current hard rock spectrum is lower than the 2012 hard rock spectrum from PGA to 0.2 sec but higher at longer periods. As a result, the 2021 spectrum at the ground surface is lower than the 2012 spectrum from PGA to 0.3 sec but higher at longer periods. The net result is a shift in the current spectrum to longer periods so it peaks around 0.7 sec compared to the peak in the 2012 spectrum at 0.2 sec. The comparison at 500 years is similar.

Spectra were also calculated at 2% and 10% damping. The damping ratio factors of Rezaeian *et al*. (2012) were used to compute the 2% and 10%-damped UHS for the return periods of 500 and 2,500 years. The factors are a function of magnitude and distance. The deaggregation of the 500 and 2,500 years UHS from the PSHA was used to compute the damping ratio factors. The 2% and 10%-damped spectra are provided in Table 7. The 2% and 10%-damped spectra are shown along with the 5%-damped spectra in Figures 22 and 23 for the 500 and 2,500-year return periods, respectively.

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Tables

Table 1 Seismic Source Zones Incorporated Into Analysis[1]

Source Zone	Symbol	Mmax (M)2	Seismogenic Depth (km)	Area (km2)
Seismotectonic Zones				
Extended Continental Crust–Gulf Coast	ECC-GC	6.0 6.7 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	1239288
Gulf Highly Extended Crust	GHEX	6.0 6.7 7.2 7.7 8.1	8 (0.5) 15 (0.5)	509090
Midcontinent Craton Zone (all alternatives)	MidC	5.6 6.1 6.6 7.2 8.0	13 (0.4) 17 (0.4) 22 (0.2)	4258598 4246625 4025001 4013028
Oklahoma Aulacogen Zone	OKA	5.8 6.4 6.9 7.4 8.0	15 (0.5) 20 (0.5)	53583
Paleozoic Extended Crust (Narrow and Wide alternatives)	PEZ	5.9 6.4 6.8 7.2 7.9	13 (0.4) 17 (0.4) 22 (0.2)	365395 598992
Reelfoot Rift Zone	RR	6.2 6.7 7.2 7.7 8.1	13 (0.4) 15 (0.4) 17 (0.2)	69479
Reelfoot Rift with Rough Creek Graben Zone	RR and RR_RCG	6.1 6.6 7.1 7.6 8.1	13 (0.4) 15 (0.4) 17 (0.2)	81452

Source Zone	Symbol	Mmax (M)2	Seismogenic Depth (km)	Area (km2)
Mmax Zones				
Mesozoic and Younger Extended Crust - Narrow	MESE-N	6.4 6.8 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	3616923
Mesozoic and Younger Extended Crust - Wide	MESE-W	6.5 6.9 7.3 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	4342413
Non-Mesozoic and Younger Extended Crust - Narrow	NMESE-N	6.4 6.8 7.1 7.5 8.0	13 (0.4) 17 (0.4) 22 (0.2)	4792101
Non-Mesozoic and Younger Extended Crust - Wide	NMESE-W	5.7 6.1 6.6 7.2 7.9	13 (0.4) 17 (0.4) 22 (0.2)	4066611
Study Region	Study R	6.5 6.9 7.2 7.7 8.1	13 (0.4) 17 (0.4) 22 (0.2)	8409024

Notes:

[1]Data extracted from Tables H-3-1, H-3-3, H-4-2, and H-4-4 in Appendix H of EPRI/DOE/NRC (2012)

[2] Weights for all magnitude distributions are 0.101/0.244/0.310/0.244/0.101

Table 2 New Madrid Fault System RLME Source Model[1]

Cluster?	wt	Localizing Structures	Southern Fault Geometry	wt	Northern Fault Geometry	wt	Central Fault Geometry	wt	Thickness (km)	wt	Mmax	wt	Recurrence method	wt	Recurrence Data	wt	Earthquake Recurrence Model	wt	Repeat Time Coefficient of Variation	wt	Rate (yrs)	wt
All In	0.9	NMS NMN RFT	BA-BL	0.6	NMN-S	0.7	RFT-S	0.7	13	0.4	NMS, RFT, NMN 7.9, 7.8, 7.6	0.167	Intervals	1.0	1811-1812, 1450, and 900 AD	1.0	Poisson	0.75	NA		167	0.101
																					270	0.244
																					417	0.310
																					714	0.244
																					1613	0.101
											7.8, 7.7, 7.5	0.167	same as above				Renewal	0.25	0.3	0.2	286	0.101
											7.6, 7.8, 7.5	0.25									909	0.244
											7.2, 7.4, 7.2	0.085									3125	0.310
											6.9, 7.3, 7.0	0.25									15625	0.244
											6.7, 7.1, 6.8	0.085									212766	0.101
									15	0.4	same as above								0.5	0.5	208	0.101
									17	0.2											455	0.244
							RFT-L	0.3	same as above												1124	0.310
					NMN-L	0.3	same as above														3846	0.244
																					32258	0.101
			BA-BFZ	0.4	same as above														0.7	0.3	227	0.101
																					455	0.244
																					1000	0.310
																					2941	0.244
																					21277	0.101
All out except RFT	0.05	RFT	NA		NA		RFT-S	0.7	13	0.4	7.8	0.167	Intervals	1.0	2000 BC and 1000 AD	1.0	Poisson	1.0	NA		769	0.101
																					1389	0.244
																					2381	0.310
																					4545	0.244
																					12500	0.101
											7.7	0.167	same as above									
											7.8	0.25										

Cluster?	wt	Localizing Structures	Southern Fault Geometry	wt	Northern Fault Geometry	wt	Central Fault Geometry	wt	Thickness (km)	wt	Mmax	wt	Recurrence method	wt	Recurrence Data	wt	Earthquake Recurrence Model	wt	Repeat Time Coefficient of Variation	wt	Rate (yrs)	wt
											7.4	0.085										
											7.3	0.25										
											7.1	0.085										
									15	0.4	same as above											
									17	0.2												
							RFT-L	0.3	same as above													
All Out	0.05	None	Revert to background																			

Note:
[1]Data extracted from Figure 6.1.5-1 and Tables 6.1.5-5 through 6.1.5-7 in EPRI/DOE/NRC (2012)

Table 3 RLME (Fault) Sources Incorporated Into Analysis[1]

Fault	Geometry	Style of Faulting 1	Mmax (M)	Dip (deg)	Seismogenic Thickness (km)	Recurrence Data3	Recurrence Interval (yr)4
Reelfoot Rift - Eastern Rift Margin Fault (ERM)							
ERM-N	ERM-N (1.0)	SS	6.7 (0.3) 6.9 (0.3) 7.1 (0.3) 7.4 (0.1)	90	13 (0.3) 15 (0.5) 17 (0.2)	1 event in 12-35 kyr (0.9)	3448 6667 12500 25000 71429
						2 events in 12-35 kyr (0.1)	2564 4545 7692 13889 31250
ERM-S	ERM-SCC (0.6)	SS	6.7 (0.15) 6.9 (0.2) 7.1 (0.2) 7.3 (0.2) 7.5 (0.2) 7.7 (0.05)	90	same as above	2 events in 17.7-21.7 kyr (0.333)	2857 4762 7143 12500 27778
						3 events in 17.7-21.7 kyr (0.334)	2326 3571 5263 8333 16129

Fault	Geometry	Style of Faulting 1	Mmax (M)	Dip (deg)	Seismogenic Thickness (km)	Recurrence Data3	Recurrence Interval (yr)4
						4 events in 17.7-21.7 kyr (0.333)	2000 2941 4167 6250 11111
	ERM-SRP (0.4)	same as above	same as above	same as above	same as above	same as above	same as above
Reelfoot Rift-Marianna In cluster (0.5) [Out of cluster (0.5) - default to background]	Marianna NW-strike (0.5)	SS	6.7 (0.15) 6.9 (0.2) 7.1 (0.2) 7.3 (0.2) 7.5 (0.2) 7.7 (0.05)	90	13 (0.3) 15 (0.5) 17 (0.2)	3 events in 9.6-10.2 kyr	 1449 2381 3704 6250 13889
						4 events in 9.6-10.2 kyr	1190 1818 2703 4167 8333
	Marianna NE-strike (0.5)	same as above	same as above	same as above	same as above	same as above	same as above
Reelfoot Rift - Commerce Fault Zone	Commerce fault (1.0)	SS	6.7 (0.15) 6.9 (0.35) 7.1 (0.35) 7.3 (0.1) 7.7 (0.05)	90	13 (0.3) 15 (0.5) 17 (0.2)	2 events in 18.9-23.6 kyr	4000 7143 12500 25000 71429

Fault	Geometry	Style of Faulting 1	Mmax (M)	Dip (deg)	Seismogenic Thickness (km)	Recurrence Data3	Recurrence Interval (yr)4
						3 events in 18.9-23.6 kyr	3030 5000 7692 13158 29412
New Madrid Fault System (NMFS)	see Table 2						

Note: Values in parentheses are weights. All faults are modeled with the Characteristic recurrence model

[1] Data extracted from Sections H.5.5 through H.5.8 and Tables H.5.5-1 through H.5.8-3 in Appendix H of EPRI/DOE/NRC (2012)

[2] SS Strike-slip

[3] "Recurrence Data" describes datasets used to calculate recurrence intervals.

[4] Weights for all distributions are: 0.101/0.244/0.310/0.244/0.101

Table 4 Hard Rock Mean UHS in g's

PERIOD (SEC)	500-YEAR	2,500-YEAR
0.005 (PGA)	0.018	0.054
0.01	0.019	0.058
0.02	0.023	0.072
0.025	0.025	0.077
0.03	0.026	0.081
0.04	0.028	0.089
0.05	0.029	0.092
0.075	0.029	0.087
0.1	0.030	0.088
0.15	0.031	0.087
0.2	0.031	0.086
0.25	0.031	0.088
0.3	0.030	0.087
0.4	0.029	0.082
0.5	0.027	0.076
0.75	0.022	0.066
1	0.018	0.056
1.5	0.013	0.042
2	0.010	0.035
3	0.006	0.023
4	0.004	0.017
5	0.003	0.013
7.5	0.002	0.009
10	0.001	0.006

Table 5 Modal Magnitude and Distance Values

	PGA	1.0 SEC SA
500-Year Return Period		
Mean M	6.80	7.33
Mean D (km)	349	506
Modal M	7.75	7.75
Modal D (km)	480	480
2,500-Year Return Period		
Mean M	6.68	7.55
Mean D (km)	263	487
Modal M	7.75	7.75
Modal D (km)	480	480

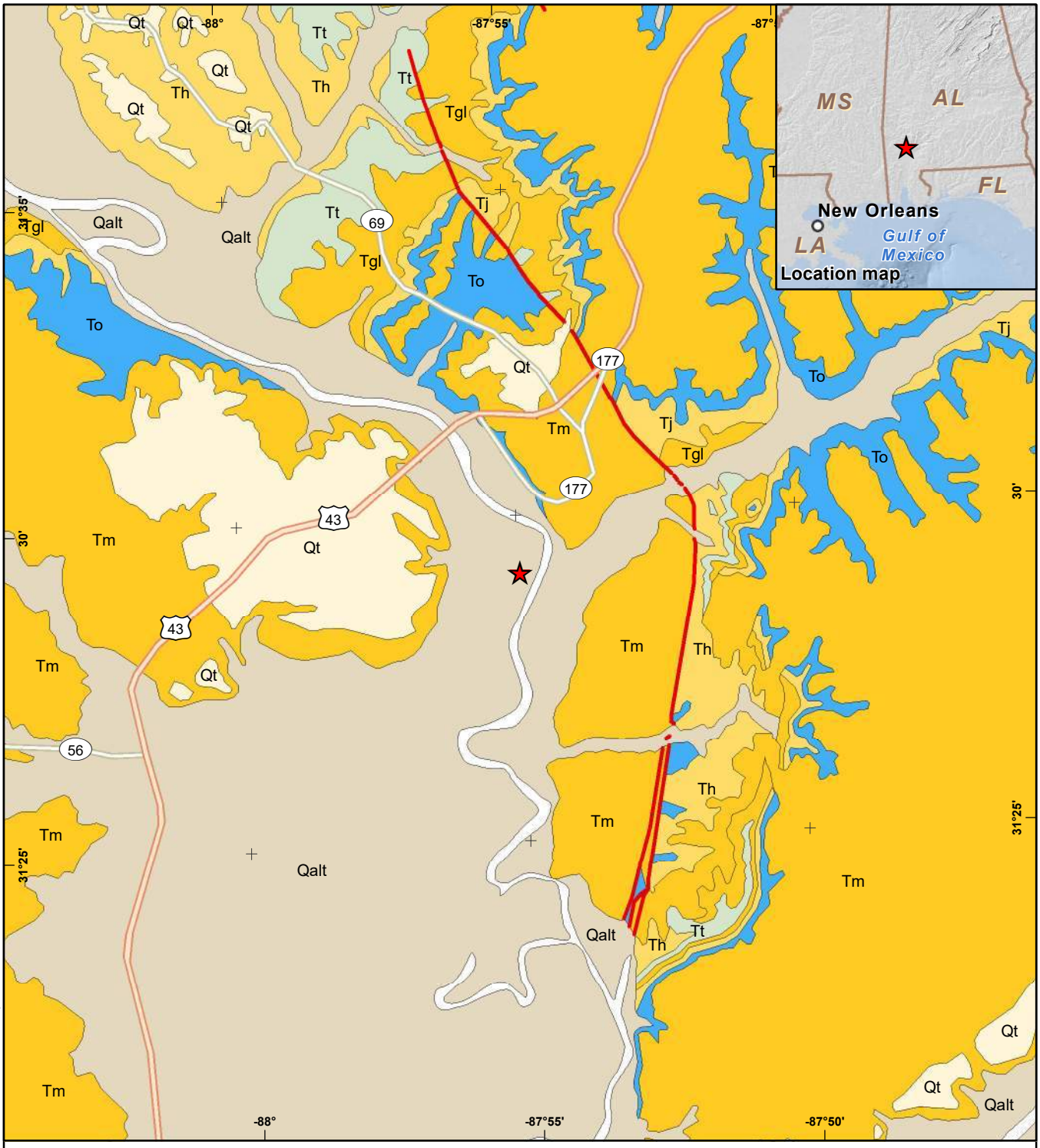
Table 6 5%-Damped Ground Surface Mean UHS in g's

PERIOD (SEC)	500-YEAR	2,500-YEAR
0.005 (PGA)	0.027	0.079
0.01	0.028	0.085
0.02	0.028	0.085
0.025	0.028	0.085
0.03	0.028	0.085
0.04	0.028	0.085
0.05	0.028	0.085
0.075	0.029	0.087
0.1	0.038	0.110
0.15	0.057	0.159
0.2	0.062	0.173
0.25	0.064	0.185
0.3	0.069	0.197
0.4	0.077	0.220
0.5	0.083	0.237
0.75	0.082	0.246
1	0.061	0.193
1.5	0.032	0.108
2	0.024	0.082
3	0.014	0.054
4	0.010	0.039
5	0.007	0.030
7.5	0.005	0.021
10	0.003	0.015

Table 7 2% and 10%-Damped Ground Surface Mean UHS in g's

Period (sec)	500-year		2,5000-year	
	2% Damped Spectrum	10% Damped Spectrum	2% Damped Spectrum	10% Damped Spectrum
0.005 (PGA)	0.027	0.027	0.079	0.079
0.01	0.028	0.028	0.085	0.085
0.02	0.028	0.028	0.085	0.085
0.025	0.028	0.028	0.086	0.085
0.03	0.028	0.028	0.086	0.084
0.04	0.029	0.028	0.089	0.083
0.05	0.030	0.027	0.092	0.082
0.075	0.034	0.026	0.102	0.079
0.1	0.048	0.032	0.140	0.094
0.15	0.077	0.045	0.214	0.126
0.2	0.086	0.048	0.240	0.134
0.25	0.089	0.049	0.255	0.140
0.3	0.098	0.052	0.279	0.148
0.4	0.109	0.057	0.313	0.163
0.5	0.118	0.061	0.338	0.173
0.75	0.115	0.061	0.346	0.183
1	0.088	0.044	0.279	0.140
1.5	0.046	0.024	0.155	0.078
2	0.034	0.017	0.119	0.059
3	0.020	0.010	0.078	0.038
4	0.015	0.007	0.056	0.028
5	0.010	0.005	0.042	0.021
7.5	0.007	0.004	0.029	0.015
10	0.004	0.003	0.019	0.011

Figures



EXPLANATION

Site location

Geologic data (USGS, 1998)

- Fault
- beach sand
- clay or mud
- claystone
- limestone
- sand
- terrace

Source:
Hillshaded DEM from GEBCO, 2020.

N W E S

0 2 4 mi

0 2 4 km

Map projection and scale: NAD 1983 UTM Zone 15N, 1:165,000

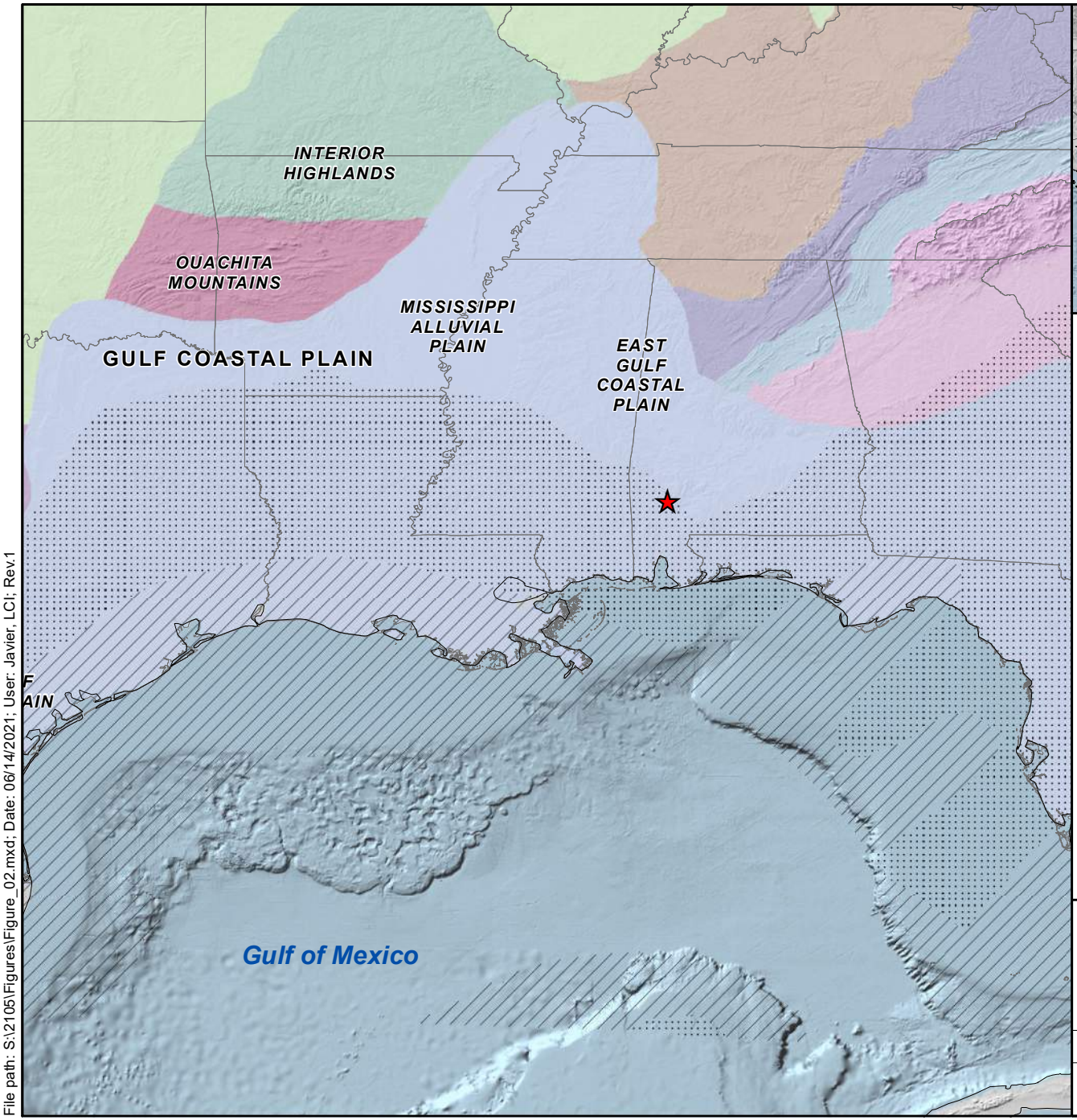
Geologic Map of Site Location

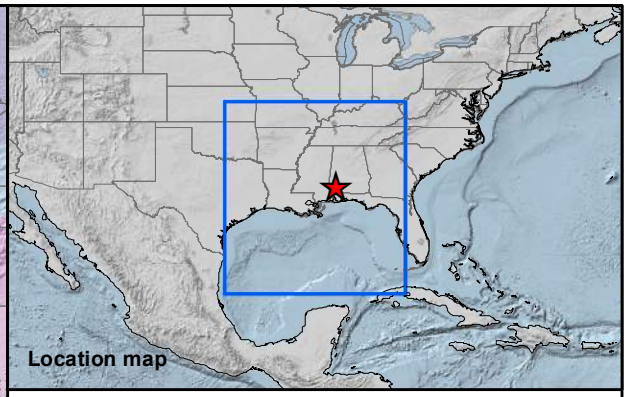
LOWMAN POWER PLANT, ALABAMA

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Figure 1

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EXPLANATION

Site location

Provinces

- Gulf Coastal Plain
- Basin and Range
- Great Plains
- Ozark Plateaus
- Central Lowland
- Interior Low Plateaus
- Ouachita

Extended areas

- Slightly extended
- Highly extended

Sources:
EPRI/DOE/NRC (2012); after Fenneman and Johnson (1946)
Gridded bathymetry data from GEBCO (2014)

0 100 200 mi
0 100 200 km

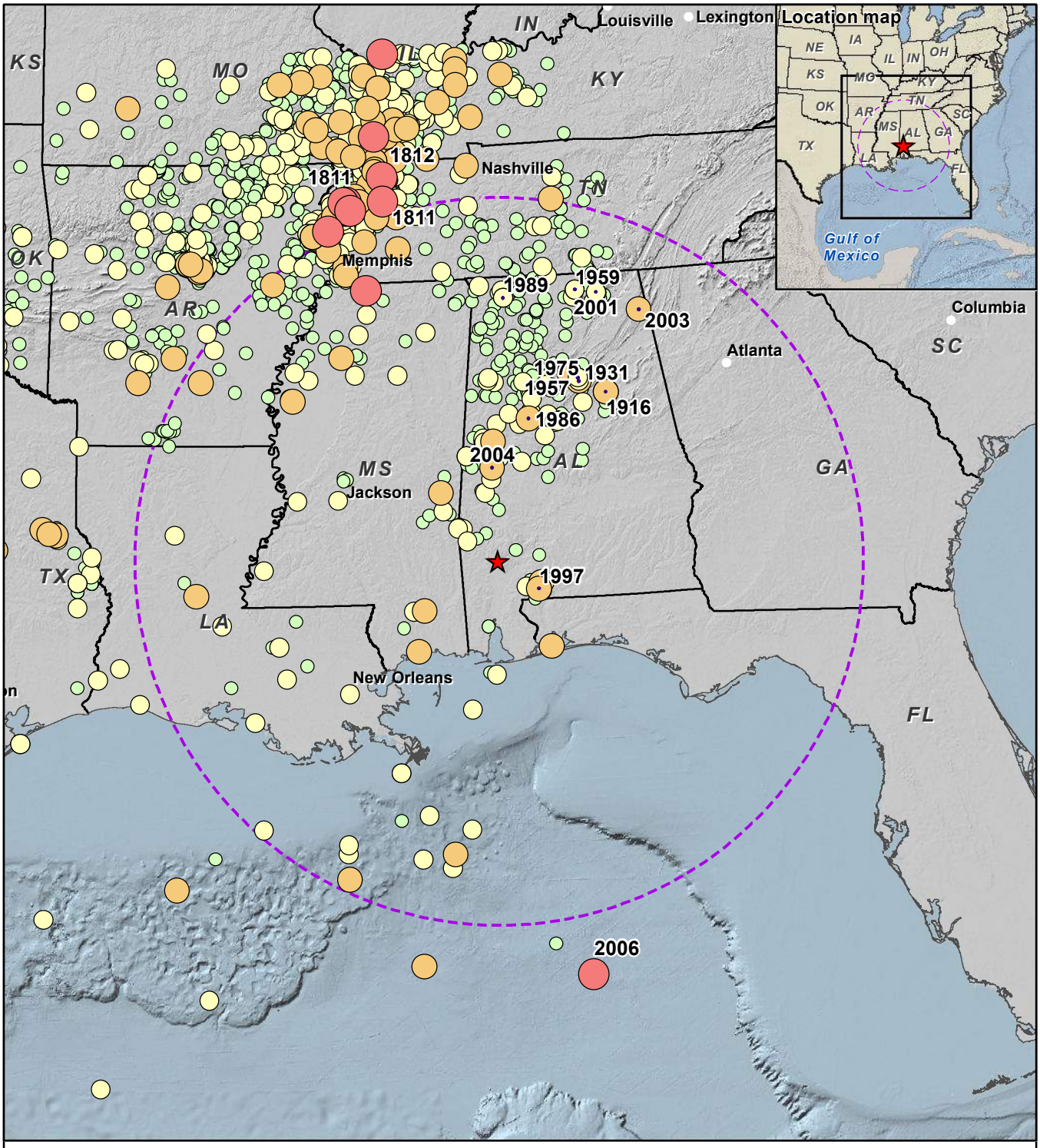
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Physiographic Setting

LOWMAN POWER PLANT, ALABAMA

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EXPLANATION

- Site location
- 500 km site buffer

Seismicity [M]

- 2.00 - 2.99
- 3.00 - 3.99
- 4.00 - 4.99
- 5.00 - 5.80

Sources:
- Seismicity from EPRI/DOE/NRC (2012), USGS (2014), and USGS (2018)

