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Saltstone Disposal Unit No. 1

Updated Settlement Analysis Report

Prepared by
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SALTSTONE VAULT No. 1 GEOTECHNICAL REPORT – UPDATED SETTLEMENT (U)

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DISCLAIMER

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1.0 INTRODUCTION

The purpose of this report is to summarize the results of our investigation in support of SRR's permit application for storage of saltstone in Vault No. 1.

The subsurface conditions in Z-Area were investigated and presented in reports prepared by WSRC and geotechnical engineering consultants as well as geotechnical engineering calculations performed by WSRC. In addition, a comprehensive geotechnical investigation was conducted at the original Vault No. 2 project site and the results relevant to Vault No. 1 are included in this report.

This investigation consists of performing subsurface explorations to collect site-specific data; characterizing subsurface conditions based on site-specific data as well as existing data from the surrounding areas; determining engineering properties required for the design of foundations and underground facilities; evaluations of bearing capacity and liquefaction potential; and estimation of settlement due to static load, liquefaction, and compression of soft zones.

This report is organized into six sections. Section 1 is the introduction, Section 2 describes the subsurface exploration, Section 3 describes the subsurface conditions, Section 4 discusses the engineering properties, Section 5 discusses engineering evaluations, and Section 6 provides the summary and conclusions.

1.1 Background Information

The Saltstone Production Facility (SPF) immobilizes salt solution by blending it with a dry mixture of cement slag, and flyash to form a grout. The grout is pumped to the Saltstone Disposal Facility (SDF), which are large storage vaults, where it is allowed to harden into a concrete-like solid waste called saltstone. Vault No. 1 was constructed in the late 1980's to store the saltstone. The Vault No. 1 was designed as a rectangular concrete vault. The vault walls were constructed of cast-in-place concrete erected on a cast-in-place concrete base slab (Ref. 1). The concrete vault was constructed in a trench formed by excavating below the original ground surface. After filling, the vault will be backfilled with soil and encased in a High Density Polyethylene (HDPE) and Geosynthetic Clay Liner (GCL) system.

The overall dimension is 100 feet wide by 600 feet long by 27 feet deep (Ref. 1). It is divided into two units, each 100 by 300 feet, with a 3-inch separation gap between the units. Each unit is further divided into three cells of 100 by 100 feet. The top of concrete wall is 27 feet above the bottom of slab. The bottom of vault is at El. 281.50 and is approximately 31.5 feet above the maximum recorded groundwater table at El. 250.00.

Three cells have been filled with grout, clean capped, and a permanent roof sealed with ethylene propylene diene monomer (EPDM) Elastomeric Sheet Roofing (fully adhered) installed. The roof is designed for a live load of 25 psf, and the platforms are designed for a live load of 100 psf. The remaining three cells are empty and available for disposal of failed equipment. To minimize leaching and migration of chemicals and radionuclides from the saltstone grout, blast furnace slag was used in the concrete mix for construction of the vaults. The concrete used in vault construction has a minimum design compressive strength of 4,000 psi. The walls and slab of the vaults are of reinforced concrete construction and are designed for gravity loads plus hydrostatic pressure due to the 8-foot-high saltstone grout. Foundation

design is based on a maximum allowable soil bearing pressure of 6,000 psf. The interior and perimeter walls are 18 inches thick, and the bottom slab is 24 inches thick (Ref. 1).

2.0 SUBSURFACE EXPLORATION

Subsurface exploration was initially conducted at the Saltstone Vault No. 1 project site using geotechnical boreholes and laboratory testing. Piezocone Penetrometer Test (CPTu) soundings were conducted in August 2010 to supplement the existing data and upgrade it to current technology. Data from the previously conducted subsurface explorations in the surrounding areas were also evaluated and included in the evaluation as appropriate. Groundwater elevation was obtained from (Ref. 4).

Table 1 summarizes the locations of the subsurface explorations performed at the project site, including the CPTu's, geotechnical boreholes, and test pits. Figure 1 shows the location of the project site as well as the locations of the subsurface explorations.

A brief summary of the evaluation and results of the subsurface exploration is provided in the following sections.

2.1 Piezocone Penetration Test Soundings

Ten CPTu soundings, four of which were seismic soundings (SCPTu), were performed in August 2010 to supplement and modernize the previous investigations; the CPTu soundings were performed in general accordance with ASTM D5778. The coordinates, ground elevations, and depths of these CPTu soundings are provided in Table 1. CPTu logs including tip resistances, sleeve resistances, friction ratios, and pore pressures are provided in Appendix B, along with dissipation test results. SCPTu shear wave velocities are also included.

2.2 Geotechnical Boreholes

In two previous investigations, seven geotechnical boreholes were drilled in the area of Vault 1; three were drilled along the centerline of Vault No. 1 in 1986 (Ref. 2) and four were drilled between the centerline of Vault No. 1 and the original planned location of Vault No. 2 in 1992 (Ref. 3). The Standard Penetration Test (SPT) was conducted in each of the boreholes and undisturbed (UD) soil samples were obtained from the other three boreholes. The coordinates, surface elevations, and depths of these boreholes are shown in Table 1. Geotechnical borehole logs including the SPT N-values, sample locations, field classifications, and soil descriptions are provided in References 2 and 3.

2.2.1 Standard Penetration Test

SPT testing was performed in all the boreholes referred to in this report: Z-215, Z-216, Z-217, ZB-1, ZB-3, ZB-5, and ZB-7. Boreholes Z-215, Z-216, and Z-217 (1986) were sampled at five-foot intervals the entire depth of the boreholes, while ZB-1, ZB-3, ZB-5, and ZB-7 (1992) were sampled at 2.5-foot intervals in the upper 10 feet and at five-foot centers for the remaining depth of the boreholes. The termination depths of the boreholes are shown in Table 1; the depths ranged between 142 feet and 168 feet.

Tests were performed in general accordance with ASTM D1586. SPT N-values in boreholes Z-215, Z-216, and Z-217 were determined by adding the number of blows required to drive the split-spoon sampler the last 12 inches of the standard 18-inch drive. The striking energy was

not measured on the SPT equipment (hammer and the rods). SPT blow counts obtained from the field are preferably converted to appropriate energy levels for evaluations. A 24-inch split-spoon was used in boreholes ZB-1, ZB-3, ZB-5, and ZB-7; SPT N-values in these boreholes were determined by adding the number of blows required to drive the split-spoon sampler the middle 12 inches.

2.2.2 Soil Samples

No soil samples were obtained in this investigation. In both previous investigations, undisturbed and disturbed soil samples were obtained for laboratory testing. Undisturbed soil samples were obtained (in accordance with ASTM D1587) from borehole Z-216U for laboratory testing using direct push Shelby tubes. Disturbed soil samples were obtained using split-spoon samplers from boreholes Z-215, Z-216, Z-217, ZB-1, ZB-3, ZB-5, and ZB-7. Disturbed soil samples are those in which the soil fabric has been rearranged sufficiently to be no longer representative of the in-situ soil fabric. Undisturbed soil samples are those in which rearrangement of the soil fabric is not extensive enough to prevent the tests from being interpretive of the in-situ soil properties.

2.3 Groundwater Monitoring Wells

No groundwater monitoring wells were specifically installed for this investigation.

2.4 Laboratory Tests

Laboratory tests were performed in 1985 on undisturbed soil samples obtained from borehole Z-216U and disturbed samples collected from boreholes Z-215, Z-216, Z-217, ZB-1, ZB-3, ZB-5, and ZB-7. Laboratory tests included index tests, strength tests, consolidation tests, and Modified Proctor moisture-density tests. The results of the laboratory tests are provided in References 2 and 3.

3.0 SUBSURFACE CONDITIONS

3.1 Engineering Stratigraphy

The subsurface conditions were determined based upon the previous and current investigations, as well as knowledge of the general and specific subsurface conditions in the General Separations Area (GSA) of the SRS. The GSA includes F-Area, H-Area, S-Area; as well as Z-Area, where the Saltstone Vault No. 1 is located.

Subsurface conditions at the project site can be described using two systems of stratigraphic nomenclature. One system uses the nomenclature developed by Mueser Rutledge Consulting Engineers (MRCE), and the other system uses the nomenclature developed at the SRS for the many investigations in the GSA. Table 2 shows the stratigraphy using both systems.

The subsurface conditions at the project site are similar to the general subsurface conditions in the GSA. Figure 1 shows the exploratory map including the locations of the CPTu's and borings. CPTu soundings are provided in Appendix B. Geotechnical boring logs are provided in References 2 and 3.

Appendix B defines engineering layers based on CPTu soundings. Table 3 provides the elevation of each engineering layer at each CPTu location. Table 4 provides the statistics of SPT N-value for each engineering layer. Subsurface conditions of each engineering layer are also described in the following sections in descending order from the ground surface. Table 5 provides the statistics of CPTu tip resistance, friction ratio, pore pressure, and SCPTu wave velocities for each engineering layer.

3.1.1 Upland Formation (S1)

The Upland formation (S1) generally consists of red-brown and gray medium dense to dense medium to fine sand of the upper S1 layer, with some clay and occasional interbedding of fine sandy clay layers. It generally classifies as a SC in the Unified Soil Classification System (USCS). Liquefaction was not considered since the layer is above the water table and therefore not liquefiable.

3.1.2 Tobacco Road/Dry Branch Formation (Lower S1, S2, S3a, S3b and C2)

Within the GSA of the SRS, it is difficult to ascertain the contact between the Tobacco Road (TR) and Dry Branch (DB) formations without visually observing the individual samples. However, for liquefaction and settlement evaluation, being able to distinguish between the TR and DB is relatively unimportant compared to determining the USCS soil type and the associated engineering properties. Descriptions of the TR and DB follow.

The TR formation (S2 layer) consists of medium dense to very dense yellow-brown to red fine to medium sand, with a trace of clay or silt. It generally classifies as a SM or SP-SM in the USCS.

The DB formation consists of the Tan Clay (TC) unit (C2 layer) and layers S3a and S3b. The C2 consists of medium dense yellow-brown and light green clayey fine sand interlayered with stiff yellow-brown silty clay. The material C2 classifies as a borderline CH soil in the USCS.

The DB formation also consists of sands associated with the lower S1, S3a, and S3b layers. The sands are intermittent and in some cases isolated pockets. However, they generally extend to the top of the Santee/Tinker formation.

The lower S1, S3a, and S3b layers consist of medium dense to dense light brown to gray fine to medium sand with some clay and sandy clay layers and pockets. The material generally classifies as a SC in the USCS. The S3b layer consists of dense to very dense light brown and yellow-brown fine to medium sand with a trace of clay and silt. The material generally classifies as a SP, SP-SM, or SP-SC in the USCS.

3.1.3 Santee/Tinker Formation (S4)

The Santee/Tinker (ST) formation (layer S4) extends from the bottom of the TR/DB formation to the top of the Warley Hill formation (layer M1). The material consists of dense to very dense light gray-green calcareous fine to medium sand with some clay and silt and occasional limestone and shell fragments. The material generally classifies as a SC or SM in the USCS. The formation is characterized by alternating low and high penetration resistances indicating the presence of limestone layers within the calcareous sands.

3.1.4 Warley Hill Formation (M1)

The Warley Hill (WH) formation (M1) consists of a hard dark gray-green clayey silt to a very dense dark gray fine to medium sand with some clay or silt. This soil generally has a USCS classification of MH.

3.2 Groundwater

The current groundwater elevation at the project site is 230 feet, MSL with estimated seasonal fluctuation of + 5 feet (Ref. 4). For this calculation a conservative groundwater elevation of 240 feet, MSL is used. Only saturated soils are analyzed for potential liquefaction.

3.3 Soft Zones

Across the SRS, the soil from approximately 100 to 250 feet below the ground surface is a marine deposit laid down during the Middle Eocene epoch, about 35 to 50 million years ago. At the location of Saltstone Vault No. 1, these sediments occur within the Lower Dry Branch and Santee/Tinker Formations. Often found within these sediments are weak zones interspersed in stronger matrix materials. These weak zones, which vary in thickness and lateral extent, have been termed "soft zones". These soft zones typically occur in the carbonate-bearing sediments of the Santee Limestone, the Utley Limestone, and the Griffins Landing Member of the lower Dry Branch Formation.

The prevailing assumption about the origin of soft zones involves dissolution of carbonate-rich, clastic sediments, resulting in vugular porosity (open pore space). When drilling into these zones, the drill rod meets little resistance and drops. Occasional rod drops have been described in numerous drilling reports for monitoring wells and geotechnical boreholes located in the central part of the SRS. Early subsurface investigations performed by the United States Army Corps of Engineers frequently described these zones as soft zones, or voids, and numerous subsequent subsurface investigations have described these same conditions at the SRS. However, much of the time, recovery of soil in the sampler precludes the zone from being characterized as a void.

For this project site, soft zones are indicated from SPT N-values less than 5 or CPT tip resistances (q_t) less than 15 tons per square foot (tsf) over a continuous interval of two feet or greater. Of the boreholes and deep CPTu's performed at the project site, a vast majority of the soft soils found are thin (i.e., less than 2 feet thick), without significant effect on the surface settlement. However, the soft zone at elevation 194.7 feet, MSL, encountered in SCPTu Z-V1-C09, along the west side of the vault, is approximately 2.8 feet thick. The effect of this soft zone on the surface settlement is discussed in Appendix D.

4.0 ENGINEERING PROPERTIES

The engineering properties of the subsurface materials encountered were assessed based on the results of the field exploration, laboratory testing, theoretical relations, and empirical formulas.

As presented in Section 2, field exploration included CPTu's and SPT's. CPTu's were conducted to acquire tip resistance, sleeve friction, pore pressure, and shear and compression wave velocities. Standard Penetration Test's were previously conducted (Ref. 2 and Ref. 3) to

acquire blow counts and soil samples for classification purposes. Undisturbed soil samples were obtained from those geotechnical boreholes for strength and consolidation tests. Laboratory tests were performed on selected soil samples obtained from undisturbed sampling, from SPT sampling, and from test pit sampling. Laboratory tests included sieve analysis, determination of Atterberg Limits, unit weight, strength tests, and consolidation tests. In addition to the new and previous test data from the project site, previous SRS studies were utilized, where appropriate.

Engineering properties are summarized in Reference 2 and Reference 3.

5.0 ENGINEERING EVALUATIONS

Engineering evaluations were performed to determine liquefaction potential, and subsequent soft zone settlement potential. Appendix A provides the evaluation of liquefaction potential using SPT data, and Appendix B provides the evaluation of liquefaction potential using CPT data. The potential settlement due to static load was evaluated prior to construction; this evaluation is contained in Reference 2. The potential settlement due to liquefaction and partial liquefaction is evaluated in Appendix B. The potential settlement due to the compression of soft zones liquefaction is evaluated in Appendix D. Evaluation methodology and results are also summarized in the following sections.

5.1 Static Settlement and Heave

Static settlement and heave occur when the overburden changes. Heave occurs as excavation progresses due to the reduction of the soil overburden pressure. Static recompression occurs during construction, vault filling, backfilling, and during the installation of the closure cap as the overburden pressure is restored and the heave is reversed. Static settlement occurs after recompression when added weight exceeds the overburden pressure removed during excavation. Static settlements were computed during the initial investigation for Vault No. 1 and are contained in Reference 2.

5.2 Dynamic Settlement

Dynamic settlement includes the settlement due to liquefaction or partial liquefaction and the settlement due to the compression of soft zones. To estimate settlement due to liquefaction or partial liquefaction, the potential for liquefaction needs to be evaluated. To estimate settlement due to the compression of the soft zones, the compression of soft zone needs to be evaluated as well as propagation of compression to ground surface.

The potential settlement due to liquefaction and partial liquefaction is evaluated in Appendix B.

5.2.1 Liquefaction Potential

The liquefaction potential was evaluated using both a SPT-based procedure (Appendix A) and a modified version of the simplified CPT-based procedure (Appendix C). The modification of the CPT-based procedure was the use of SRS site-specific CRR data (as described in Appendix C, pages 5 and 6) rather than generic CRR data. This procedure computes the factor of safety:

$FS = MSF\ K_\sigma\ K_\alpha\ CRR_{7.5}/CSR$

where MSF is the magnitude scaling factor

K_σ is a correction for overburden pressure

K_α is a correction for static shear stress

$CRR_{7.5}$ is the cyclic resistance ratio for a magnitude 7.5 earthquake

CSR is the calculated cyclic stress ratio generated by the earthquake

The minimum acceptable safety factor for liquefaction analyses at the SRS has been established previously at 1.15. Once the safety factor is determined, dynamic settlements are determined based on the computed factor of safety and SRS site specific strain curves (see Appendix C, Figure 4).

The liquefaction analyses were performed using the Design Basis Earthquake (DBE) having peak ground acceleration (PGA) of 0.21g.

Liquefaction analysis using both the SPT and the CPTu tip stress suggests that the soils at the project site are not susceptible to significant liquefaction for the 2,500 year earthquake having a PGA of 0.21g. Appendix B of this report's Appendix C shows plots of the factor of safety versus the depth for the CPTu locations.