N W E S

0 100 200 mi

0 200 400 km

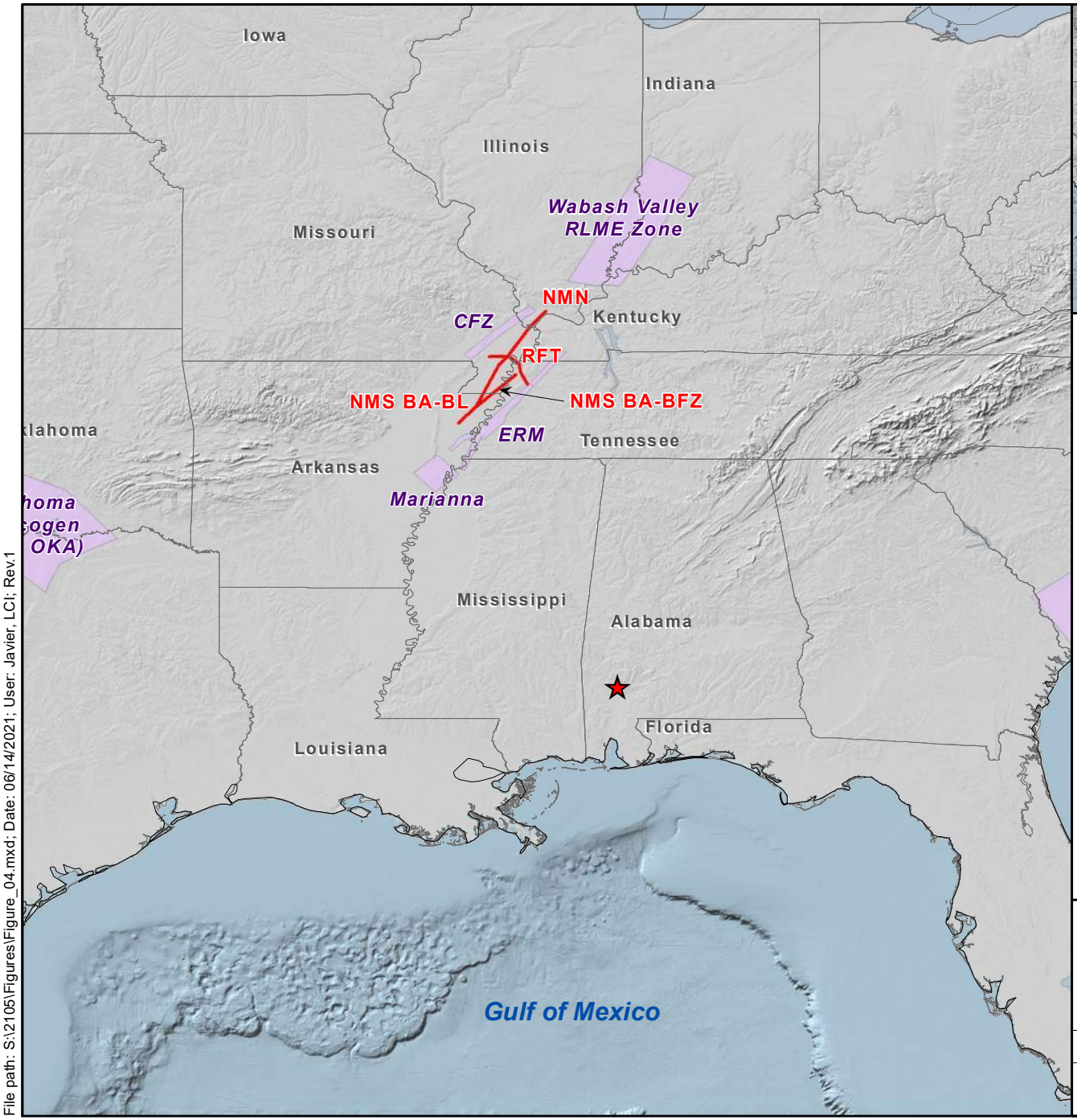
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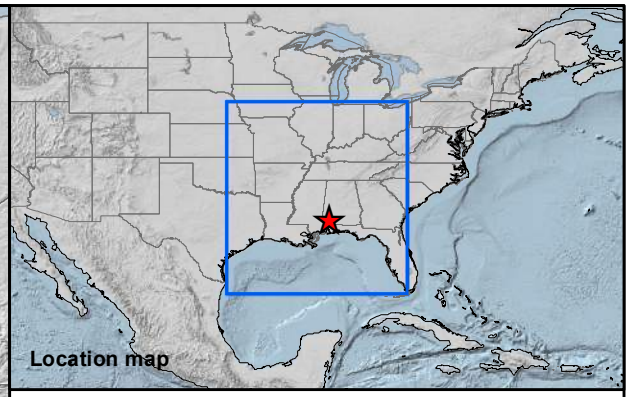
Historical Seismicity of the Site Region 1699 to August 2018

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EXPLANATION

- Site location
- RLME zone
- RLME fault

Notes:
BA - Blytheville arch
BL - Bootheel lineament
BFZ - Blytheville fault zone
CFZ - Commerce fault zone
ERM - Eastern Rift Margin
MF - Meers fault
NMN - New Madrid North
NMS - New Madrid South
RFT - Reelfoot thrust

Sources:
EPRI/DOE/NRC (2012)
Gridded bathymetry data from GEBCO (2014)

N W E S

0 100 200 mi
0 100 200 km

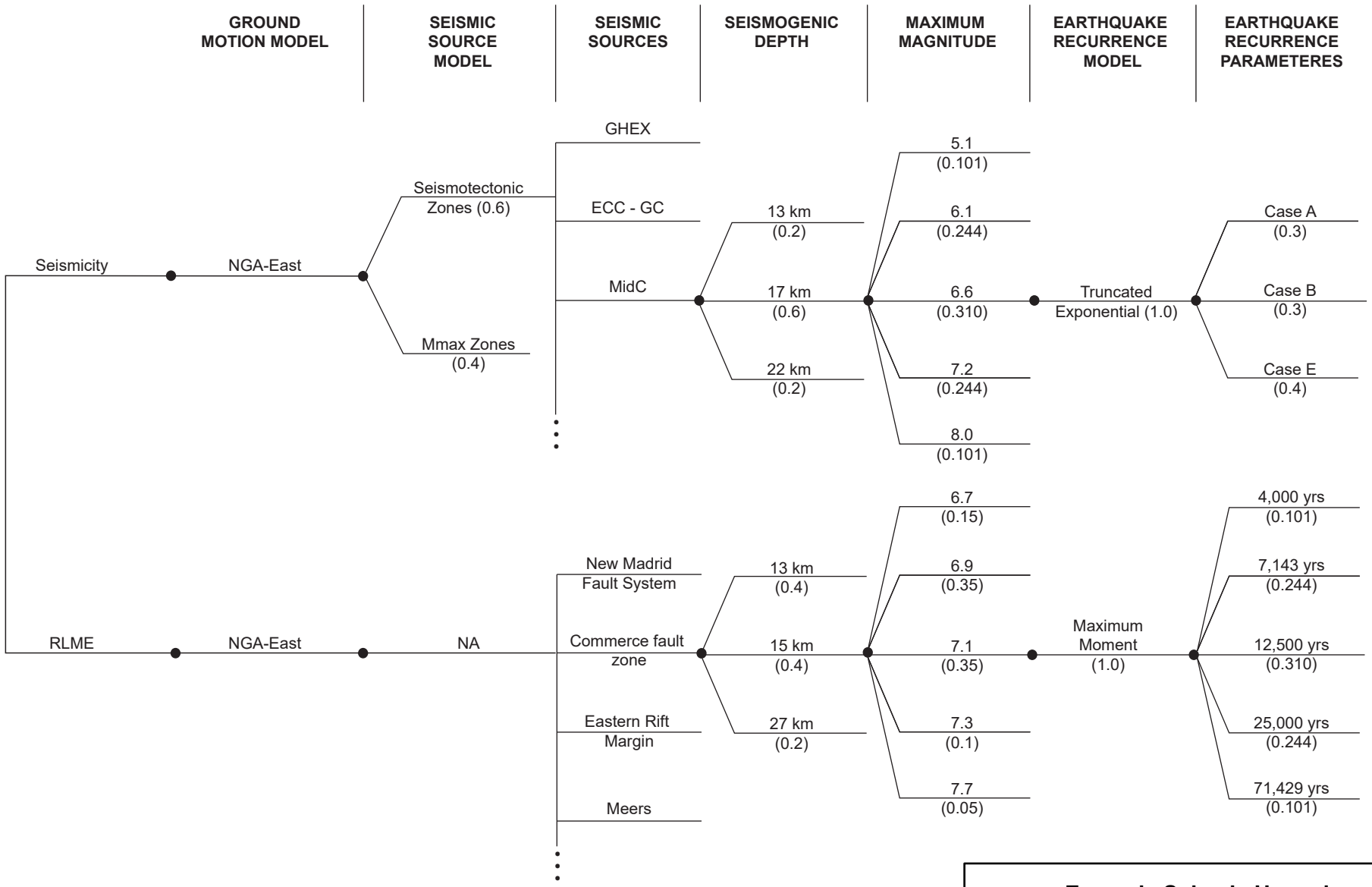
Map projection and scale: GCS North American 1983, 1:10,400,000

Quaternary Faults and RLMEs in the Central U.S.

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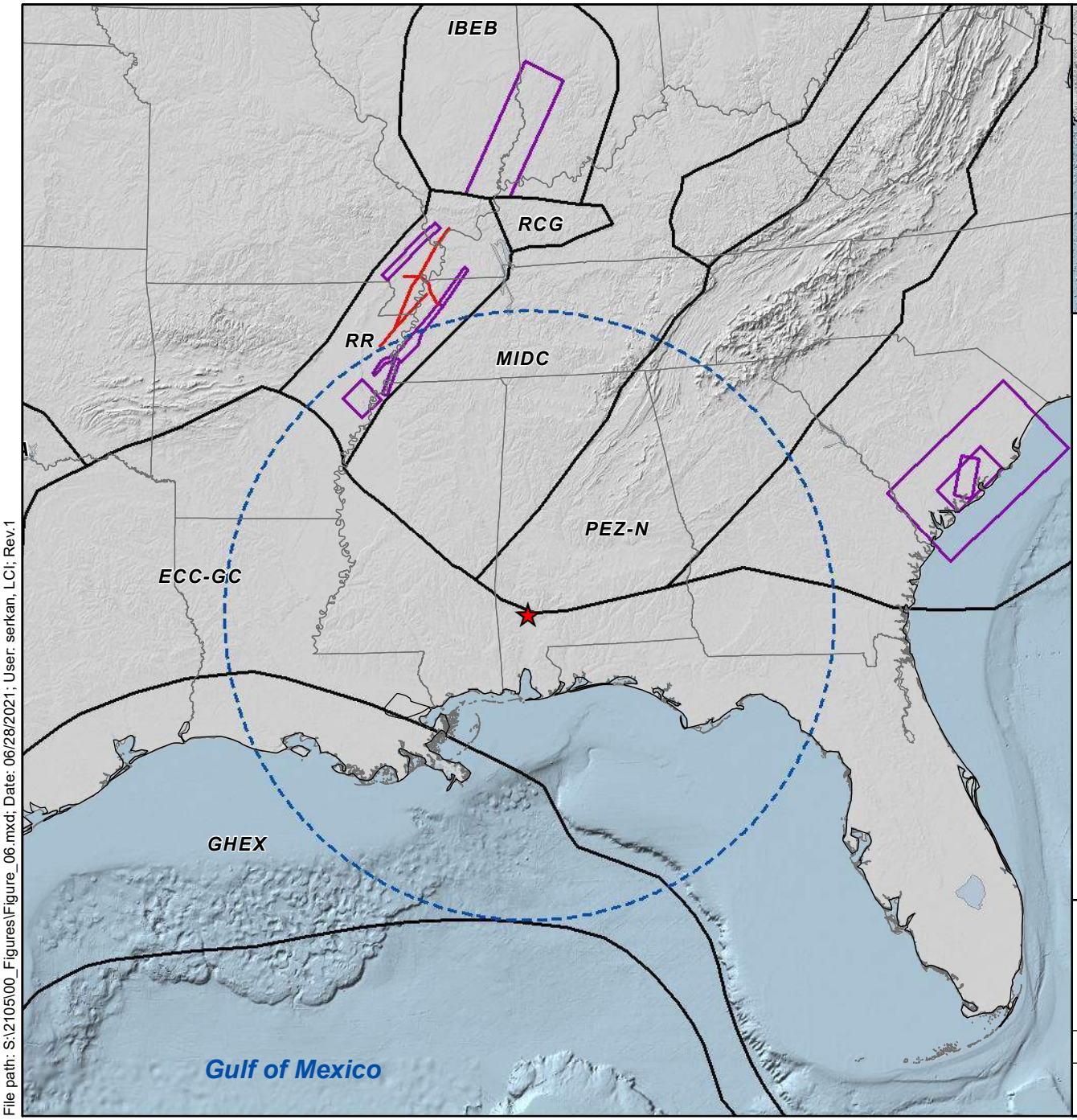


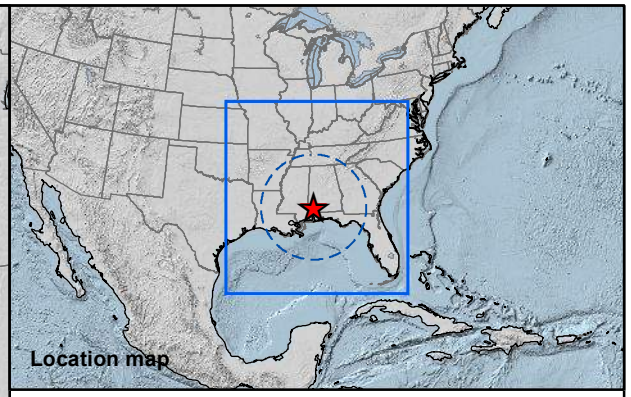
Example Seismic Hazard Model Logic Tree

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EXPLANATION

- Site location
- RLME fault
- RLME zone boundary
- Seismotectonic zone boundary
- 500 km site buffer

Notes:
ECC-GC - Extended Continental Crust - Gulf Coast
GHEX - Gulf Coast Highly Extended Crust
IBEB - Illinois Basin Extended Basement
MIDC - Midcontinent-Craton
OKA - Oklahoma Aulacogen
PEZ-N - Paleozoic extended zone narow
RCG - Rough Creek Graben
RR - Reelfoot Rift

Sources:
EPRI/DOE/NRC (2012)
Gridded bathymetry data from GEBCO (2014)

0 100 200 mi
0 100 200 km

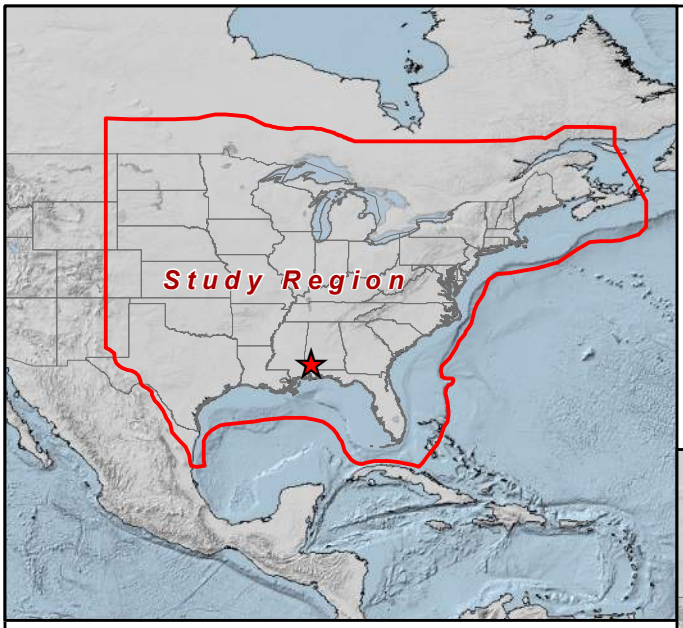
Map projection and scale: NAD 1983 UTM Zone 15N, 1:10,000,000

Seismotectonic Zones and RLMEs

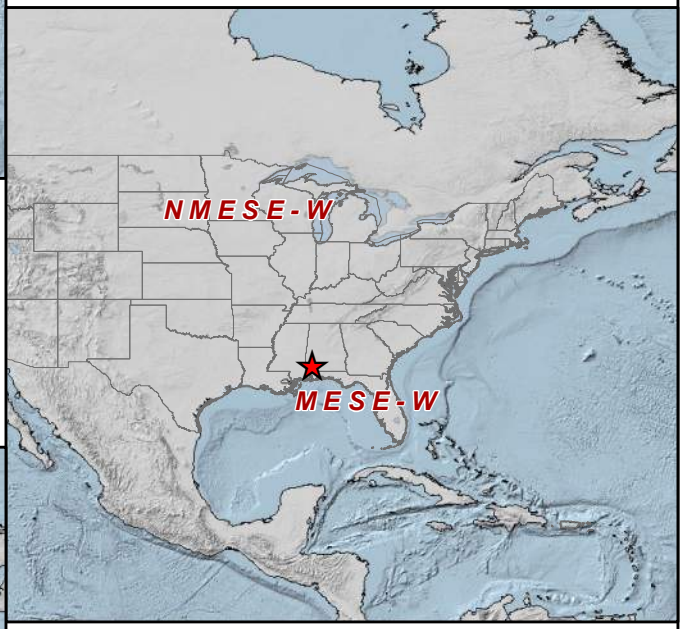
LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure 6

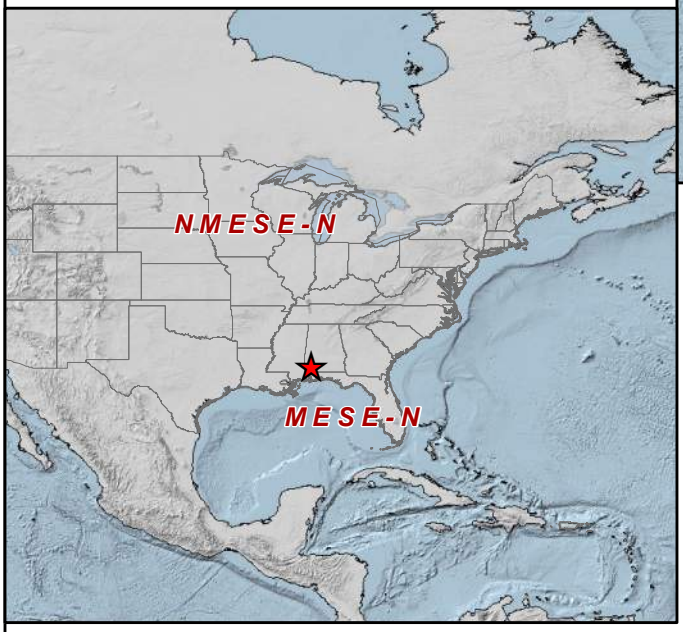
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1 Zone Model



2-Zone Model
MESE-Wide



2-Zone Model
MESE-Narrow

EXPLANATION

- Site location
- Seismotectonic zones

Notes:
MESE - Mesozoic and younger extended
NMESE - Non-Mesozoic and younger extended
N - Narrow
W - Wide

Sources:
EPRI/DOE/NRC (2012)
Gridded bathymetry data from GEBCO (2014)

N W E S

0 1,000 2,000 mi
0 1,000 2,000 km

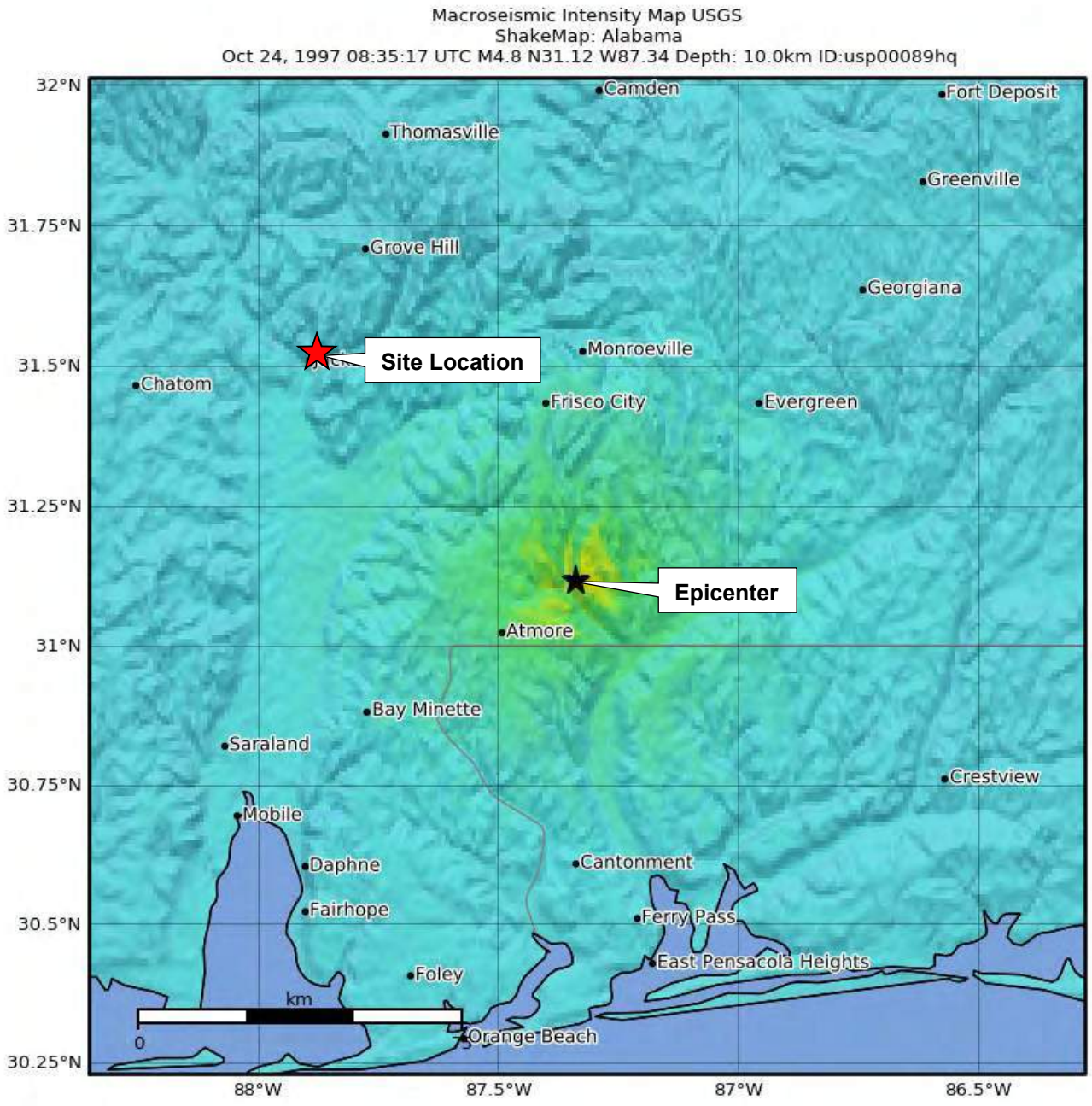
Map projection and scale: GCS WGS 1984, 1:70,000,000

Mmax Zones

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure 7

File path: S:\2105\Figures\Figure_07.mxd; Date: 06/14/2021; User: Javier, LCI; Rev.1



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(%g)	<0.0066	0.0795	0.954	4.99	8.76	15.4	27	47.4	>83.2
PGV(cm/s)	<0.0028	0.0383	0.524	3.03	6.48	13.9	29.6	63.4	>136
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Scale based on Atkinson and Kaka (2007) Version 1: Processed 2020-06-06T00:59:30Z

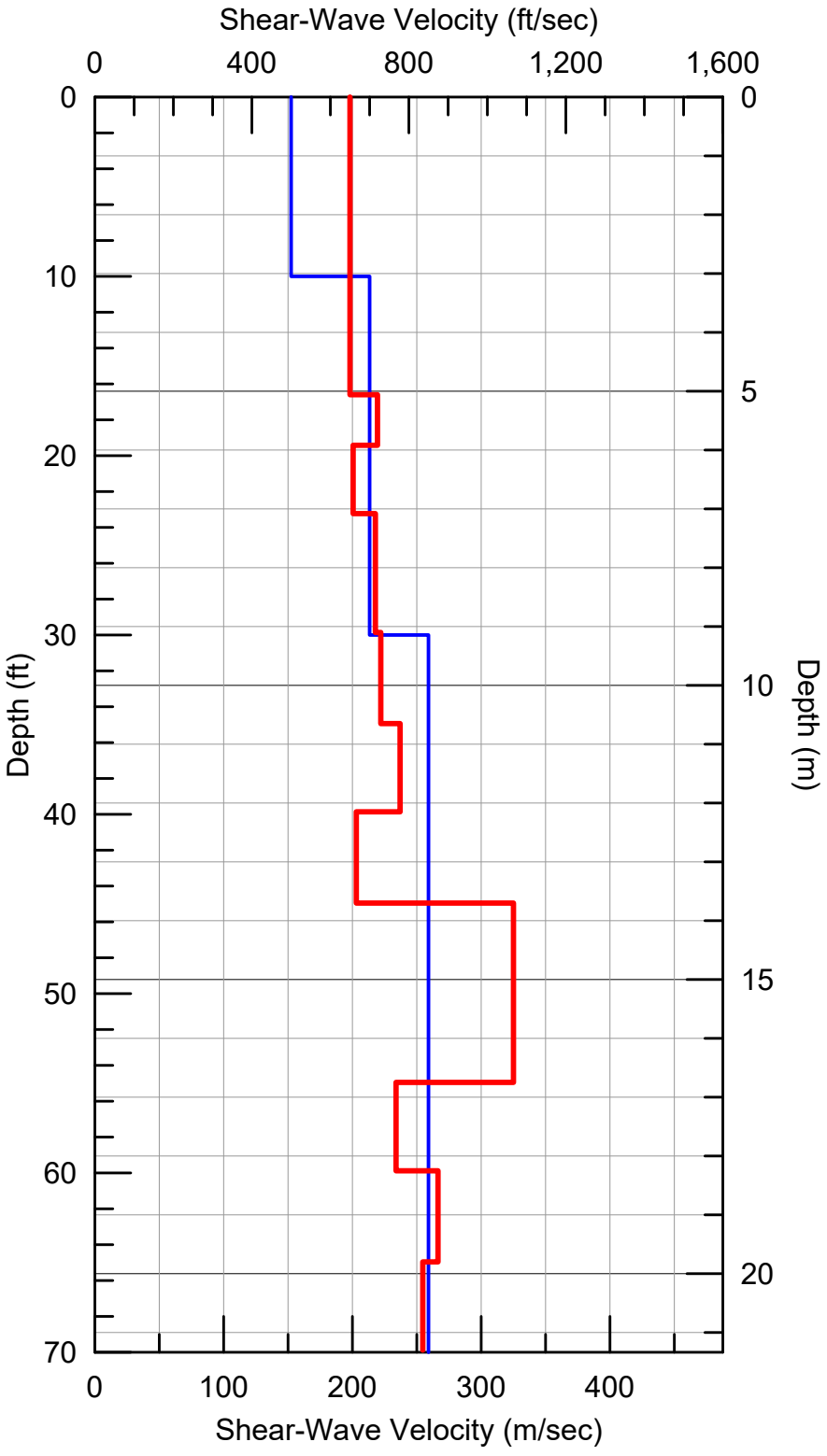
△ Seismic Instrument ○ Reported Intensity

Intensity Map of the
24 October 1997 **M** 4.8
Escambia Earthquake

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **8**

File path: S:\2105\00_Figures\LFigure_08.ai; Date: 06/24/2021; User: Jch, LCI.



Smoothed Lognormal Average

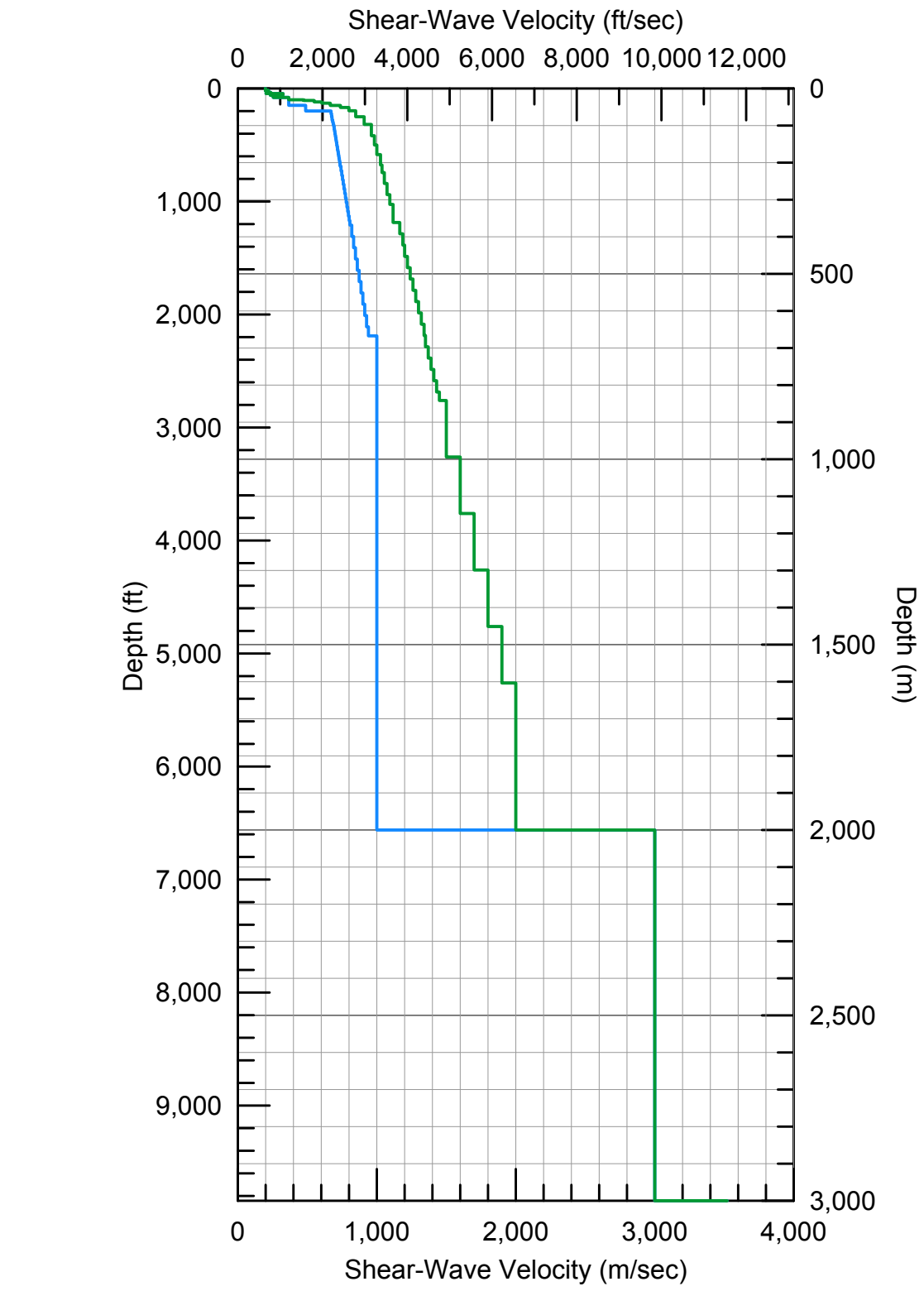
PE&A Profile 1

Smoothed Lognormal Average Vs Profile and PE&A Profile 1

LOWMAN POWER PLANT, ALABAMA

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Figure 9

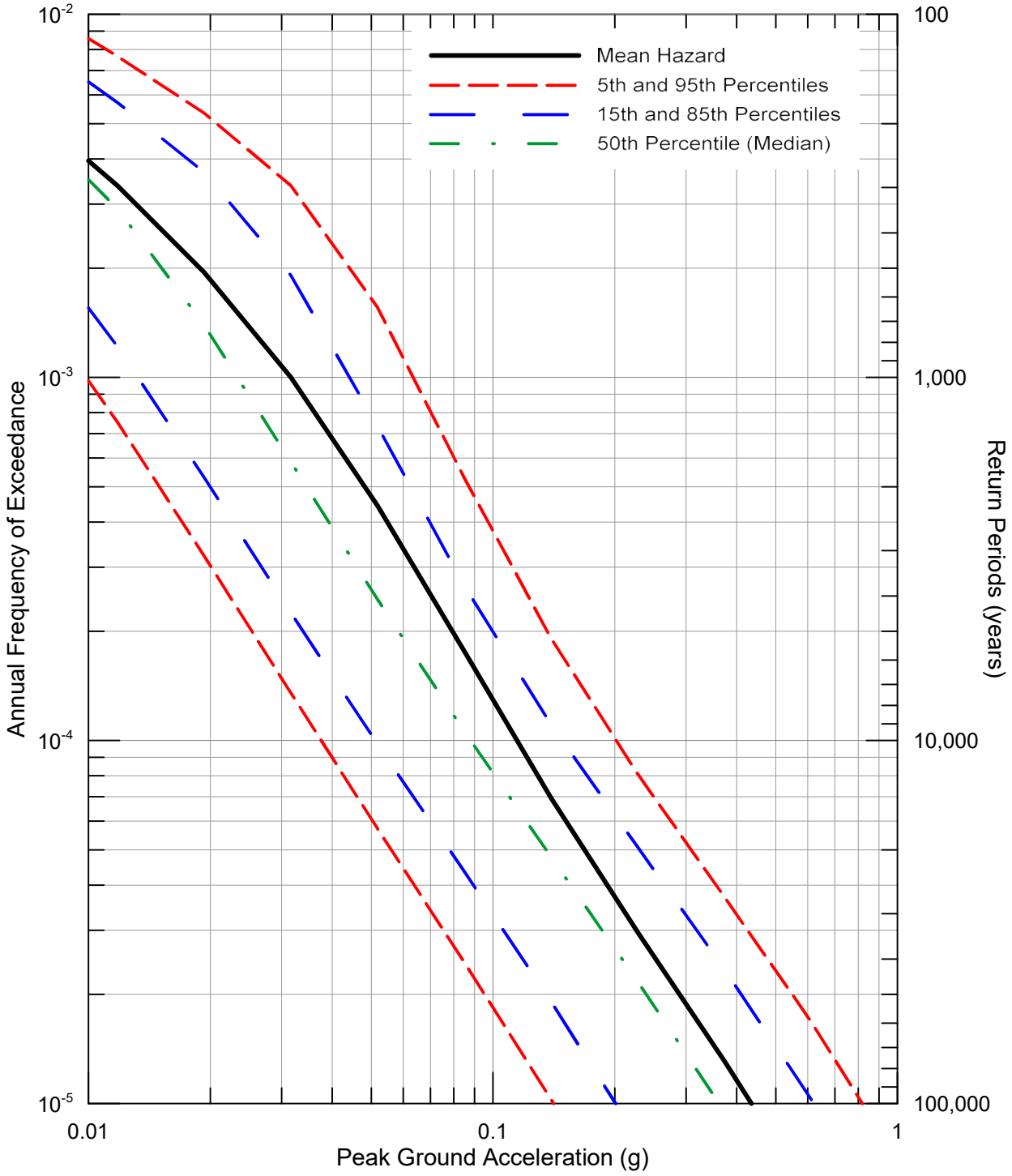


Basecase Profiles P1 and P2

LOWMAN POWER PLANT, ALABAMA

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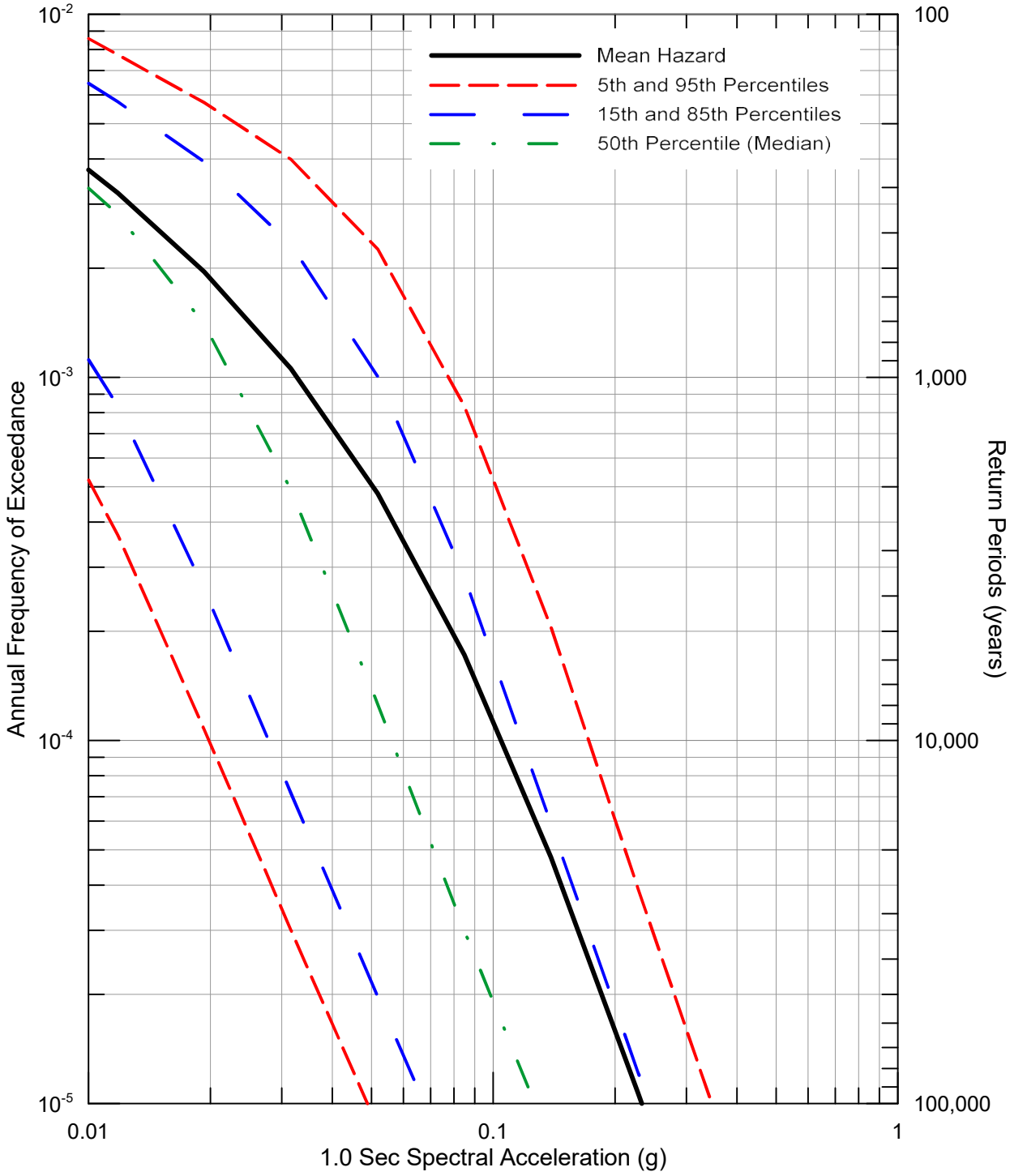
Figure **10**



Seismic Hazard Curves for Peak Horizontal Acceleration on Hard Rock

LOWMAN POWER PLANT, ALABAMA

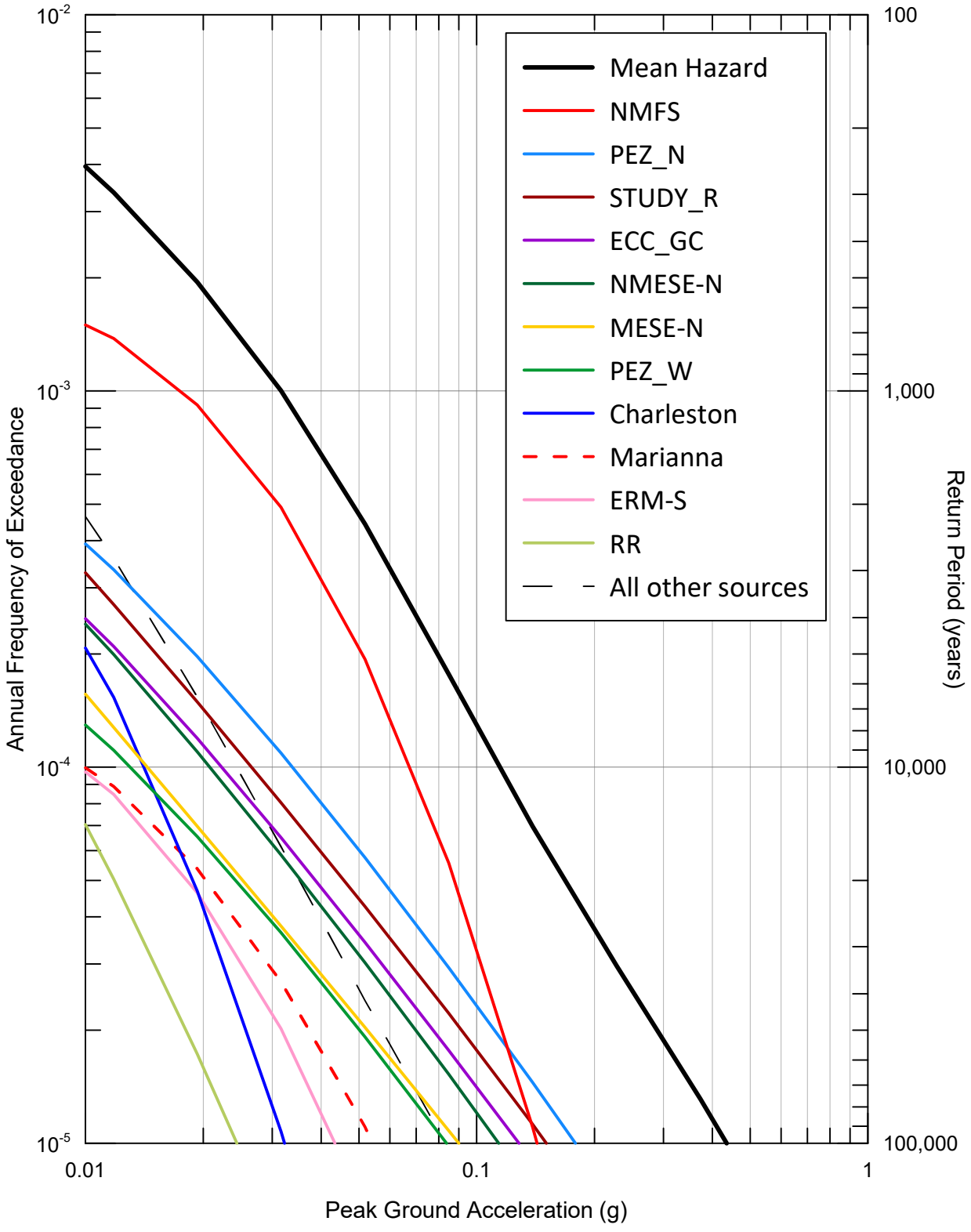
Lettis Consultants International, Inc. | Figure **11**



Seismic Hazard Curves for 1.0 Sec Horizontal Spectral Acceleration on Hard Rock

LOWMAN POWER PLANT, ALABAMA

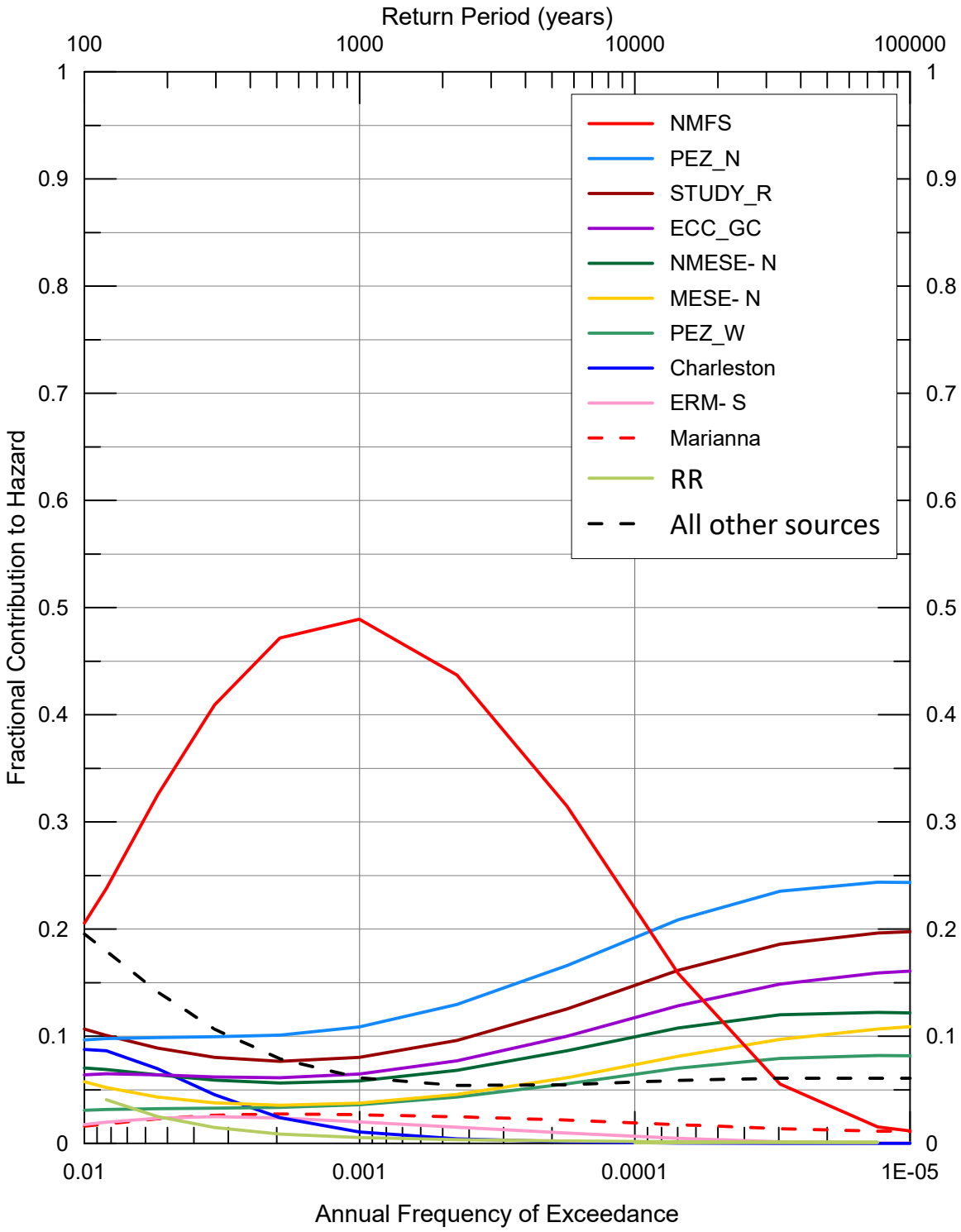
Lettis Consultants International, Inc. | Figure **12**



Seismic Source Contributions to Mean Peak Horizontal Acceleration on Hard Rock

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **13**

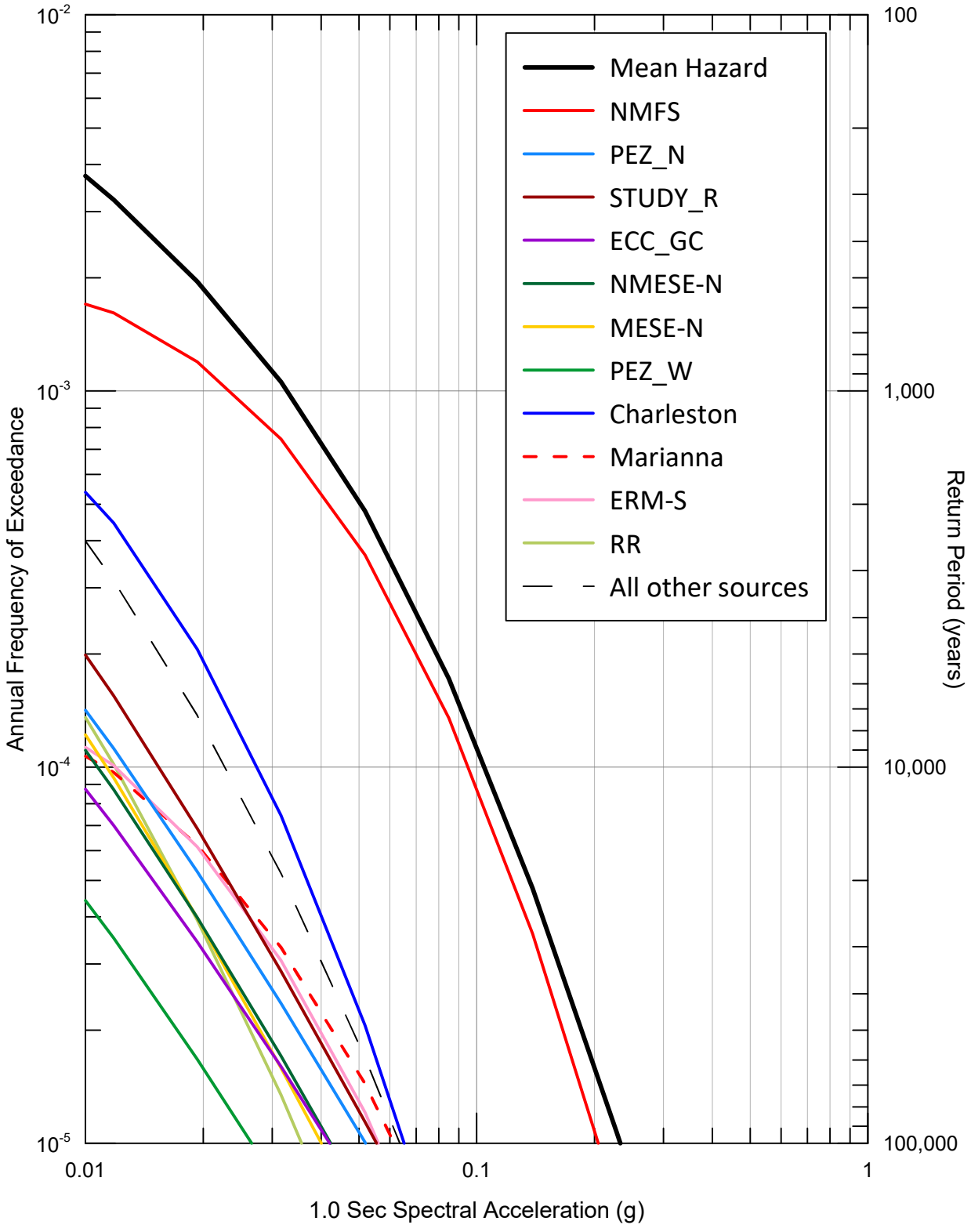


Seismic Source Fractional Contributions to Mean Peak Horizontal Acceleration on Hard Rock

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc.