5.2.2 Settlement due to Liquefaction and Partial Liquefaction

Settlements were calculated using the SRS volumetric strain relationship for each of the CPTu soundings and for all magnitudes in the USGS seismic hazard for SRS. Appendix C, Figure 4 shows the SRS relationship between the volumetric strain and the factor of safety. Settlement versus depth for the CPTu locations was computed for several different magnitudes.

Settlement estimates due to liquefaction and partial liquefaction ranged to 1.39 inches depending on CPTu location and earthquake magnitude. The maximum liquefaction-induced settlement was conservatively estimated to be 1.5 inches. Assuming that the differential settlement is the difference between the minimum and maximum settlement for a given magnitude, the maximum differential settlement that can be expected is approximately 1 inch.

The magnitude weighted average using the USGS PGA hazard disaggregation is less than an inch for the 2,500-year earthquake.

5.2.3 Compression of Soft Zones

It is assumed that the soft zone is under-consolidated and the full overburden pressure will eventually be acting on the soft zone after a DBE event. The compression of the soft zone s_S at depth is estimated as:

$$s_S = H\,[C_c/(1 + e_o)]\,\log\,[(P_o + \Delta P)/P_o]$$

Where s_S is the compression of the soft zone

H is the thickness of the soft zone.

C_c is the compression index

e_0 is the initial void ratio

When the arch above the soft zone is weakened, the $P_o + \Delta P$ term is equal to the overburden pressure and the P_o term in the denominator is the soft zone preconsolidation pressure. In this instance the equation becomes:

$$s_S = H\,[C_c/(1 + e_o)]\,\log(1/OCR)$$

where OCR is the over-consolidation ratio of the soft zone.

Using the project site-specific soil properties, the compression of the soft zone is computed to be approximately 1 inch.

5.2.4 Settlement due to the Compression of Soft Zone

It is conservatively assumed that the compression of the soft zone will be propagated upward to the ground surface. Calculation was performed to determine the settlement profile at the ground surface including the settlement, slope, and the curvature of the slope (see Appendix D). A vertical slice of subsurface with unit thickness perpendicular to the longitudinal direction of the soft zone was considered. Ground settlement was computed considering the settlement profile resembles the shape of an error or normal probability curve. For settlement due to compression of a soft zone with short width, the surface settlement s(x) at any point x is:

$$s(x) = R_{S/L}\ s_S\ W_{SZ}\ /W\ \mathrm{Exp}[-x^2/(2i^2)]$$

where $R_{S/L}$ is the ratio of the volume of the settlement to the volume lost at-depth

s_S is the compression of the soft zone computed in the previous section

W_{SZ} is the width of the soft zone

$i = W/(2\pi)^{1/2}$ is the distance from center of the probability curve to the point of inflection

W is the half width of the normal probability curve and estimated as

$$W = z \tan\beta + W_{SZ}/2$$

where z is the soft zone depth

$\beta = \tan^{-1}\{(W - W_{SZ}/2)/Z\}$ based on soil type

Surface settlement due to a wide soft zone was computed by superimposing the surface settlement profile computed for narrow soft zone many times to simulate the desired width. For the computation at the saltstone project site, a series of surface settlement profiles for 5-foot width soft zones were superimposed to simulate soft zones with sufficient large range of widths, such that the results can be extrapolated to all possible widths between 0 to infinite.

The results indicate that the maximum settlement at the ground surface due to the compression of the soft zone is ¼ inch, the maximum differential settlement is ¼ inch, the maximum slope is 0.0003 feet/feet, and the maximum curvature of the slope of 0.00002 ft/ft per foot. Appendix D provides the detailed calculation.

6.0 SUMMARY AND CONCLUSIONS

Extensive geotechnical investigation was performed for the Saltstone Vault No. 1 project in 1986, but did not include a liquefaction analysis. The purpose of this work was to evaluate liquefaction potential and its effects at Saltstone Vault No. 1. Additional geotechnical exploration was conducted around Vault 1 using CPTu soundings. The CPTu data were incorporated into

the previously obtained data from geotechnical boreholes and laboratory tests. Subsurface conditions were characterized and soil properties were determined using site-specific data as well as existing data from the surrounding areas. Liquefaction potential was evaluated using the SPT-based procedure on the data from the investigations in 1986 and 1992, as well as the CPTu-based procedure that is the current state of practice at SRS. Settlement due to liquefaction and compression of soft zones was estimated.

There are two types of settlement:

1. Static settlement due to static load.
2. Dynamic settlement including the settlement due to liquefaction or partial liquefaction and the settlement due to the compression of soft zones.

Static settlement was estimated in the 1986 geotechnical evaluation that was used for construction of Vault No. 1. This report only considered settlement due to liquefaction and due to compression of soft zones.

The dynamic settlement will only occur after the design basis earthquake, which has a low probability of occurrence. The project site is not susceptible to significant liquefaction for the 2,500 year earthquake having a PGA of 0.21g. The dynamic settlement due to liquefaction and partial liquefaction was conservatively estimated to not exceed 1.5 inches. The maximum differential settlement that can be expected is approximately 1 inch.

After the design basis earthquake, compression of the soft zones may occur. This compression will propagate to the ground surface and cause the ground to settle. The estimated maximum settlement and the estimated maximum differential settlement are ¼ inch, respectively, the estimated maximum slope is 0.00075, and the estimated maximum curvature of the slope is 0.000035 per foot.

REFERENCES

(1) G-SD-Z-00002 System Design Description (U) for Saltstone Structures and Vaults, rev. 5, April 2010.

(2) Z-SDF-2, Report: "Saltstone Disposal Investigation," Mueser Rutledge Consulting Engineers (MRCE), 1986.

(3) Z-SDF-9, Report: "Z-Area Vault No. 2 Geotechnical Investigations Report," C-ESR-Z-00001, BSRI, 1992

(4) WSRC-2003-00250, Rev. 0, "An Updated Regional Water Table of the Savannah River Site and Related Coverages," December 2003

(5) Z-SDF-3, Report: "Preliminary Foundation Recommendations, Saltstone Disposal – Z-area," Mueser Rutledge Consulting Engineers (MRCE), 1986.

TABLES

Table 1: Subsurface Exploration Locations

Type of Exploration	CPTu or SCPTu I.D.	Date	SRS North (feet)	SRS East (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
CPTu	Z-V1-C-01	17-Aug-2010	76,084	66,243	284.1	152.1
CPTu	Z-V1-C-02	24-Aug-2010	76,005	66,333	285.7	126.7
SCPTu	Z-V1-C-03	24-Aug-2010	75,895	66,369	285.7	156.7
CPTu	Z-V1-C-04	24-Aug-2010	75,750	66,417	285.7	156.5
SCPTu	Z-V1-C-05	23-Aug-2010	75,620	66,460	285.4	110.2
CPTu	Z-V1-C-06	23-Aug-2010	75,488	66,442	283.7	123.3
CPTu	Z-V1-C-07	20-Aug-2010	75,575	66,346	283.5	109.9
SCPTu	Z-V1-C-08	23-Aug-2010	75,689	66,309	283.5	119.2
CPTu	Z-V1-C-09	19-Aug-2010	75,833	66,261	283.4	134.7
SCPTu	Z-V1-C-10	19-Aug-2010	75,961	66,218	283.5	122.9
SPT	ZB-1	26-Mar-1992	77,422	66,957	283	142
SPT	ZB-3	24-Mar-1992	77,271	67,018	285	147
SPT	ZB-5	19-Mar-1992	77,151	66,979	282	142
SPT	ZB-7	30-Mar-1992	77,430	66,953	285	142
SPT	Z-215	04-Apr-1986	75,987	67,020	294.7	161.5
SPT	Z-216	01-Apr-1986	77,272	67,024	294.5	146.5
SPT	Z-217	18-Apr-1986	77,353	66,983	295.5	167.5

Table 2: Stratigraphy at the Project Site

General Description	USCS	MRCE Strata	SRS Strata
Medium dense to dense red-brown clayey fine sand	SC	Upper S1	Upland Formation
Medium dense to very dense fine to medium sand, some silt	SM, SP-SM	S2	Tobacco Road Formation
Medium dense clayey fine sand interlayered with stiff silty clay (C2)	CH	C2	Dry Branch Formation
Medium dense to dense medium sand with some clay and sandy clay layers (S1, S3a & S3b)	SC	Lower S1, S3a, S3b	Dry Branch Formation
Dense to very dense calcareous fine to medium sand with some clay and silt	SC, SM	S4	Santee/Tinker Formation
Hard clayey silt to very dense fine to medium sand	MH	M1	Warley Hill Formation

Table 3: Engineering Layers at Test Locations

Exploratory I.D.	Ground Elevation (feet MSL)	Contact between Layers S1/2 and C2 (ft, MSL)	Contact between Layers C2 and S3 (ft, MSL)	Contact between Layers S3 and S4 (ft, MSL)	Contact between Layers S4 and M1 (ft, MSL)
Z-V1-C01	284.1	210	201	194	142
Z-V1-C02	285.7	217	205	192	-
Z-V1-C03	285.7	214	-	195	145
Z-V1-C04	285.7	220	208	194	138
Z-V1-C05	285.4	222	213	197	-
Z-V1-C06	283.7	216	202	189	-
Z-V1-C07	283.5	221	210	204	-
Z-V1-C08	283.5	219	206	200	-
Z-V1-C09	283.4	215	194	190	-
Z-V1-C10	283.5	216	198	191	-
ZB-1	283	214	205	176	141
ZB-3	285	220	197	181	138
ZB-5	282	218	195	178	143
ZB-7	285	216	210	180	145
Z-215	294.7	-	217	172	141
Z-216	294.5	217	207	167	142
Z-217	295.5	223	217	182	137

Table 4: SPT Data

Description	S1	S2	C2	S3	S4
Average	25	25	6	23	N/A
Minimum	17	8	4	5	Weight of rod
Maximum	31	50	8	59	50 in 2 inches
Standard Deviation	6.0	8.9	1.3	14.7	N/A
No. of Data Points	6	35	11	15	16
Average Corrected N_{60}	31	31	8	30	N/A

Table 5: CPTu and SCPTu Data

Layer	Description	Sleeve Friction (tsf)	Tip Resis-tance (tsf)	Friction Ratio (%)	Pore Pressure (tsf)	P Wave (fps)	S Wave (fps)	
S1	Average	2.37	126.7	2.68	0.87	-	-	
	Minimum	0.01	0.97	0.01	-0.64	-	-	
	Maximum	7.01	425.11	117.63	22.71	-	-	
	Standard Deviation	0.43	30.55	3.38	2.11	-	-	
	No. of Data Points	1036	1036	1036	1036	-	-	
S2	Average	1.58	146.65	1.57	0.18	-	-	
	Minimum	0.01	1.33	0.06	-0.83	-	-	
	Maximum	5.91	478.16	19.16	10.5	-	-	
	Standard Deviation	0.11	13.92	0.14	0.35	-	-	
	No. of Data Points	5,016	5,016	5,016	5,016	-	-	
C2	Average	0.83	41.1	2.04	9.74	-	-	
	Minimum	0.01	5.50	0.18	-0.59	-	-	
	Maximum	5.35	303.32	5.28	35.72	-	-	
	Standard Deviation	0.39	18.93	0.18	2.39	-	-	
	No. of Data Points	1,221	1,221	1,221	1,221	-	-	
S3	Average	1.44	177.81	1.13	1.75	-	-	
	Minimum	0.20	19.27	0.24	-0.42	-	-	
	Maximum	5.93	450.19	6.52	17.46	-	-	
	Standard Deviation	0.35	28.48	0.44	1.90	-	-	
	No. of Data Points	757	757	757	757	-	-	
S4	Average	1.80	140.87	1.78	12.15	-	-	
	Minimum	0.0	3.86	0.0	-0.41	-	-	
	Maximum	18.83	692.41	15.84	72.14	-	-	
	Standard Deviation	0.48	19.59	0.17	2.86	-	-	
	No. of Data Points	3,011	3,011	3,011	3,011	-	-	
All	Average	1.6	127	1.8	3.5	-	-	
	No. of Data Points	11,041	11,041	11,041	11,041	-	-	

FIGURES

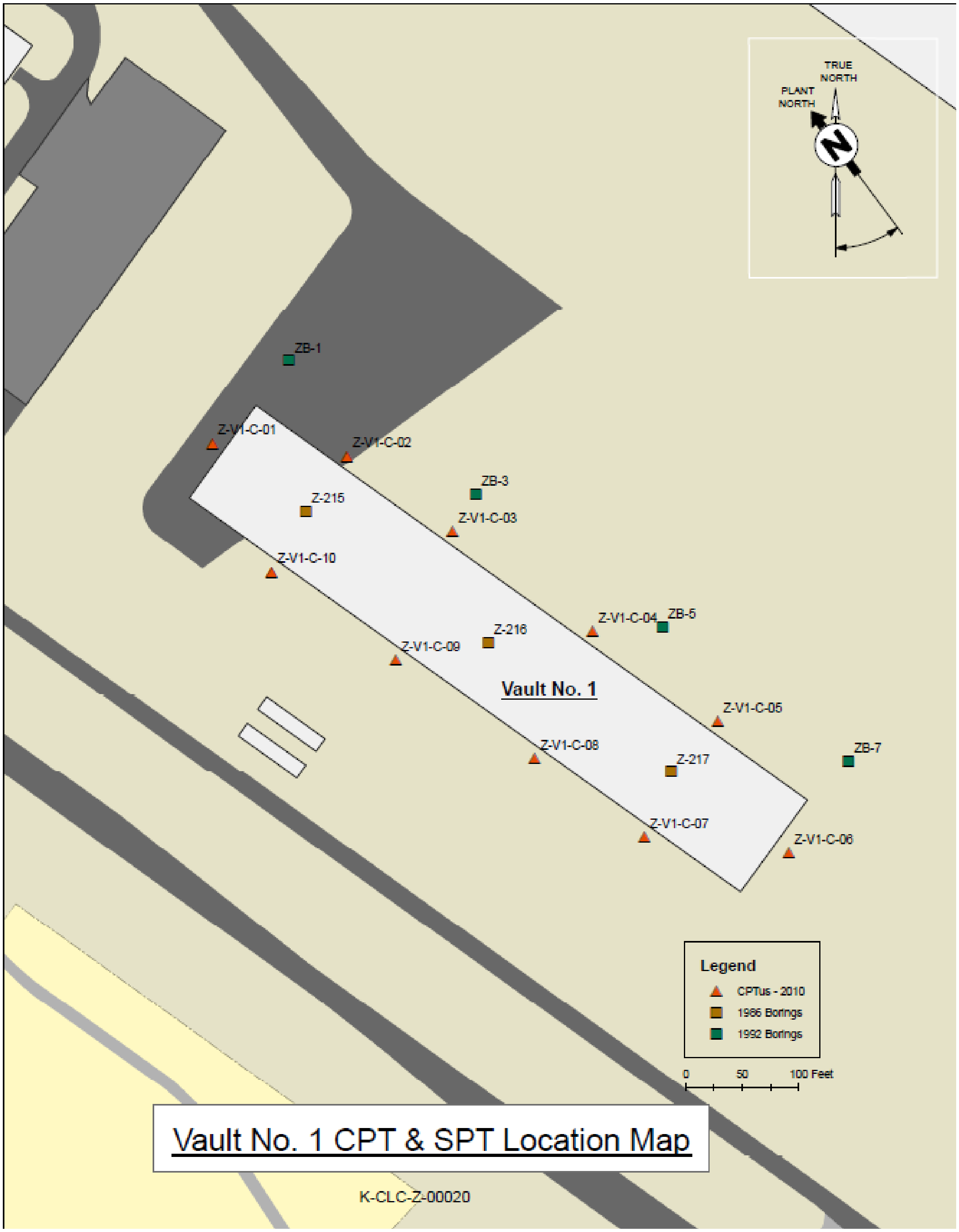

Figure 1: Geotechnical exploratory map

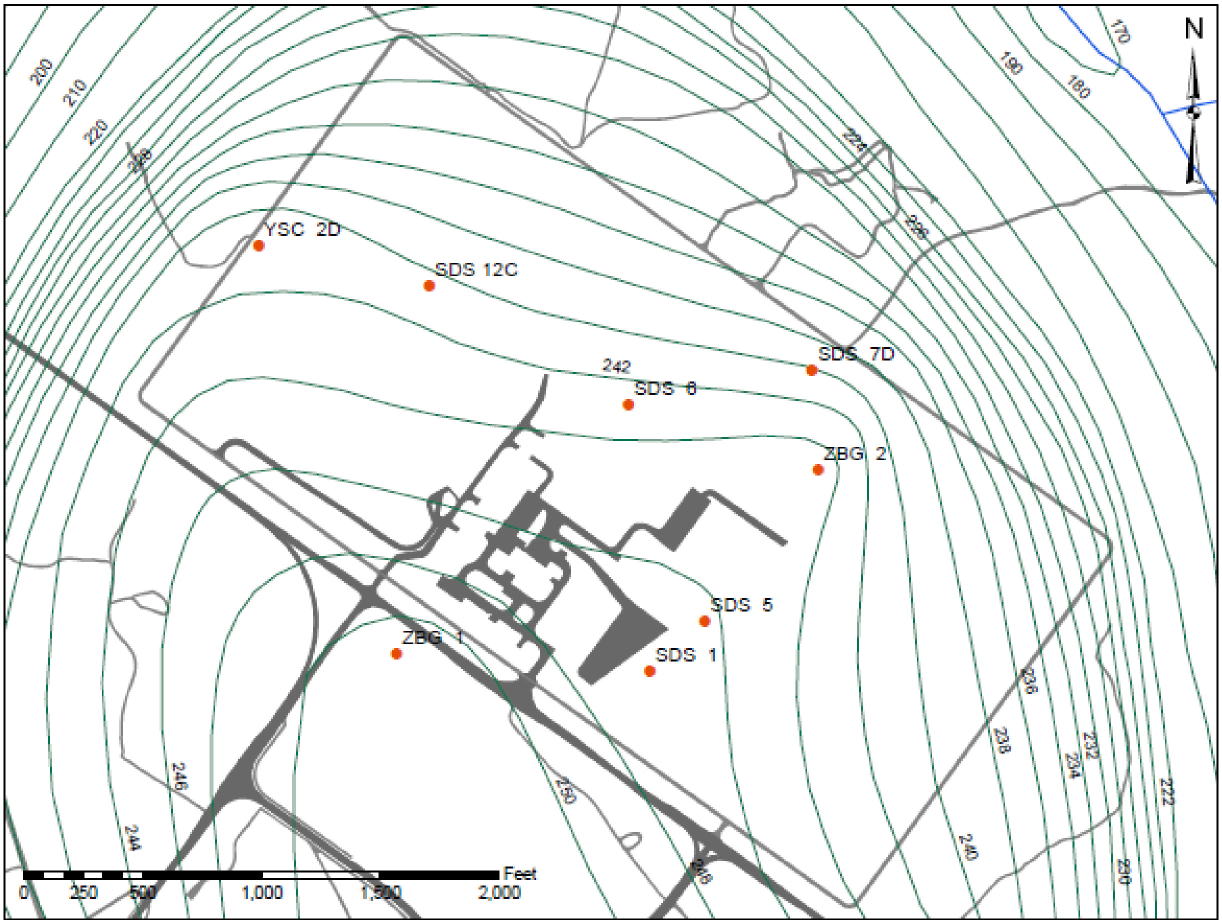

Figure 2 - Probable Maximum Water Table Elevations across the Saltstone Disposal Facility – Ref.4