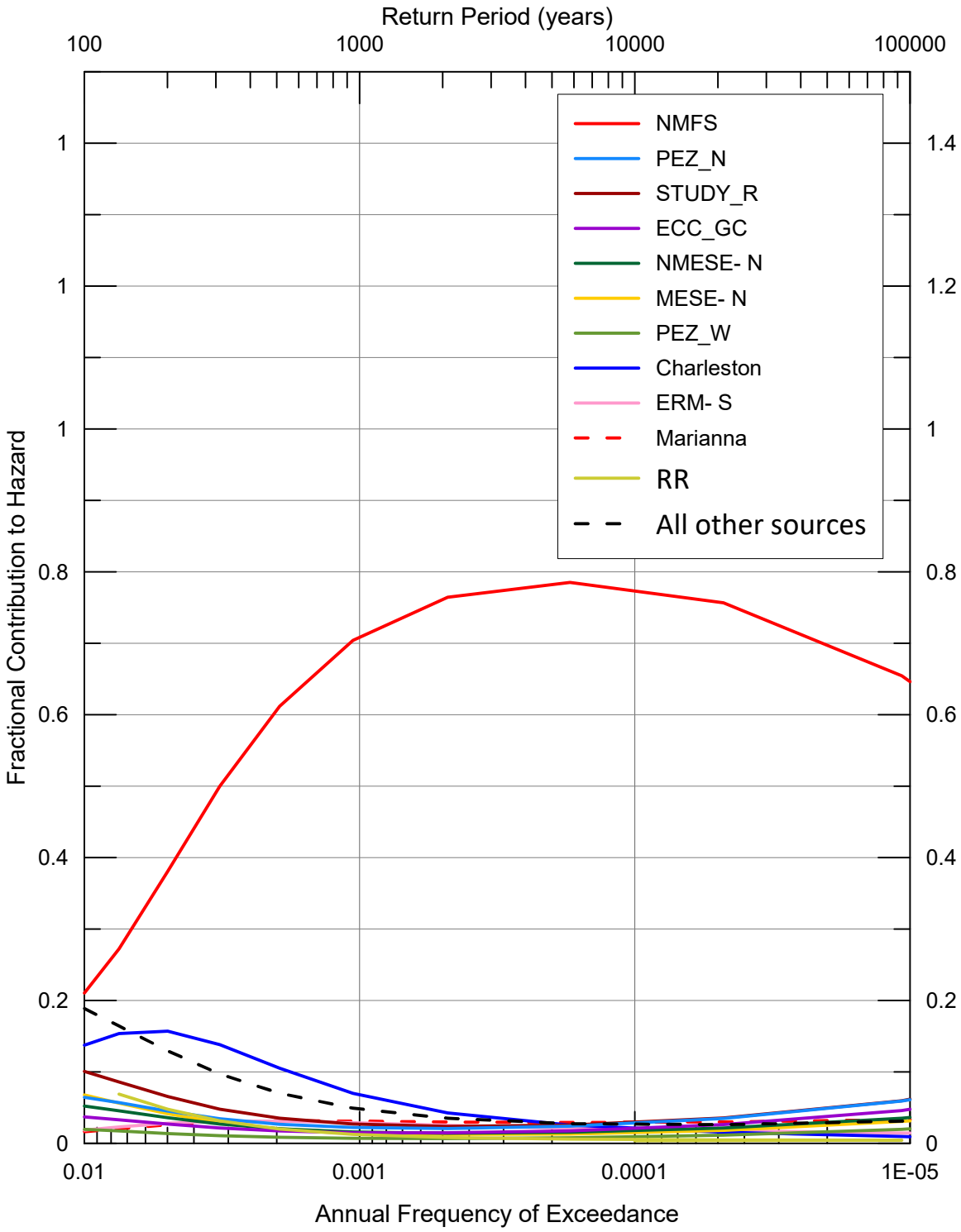
Figure **14**



Seismic Source Contributions to Mean 1.0 Sec Horizontal Spectral Acceleration on Hard Rock

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **15**



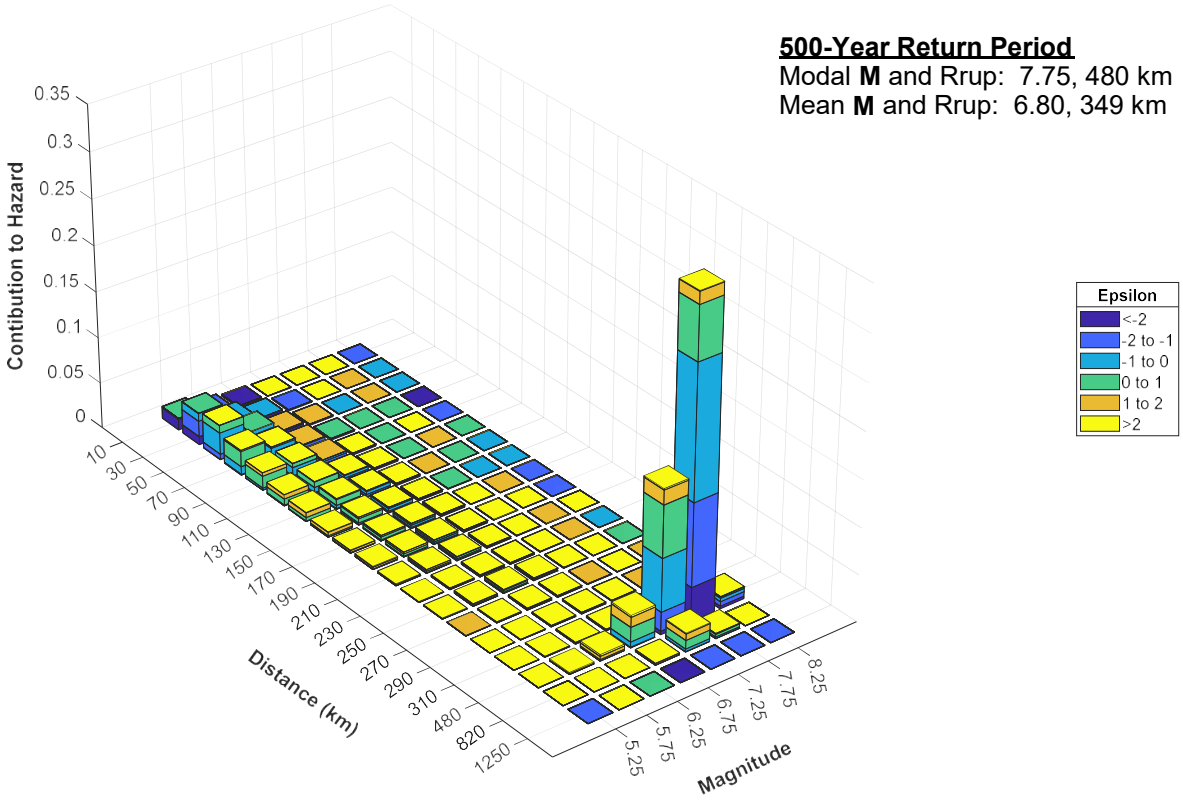
Seismic Source Fractional Contributions to Mean 1.0 Sec Horizontal Spectral Acceleration on Hard Rock

LOWMAN POWER PLANT, ALABAMA

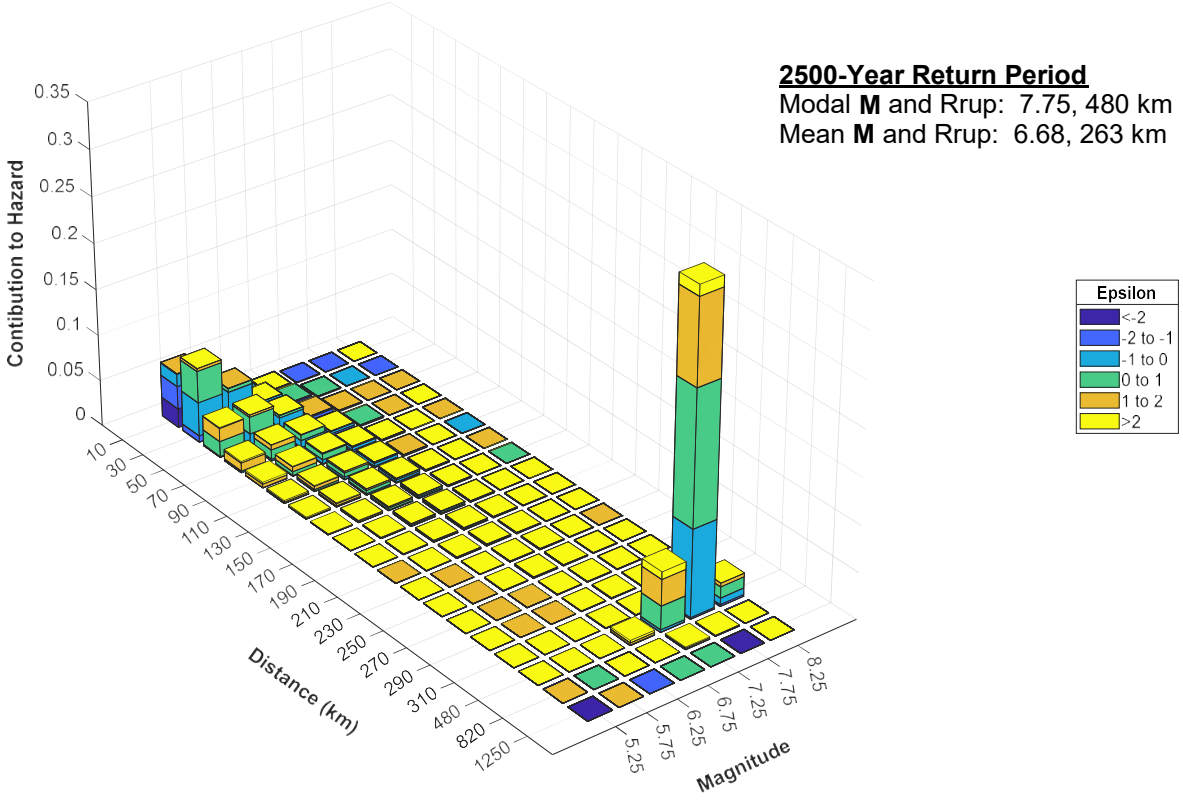
Lettis Consultants International, Inc.

Figure **16**

500-Year Return Period
Modal **M** and Rrup: 7.75, 480 km
Mean **M** and Rrup: 6.80, 349 km



2500-Year Return Period
Modal **M** and Rrup: 7.75, 480 km
Mean **M** and Rrup: 6.68, 263 km

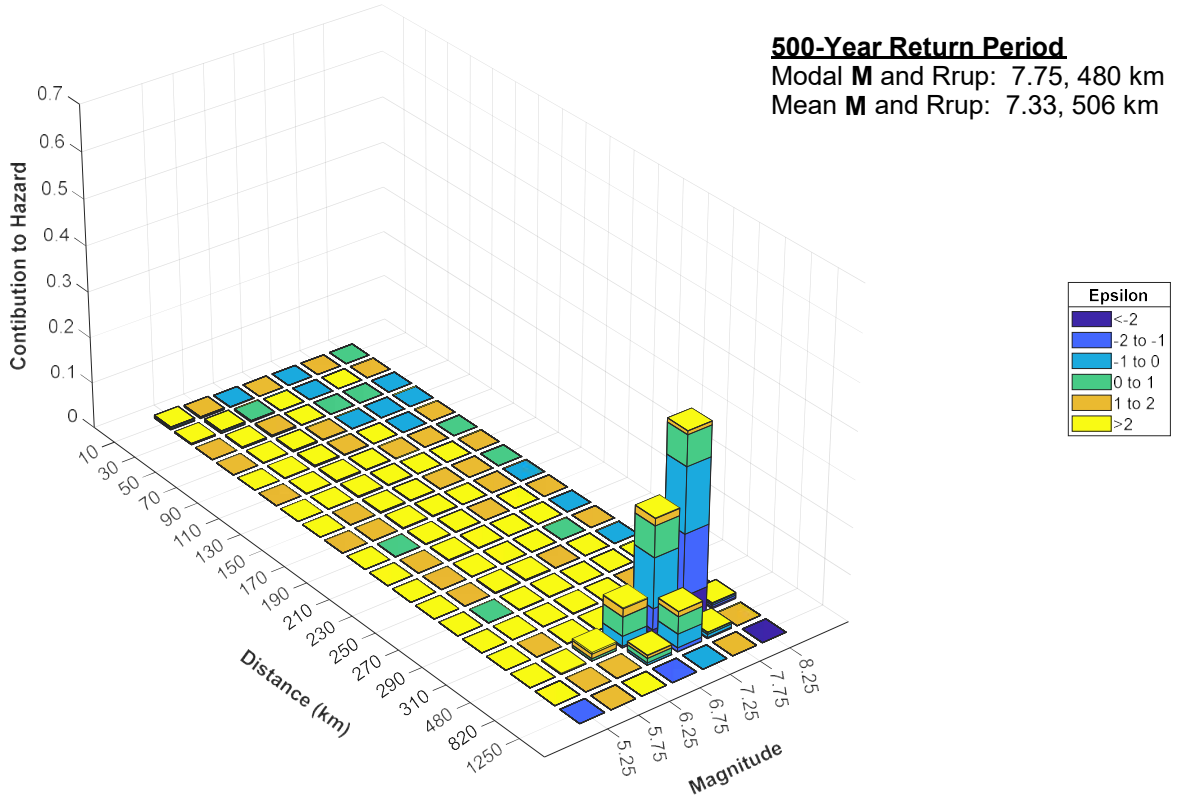


Magnitude and Distance Contributions to the Mean Peak Horizontal Acceleration Hazard at 500- and 2500-Year Return Periods on Hard Rock

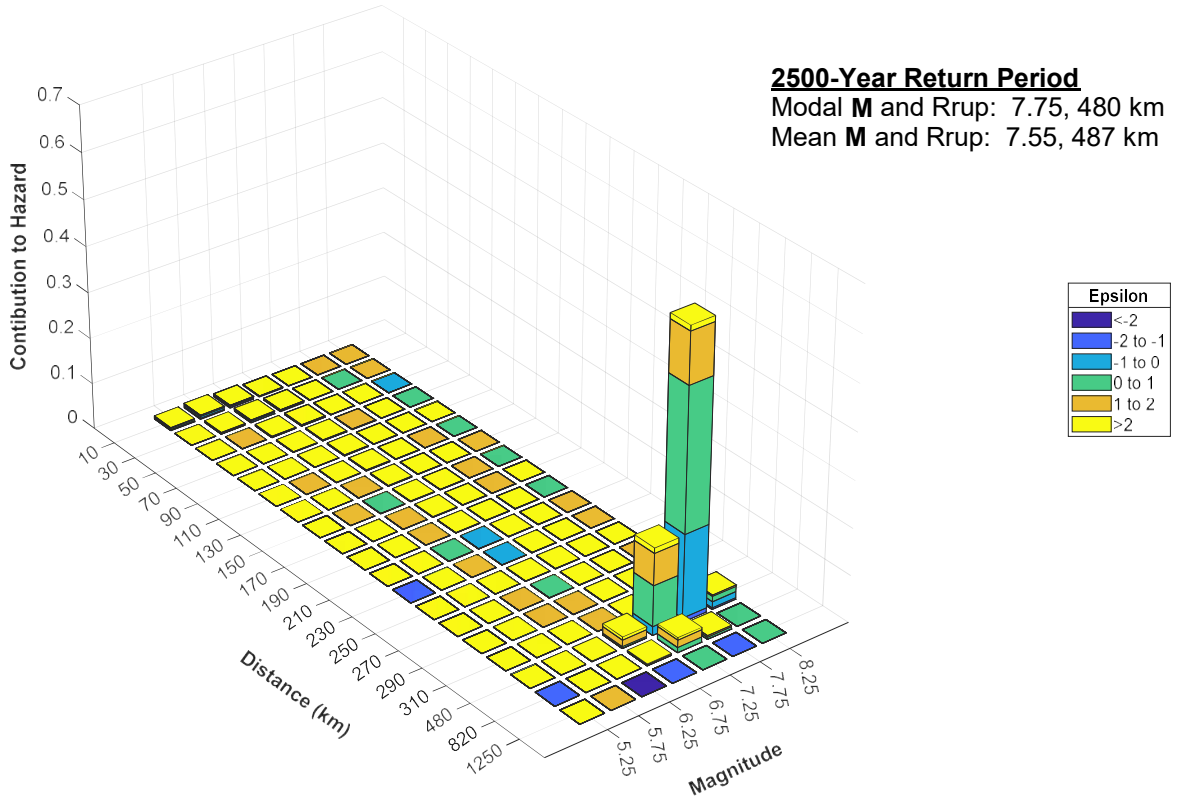
LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **17**

500-Year Return Period
Modal **M** and Rrup: 7.75, 480 km
Mean **M** and Rrup: 7.33, 506 km



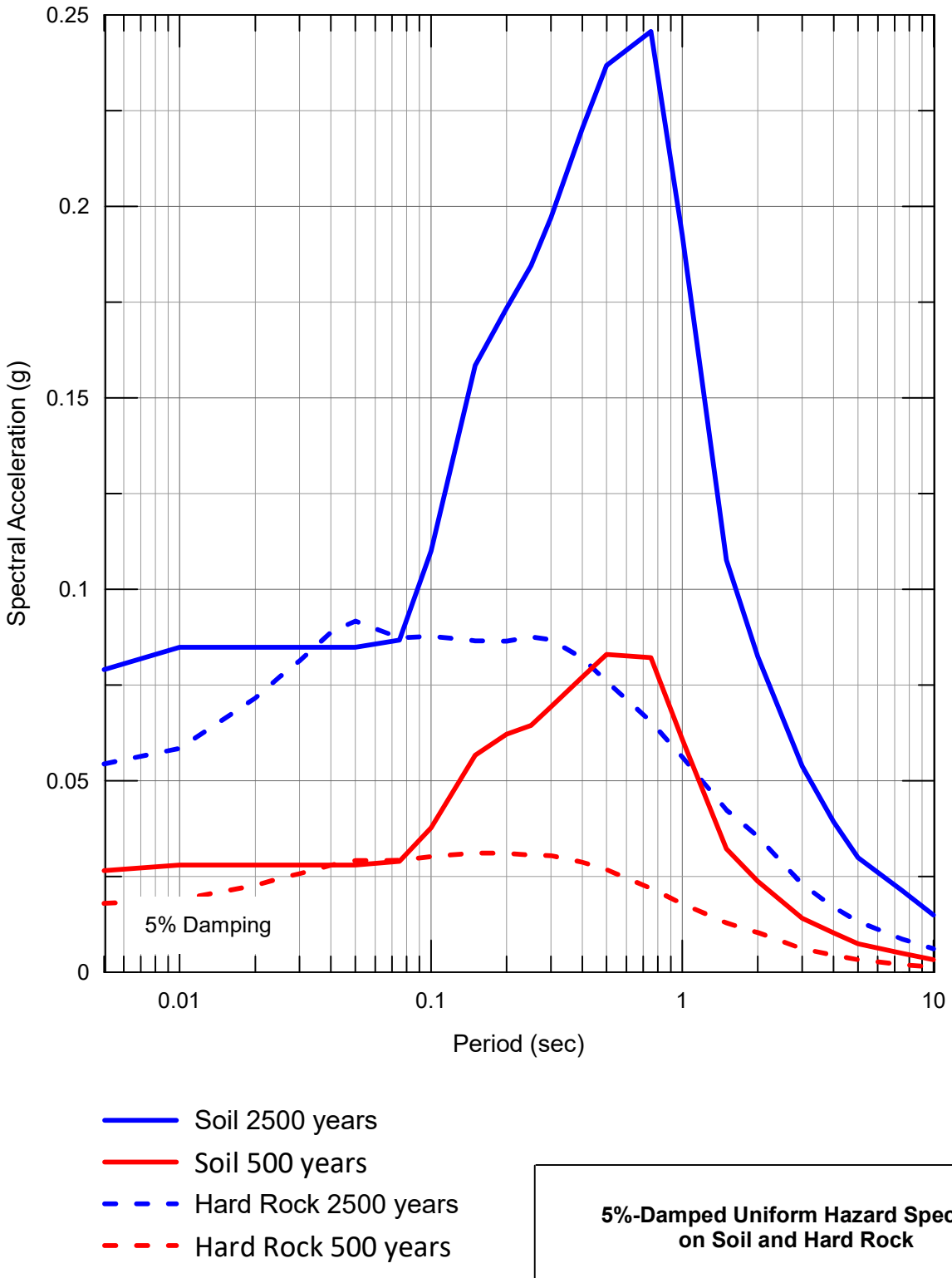
2500-Year Return Period
Modal **M** and Rrup: 7.75, 480 km
Mean **M** and Rrup: 7.55, 487 km



Magnitude and Distance Contributions to the Mean 1.0 Sec Horizontal Spectra Acceleration Hazard at 500- and 2500-Year Return Periods on Hard Rock

LOWMAN POWER PLANT, ALABAMA

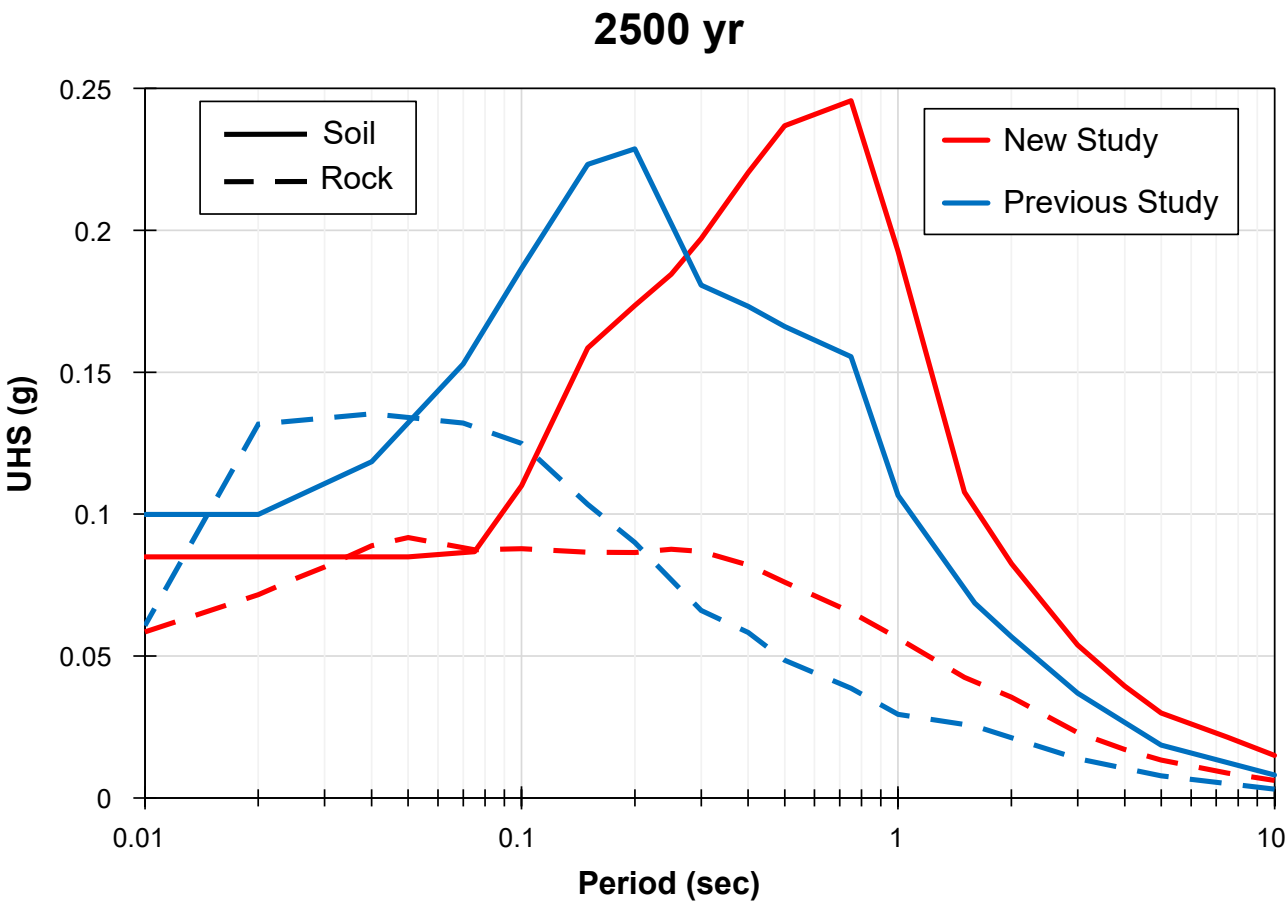
Lettis Consultants International, Inc. | Figure **18**



5%-Damped Uniform Hazard Spectra on Soil and Hard Rock

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **19**

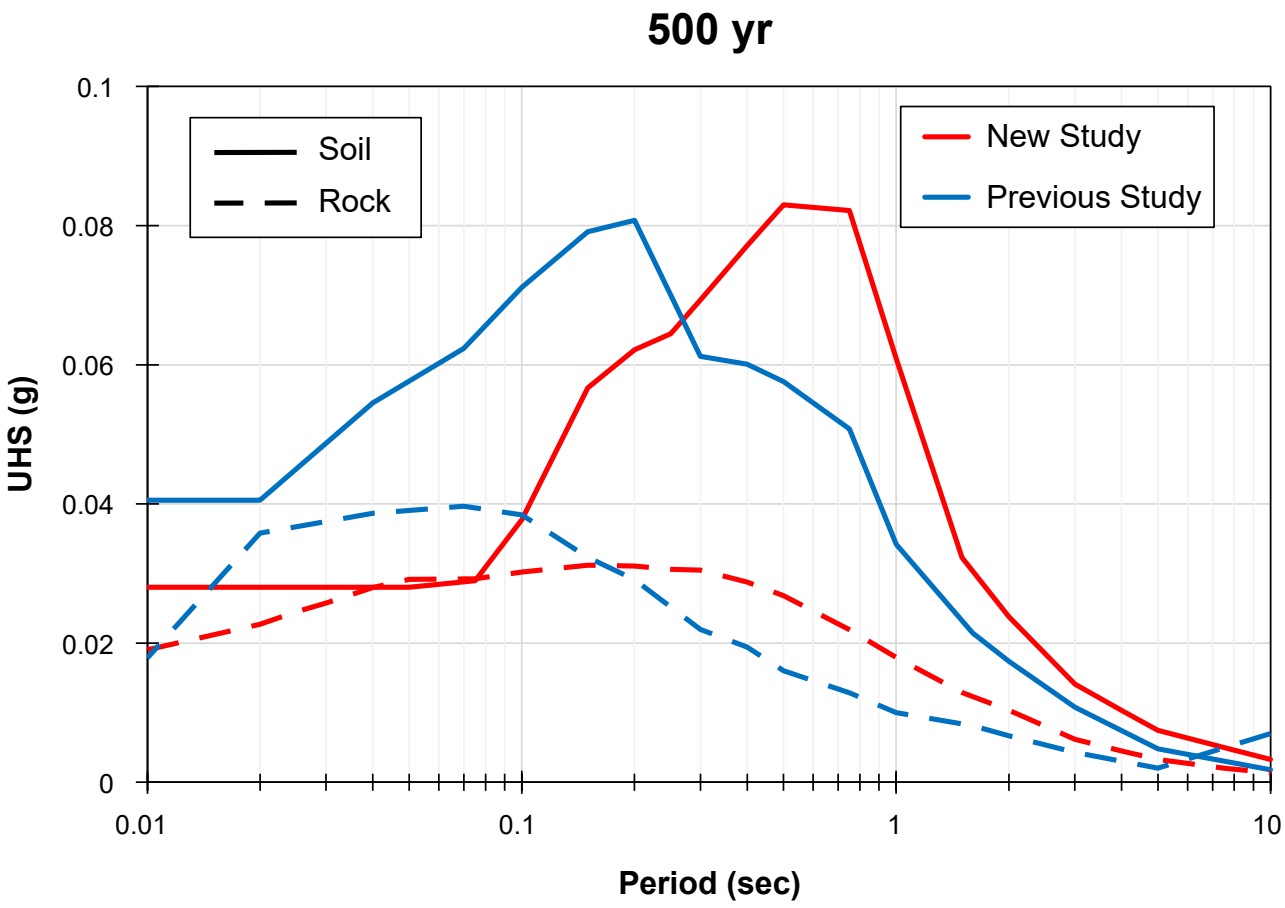


Comparing 2012 and Current 2,500 Year Return Period Rock and Soil UHS

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **20**

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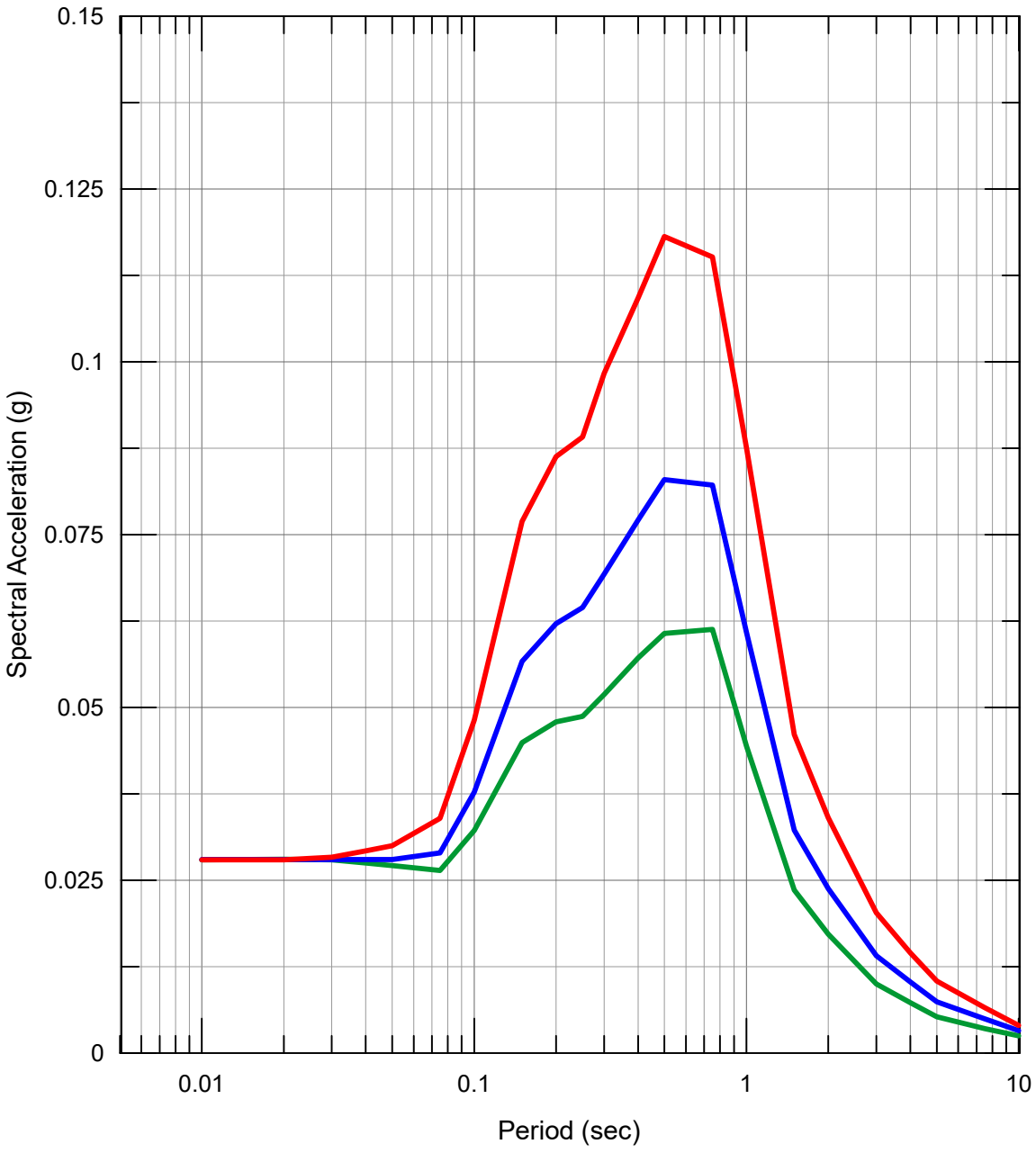


Comparing 2012 and Current 500 Year Return Period Rock and Soil UHS

LOWMAN POWER PLANT, ALABAMA

LCI Lettis Consultants International, Inc. | Figure **21**

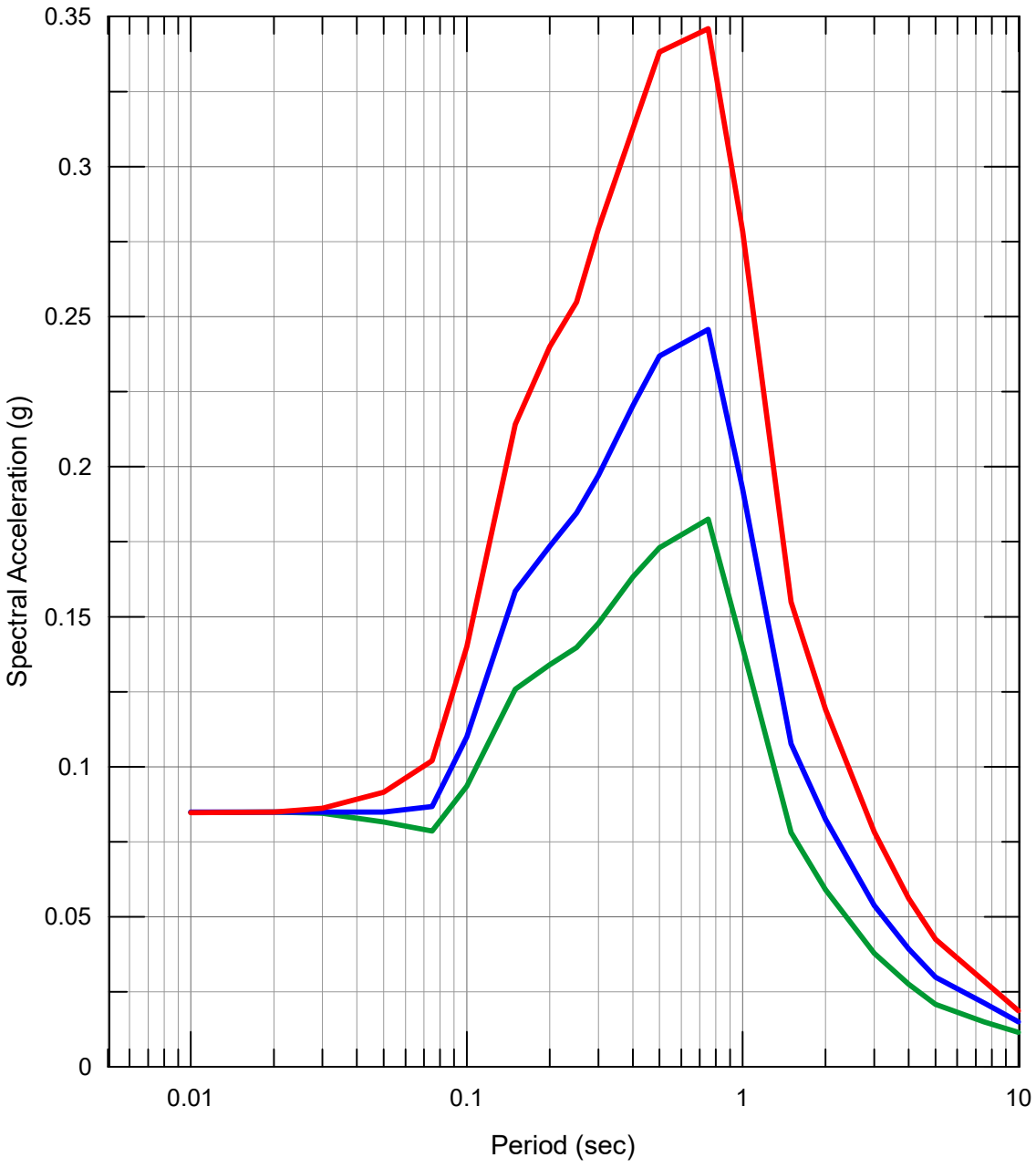
File path: S:\2105\00_Figures\LFigure_21.ai; Date: 06/30/2021; User: Jch, LCI.



2%, 5%, and 10%-Damped Uniform Hazard Spectra at the 500-Year Return Period at the Ground Surface

LOWMAN POWER PLANT, ALABAMA

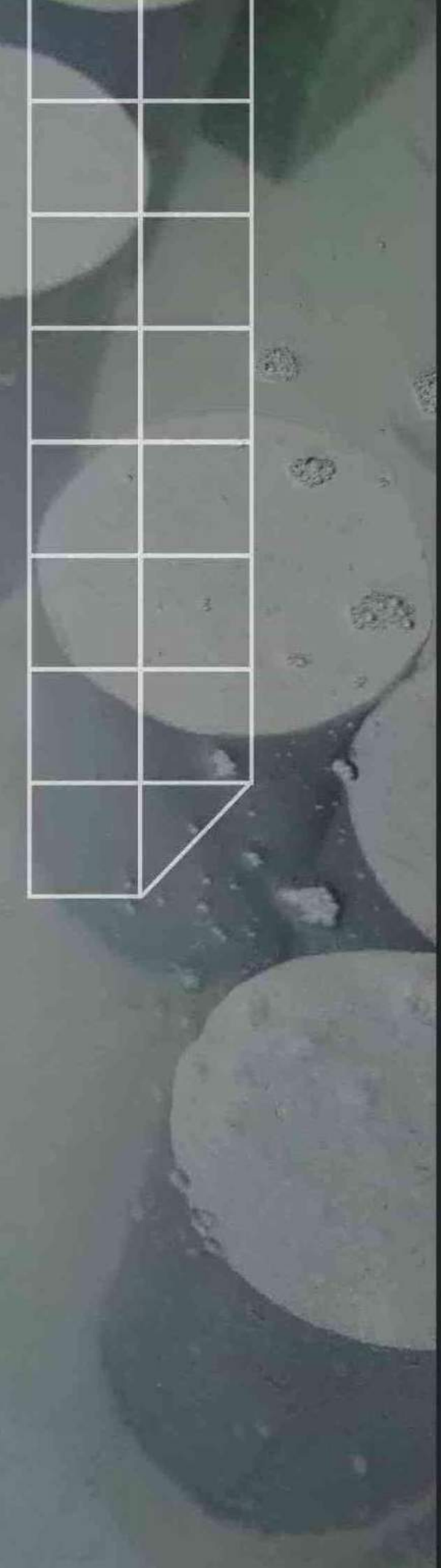
Lettis Consultants International, Inc. | Figure **22**



2%, 5%, and 10%-Damped Uniform Hazard Spectra at the 2,500-Year Return Period at the Ground Surface

LOWMAN POWER PLANT, ALABAMA

Lettis Consultants International, Inc. | Figure **23**



APPENDIX H

Seismic Stability and Deformation Analyses

DAN BROWN AND ASSOCIATES

danbrownandassociates.com

TECHNICAL MEMORANDUM

Seismic Consulting Services
Lowman Power Plant
Leroy, Alabama
DBA Project No. 26-606

To:	Justin Goss, PE Three Notch Group
From:	Timothy C. Siegel, P.E., G.E., D.GE Timothy M. O'Donnell, Ph.D., E.I.T. Benjamin Turner, Ph.D., P.E.
Date:	12 October 2026
Attachments:	Calculations

Timothy C. Siegel

ALABAMA LICENSED No. 33487 PROFESSIONAL TIMOTHY CARL SIEGEL ENGINEER

Digitally signed by Timothy C. Siegel
Date: 2026.10.13 08:47:02 -04'00'

This Technical Memorandum (TM) has been prepared by Dan Brown and Associates, LLC (DBA) to transmit the results of our latest seismic slope displacement and liquefaction analyses for the Lowman power plant in Leroy, Alabama.

We analyzed eight cross sections using the Bray and Travasarou (2007) method[1] to estimate permanent deformations of the embankments at the Lowman site. The Bray and Travasarou method is a semi-empirical procedure that estimates permanent displacement of a potential sliding mass from three inputs: the yield acceleration of the critical failure surface (k_y), the initial fundamental period of the sliding mass (T_s), and the spectral acceleration of the design ground motion at a degraded period of $1.5T_s$. The method was developed by computing the displacement of a nonlinear coupled sliding block subjected to a large suite of recorded ground motions from 41 earthquakes and fitting predictive equations to the results. The equations were then checked against the observed performance of 16 earth dams and landfills that experienced strong shaking. Because the equations are fit to a large set of real records, the method returns a range rather than a single value: the probability that displacement is negligible (less than about 0.5 inch), the median displacement if movement occurs, and the probability of exceeding a defined threshold.

We used yield accelerations shown on the pseudostatic slope stability analysis results for each cross section provided to DBA by Three Notch Group. We understand that Three Notch Group has updated their slope stability models to reflect the latest conditions; the computed yield accelerations are similar to those used in the previous evaluation by DBA in 2020. The fundamental period of each sliding mass was computed from its height and representative shear wave velocity of the embankment fill, taken as 650 ft per second. Spectral accelerations were taken from a 5-percent damping response spectrum for the 2,475-year return period (2 percent probability of exceedance in 50 years) hazard level obtained from the United States Geological Survey's latest (2023) national seismic hazard model. We note that the spectral accelerations are similar to those used in DBA's previous analyses based on a site-specific response spectrum developed for this project. The moment magnitude of the design earthquake was taken as 7.5 based on disaggregation of the computed seismic hazard, consistent with the previous study.

---

[1] Bray, J.D., and Travasarou, T. (2007). "Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 4, pp. 381–392.

Our previous analyses used the Makdisi and Seed (1978) method, which compares k_y to the maximum average acceleration of the sliding mass (k_{max}). In that method no displacement is calculated when k_{max} is less than k_y, and the ratio k_y/k_{max} was reported as an equivalent factor of safety. The Makdisi and Seed method was developed from a small number of earthquake records and treats displacement as either zero or non-zero based on a single comparison. The Bray and Travasarou method has largely replaced the Makdisi and Seed method in practice because it is based on a much larger set of recorded ground motions, uses a sliding block model that accounts for the flexibility of the embankment, and quantifies uncertainty in the estimated displacement. Notably, Bray and Travasarou compared both methods against case histories and found that the Makdisi and Seed method predicted no displacement at several embankments that moved one to two inches during the earthquake, whereas the newer method captured those cases. This is relevant to the Lowman project because for the cross sections where nonzero displacement is estimated, the magnitude of predicted settlement is small, as explained below, and the newer method provides a more reliable estimate of performance for these cases where seismic demand and resistance are of a similar order of magnitude.

Table 1 summarizes the results of the seismic displacement analysis. The table reports the median estimated displacement at each cross section and the computed probability that displacement will exceed 24 inches, which DBA understands is the maximum permanent seismic displacement allowed by the project design criteria.

Table 1: Summary of Seismic Displacement Analysis

Cross Section	k_y	Estimated Displacement, Median Prediction (inches)	Estimated Probability that Displacement Will Exceed 24 inches
D-D'	0.043	5	1%
E-E'	0.550	0	0%
F-F'	0.206	0	0%
G-G'	0.260	0	0%
H-H'	0.230	0	0%
J-J'	0.233	0	0%
K-K'	0.099	1	0%
L-L'	0.144	1	0%

For five of the eight cross sections the median estimated displacement is zero, identical to the previously reported results. At Sections D-D', K-K', and L-L', the new analyses give small median displacements of 1 to 5 inches whereas the previous analyses reported zero. This change reflects the analysis method rather than any change in the condition of the embankments. These three sections have the lowest yield accelerations relative to the design ground motion. Among the recorded motions on which the Bray and Travasarou method is based, some contain short pulses that briefly exceed k_y at this level of shaking, and the method reports the small movement that results. The Makdisi and Seed method cannot resolve movement of this size and reports zero instead. At every cross section the probability of exceeding the 24-inch design limit is 1 percent or less.

The estimated permanent displacements are well below the 24-inch design limit, with a probability of exceedance of 1 percent or less at every section. The embankments are therefore expected to perform acceptably under the design earthquake and conform to the requirements for seismic loading.

We also performed liquefaction analyses for borings SE-01, -02, and -03, and find that no liquefaction is expected. Summary results are attached.

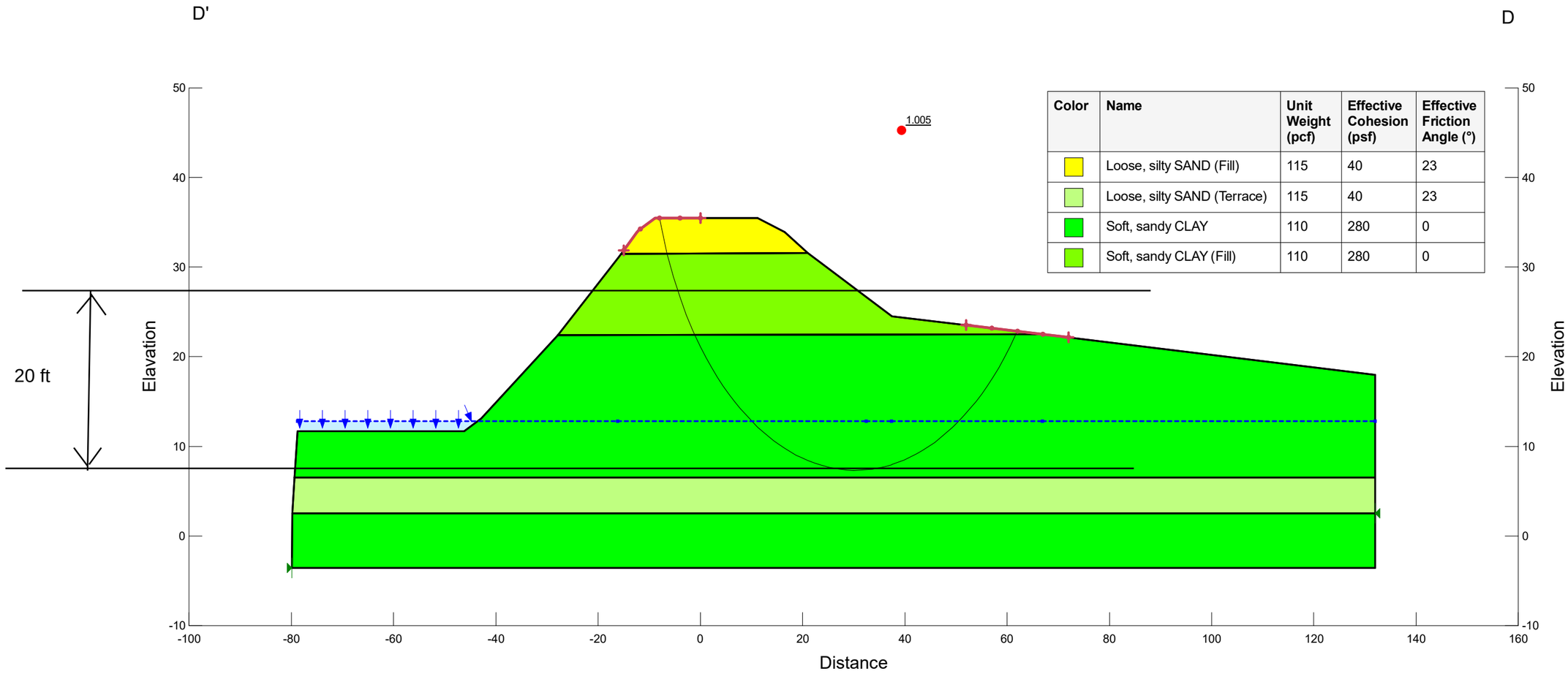
Attachments

Seismic Displacement Calculations

Liquefaction Calculations

Slope Displacement Calculations

Method: Spencer
Calculated Factor of Safety: 1.005
Yield Acceleration (ky): 0.043



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, silty SAND (Fill)	115	40	23
	Loose, silty SAND (Terrace)	115	40	23
	Soft, sandy CLAY	110	280	0
	Soft, sandy CLAY (Fill)	110	280	0

Unit 1 Impoundment
Charles R. Lowman Power Plant
CDG Project No. R021221021

Section D-D' - Pseudostatic, Maximum Storage Pool		
Section D-D' - Pseudostatic Permeter Berm Static.gsz		
10/08/2026	1:260	Vertical Exaggeration: 1.75

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

Reference: Jonathan D. Bray and Thaleia Travasarou

Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.043		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.12	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.18	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.24	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	13.01	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.000		eq. (3)
D1	6.74	cm	calc. using eq. (7)
D2	13.00	cm	calc. using eq. (7)
D3	25.07	cm	calc. using eq. (7)
P(D>d_threshold)	0.010		eq. (7)

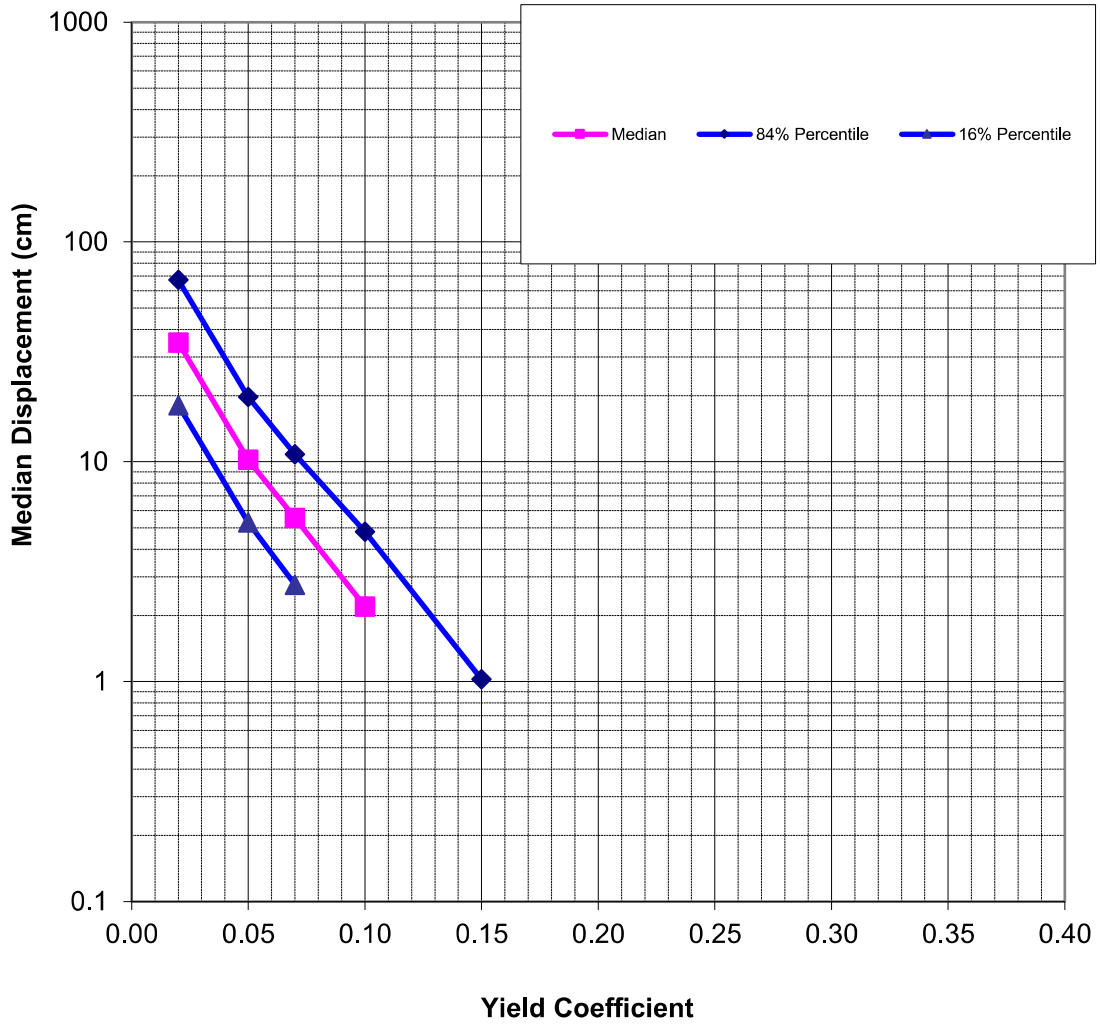
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section D-D'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	34.8	34.8	67.2	18.1
0.05	0.00	10.2	10.2	19.7	5.3
0.07	0.03	5.7	5.5	10.8	2.8
0.1	0.22	2.8	2.2	4.8	<1
0.15	0.71	1.1	<1	1.0	<1
0.2	0.93	0.6	<1	<1	<1
0.3	1.00	0.2	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