APPENDICES

Appendix A

K-CLC-Z-00018, Updated SPT Liquefaction Analysis for Saltstone Vault No. 1, Rev. 0, July 7, 2010

(18 pages)

Calculation Cover Sheet

Project	Calculation No.	Project Number
Disposal Unit 1	K-CLC-Z-00018	

Title	Functional Classification / Discipline	
Updated SPT Liquefaction Analysis for Saltstone Disposal Unit 1	Functional Classification: PS Discipline: Geotechnical	Sheet 1 of 18

Calc Level	Type 1 Calc Status
☒ Type 1 ☐ Type 2	☐ Preliminary ☒ Confirmed

Computer Program No.	Version / Release No.
☒ N/A	N/A

Purpose and Objective	DC/RO
The purpose of this calculation is to evaluate the liquefaction potential for Saltstone Disposal Unit 1 using the most recent SPT-based methodology.	UNCLASSIFIED DOES NOT CONTAIN UNCLASSIFIED CONTROLLED NUCLEAR INFORMATION ADC & Reviewing Official A.A. Siddall (ACT) (Name) Date: 7/07/2010

Summary of Conclusion

See the Results and Conclusions Section.

Revision

Rev. No.	Revision Description
0	Original

Sign Off

Rev. No.	Originator (Print) Sign / Date	Verification / Checking Method	Verifier / Checker (Print) Sign / Date	Manager (Print) Sign / Date
0	Bruce Nothdurft, PE 7/7/10	Document Review	Rucker J. Williams 7-7-10	William T. Li 7/7/10

Additional Reviewer (Print)	Signature	Date

Design Authority (Print)	Signature	Date

Release to Outside Agency (Print)	Signature	Date

Security Classification of the Calculation

Unclassified

TABLE OF CONTENTS

1. Introduction

The purpose of this calculation is to reevaluate the liquefaction potential for Saltstone Disposal Unit 1 using the most recent SPT-based methodology. See Figure 1 for the location of Disposal Unit 1.

2. Input

2.1 Site Configuration and Soil Properties

The surface elevation along Vault 1 presently ranges from approximately 265 ft to 270 ft, msl. At the time of the original geotechnical investigation in 1986, the surface elevation along Vault 1 ranged from 294.5 ft to 295.5 ft, msl. Several deep Standard Penetration Tests (SPTs) were performed in 1986 to investigate the subsurface conditions for the proposed vaults in Z-Area. Three of these borings (Z-215, Z-216, and Z-217) lay along the present centerline of Vault 1; four additional borings (ZB-1, ZB-3, ZB-5, and ZB-7) lay along a parallel line about 50ft north of the centerline borings. These borings are shown in Figure 1. The four ZB-borings were drilled in 1992. Note that the boring Z-315 shown at the edge of Vault 1 was a shallow (26-ft) boring used to detect perched groundwater and is not otherwise significant. Groundwater elevation in the borings was about 235 ft, msl at the time of the 1986 investigation; more currently, groundwater elevation ranges from 215 to 220 ft, msl (Ref. 10). Estimated seasonal fluctuation is about 5 feet. Since only saturated soils are susceptible to liquefaction, a conservative water level of 240 ft, msl was used for the computation of liquefaction potential. See Figure 1 for the layout of Vault 1.

Stratigraphic interpretations of the soil profile at Vault 1 are summarized in Table 1 (Ref. 1, 5).

Table 1: Stratigraphic Interpretations

ID	Test Type	SRS Northing	SRS Easting	Elevation Top Pick (ft, msl)			
				Elevation ft, msl	C2 Layer	S3 Layer	S4 Layer
Z-215	SPT	75,987	66,275	294.7	-	217	172
Z-216	SPT	75,797	66,337	294.5	217	207	167
Z-217	SPT	75,608	66,400	295.5	223	217	182
ZB-1	SPT	76,105	66,342	283	214	205	176
ZB-3	SPT	75,911	66,406	285	221	197	181
ZB-5	SPT	75,962	66,562	282	218	195	178
ZB-7	SPT	75,717	66,470	284.5	216	210	180

S1/S2 layer consists of the Upland Formation, Tobacco Road Formation, and the upper portion of the Dry Branch Formation. The C2 Layer is the Tan Clay Unit within the Dry Branch Formation. The S3 layer is the lower portion of the Dry Branch Formation, while the S4 layer is the Santee/Tinker Formation. For this calculation, the liquefaction analysis is performed for the saturated portions of the Tobacco Road and Dry Branch Formations (S1/S2, C2, and S3 layers). The Santee Formation (S4 layer) is typically too deep for liquefaction to pose a settlement

concern (Ref. 10). Settlement, including earthquake-induced dynamic settlement within the Santee Formation due to "soft zones," is evaluated in another calculation, K-CLC-Z-00021.

Soil unit weights were determined from undisturbed samples taken during the previous subsurface investigations. Within the Tobacco Road and Dry Branch formations, the unit weights ranged from 115 to 130 pcf. A value of 115 pcf was used for this calculation. Sieve analyses were performed on soil samples taken from geotechnical boreholes as part of the previous (1986) subsurface investigation. The locations of the boreholes are provided in the previous table. Table 2 summarizes the fines content of the soil samples. Note that grain-size distributions were not done in all the borings or at all depths in any boring.

Table 2: Fines Content (%) per Layer

ID	S1/S2	C2	S3	S4
Z-215	-	-	-	-
Z-216	-	-	-	-
Z-217	-	-	-	-
ZB-1	-	-	8.7-26.1	-
ZB-3	15.4	-	5.9	26.7
ZB-5	14.2-43.78	-	-	-
ZB-7	7.5-16.4	19	-	-
Site	16	45	17	-

2.2 Design Basis Earthquake

At the time of the initial geotechnical investigation in 1985, South Carolina used the Southern Building Code (SBCCI) under which seismic analysis was not required. At that time, seismic hazard characterization solely followed the deterministic approach. Reference 15 provided new seismic hazard models based on the probabilistic approach. Vault #1 was designed using deterministic seismic hazard models. A deterministic seismic hazard model estimates the most "likely" earthquake magnitude or intensity for a given location, and its "likely" attenuation, based on historical data, and then assumes a "likely" ground acceleration value.

The 1985 NEHRP provisions used the 1978 seismic hazard maps by Algermissen, Perkins, et.al., but building codes did not yet incorporate these; in 1967, DOE retained Dr. George W. Housner to determine a "safe-shutdown" seismic acceleration for the Savannah River Site. The acceleration Dr. Housner suggested a likely peak ground acceleration (PGA) of 0.1g and a safe-shutdown seismic acceleration of 0.2g (FS of 2) (ref. 9). URS/Blume updated the seismic criteria for SRS and retained Dr. Housner's safe-shutdown seismic acceleration of 0.2g; DOE further specified 0.2g as the design criteria in Regulatory Guide 1.60. At the time of construction of Vault #1, the Structural Specification for Building Code Requirements for All Projects, 1987, DuPont Site Specification No. 7096, called for High-Resistance structures to be designed for a

PGA of 0.13g. Vault #1 is classified as a High-Resistance structure; this calculation conservatively uses a PGA of 0.17g which is that required of current procedure.

3. Computation

3.1 Liquefaction Susceptibility

In this calculation the liquefaction potential for the Saltstone Disposal Unit 1 is evaluated using a modified version of the "Simplified Procedure for Evaluating Soil Liquefaction Potential" (Ref. 12). The simplified procedure calculates the liquefaction factor of safety as the ratio of Cyclic Resistance Ratio (CRR) to the Cyclic Stress Ratio (CSR) generated by the earthquake.

Factor of Safety = CRR / CSR

It should be noted that CRR was previously termed CSR required to induce liquefaction, but has been changed to more clearly distinguish the term from the CSR induced by the earthquake (Ref. 12). The CRR (soil capacity) and CSR (earthquake demand) are defined below

$$CRR = \tau_{ave} / \sigma'_{vo}$$

where:

τ_{ave} = average shear stress required to induce liquefaction
σ'_{vo} = effective vertical overburden stress

CSR is

$$CSR = \tau_{ave} / \sigma'_{vo} = 0.65 \times (a_{max} / g) \times (\sigma_{vo} / \sigma'_{vo}) \times r_d$$

where:

τ_{ave} = average shear stress induced by the earthquake
σ'_{vo} = effective vertical overburden stress

3.1.1 Earthquake Demand or Cyclic Stress Ratio (CSR)

The "Simplified Procedure for Evaluating Soil Liquefaction Potential" uses peak ground acceleration (PGA) to estimate shear stress (τ_{ave}) at ground surface and a stress reduction factor (r_d) to calculate τ_{ave} as a function of depth (Ref. 12). The simplified method of CSR determination is given below.

$$CSR = \tau_{ave} / \sigma'_{vo} = 0.65 \times (a_{max} / g) \times (\sigma_{vo} / \sigma'_{vo}) \times r_d$$

where

a_{max} = maximum horizontal acceleration at ground surface
g = the gravitational acceleration
σ_{vo} = total vertical stress
σ'_{vo} = effective vertical stress
r_d = stress reduction factor as a function of depth in meters
= $1.000 - 0.00765z$ for $z \leq 9.15$ m
= $1.174 - 0.0267z$ for $9.15 \text{ m} < z \leq 23$ m
= $0.744 - 0.0080z$ for $23 \text{ m} < z \leq 30$ m
= 0.500 for $z > 30$ m

3.1.2 Soil Capacity or Cyclic Resistance Ratio (CRR)

The original simplified procedure determines CRR using Standard Penetration Test (SPT) N_{160} values (corrected to clean sand) and a curve delineating liquefaction approximated by (ref 12):

$$CRR_{7.5} = \left(\frac{1}{34-(N_1)_{60}}\right)+\left(\frac{(N_1)_{60}}{135}\right)+\left(\frac{50}{(10\cdot(N_1)_{60}+45)^2}\right)-\frac{1}{200} \text{ (for } (N_1)_{60} < 30)$$

Therefore this calculation uses the original curve for $CRR_{7.5}$ using the average fines contents determined at Vault 1 and converting $(N_1)_{60}$ to clean-sand values; i.e., $(N_1)_{60cs}$.

$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$ where α and β are coefficients determined from the following equations (ref 11):

$\alpha = 0$ for $FC \leq 5\%$

$\alpha = \exp[1.76 - (190/FC^2)]$ for $5\% < FC < 35\%$

$\alpha = 5.0$ for $FC > 35\%$

$\beta = 1.0$ for $FC < 5\%$

$\beta = [0.99 + (FC^{1.5}/1000)]$ for $5\% < FC < 35\%$

$\beta = 1.2$ for $FC \geq 35\%$, where FC is the fines content.

The fines content used was that determined from laboratory gradation tests on retrieved soil samples (Ref. 1).

Current SRS liquefaction calculation procedures use a site-developed curve for CRR based on data at H-area as follows (Ref. 19):

$CRR_{7.5} = 0.0000685(N_{160})^3 - 0.0010089(N_{160})^2 + 0.0177243(N_{160}) + 0.0645105$. This curve uses site-specific data for fines < 24% (9-24%); therefore a clean-sand reduction isn't used. However, fines content determined in the borings sometimes exceeded 24%.

This calculation determined $CRR_{7.5}$ both ways.

The $CRR_{7.5}$ from the curves are further modified by several factors that have been developed over time. These factors correct for: aging, static driving shear stress, overburden pressure, and earthquake magnitude. SPT data (N_{160}) were used to determine $CRR_{7.5}$ for this calculation. Although there are several correction factors for SPT data, the most significant one is the correction factor for overburden stress, C_N. The other correction factors are for energy ratio, borehole diameter, rod length, and sampling method. Except for rod length, there was no information available to determine these correction factors, and the absence of the rod length correction did not significantly affect the results since the SPT N-values were so high.

The $CRR_{7.5}$ curve was developed from testing of SRS soils in lieu of the standard SPT and CPTu

liquefaction methods developed for Holocene and younger deposits. When applying the correction factors, the Factor of Safety is expressed as given below. Each of the correction factors used in this calculation for determining CRR is discussed in the following sections.

$$\text{Factor of Safety} = \frac{CRR_{7.5} \cdot K_\sigma \cdot K_{age} \cdot K_\alpha \cdot MSF}{CSR}$$

3.1.3 Overburden Stress Correction Factor, C_N, for SPT Data

$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a)$ where P_a = 100 kPa normalizes the effective overburden stress σ'_{vo} (at the time of the testing) to one atmosphere of pressure (ref 12).

3.1.4 Static Effective Overburden Pressure Correction Factor, K_σ

Most of the case history data used to develop the standard liquefaction curves (Ref. 12) were taken from cases of level ground with relatively small initial effective overburden stresses ($\sigma'_o \leq$ 1 tsf). However, at higher effective overburden stresses ($\sigma'_o >$ 1 tsf), the liquefaction susceptibility of the soil will increase for a given CSR (Ref. 20). Thus, the CRR must be corrected for the influence of the static overburden stresses. This is done by multiplying CRR by the correction factor K_σ. The soils at SRS are much older than the case history data typically used for liquefaction studies and are somewhat overconsolidated. Therefore testing of soils at SRS has been performed to determine appropriate K_σ for SRS soils (Ref. 22, 23). Figure 3 shows the SRS K_σ curve along with data used to develop the curve (Ref. 26). The NCEER recommended K_σ curves (Ref. 12) are also shown on Figure 3 for comparison. The polynomial representing the SRS K_σ curve shown in Figure 3 is given below.

$$K_\sigma = 1.009376 - 0.18326 \log(\sigma'_{vo}) - 0.08340 \log(\sigma'_{vo})^2$$

where:

σ'_{vo} = Effective Vertical Overburden Pressure in tsf.

Note that the K_σ used for this calculation is the site-specific relationship developed using data from investigations in H-Area (Ref. 13, 22, and 25) and not the standard K_σ relationship proposed by NCEER (Ref. 12). The SRS K_σ used is applicable for Saltstone Disposal Unit 1, but less stringent than the K_σ proposed by NCEER.

3.1.5 Age Correction Factor K_{age}

Since SRS soils are older than Holocene, their liquefaction potential would be decreased due to aging. This calculation conservatively ignored the aging effect. (i.e., K_{age} = 1.0); i.e., assuming the soils are younger than Holocene-aged.

The effect of soil aging is incorporated into SRS's CPT-based CRR curves using data from investigations in H-Area (Ref. 22, 23, 25) and after extensive field and laboratory testing programs. These curves will be used in our subsequent calculation evaluating liquefaction potential of Vault 1 using the CPT procedure.