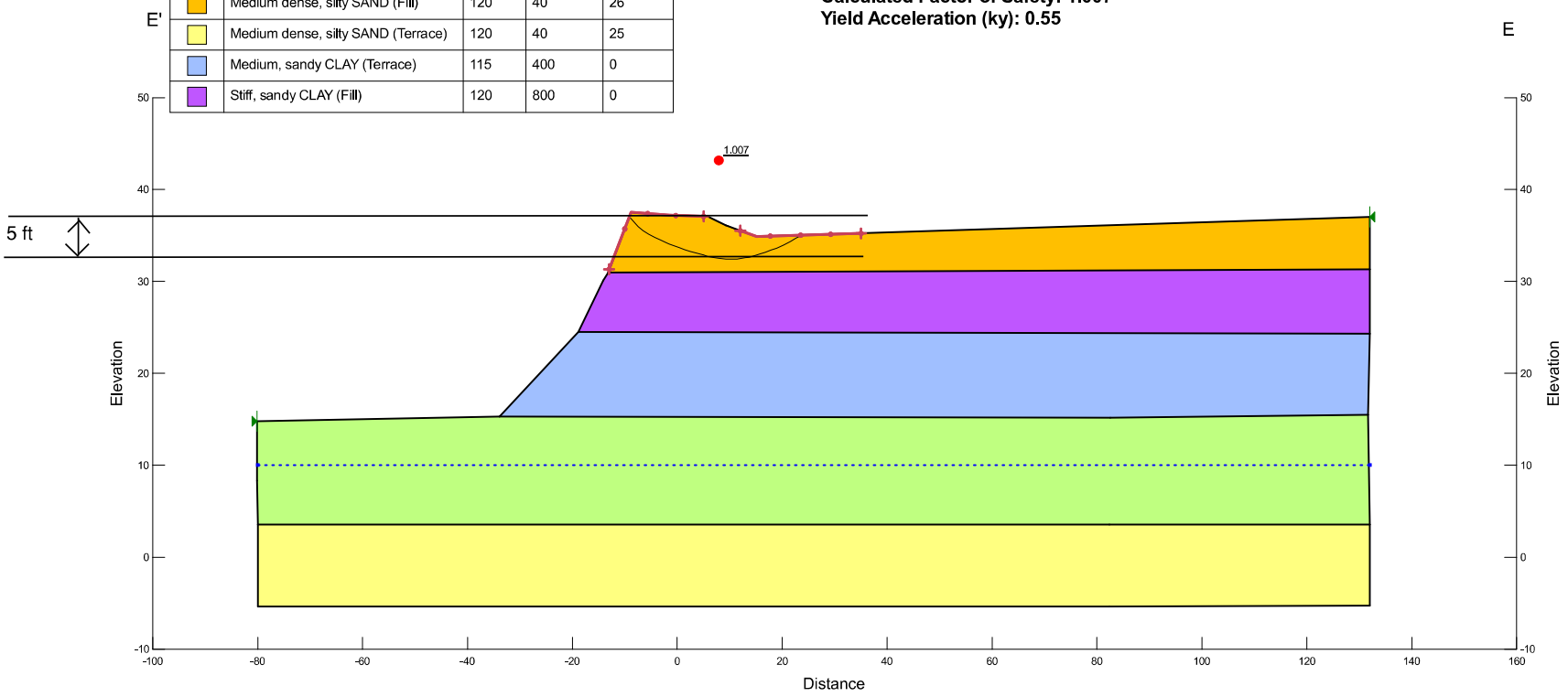
Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, silty SAND (Terrace)	115	40	23
	Medium dense, silty SAND (Fill)	120	40	26
	Medium dense, silty SAND (Terrace)	120	40	25
	Medium, sandy CLAY (Terrace)	115	400	0
	Stiff, sandy CLAY (Fill)	120	800	0

Method: Spencer
Calculated Factor of Safety: 1.007
Yield Acceleration (ky): 0.55



Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section E-E' - Pseudostatic Maximum Storage Pool		
Section E-E' - Pseudostatic Maximum Storage Pool.gsz		
08/27/2026	1:260	Vertical Exaggeration: 1.75

Cross Section E-E'
with new/proposed embankment fill

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

Reference: Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.55		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.03	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.05	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.17	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	0.02	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	1.000		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	<1	cm	calc. using eq. (7)
D3	<1	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

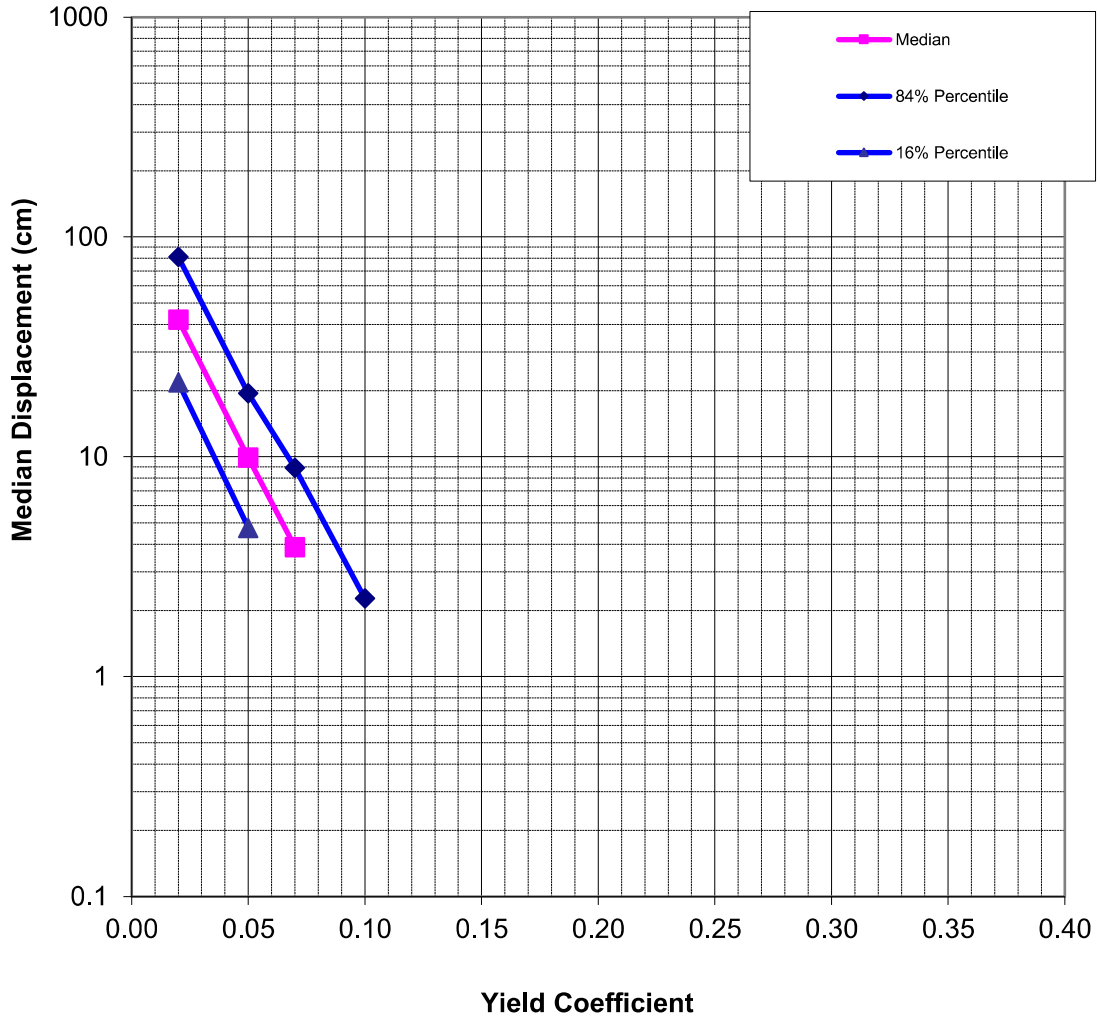
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section E-E'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	42.0	42.0	80.9	21.8
0.05	0.05	10.3	9.9	19.5	4.7
0.07	0.27	5.3	3.9	8.9	<1
0.1	0.71	2.5	<1	2.3	<1
0.15	0.97	0.9	<1	<1	<1
0.2	1.00	0.4	<1	<1	<1
0.3	1.00	0.1	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

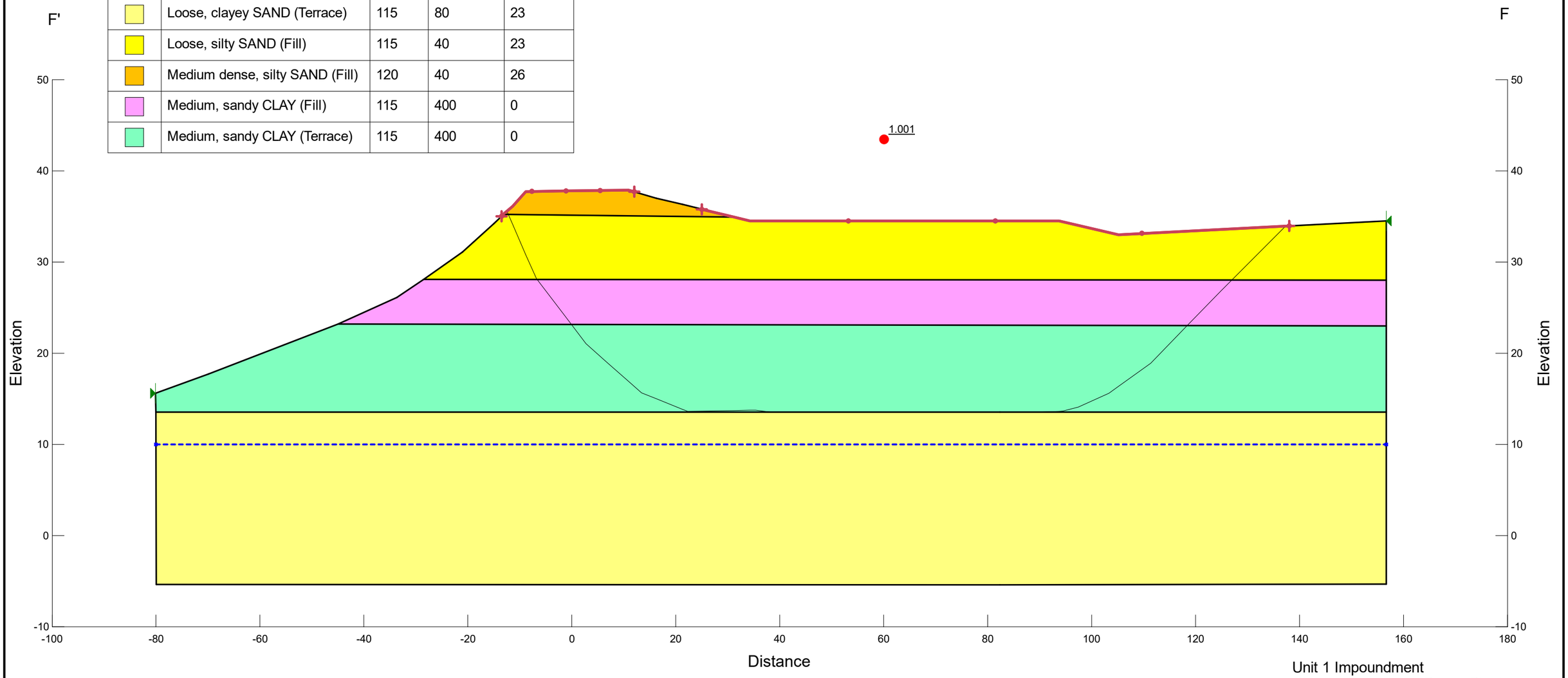
Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Method: Spencer
Calculated Factor of Safety: 1.001
Yield Acceleration (ky): 0.206

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, clayey SAND (Terrace)	115	80	23
	Loose, silty SAND (Fill)	115	40	23
	Medium dense, silty SAND (Fill)	120	40	26
	Medium, sandy CLAY (Fill)	115	400	0
	Medium, sandy CLAY (Terrace)	115	400	0



Unit 1 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021221021

Section F-F' - Pseudostatic Maximum Storage Pool

Section F-F' - Pseudostatic Maximum Storage Pool.gsz

08/27/2026 1:225 Vertical Exaggeration: 1.75

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

Reference: Jonathan D. Bray and Thaleia Travasarou

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Input Parameters			
Yield Coefficient (ky)	0.206		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.12	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.18	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.24	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	0.51	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.946		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	<1	cm	calc. using eq. (7)
D3	<1	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

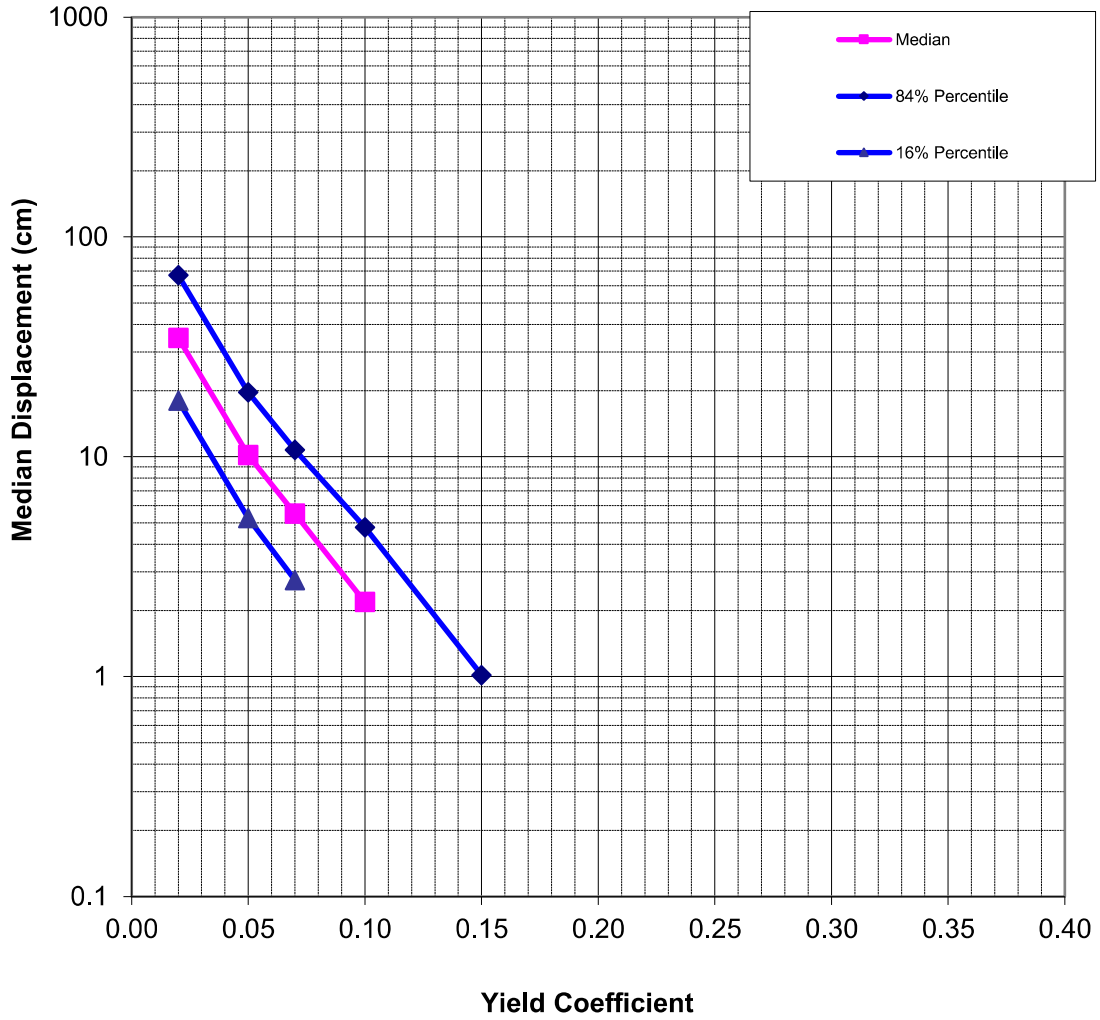
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section F-F'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	34.7	34.7	66.9	18.0
0.05	0.00	10.2	10.2	19.6	5.3
0.07	0.03	5.6	5.5	10.7	2.7
0.1	0.22	2.8	2.2	4.8	<1
0.15	0.71	1.1	<1	1.0	<1
0.2	0.93	0.6	<1	<1	<1
0.3	1.00	0.2	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

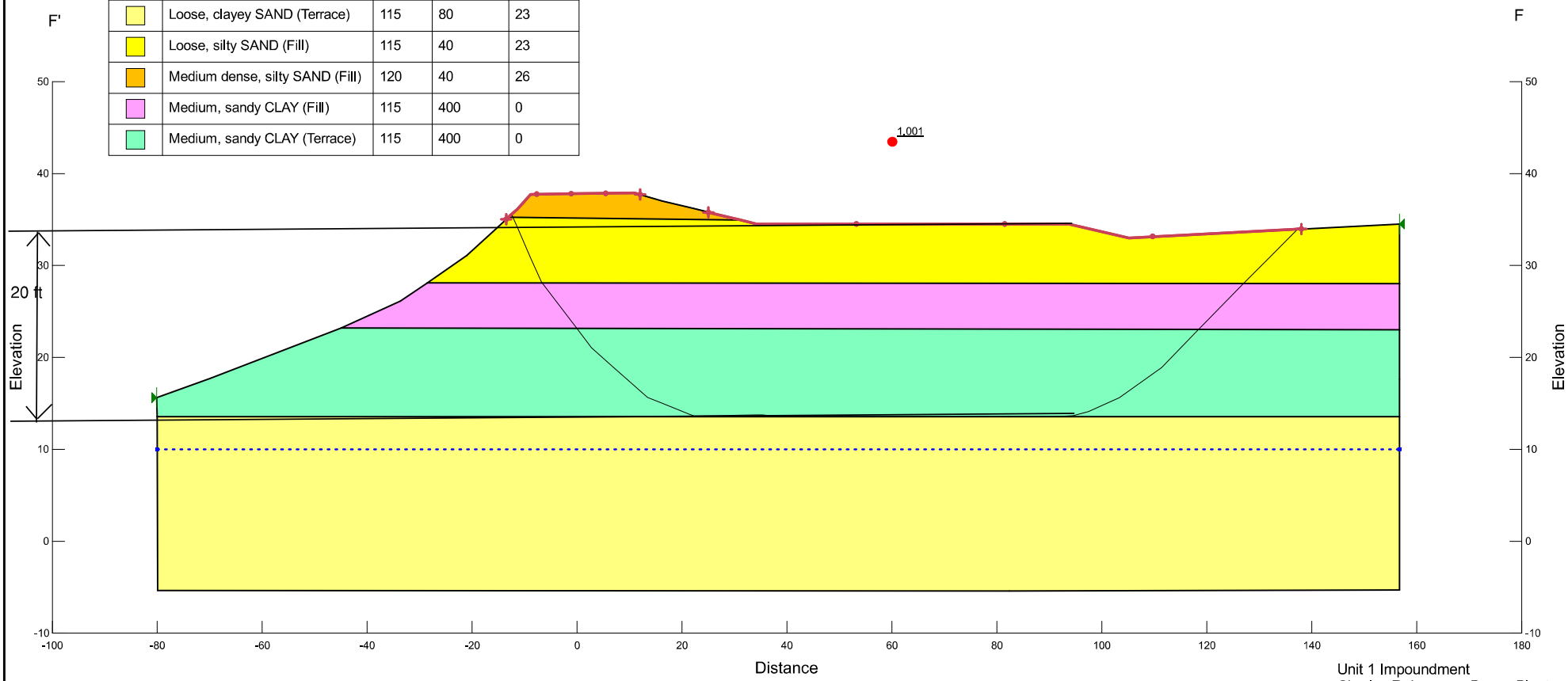
Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Method: Spencer
Calculated Factor of Safety: 1.001
Yield Acceleration (ky): 0.206

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Loose, clayey SAND (Terrace)	115	80	23
	Loose, silty SAND (Fill)	115	40	23
	Medium dense, silty SAND (Fill)	120	40	26
	Medium, sandy CLAY (Fill)	115	400	0
	Medium, sandy CLAY (Terrace)	115	400	0



Unit 1 Impoundment
Charles R. Lowman Power Plant
CDG Project No. R021221021

Section F-F' - Pseudostatic Maximum Storage Pool
Section F-F' - Pseudostatic Maximum Storage Pool.gsz
08/27/2026 1:225 Vertical Exaggeration: 1.75

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SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.26		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.06	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.09	seconds	
Moment Magnitude (M_w)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.23	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	0.22	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

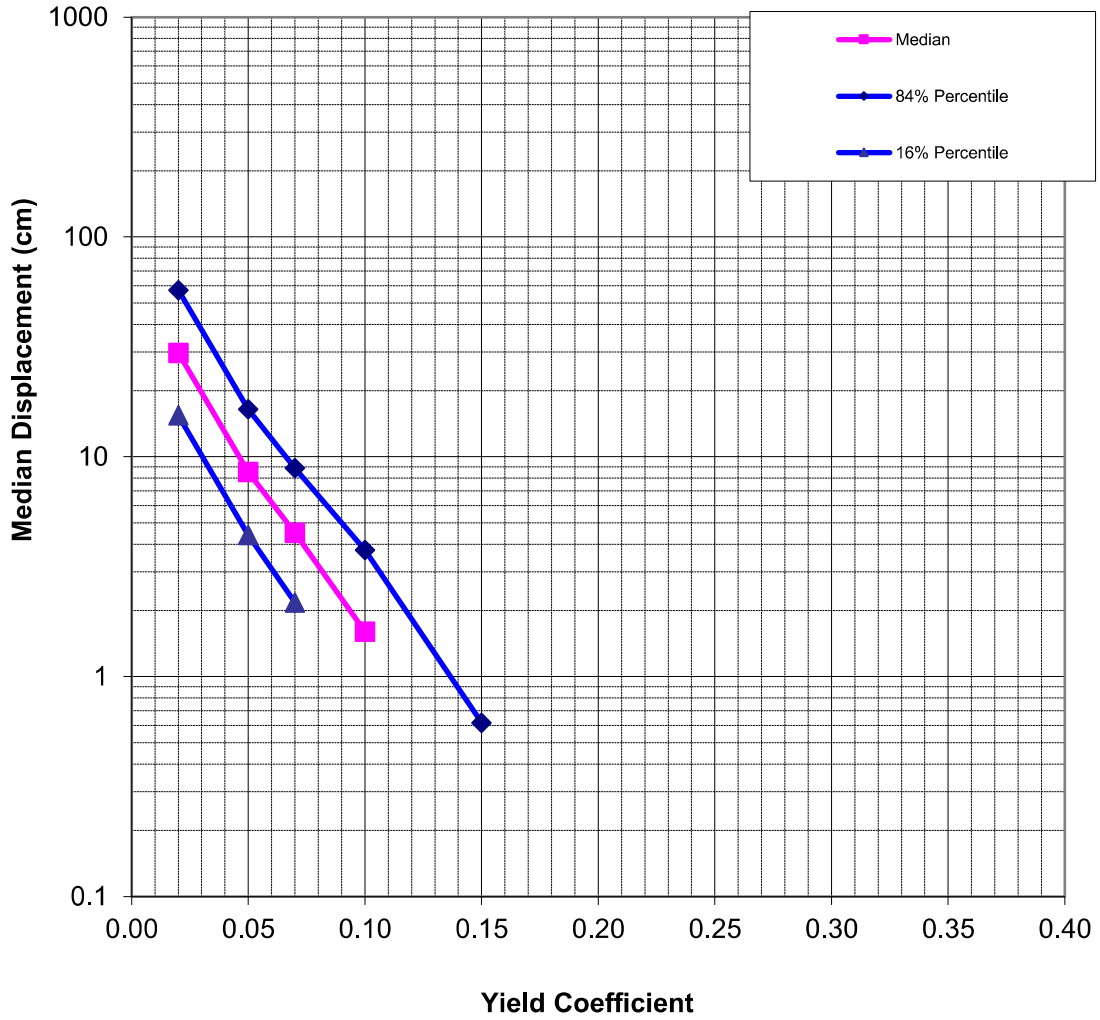
Results			
Probability of Negligible Displ. (P(D=0))	0.995		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	<1	cm	calc. using eq. (7)
D3	<1	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	29.7	29.7	57.2	15.4
0.05	0.00	8.5	8.5	16.4	4.4
0.07	0.04	4.7	4.5	8.9	2.2
0.1	0.29	2.3	1.6	3.8	<1
0.15	0.78	0.9	<1	0.6	<1
0.2	0.96	0.4	<1	<1	<1
0.3	1.00	0.1	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