3.1.6 Static Driving Shear Stress Correction Factor, K_α

Relationships proposed by Seed and Harder (Ref. 20) suggest that a static driving shear stress can increase or decrease the soil's resistance to liquefaction, depending on the magnitude of the driving stress and the relative density of the soil. Seed and Harder proposed a static driving shear stress correction factor (K_α) to correct CRR (Ref. 20). However, the proposed chart to estimate K_α is preliminary and this correction factor is a subject of current research (Ref. 21, pp. 172-176). For this calculation, no K_α correction was used (i.e., $K_\alpha = 1.0$).

3.1.7 Earthquake Magnitude Scaling Factor, MSF

The CRR curves used for liquefaction analysis are only valid for M = 7.5 earthquakes. For earthquakes with differing magnitudes, the CRR values must be multiplied by a magnitude scaling factor (MSF).

The earthquake magnitudes from the PSHA and the appropriate MSF for each magnitude are given below. These MSFs represent the middle of the NCEER (1997) recommended values shown on Figure 4 and were used to fit an exponential curve.

Earthquake Magnitude (Mw)	Magnitude Scaling Factor
5.5	2.5
6.0	2.0
6.5	1.6
7.0	1.25
7.5	1.0
8.0	0.8
8.5	0.7

The MSF equation,

$MSF = 27.3e^{-0.44 \times Mw}$ is rounded to the nearest tenth (0.1) for use in calculations.

3.1.8 Percent Fines

The fines content determined from samples taken around Vault 1 during previous subsurface investigations (Ref. 1, 5) were previously summarized.

3.2 Liquefaction Hazard Weighting

Table 3 summarizes the PGA hazard for the 2,500 year earthquake. (Ref. 24). The weight for a given magnitude range is the sum of the weights for the various distances for the given magnitude Settlement due to liquefaction and partial liquefaction, is calculated using different magnitudes. The magnitudes selected are the midpoints of the magnitude ranges given in Table 3 (i.e. 4.75, 5.25, 5.75, 6.25, 6.75, and 7.5). The results are then weighted according to the PGA hazard disaggregation.

Table 3: USGS Rock Seismic Hazard: 100 Hz Oscillator Frequency, 2,500 Year Return Period, S_a = 0.15 g

Distance	Mw = 4.5 to 5.0	Mw = 5.0 to 5.5	Mw = 5.5 to 6.0	Mw = 6.0 to 6.5	Mw = 6.5 to 7.0	Mw = 7.0 to 8.0
7.5 km	7.13	3.56	1.73	0.78	0.42	0.00
20 km	4.93	3.64	2.35	1.28	0.78	0.00
37.5 km	3.11	3.90	4.04	3.24	2.72	0.00
75 km	0.33	0.93	2.05	3.24	0.00	16.45
150 km	0.03	0.14	0.59	1.68	0.00	29.25
250 km	0.00	0.00	0.00	0.04	0.00	1.63
550 km	0.00	0.00	0.00	0.00	0.00	0.08
Mw Bin Sum =	15.52	12.16	10.76	10.25	3.92	47.40

Note: ⅓ wt Frankel et al. attenuation model, ⅓ wt Toro et al. attenuation model and ⅓ wt AB95 attenuation model

Figure 2 provides a graphical view of this information with the relative weighting of the various earthquake magnitudes.

4. Results and Conclusions

Liquefaction potential based on the CRR determination using Standard Penetration Testing indicates the soil is generally only slightly susceptible to liquefaction. The clean-sand CRR calculation resulted in the most conservative values. The minimum factors of safety from the SPT data using the clean-sand procedure range from 0.81 to well over 2.5 (even though some safety factors were much greater than 2.5, we used 2.5 as a reporting ceiling. Generally the factors of safety are in excess of 3.0 for the vast majority of the soil column. See the attached Figures. Settlements will be estimated in a subsequent calculation.

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Figures

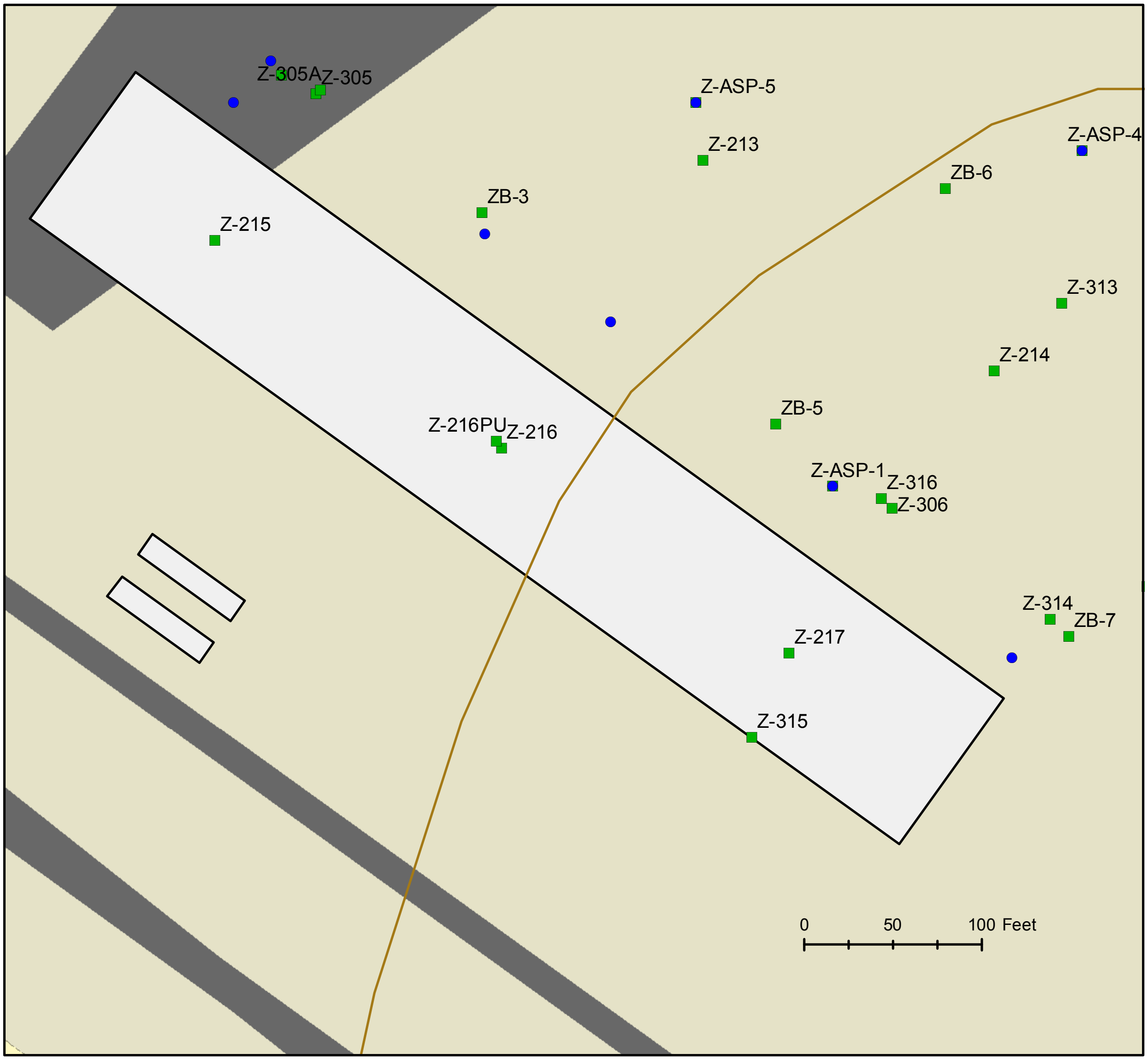


Figure 1: Saltstone Disposal Unit 1 Project Site Showing 1986 SPT Locations

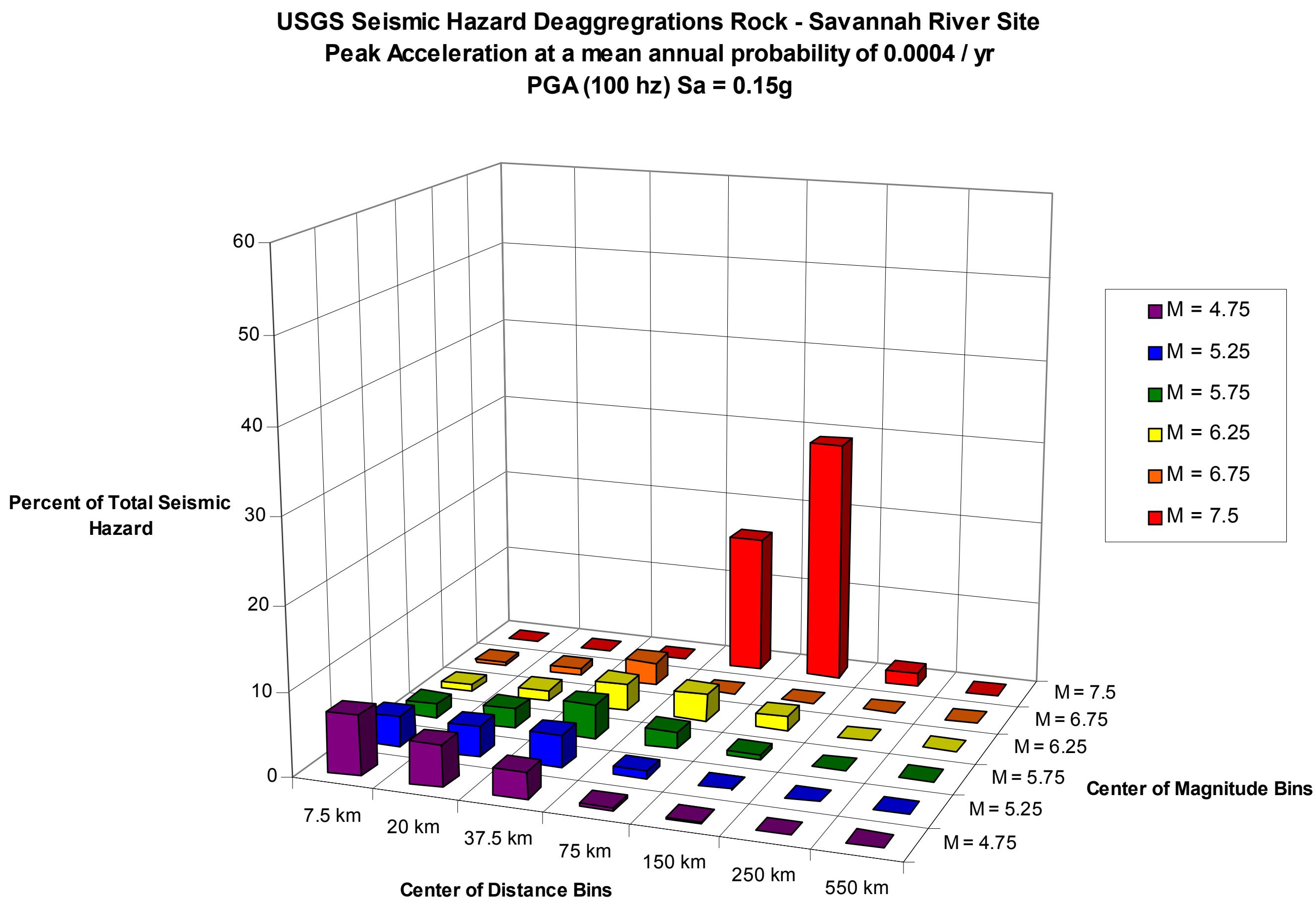


Figure 2: USGS Rock Seismic Hazard Disaggregation: 100 Hz, 2,500 Year Return, S_a = 0.15g

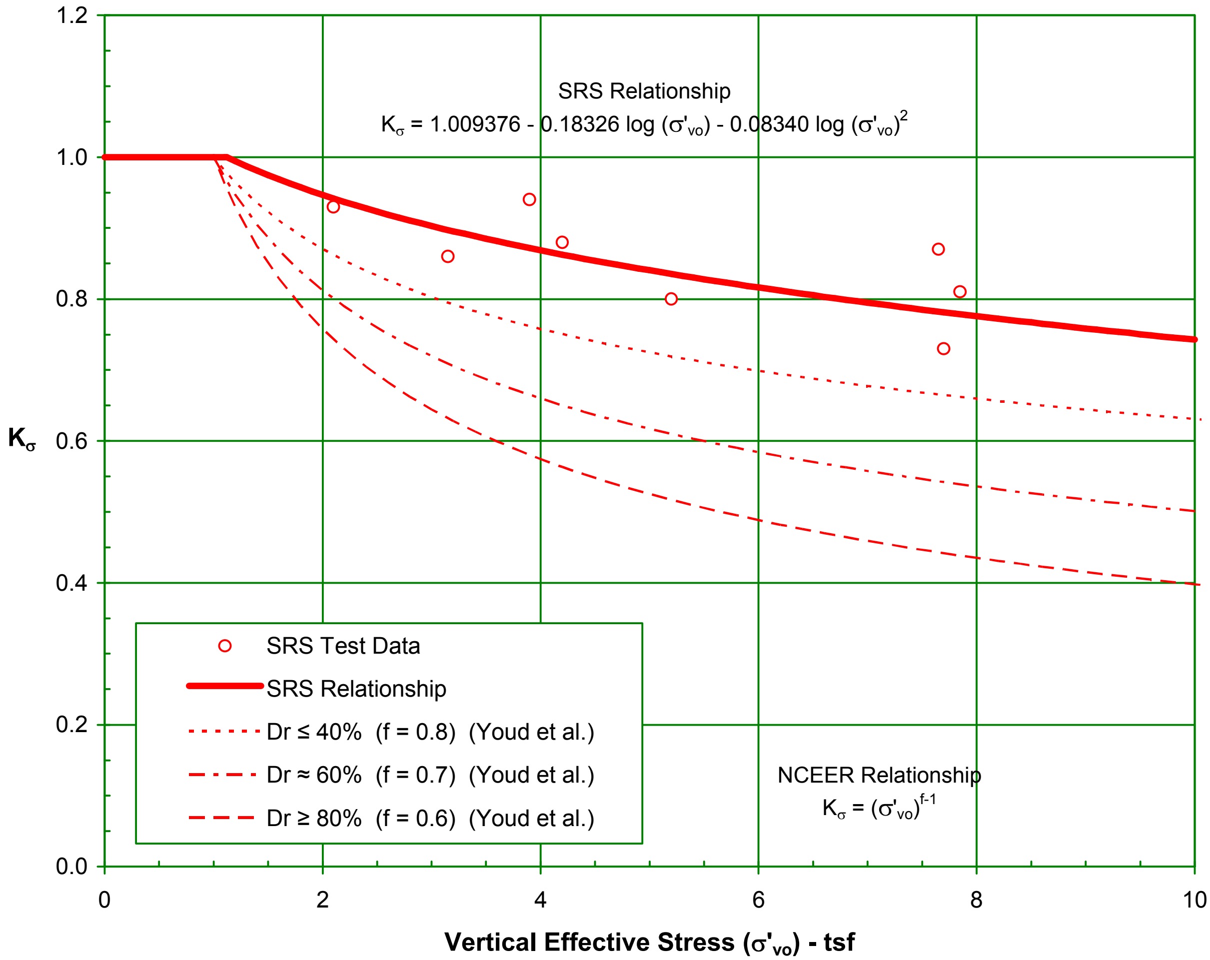


Figure 3: Comparison of NCEER Kσ with SRS Kσ

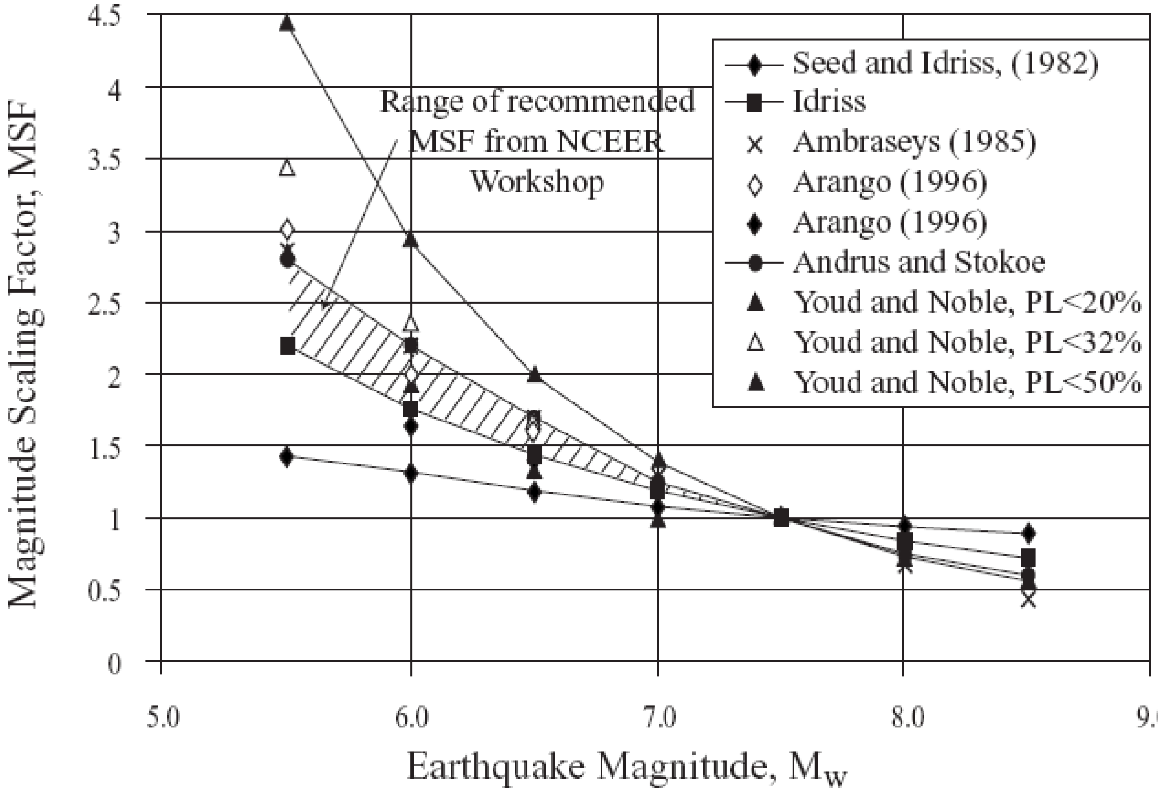


Figure 4: Magnitude Scaling Factor Recommendations Presented by NCEER

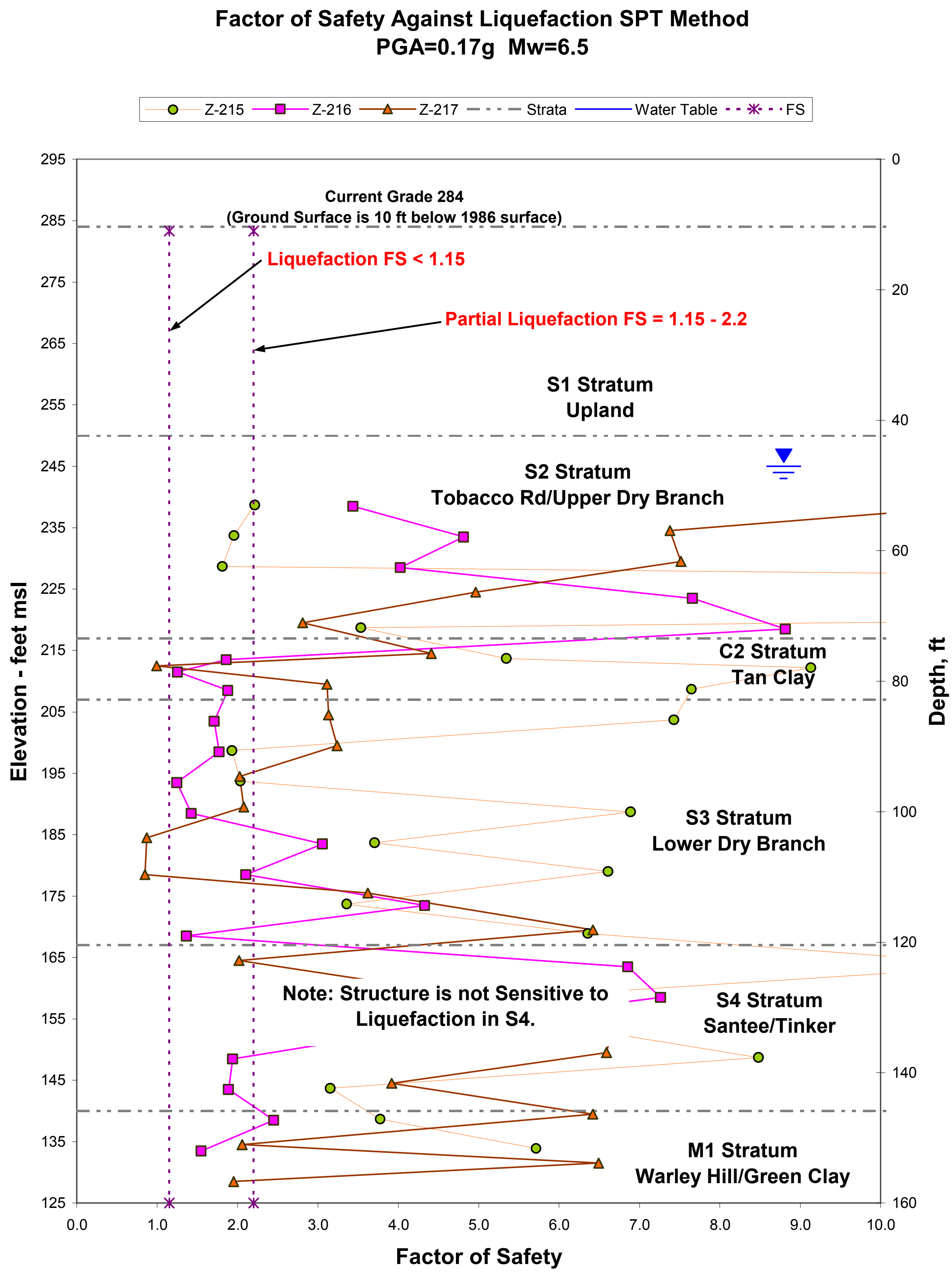


Figure 5: Factor of Safety against Liquefaction, SPT Method, DBE – Borings Z-215 to Z-217.

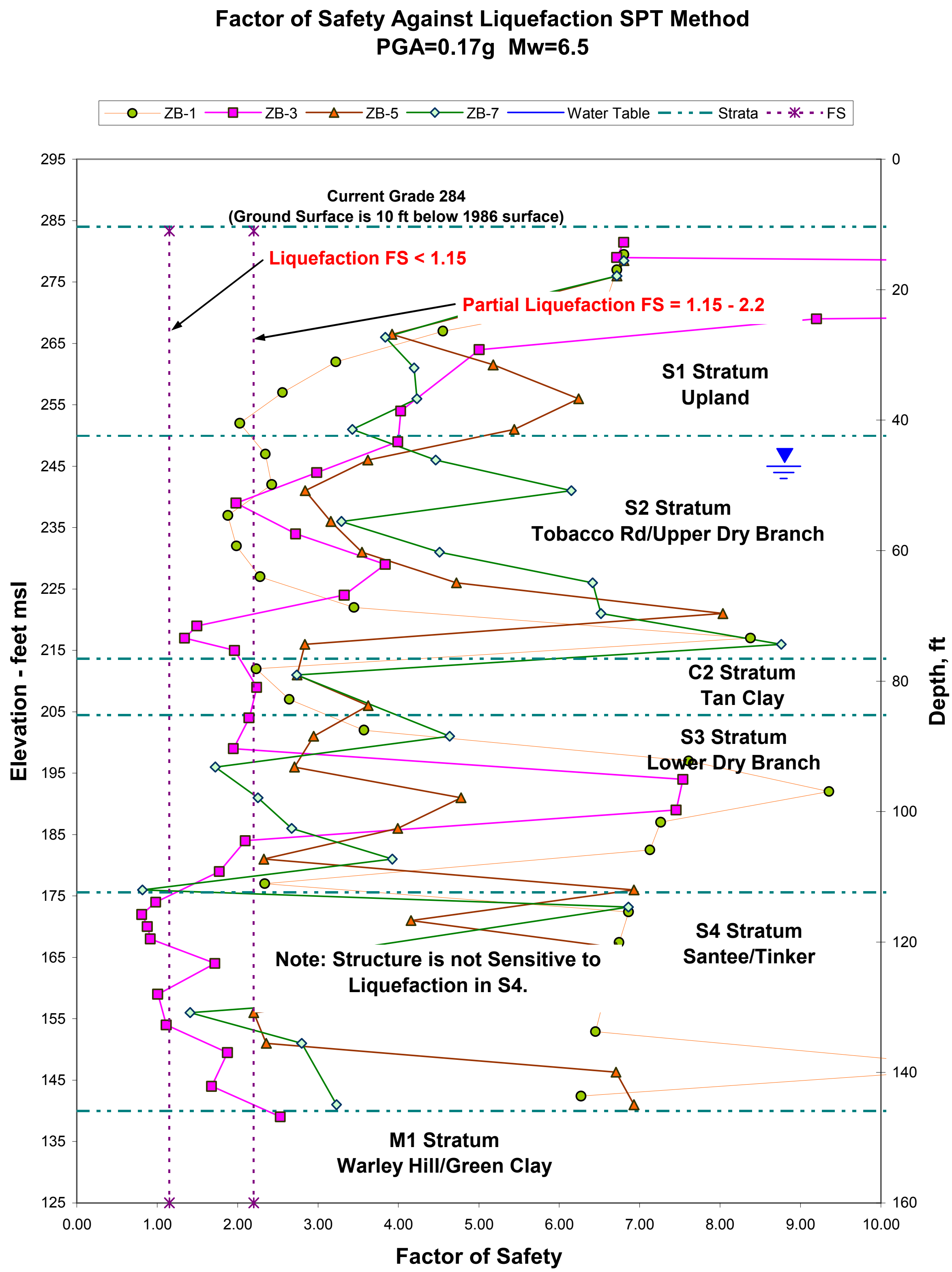


Figure 6: Factor of Safety against Liquefaction, SPT Method, DBE – Borings ZB-1, 3, 5, & 7

Appendix B

K-CLC-Z-00020, Stratigraphy for the Saltstone Vault No. 1, Rev. A, October 27, 2010

(27 pages)

OSR 45-24 (Rev 1-20-2009)

CALCULATION COVER SHEET

Project/Task	Calculation No.	Project/Task No.
Saltstone Vault No. 1	K-CLC-Z-00020	N/A
Title: Stratigraphy for Saltstone Vault No. 1	Functional Classification: P/S; Discipline:	Sheet 1 of 27
Calculation Type: ☒ Type 1 ☐ Type 2	Type 1 Calc Status: ☒ Preliminary ☐ Confirmed	
Computer Program No. ☒ N/A	Version/Release No.	
Purpose and Objective: Develop engineering stratigraphy for the Saltstone Vault No. 1 settlement analysis update.	DC/RO Date UNCLASSIFIED DOES NOT CONTAIN UNCLASSIFIED CONTROLLED NUCLEAR INFORMATION ADC & Reviewing Official: A.A. Siddall for ACT (Name) Date: 10/14/2010	

Summary of Conclusion

See results and conclusions section.

Revisions

Rev #	Revision Description
A	Original

Sign Off

Rev #	Originator (Print) Sign/Date	Verification/Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign/Date
A	D. Bruce Nothdurft, PE	☐ Design Check (GS/PS only) ☒ Document Review ☐ Qualification Testing ☐ Alternative Calculation ☐ Operational Testing	Patti Bennett Patti Bennett 10/13/10	William T. Li 10/13/10
		☐ Design Check (GS/PS only) ☐ Document Review ☐ Qualification Testing ☐ Alternative Calculation ☐ Operational Testing		

Additional Reviewer (Print)	Signature	Date
Jim Mason	James J. Mason, Jr.	10/14/2010
Design Authority (Print): N/A	Signature	Date
Release to Outside Agency (Print): N/A	Signature	Date

Security Classification of the Calculation

Unclassified

1. INTRODUCTION

This calculation identifies engineering stratigraphic layers for SPT and CPTU soundings performed for the Saltstone Vault No. 1. Engineering stratigraphic layers and nomenclature were based on previous subsurface investigations performed within Z Area generally, and the Vault No. 1 area particularly, by Mueser Rutledge Consulting Engineers (MRCE) (Ref 1) and SRS (Ref 2). Figure 1 depicts the location of Vault 1 within Z Area.

2. INPUT DATA

Recent subsurface data acquired for the Saltstone Vault No. 1 investigation included four (4) seismic piezocone penetration test (SCPTU) soundings and six (6) piezocone penetration test soundings (CPTU) (Ref. 4). In addition seven (7) borings were drilled in two previous explorations in 1986 (Ref 1) and 1992 (Ref 2). These locations are depicted in Figure 1.

3. METHOD FOR DETERMINATION OF ENGINEERING LAYERS

Engineering layers developed for the Saltstone Vault No. 1 area followed a scheme previously used by other investigations in Z Area. Using this same layering scheme provided a means to compare subsurface conditions between investigation sites. An S or C designation was used to divide layers into predominantly sand and clay units, respectively. The upper layer S1/2 consists of the Upland Formation, Tobacco Road Formation, and the upper portion of the Dry Branch Formation; layer S1/2 extends from the surface to the C2 layer. The C2 layer corresponds to what has commonly been termed the "Tan Clay" layer which occurs within the Dry Branch Formation. The S3 layer is the lower portion of the Dry Branch Formation and occurs between the C2 layer and the S4 layer. The S4 layer corresponds to the Santee/Tinker Formation. The Warley Hill (sometimes referred to as the “green clay”) is denoted as M1 because it typically occurs as a predominately silty layer.

CPT and SCPT explorations and SPT boring data were used to determine the thickness and presence of the engineering layers. Patterns in tip stress, sleeve stress, ratio of tip stress to sleeve stress, and pore pressure responses were used in differentiating the soil layers. Soil properties exhibited in the CPT and SCPT soundings were compared to descriptions of soil samples collected from SPT borings advanced in 1986 and 1992, and information published from previous Saltstone vault studies (Ref 3) was used to correlate engineering soil layers at the site to regional geologic units and soil stratigraphy at the SRS.

Two engineering layers, C2 and S4, were identified as critical surfaces from a review of the existing data. These layers were developed from previous geotechnical studies performed in the area. The first layer, C2, was historically identified on boring logs from low N-value measurements and visually from field procedures on split-spoon contents. This layer is easily identifiable in CPT logs from a sudden drastic decrease in tip resistance accompanied by a sudden drastic increase in sleeve friction and pore pressure. Field and laboratory data suggest this layer contains considerable fines. The second layer, S4, has been identified as the Santee/Tinker surface.

Geologic stratigraphy (specifically the top of the Santee/Tinker Formation) was determined from the ten CPT pushes advanced as part of this investigation, and coordinated with that reported in the two previous SPT investigations. The top of the Santee/Tinker Formation (designated S4) was determined solely for the purpose of the liquefaction potential analysis. The Altamaha, Tobacco Road and Dry Branch (except for C2) formations overlying the Santee/Tinker Formation were not differentiated as part of this study. Copies of the CPT logs are provided in the Attachment.

The top of the Santee/Tinker Formation was visually determined in previous borings from soil samples as a marked change in the gradation, color, mineralogy, and reaction to dilute hydrochloric acid (HCl) solution. The gradational change from Dry Branch to Santee/Tinker formations was noted as becoming more fine grained while the color became more greenish and the mineralogy transitioned from an abundance of manganese oxide to the presence of calcite (positive HCl reaction) and glauconite.

Table 1 summarizes the contacts between stratigraphic layers based on the SPT boring logs conducted in 1986 and 1992.

Table 1 – Engineering Stratigraphy Based on Previous (1986 & 1992) SPT Borings

Boring ID	Ground Elevation	Total Depth	C2	S3	S4	Congaree (M1)	SRS_N	SRS_E
Z-215	294.7	161.5	-	217	172	141	75,987	66,275
Z-216	294.5	146.5	217	207	167	142	75,797	66,337
Z-217	295.5	167.5	223	217	182	137	75,608	66,400
ZB-1	283	142	214	205	176	141	76,105	66,342
ZB-3	285	147	220	197	181	138	75,911	66,406
ZB-5	282	142	218	195	178	143	75,962	66,562
ZB-7	294.5	142	216	210	180	145	75,717	66,470

Tops of Stratigraphic Layers in feet msl.

The Congaree Formation (M1), which consists of dense, variably clayey coarse sands, was identified in the borings and on a few CPT signatures. This formation appears to be relatively flat across the site and exists at an elevation of approximately 142 ft, msl. CPTs Z-V1-C-01, Z-V1-C-03, and Z-V1-C-04 refused on (did not penetrate) the top of the Congaree Formation.

4. SOFT ZONES

Since the early 1950s, numerous subsurface and geotechnical studies at SRS have determined that the soils within 200 feet of the surface consist mainly of sands, clayey sands, and silty sands. Several SRS geotechnical explorations have encountered weak zones within stronger materials in the lower Dry Branch and the upper Tinker-Santee Formations. Low tip resistance and SPT values and drilling rod drops are common indicators of these weak zones, which are referred to as "soft zones". With regard to CPT penetration, “soft zones” are defined as soils with a continuous corrected tip stress of 15 tons per square foot (tsf) or less over elevation intervals of at least 1.5 feet within the lower Dry Branch or the Tinker-Santee Formations (engineering layers S3 and S4 respectively).

5. RESULTS AND CONCLUSIONS

Engineering stratigraphy for Saltstone Vault No. 1 SPT borings and CPT soundings is summarized in Tables 1 and 2 respectively. The elevations shown for the top of S3 are also the elevations for the bottom of C2. Note that the elevations and locations shown in Table 2 are correct; the preliminary CPT logs contain some typographic errors in that regard.