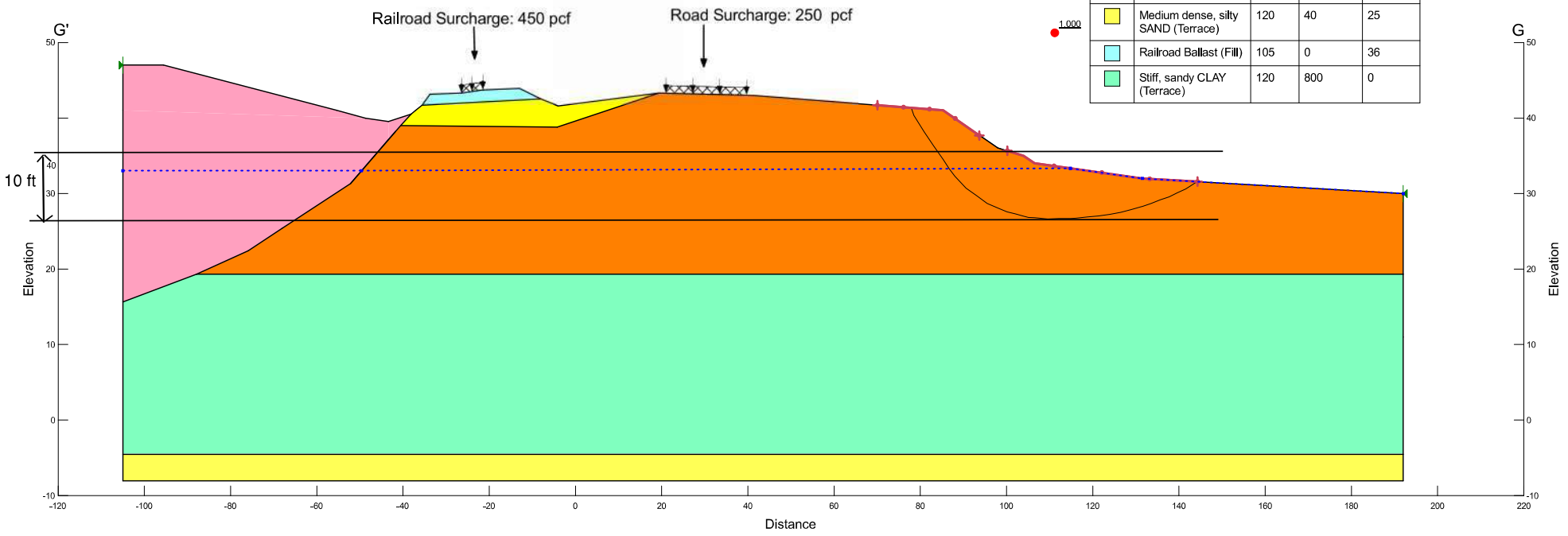
Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Method: Spencer
Calculated Factor of Safety: 1.000
Yield Acceleration (ky): 0.26

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	40	28
	Loose, silty SAND (Fill)	115	40	23
	Medium dense, silty SAND (Terrace)	120	40	25
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	800	0



Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section G-G' - Pseudostatic, Maximum Storage Pool		
Section G-G' - Psuedostatic, Maximum Storage Pool.gsz		
09/02/2026	1:275	Vertical Exaggeration: 1.75

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

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Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.23		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.11	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.17	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.24	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	0.38	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.976		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	<1	cm	calc. using eq. (7)
D3	<1	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

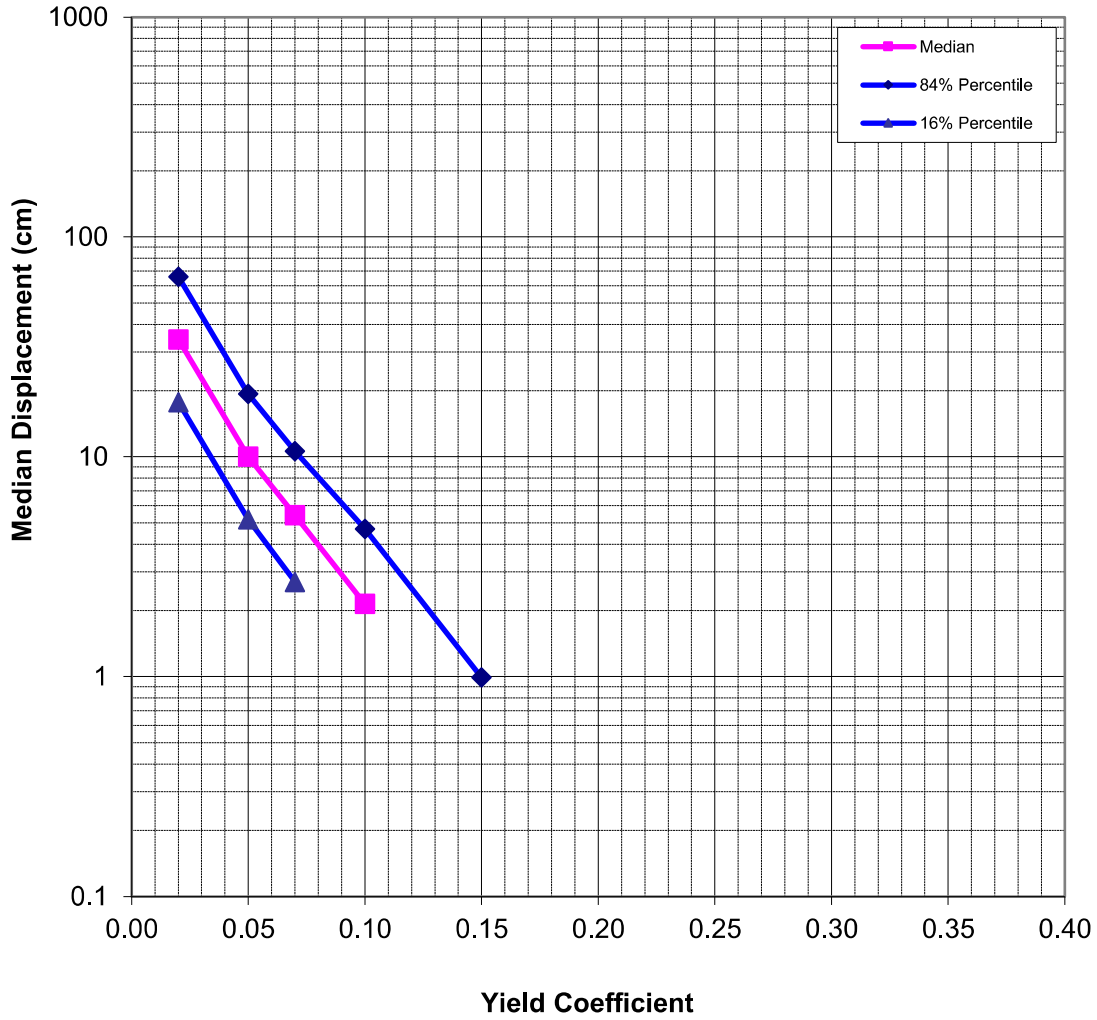
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section H-H'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	34.2	34.2	65.9	17.7
0.05	0.00	10.0	10.0	19.3	5.2
0.07	0.03	5.6	5.4	10.6	2.7
0.1	0.23	2.7	2.1	4.7	<1
0.15	0.72	1.1	<1	1.0	<1
0.2	0.94	0.5	<1	<1	<1
0.3	1.00	0.2	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

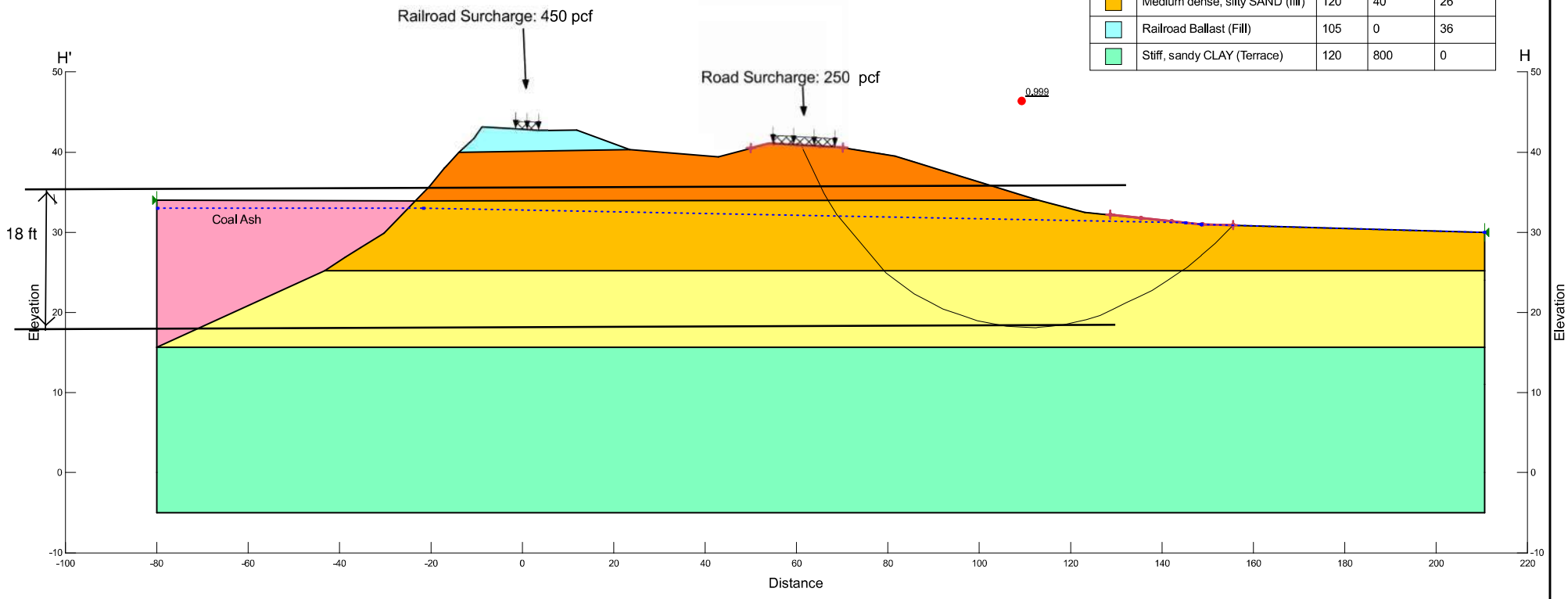
Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Method: Spencer
Calculated Factor of Safety: 0.999
Yield Acceleration (ky): 0.23

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	40	28
	Loose, silty SAND (Terrace)	115	40	23
	Medium dense, silty SAND (fill)	120	40	26
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	800	0



Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section H-H' - Pseudostatic, Maximum Storage Pool		
Section H-H' - Psuedostatic, Maximum Storage Pool.gsz		
09/02/2026	1:255	Vertical Exaggeration: 1.75

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

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Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.233		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.08	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.12	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.24	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	0.35	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.979		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	<1	cm	calc. using eq. (7)
D3	<1	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

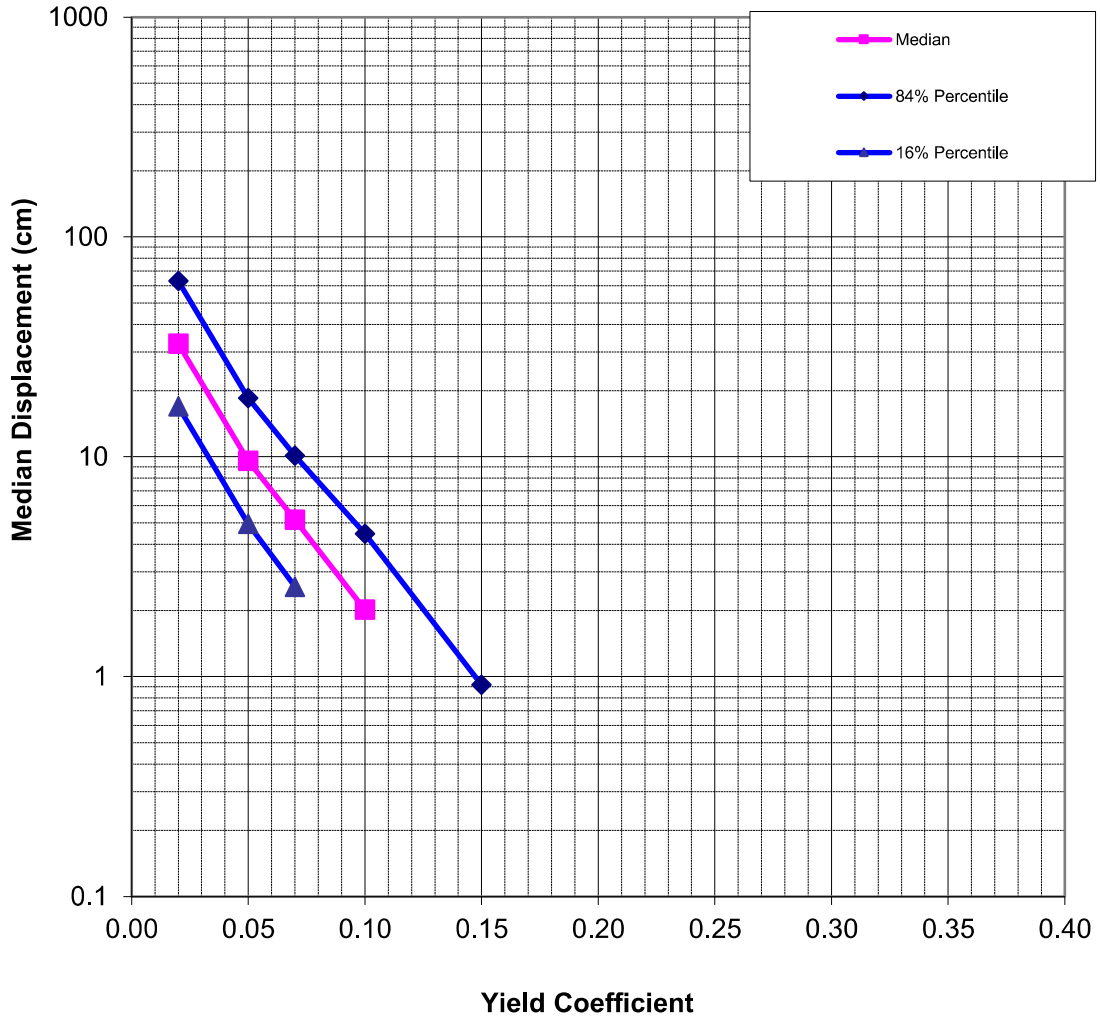
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section J-J'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	32.7	32.7	63.0	16.9
0.05	0.00	9.6	9.6	18.5	5.0
0.07	0.03	5.3	5.2	10.1	2.6
0.1	0.24	2.6	2.0	4.5	<1
0.15	0.73	1.1	<1	0.9	<1
0.2	0.94	0.5	<1	<1	<1
0.3	1.00	0.2	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

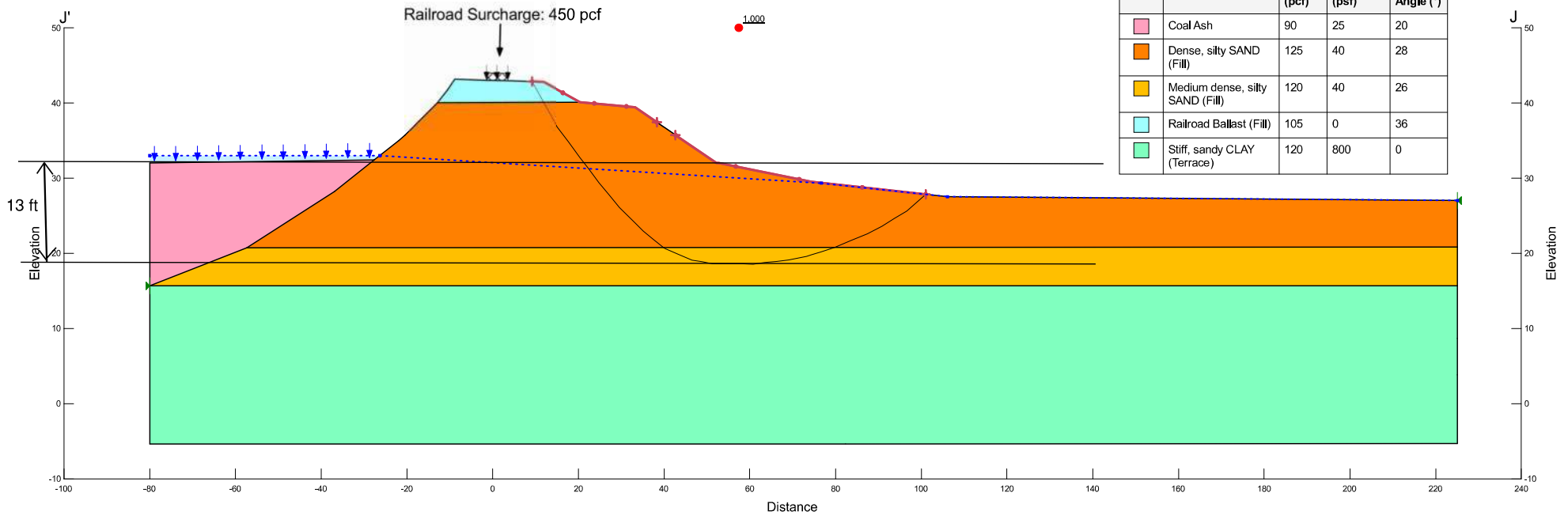
a	-1.760
b	-3.220
c	-0.484
d	3.520

Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Method: Spencer
Calculated Factor of Safety: 1.000
Yield Acceleration (ky): 0.233



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Coal Ash	90	25	20
	Dense, silty SAND (Fill)	125	40	28
	Medium dense, silty SAND (Fill)	120	40	26
	Railroad Ballast (Fill)	105	0	36
	Stiff, sandy CLAY (Terrace)	120	800	0

Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section J-J' - Pseudostatic, Maximum Storage Pool		
Section J-J' - Psuedostatic, Maximum Storage Pool.gsz		
09/02/2026	1:275	Vertical Exaggeration: 1.75

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

Reference: Jonathan D. Bray and Thaleia Travasarou

Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.099		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.20	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.30	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.24	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	3.20	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.187		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	2.64	cm	calc. using eq. (7)
D3	5.62	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

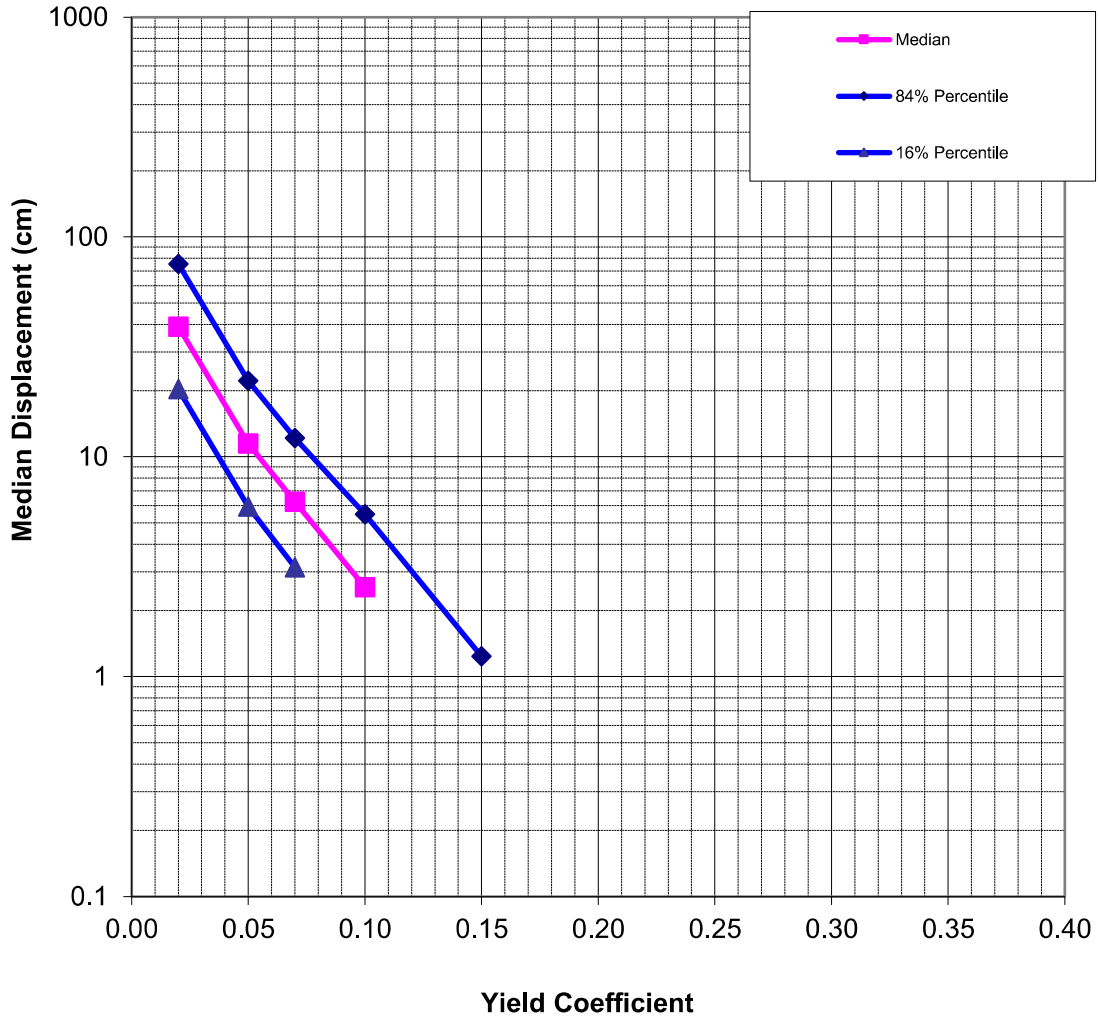
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section K-K'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	39.1	39.1	75.4	20.3
0.05	0.00	11.5	11.5	22.1	5.9
0.07	0.02	6.4	6.3	12.2	3.1
0.1	0.20	3.1	2.6	5.5	<1
0.15	0.69	1.3	<1	1.2	<1
0.2	0.93	0.6	<1	<1	<1
0.3	1.00	0.2	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

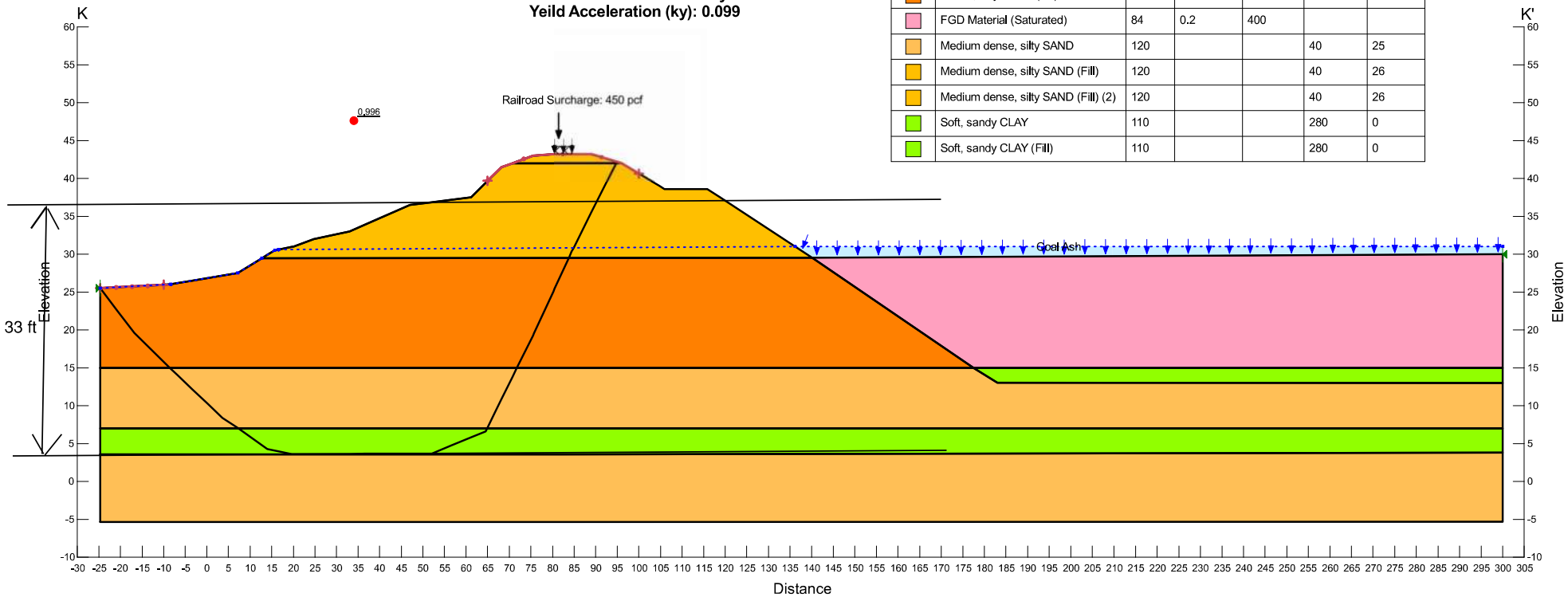
Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	

Method: Spencer
Calculated Factor of Safety: 0.996
Yeild Acceleration (ky): 0.099

Color	Name	Unit Weight (pcf)	Tau/Sigma	Minimum Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense, silty SAND (Fill)	125			40	28
	FGD Material (Saturated)	84	0.2	400		
	Medium dense, silty SAND	120			40	25
	Medium dense, silty SAND (Fill)	120			40	26
	Medium dense, silty SAND (Fill) (2)	120			40	26
	Soft, sandy CLAY	110			280	0
	Soft, sandy CLAY (Fill)	110			280	0



Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

Section K-K' - Pseudostatic, Maximum Storage Pool
Section K-K' Pseudostatic, Maximum Storage Pool.gsz
09/03/2026 1:275 **Vertical Exaggeration: 1.75**

Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements

Reference: Jonathan D. Bray and Thaleia Travasarou

Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.144		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.15	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.23	seconds	
Moment Magnitude (Mw)	7.6		
Spectral Acceleration (Sa(1.5Ts))	0.24	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	61	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	1.29	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.656		eq. (3)
D1	<1	cm	calc. using eq. (7)
D2	<1	cm	calc. using eq. (7)
D3	1.37	cm	calc. using eq. (7)
P(D>d_threshold)	0.000		eq. (7)