Table 2 – Engineering Stratigraphy Based on CPTs

CPT ID	Ground Elevation	Total Depth	C2	S3	S4	M1	SRS_N	SRS_E
Z-V1-C-01	284.1	152.1	210	201	194	142	76084.0	66243.0
Z-V1-C-02	285.7	126.7	217	205	192	NDE	76005.0	66333.1
Z-V1-C-03	285.7	156.7	214	NP	195	145	75895.5	66369.2
Z-V1-C-04	285.7	156.5	220	208	194	138	75750.1	66417.3
Z-V1-C-05	285.4	110.2	222	213	197	NDE	75620.0	66459.9
Z-V1-C-06	283.7	123.3	216	202	189	NDE	75488.0	66441.9
Z-V1-C-07	283.5	109.9	221	210	204	NDE	75574.8	66346.5
Z-V1-C-08	283.5	119.2	219	206	200	NDE	75688.8	66308.7
Z-V1-C-09	283.4	134.7	215	194	190	NDE	75832.6	66261.2
Z-V1-C-10	283.5	122.9	216	198	191	NDE	75960.9	66218.0

Tops of Stratigraphic Layers in feet msl.
NDE – Not Deep Enough
NP – Not Present

6. REFERENCES

1. Mueser Rutledge Consulting Engineers (MRCE), Report: “Saltstone Disposal Investigation,” 1986, File Z-SDF-2
2. Bechtel Savannah River, Inc. (BSRI), Report: “Z-Area Vault No. 2 Geotechnical Investigations Report,” C-ESR-Z-00001, BSRI, 1992, File Z-SDF-9
3. K-CLC-Z-00003, Subsurface Stratigraphy for the Saltstone Vault No. 4, Rev 0.
4. LanKelma, Inc.; SRNS Strategic Agreement SAQ003; Purchase Order AC76230N; Task Order 03; SOW 4T5310, July 27, 2010

7. LIST OF FIGURES

8. ATTACHMENTS

Attachment 1: CPT Data Curves (engineering strata indicated)

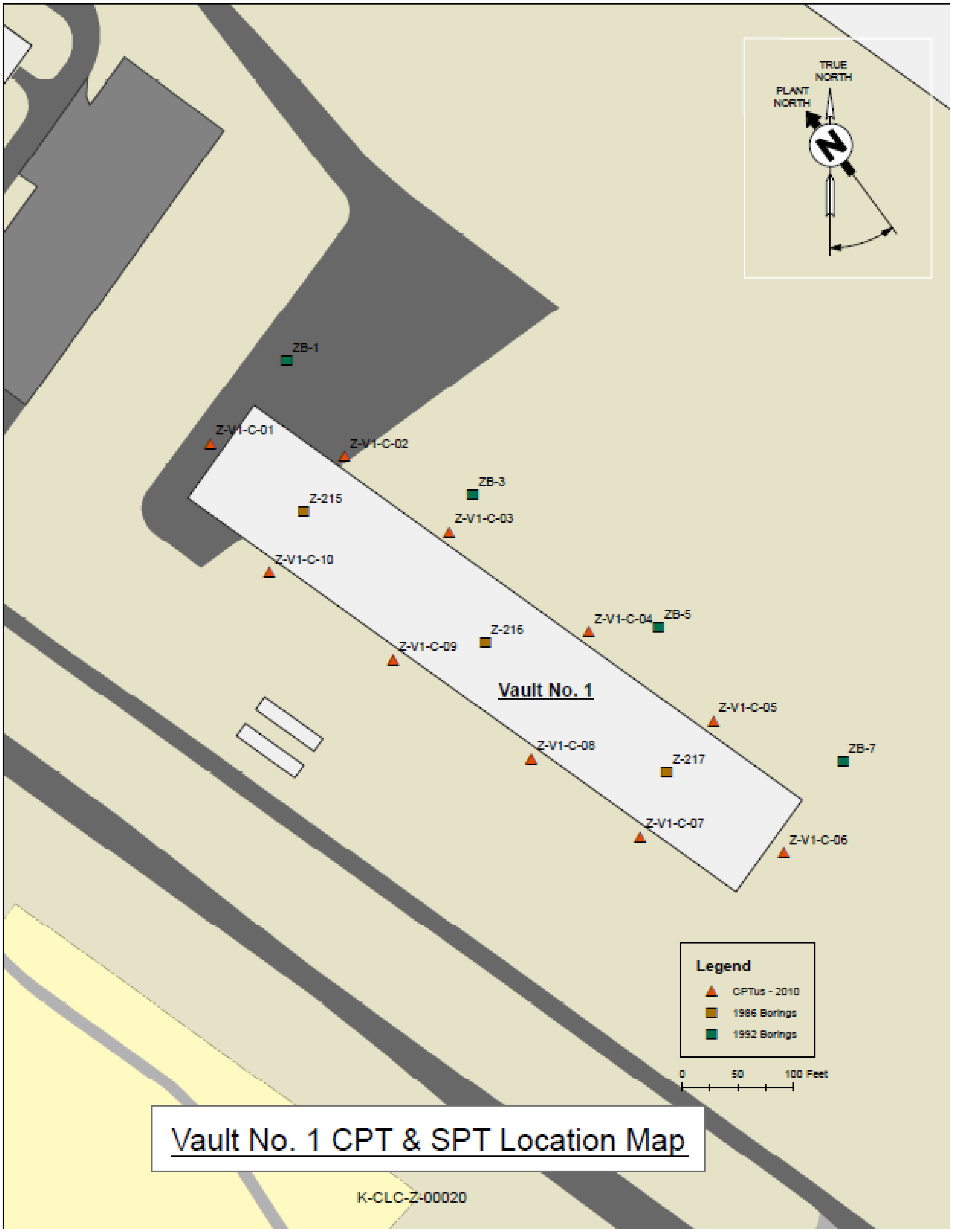


Figure 1: Vault 1 Location Map with Geotechnical Investigation Locations

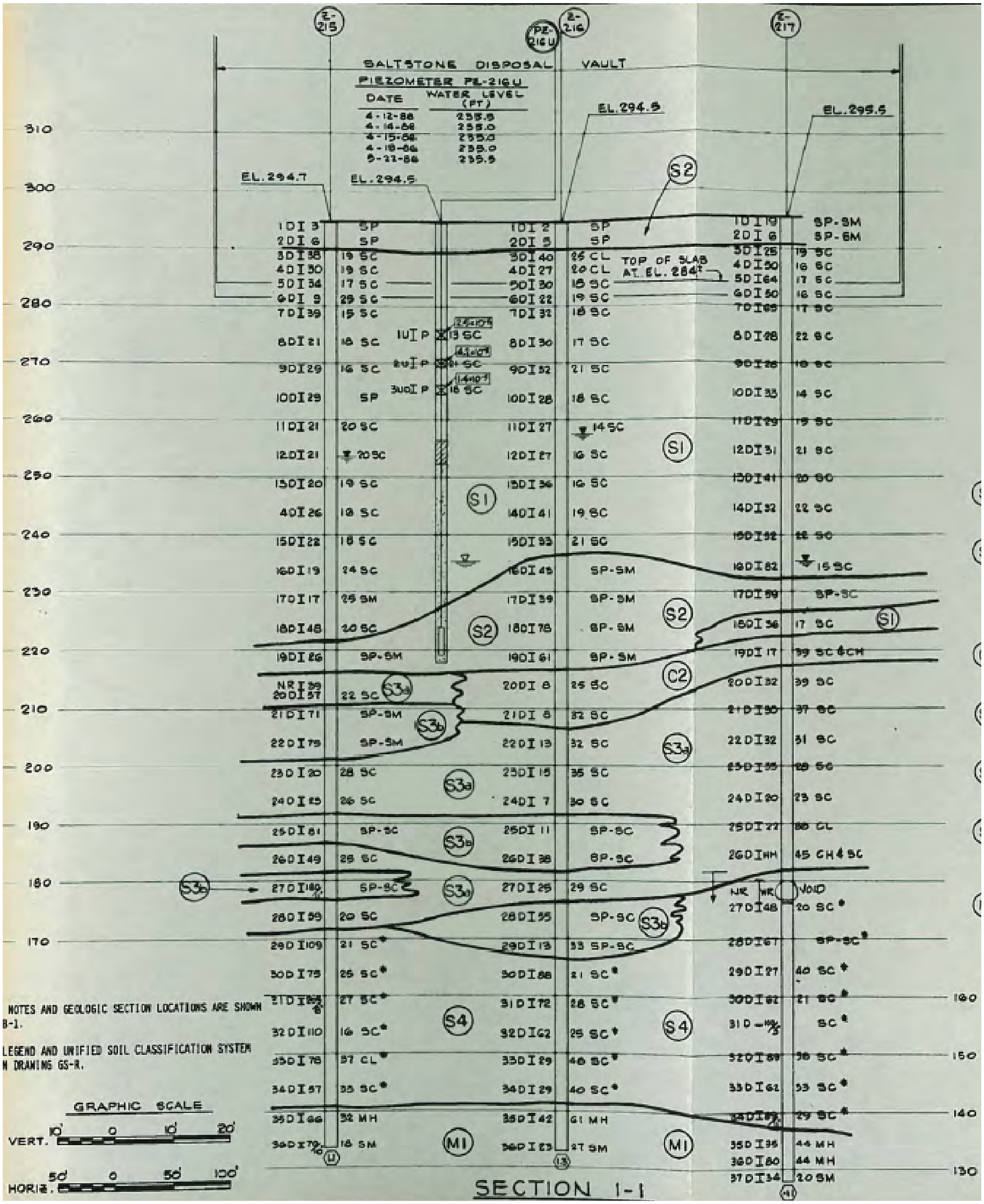


Figure 2: Section 1-1 from 1986 Geotechnical Investigation (ref. 1)

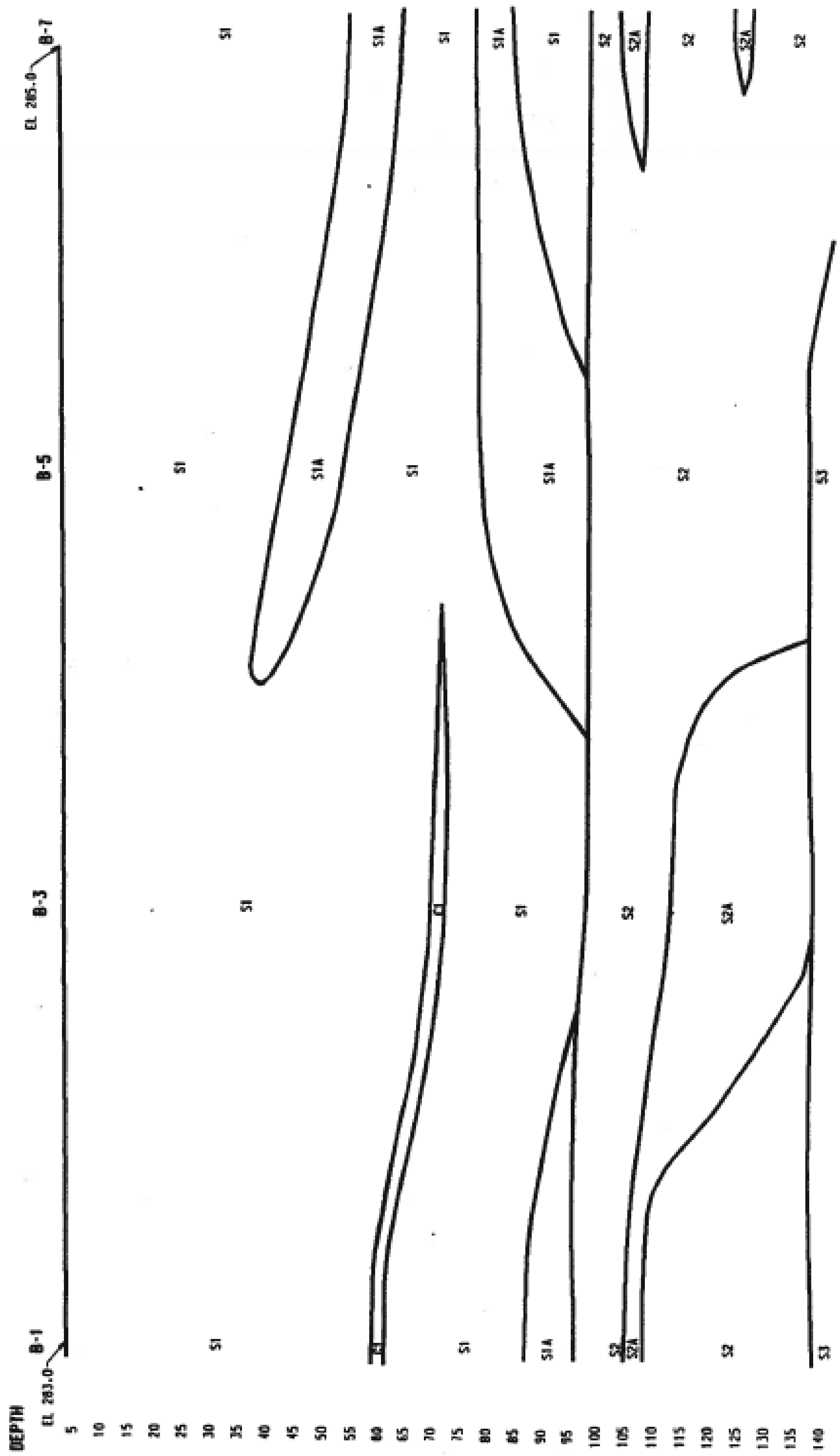
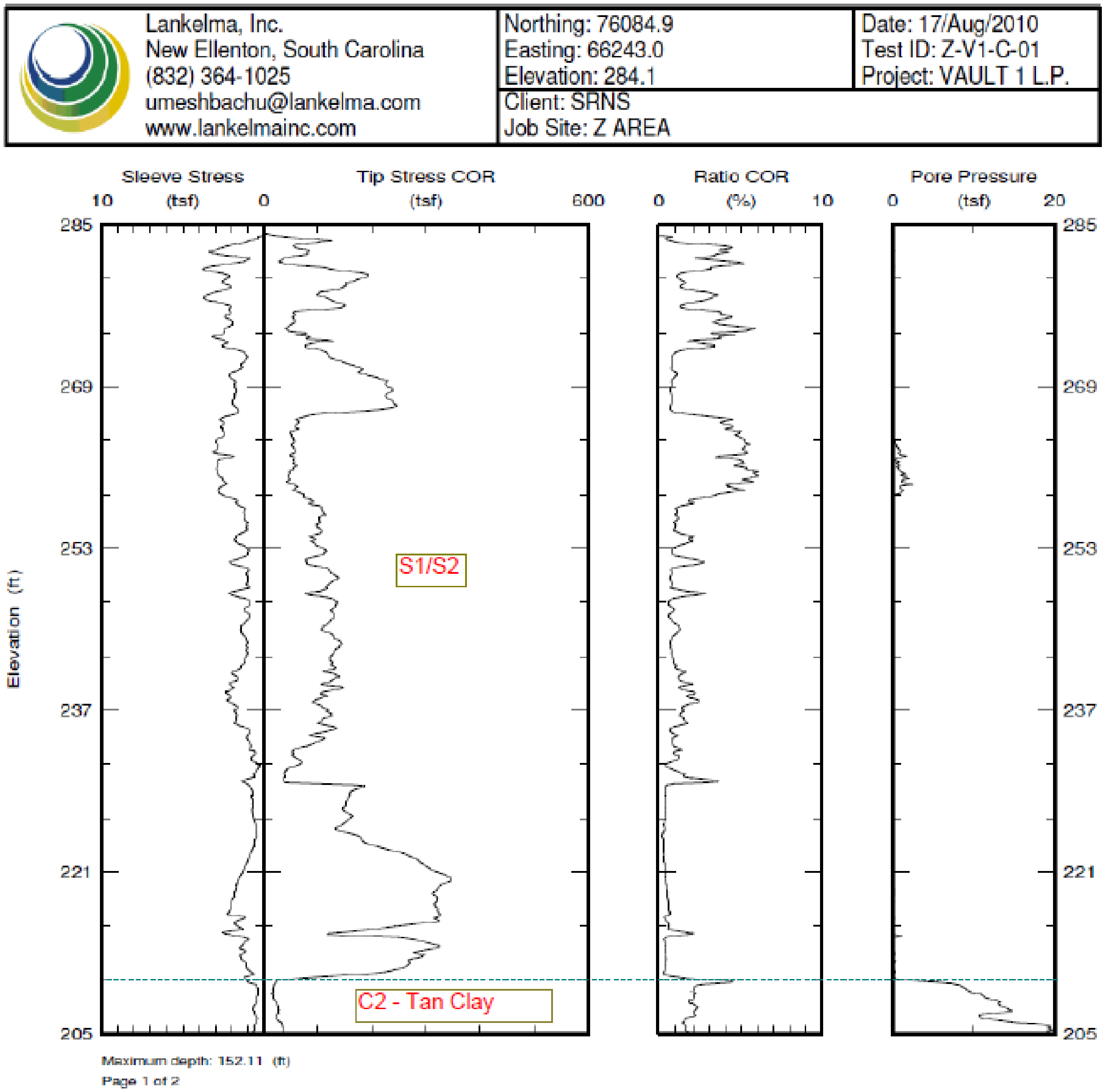
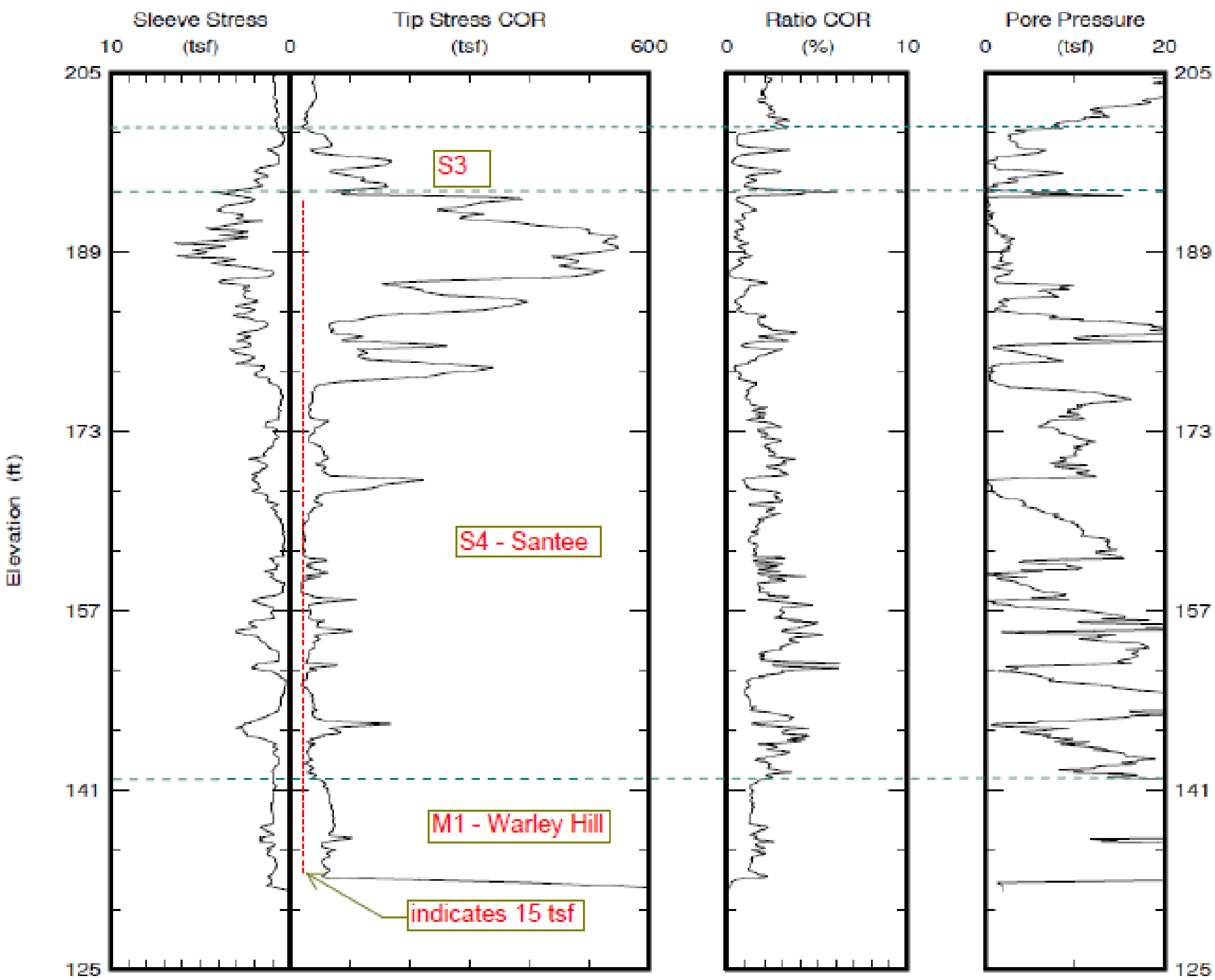