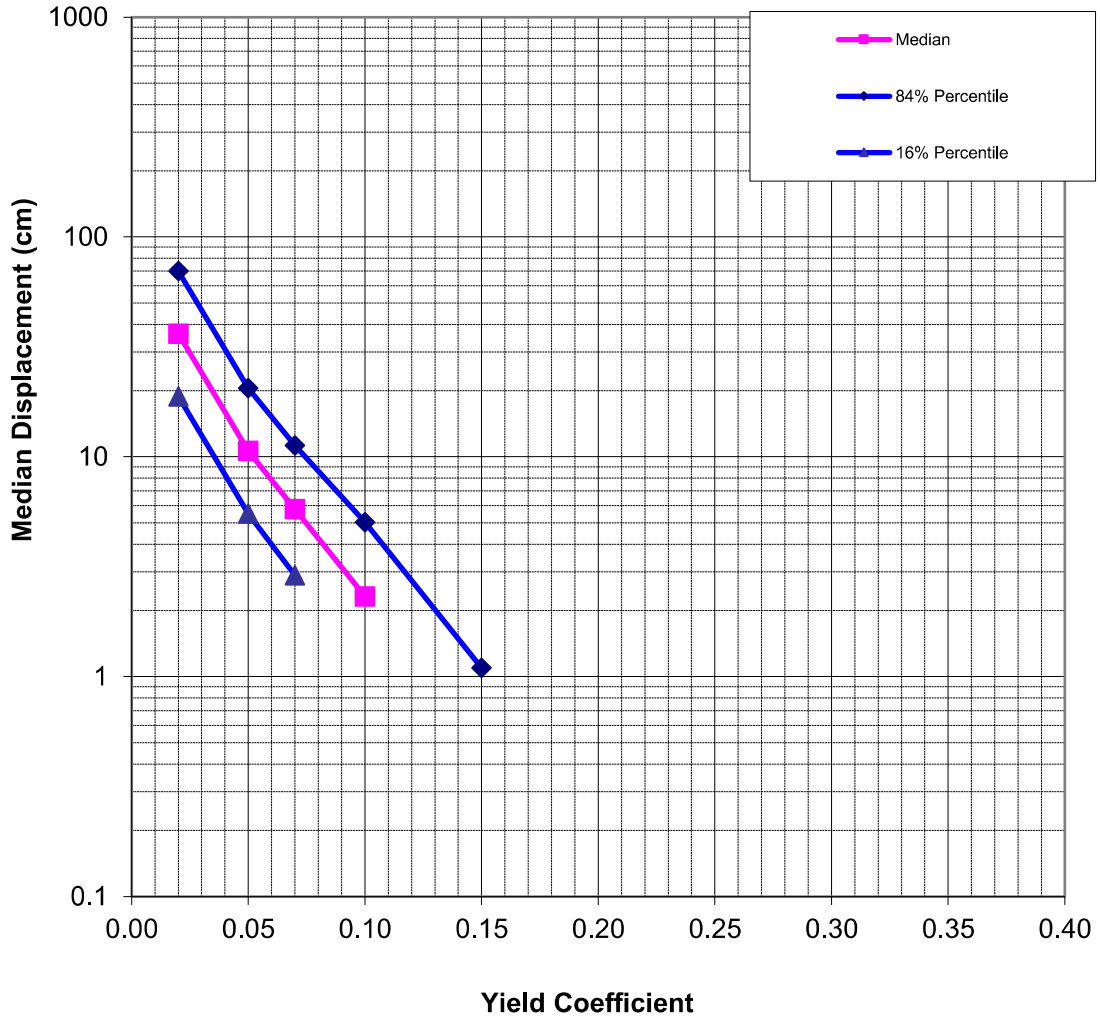
Notes

1. Values highlighted in blue are input parameters
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
5. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
6. Rigid slope is assumed for Ts < 0.05 s
7. When a value for D is not calculated, D is < 1cm
8. ky may be estimated using the simplified equations shown below.
9. Examples of how Ts is estimated are shown below.
10. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Cross Section L-L'
with new/proposed embankment fill

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D1 (cm)	D3 (cm)
0.020	0.00	36.3	36.3	69.9	18.8
0.05	0.00	10.7	10.6	20.5	5.5
0.07	0.02	5.9	5.8	11.3	2.9
0.1	0.21	2.9	2.3	5.0	<1
0.15	0.70	1.2	<1	1.1	<1
0.2	0.93	0.6	<1	<1	<1
0.3	1.00	0.2	<1	<1	<1
0.4	1.00	0.1	<1	<1	<1



Model for Probability of Zero

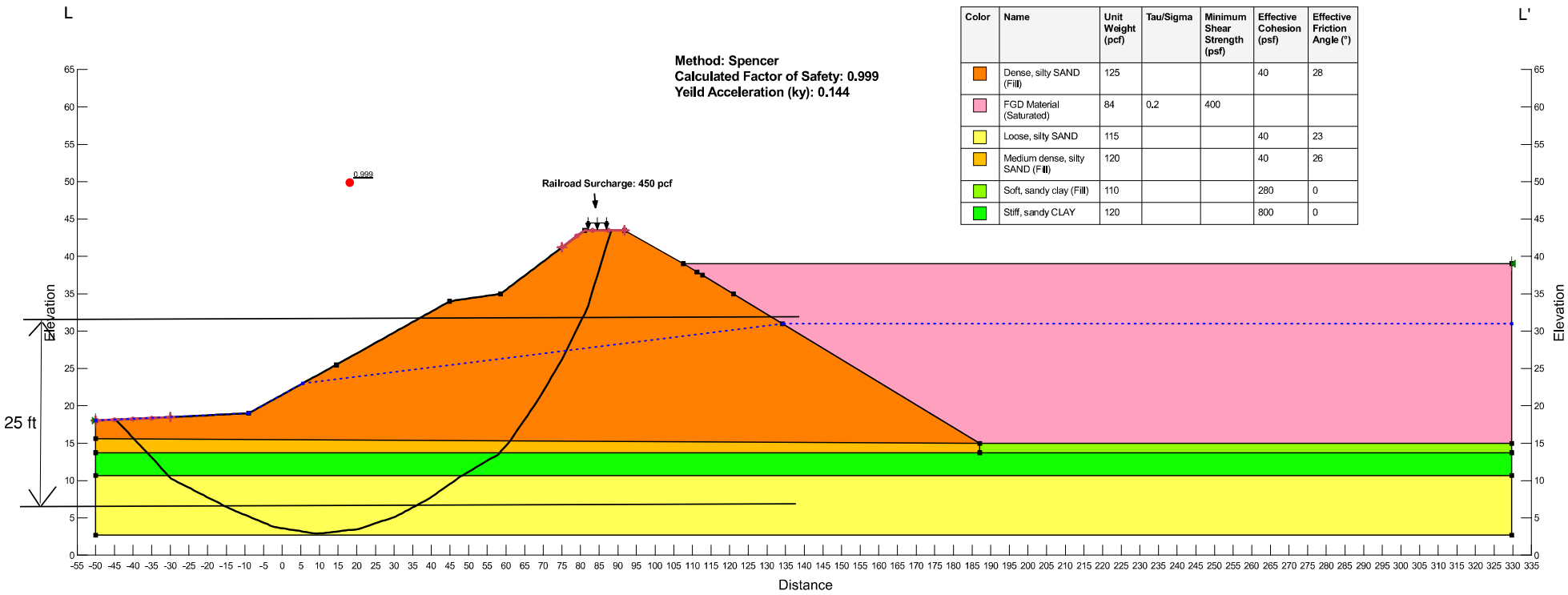
P("0")= 1-Φ(a+blnky+clnkyTs + dlnSA15)

a	-1.760
b	-3.220
c	-0.484
d	3.520

Model for Displacement Estimate

ln(D)=a+blnky + clnky2 + dlnky*lnSA15 + e*ln(SA15) + f*[ln(SA15)]^2 + g*Ts + h*(M-7)

Coefficients	Values	Rigid Block (Ts=0)
a	-1.1	-0.22
b	-2.83	
c	-0.333	
d	0.566	
e	3.04	
f	-0.244	
g	1.5	
h	0.278	
σ	0.660	



Color	Name	Unit Weight (pcf)	Tau/Sigma	Minimum Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Dense, silty SAND (Fill)	125			40	28
	FGD Material (Saturated)	84	0.2	400		
	Loose, silty SAND	115			40	23
	Medium dense, silty SAND (Fill)	120			40	26
	Soft, sandy clay (Fill)	110			280	0
	Stiff, sandy CLAY	120			800	0

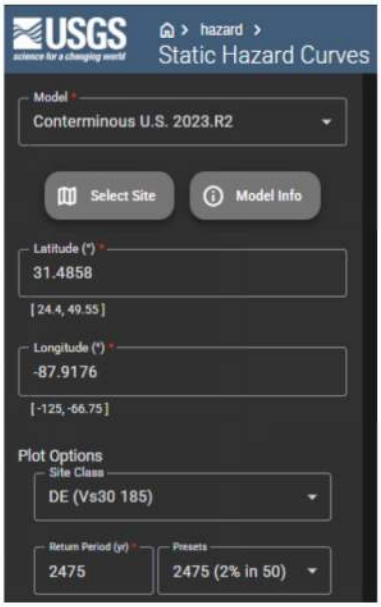
Unit 2/3 Impoundment
Charles R. Lowman Power Plant
Three Notch Project No. R021225012

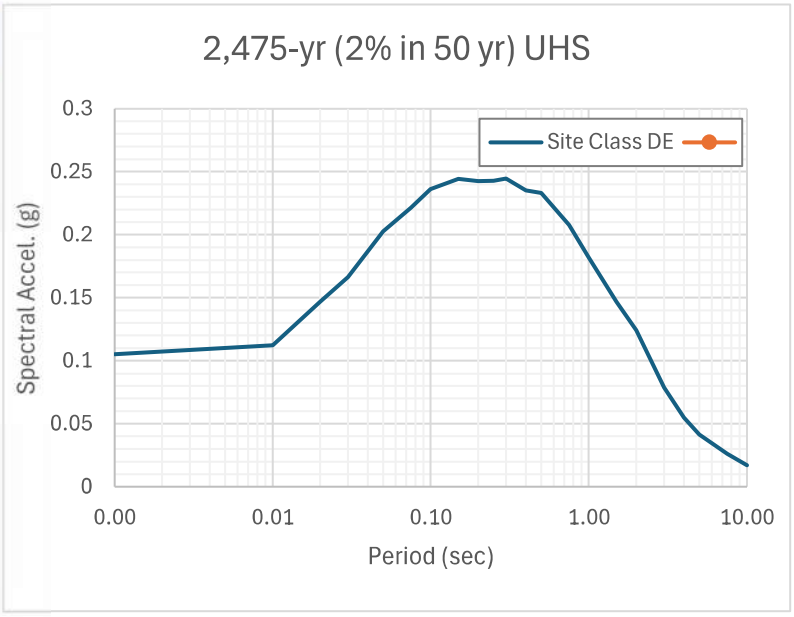
Section L-L' - Pseudostatic, Maximum Storage Pool		
Section L-L' Maximum Storage Pool.gsz		
09/03/2026	1:320	Vertical Exaggeration: 2

1.) Uniform hazard response spectrum:

Using USGS earthquake hazard toolbox:

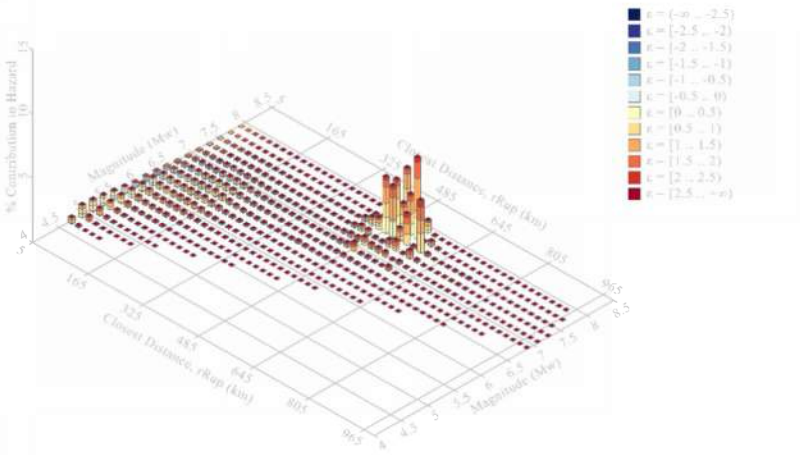
https://staging-earthquake.usgs.gov/nshmp/hazard/static?commonReturnPeriods=2475&latitude=31.4858&longitude=-87.9176&maxDirection=false&model=STATIC_CONUS_2023_R2&returnPeriod=2475&siteClass=DE&truncate=false





2.) Hazard Disaggregation to determine Earthquake Moment Magntiude





Mean (over all sources)	Mode (largest m-r bin)
m : 6.98	m : 7.76

r : 344.23 km
ε_0 : 1 σ

r : 555.41 km
ε_0 : 1.1 σ
Contribution : 6.31 %

.

Source Set Source	Type	r	m	ε0
Grid Sources (reduced) (opt)	Grid			
PointSourceFinite: -87.918, 31.598		12.84	5.47	0.23
PointSourceFinite: -87.918, 31.643		17.27	5.57	0.38
PointSourceFinite: -87.918, 31.688		21.78	5.67	0.48
PointSourceFinite: -87.918, 31.553		8.7	5.39	0.04
PointSourceFinite: -87.918, 31.733		26.33	5.77	0.56
PointSourceFinite: -87.918, 31.778		30.89	5.86	0.61
PointSourceFinite: -87.918, 31.868		40.04	6.03	0.69
PointSourceFinite: -87.918, 32.722		131.94	6.47	0.88
New Madrid - USGS (center, all) : R1-500-yr	FaultCluster			
New Madrid - USGS (center, all) : R1-500-yr		532.56	7.62	1.12
New Madrid - USGS (center, all) : R2-750-yr	FaultCluster			
New Madrid - USGS (center, all) : R2-750-yr		532.56	7.62	1.12
New Madrid - SSCn (AxS-BL : RFT-S : NMN-S) : R2-270-yr	FaultCluster			
New Madrid - SSCn (AxS-BL : RFT-S : NMN-S) : R2-270-yr		533.62	7.66	1.11
New Madrid - SSCn (AxS-BL : RFT-S : NMN-S) : R3-417-yr	FaultCluster			
New Madrid - SSCn (AxS-BL : RFT-S : NMN-S) : R3-417-yr		533.62	7.66	1.11

Ground Motion Parameters

Site Class DE

	(PGA)						
T:	0.00	0.01	0.02	0.03	0.05	0.075	0.1
Sa (g)	0.105222	0.112128	0.146766	0.166401	0.202563	0.221519	0.236085

Ground Motion Parameters

		az	%	
lon	lat			51.59
			0	1.93
87.918°W	31.598°N		0	1.78
87.918°W	31.643°N		0	1.67
87.918°W	31.688°N		0	1.58
87.918°W	31.553°N		0	1.5
87.918°W	31.733°N		0	1.41
87.918°W	31.778°N		0	1.15
87.918°W	31.868°N		0	1.11
87.918°W	32.722°N			5.11
		330.97		5.11
90.614°W	35.419°N			3.41
		330.97		3.41
90.614°W	35.419°N			2.49
		332		2.49
90.503°W	35.425°N			2.05
		332		2.05
90.503°W	35.425°N			

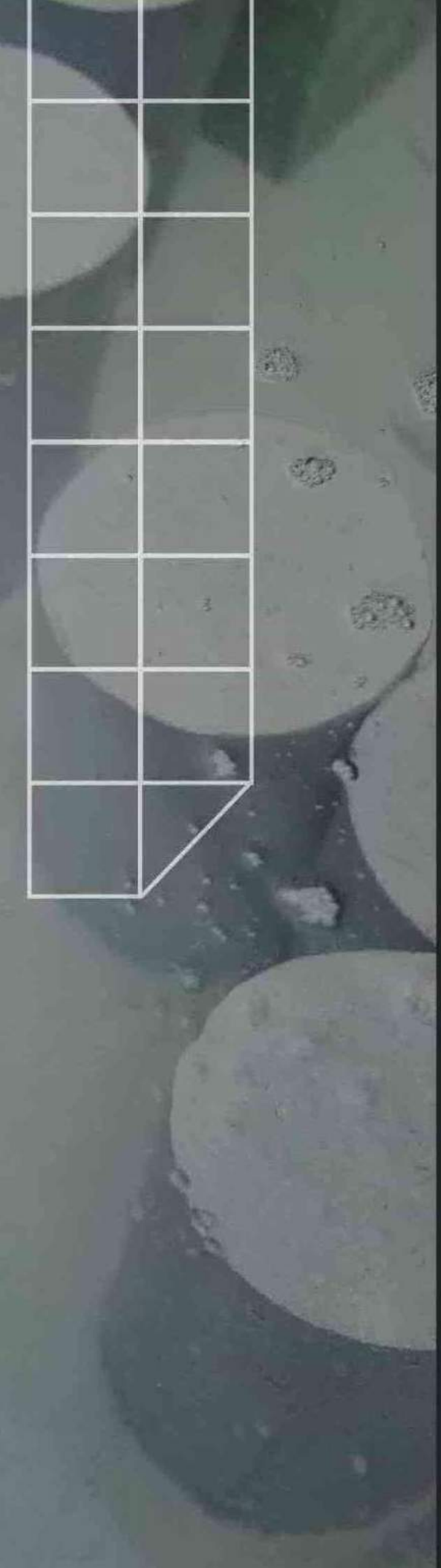
Ground Motion Parameters

0.15	0.2	0.25	0.3	0.4	0.5	0.75	1	1.5	2
0.24427	0.242479	0.24287	0.244561	0.235089	0.233242	0.207682	0.18217	0.146708	0.124123

.

Ground Motion Parameters

3	4	5	7.5	10
0.078815	0.054957	0.04146	0.026033	0.017101



APPENDIX I

Liquefaction Potential Analyses

Project Lowman PP Sec D EL 25
Project No. 16-099
Sheet 1 **of** 4
Date 9 / 1 / 2021
Engineer TCS
Checked by DD

Reviewed October 2026: results are appropriate for current conditions.

The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 115pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface $d_w := 12ft$

sample depth Boring T-1 Depth 15 $z := 15ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 1725 \cdot psf$

static water pressure $u_o := (z - d_w) \cdot \gamma_{water}$ $u_o = 187 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 1538 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value $N_m := 4$

fines content $FC := 72.3$

energy correction $C_E := 1.46$

borehole diameter correction $C_B := 1.00$

rod length correction $C_R := 0.85$

sampler correction $C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3

(Skempton, 1986)

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15

(Skempton, 1986)

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 5$

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v} \right)^{0.5}$

$C_{N\_a} = 1.173$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 5.8$

Project Lowman PP Sec D EL 25
Project No. 16-099
Sheet 2 **of** 4
Date 9 / 1 / 2021
Engineer TCS
Checked by DD

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.209$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.2$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 6$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.209$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.6$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 11.5$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.079 \cdot g$$

earthquake moment magnitude

$$M_w := 7.75$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126 \cdot \sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118 \sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

Project Lowman PP Sec D EL 25
Project No. 16-099
Sheet 3 **of** 4
Date 9 / 1 / 2021
Engineer TCS
Checked by DD

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.972$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.056$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 0.936$

coefficient for overburden correction factor (maximum Cσ=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.079$

overburden correction factor (maximum Kσ=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.025$

estimated static shear stress on horizontal plane $\tau_{ho} := 35psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 32.111 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.468$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

Project Lowman PP Sec D EL 25
Project No. 16-099
Sheet 4 **of** 4
Date 9 / 1 / 2021
Engineer TCS
Checked by DD

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot\left(1 + 2K_o\right)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.197$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0.023$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 0.966$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 2.13$$

A LL=29 and PI=11 classify this material as "clay-like" behavior according to Idriss and Boulanger.

Project Lowman PP Sec G EL 29
Project No. 16-099
Sheet 1 **of** 4
Date 9 / 1 / 2021
Engineer TCS
Reviewed October 2026: results are appropriate for current conditions. **Checked by** DD

The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 115pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface $d_w := 8ft$

sample depth Boring S-6 Depth 14 $z := 14ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 1610 \cdot psf$

static water pressure $u_o := (z - d_w) \cdot \gamma_{water}$ $u_o = 374 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 1236 \cdot psf$

Standard Penetration N-value Correction -

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Standard Penetration Test N-value $N_m := 48$

fines content $FC := 25$

energy correction $C_E := 1.46$

borehole diameter correction $C_B := 1.05$

rod length correction $C_R := 0.85$

sampler correction $C_S := 1$

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 62.5$

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$

$C_{N\_a} = 1.309$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 81.9$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.043$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.08$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 67.6$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.08$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.1$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 72.6$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.079 \cdot g$$

earthquake moment magnitude

$$M_w := 7.75$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126 \cdot \sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118 \sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.975$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.065$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 0.936$

coefficient for overburden correction factor (maximum Cσ=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = -0.486$

overburden correction factor (maximum Kσ=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 0.739$

estimated static shear stress on horizontal plane $\tau_{ho} := 35psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 61.023 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.125$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot\left(1 + 2K_o\right)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -1.065$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0.028$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 6.729$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 2.43 \times 10^{19}$$

This sample is classified as non-plastic.

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The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 115pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface $d_w := 5ft$

sample depth Boring T-2 Depth 11 $z := 11ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 1265 \cdot psf$

static water pressure $u_o := (z - d_w) \cdot \gamma_{water}$ $u_o = 374 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 891 \cdot psf$

Standard Penetration N-value Correction -

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3

(Skempton, 1986)

Standard Penetration Test N-value $N_m := 15$

fines content $FC := 20.1$

energy correction $C_E := 1.46$

borehole diameter correction $C_B := 1.00$

rod length correction $C_R := 0.80$

sampler correction $C_S := 1$

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15

(Skempton, 1986)

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 17.5$

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$

$C_{N\_a} = 1.541$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 27$

overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.388$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.4$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 24.5$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.411$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 4.5$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 29$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.079 \cdot g$$

earthquake moment magnitude

$$M_w := 7.75$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126 \cdot \sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118 \sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.983$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.072$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 0.936$

coefficient for overburden correction factor (maximum Cσ=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.159$

overburden correction factor (maximum Kσ=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.138$

estimated static shear stress on horizontal plane $\tau_{ho} := 35psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 50.785 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.225$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot\left(1 + 2K_o\right)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.587$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0.039$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1.22$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 7.79$$

This sample is classified as non-plastic.

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The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil		$\gamma_{total} := 115pcf$	
unit weight of water		$\gamma_{water} := 62.4pcf$	
water table depth below ground surface		$d_w := 5ft$	
sample depth	Boring S-10 Depth 24	$z := 24ft$	
total vertical stress		$\sigma tot_v := \gamma_{total} \cdot z$	$\sigma tot_v = 2760 \cdot psf$
static water pressure		$u_o := (z - d_w) \cdot \gamma_{water}$	$u_o = 1186 \cdot psf$
effective vertical stress		$\sigma eff_v := \sigma tot_v - u_o$	$\sigma eff_v = 1574 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value		$N_m := 17$	
fines content		$FC := 13$	
energy correction		$C_E := 1.46$	
borehole diameter correction		$C_B := 1.05$	
rod length correction		$C_R := 0.95$	
sampler correction		$C_S := 1$	
SPT N-value for an energy ratio of 60%		$N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$	$N_{60} = 24.8$
overburden correction factor (based on overburden stress only)		$C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$	$C_{N\_a} = 1.159$
normalized SPT N-value (based on overburden stress only)		$N_{1\_60} := C_{N\_a} \cdot N_{60}$	$N_{1\_60} = 28.7$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.114$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.11$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 27.5$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.117$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 2.5$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 30$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.079 \cdot g$$

earthquake moment magnitude

$$M_w := 7.75$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126 \cdot \sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118 \sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.944$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.085$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 0.936$

coefficient for overburden correction factor (maximum Cσ=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.181$

overburden correction factor (maximum Kσ=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.053$

estimated static shear stress on horizontal plane $\tau_{ho} := 35psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 51.638 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.216$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot\left(1 + 2K_o\right)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.617$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0.022$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1.14$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 6.4$$

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Reviewed October 2026: results are appropriate for current conditions.

The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 115pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface $d_w := 7ft$

sample depth Boring T-3 Depth 30 $z := 30ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 3450 \cdot psf$

static water pressure $u_o := (z - d_w) \cdot \gamma_{water}$ $u_o = 1435 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 2015 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value $N_m := 16$

fines content $FC := 10.3$

energy correction $C_E := 1.46$

borehole diameter correction $C_B := 1.00$

rod length correction $C_R := 0.95$

sampler correction $C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 22.2$

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v} \right)^{0.5}$

$C_{N\_a} = 1.025$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 22.7$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.02$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 22.2$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.021$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 1.3$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 23.5$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.079\cdot g$$

earthquake moment magnitude

$$M_w := 7.75$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot\sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118\sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.924$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.081$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 0.936$

coefficient for overburden correction factor (maximum Cσ=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.145$

overburden correction factor (maximum Kσ=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.007$

estimated static shear stress on horizontal plane $\tau_{ho} := 35psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 48.521 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.251$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot\left(1 + 2K_o\right)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.532$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0.017$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1.039$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 3.11$$

This sample is classified as non-plastic.

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Reviewed October 2026: results are appropriate for current conditions. **Checked by** DD

The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 115pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface $d_w := 8ft$

sample depth Boring S-12 Depth 14 $z := 14ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 1610 \cdot psf$

static water pressure $u_o := (z - d_w) \cdot \gamma_{water}$ $u_o = 374 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 1236 \cdot psf$

Standard Penetration N-value Correction -

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Standard Penetration Test N-value $N_m := 36$

fines content $FC := 8.4$

energy correction $C_E := 1.46$

borehole diameter correction $C_B := 1.05$

rod length correction $C_R := 0.85$

sampler correction $C_S := 1$

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 46.9$

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$

$C_{N\_a} = 1.309$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 61.4$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.097$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.1$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 51.6$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784 - 0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.128$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 0.5$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 52.1$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.079 \cdot g$$

earthquake moment magnitude

$$M_w := 7.75$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126 \cdot \sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118 \sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.975$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.065$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 0.936$

coefficient for overburden correction factor (maximum Cσ=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 1.717$

overburden correction factor (maximum Kσ=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.924$

estimated static shear stress on horizontal plane $\tau_{ho} := 35psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 59.128 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.142$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot\left(1 + 2K_o\right)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.912$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0.028$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 2.906$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 2.4\times 10^5$$



THREE NOTCH GROUP

Environment & Infrastructure

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