Figure 3: Cross-section from Bechtel Vault 2 Geotechnical Rpt C-ESR-Z-00001 Z-SDF-9 (ref.2)

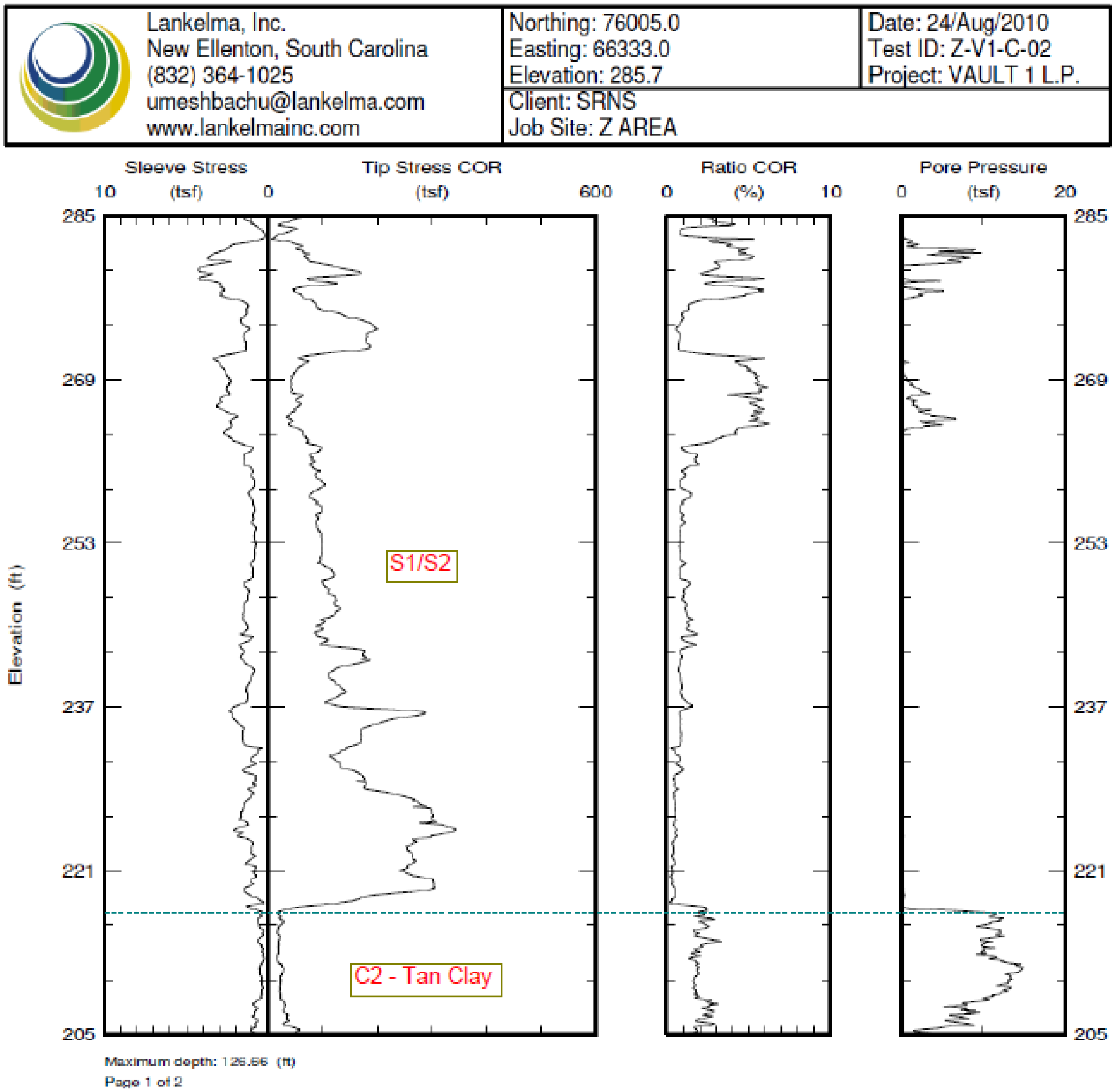
Attachment: CPT Data Curves with engineering strata indicated

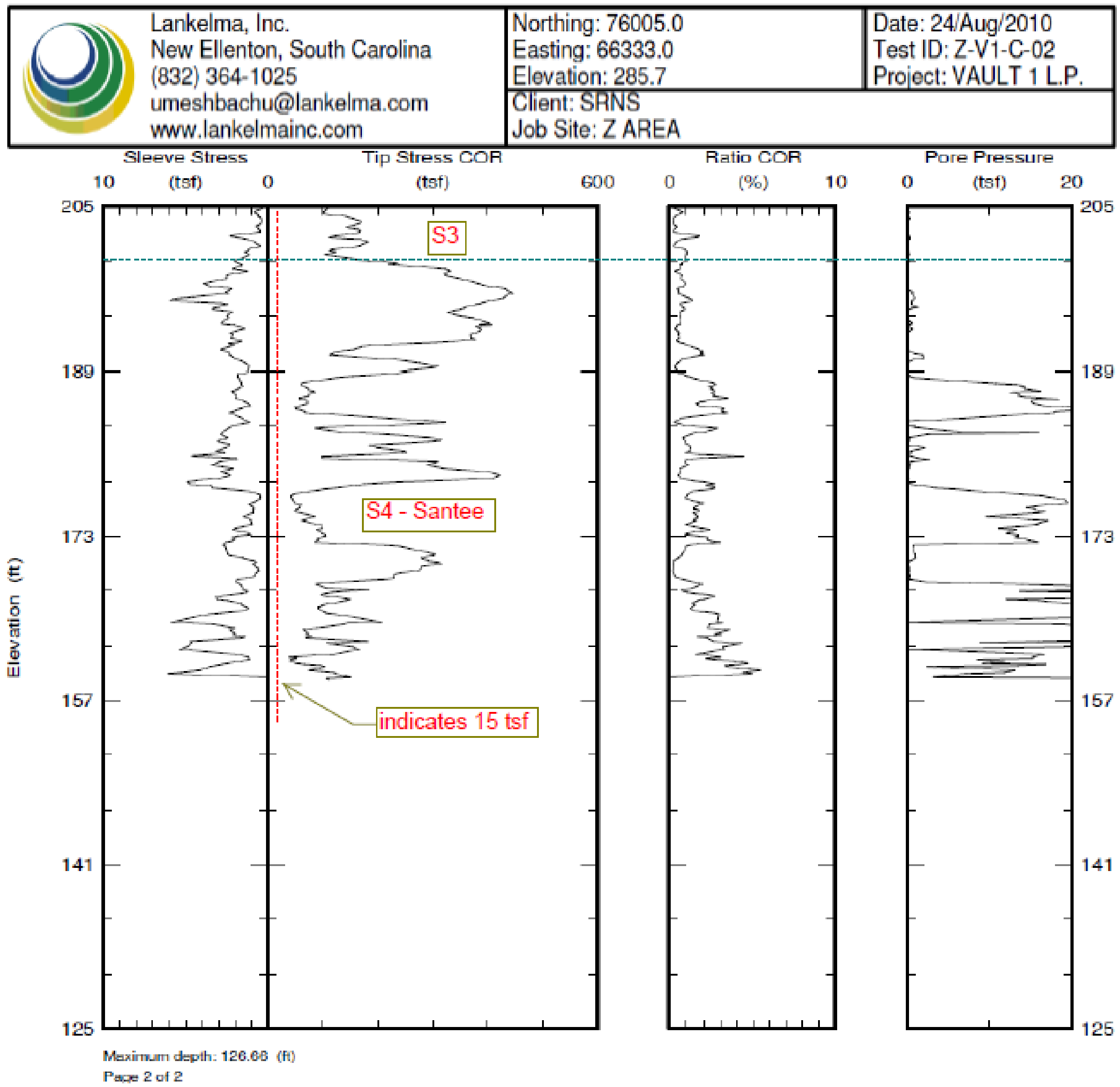


Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 76084.9 Easting: 66243.0 Elevation: 284.1	Date: 17/Aug/2010 Test ID: Z-V1-C-01 Project: VAULT 1 L.P.
	Client: SRNS Job Site: Z AREA	

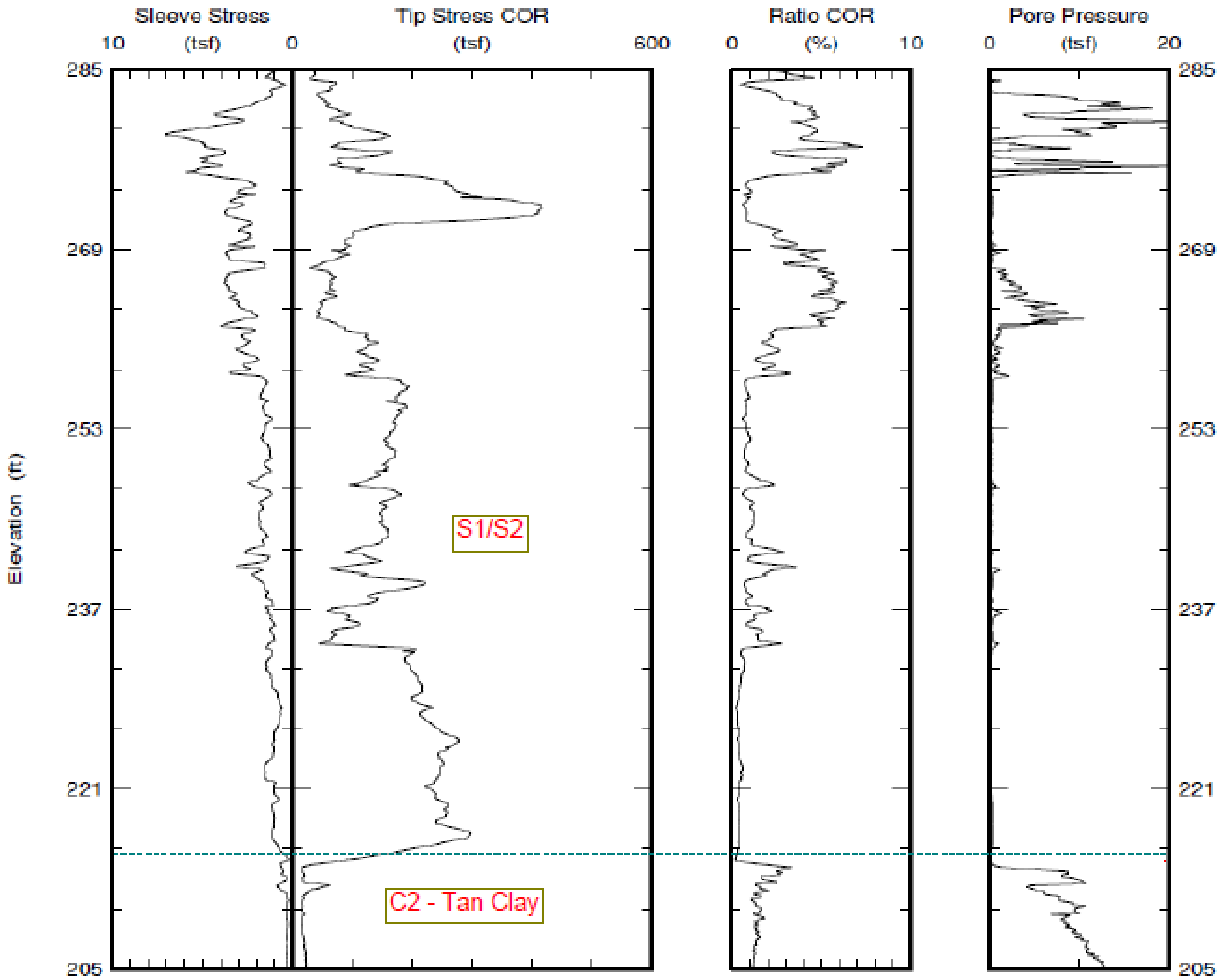


Maximum depth: 152.11 (ft)
Page 2 of 2

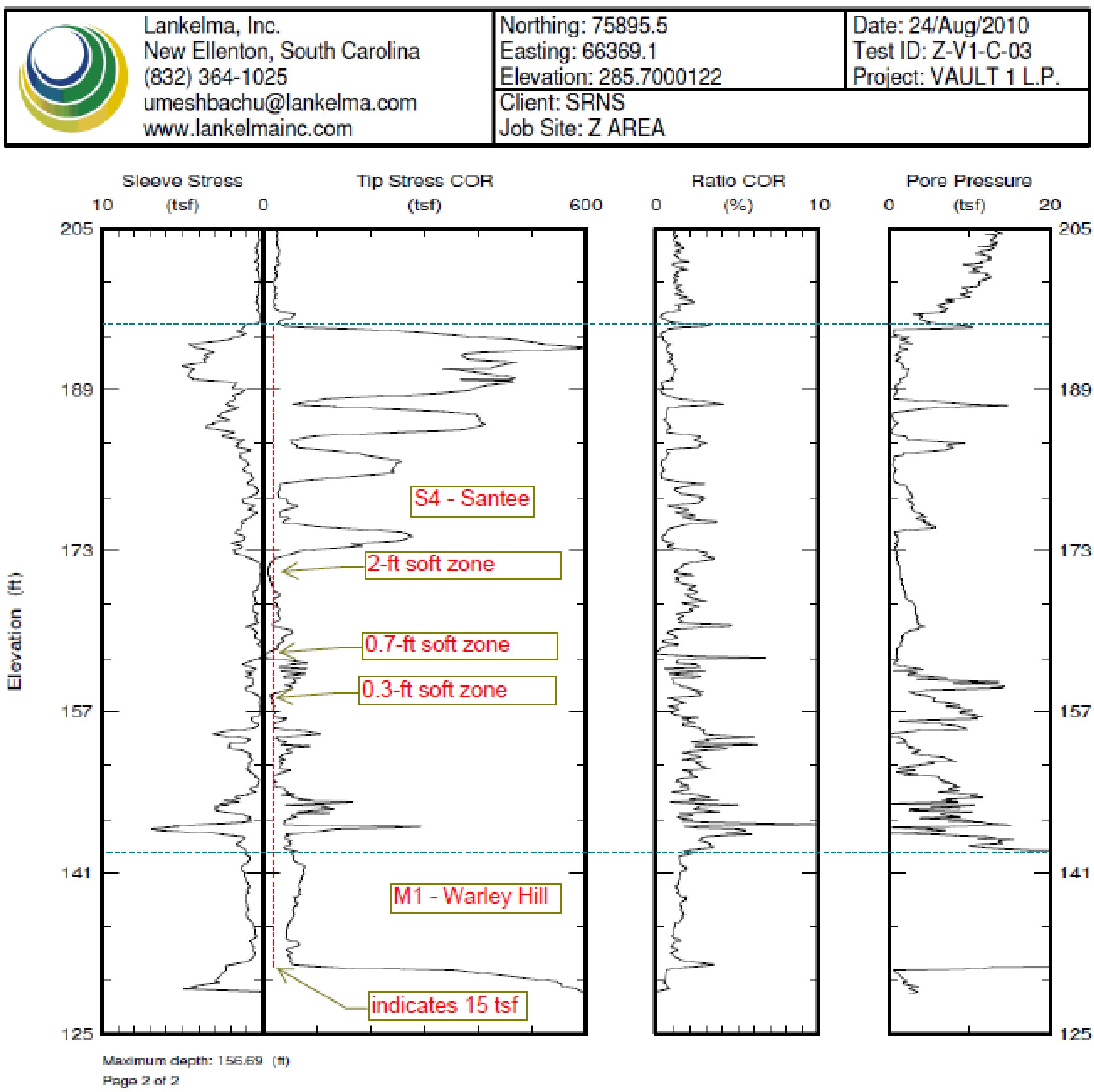

Lankelma, Inc.
New Ellenton, South Carolina
(832) 364-1025
umeshbachu@lankelma.com
www.lankelmainc.com
Northing: 76005.0
Easting: 66333.0
Elevation: 285.7
Date: 24/Aug/2010
Test ID: Z-V1-C-02
Project: VAULT 1 L.P.
Client: SRNS
Job Site: Z AREA
Sleeve Stress
10 (tsf) 0
Tip Stress COR
(tsf) 600
Ratio COR
0 (%) 10
Pore Pressure
0 (tsf) 20
Elevation (ft)
285
269
253
237
221
205
S1/S2
C2 - Tan Clay
Maximum depth: 126.66 (ft)
Page 1 of 2


Lankelma, Inc.
New Ellenton, South Carolina
(832) 364-1025
umeshbachu@lankelma.com
www.lankelmainc.com
Northing: 76005.0
Easting: 66333.0
Elevation: 285.7
Date: 24/Aug/2010
Test ID: Z-V1-C-02
Project: VAULT 1 L.P.
Client: SRNS
Job Site: Z AREA
Sleeve Stress
10 (tsf) 0
Tip Stress COR
(tsf) 600
Ratio COR
0 (%) 10
Pore Pressure
0 (tsf) 20
Elevation (ft)
205
189
173
157
141
125
S3
S4 - Santee
indicates 15 tsf
Maximum depth: 126.66 (ft)
Page 2 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75895.5 Easting: 66369.1 Elevation: 285.7000122	Date: 24/Aug/2010 Test ID: Z-V1-C-03 Project: VAULT 1 L.P.
	Client: SRNS Job Site: Z AREA	



Maximum depth: 156.69 (ft)
Page 1 of 2


Lankelma, Inc.
New Ellenton, South Carolina
(832) 364-1025
umeshbachu@lankelma.com
www.lankelmainc.com
Northing: 75895.5
Easting: 66369.1
Elevation: 285.7000122
Client: SRNS
Job Site: Z AREA
Date: 24/Aug/2010
Test ID: Z-V1-C-03
Project: VAULT 1 L.P.
Sleeve Stress
10 (tsf) 0
Tip Stress COR
(tsf) 600
Ratio COR
0 (%) 10
Pore Pressure
0 (tsf) 20
Elevation (ft)
205
189
173
157
141
125
S4 - Santee
2-ft soft zone
0.7-ft soft zone
0.3-ft soft zone
M1 - Warley Hill
indicates 15 tsf
Maximum depth: 156.69 (ft)
Page 2 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75750.1 Easting: 66417.3 Elevation: 285.7000122	Date: 24/Aug/2010 Test ID: Z-V1-C-04 Project: VAULT 1 L.P.
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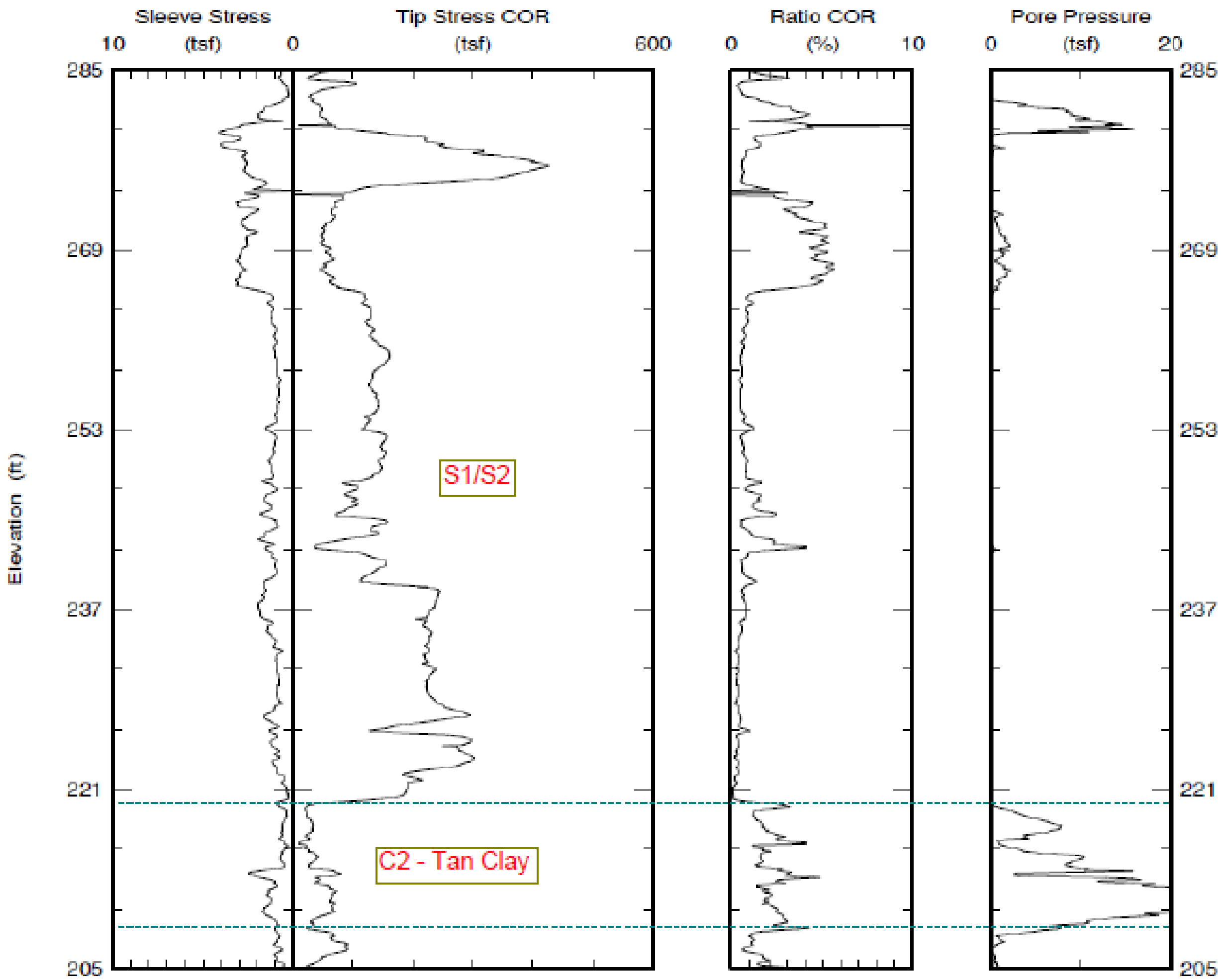
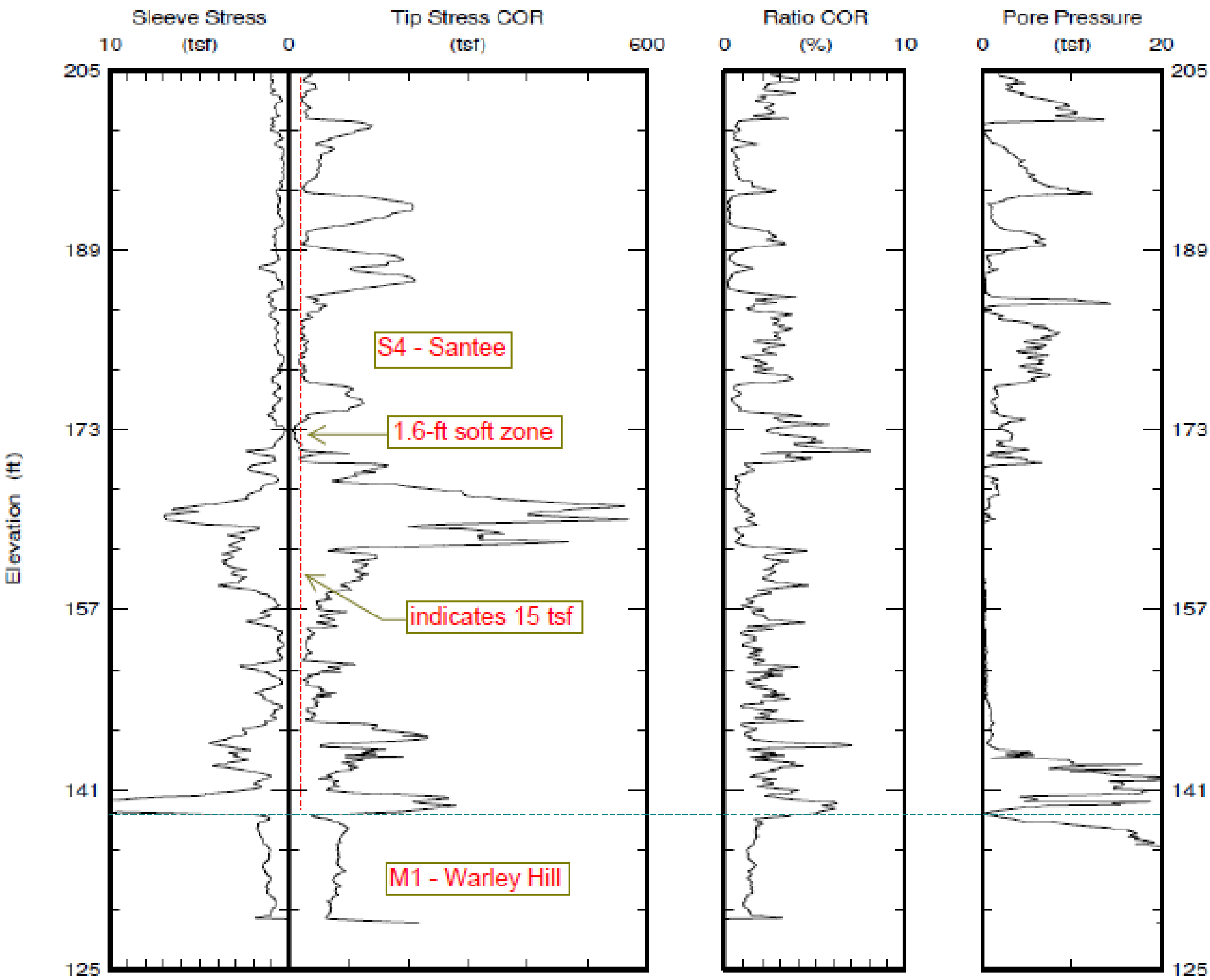


Maximum depth: 156.51 (ft)
Page 1 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75750.1 Easting: 66417.3 Elevation: 285.7000122	Date: 24/Aug/2010 Test ID: Z-V1-C-04 Project: VAULT 1 L.P.
	Client: SRNS Job Site: Z AREA	



Maximum depth: 156.51 (ft)
Page 2 of 2

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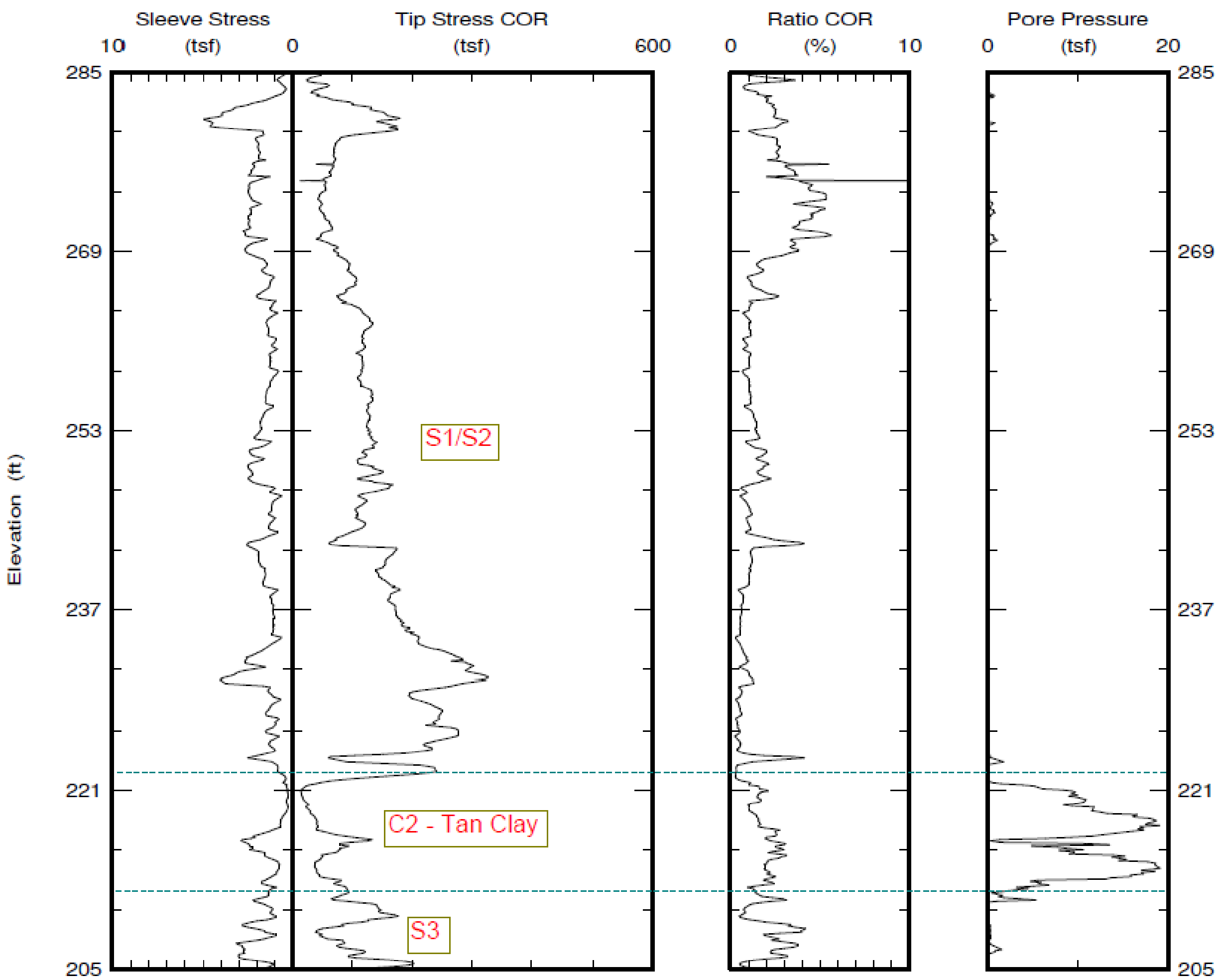

Maximum depth: 110.22 (ft)
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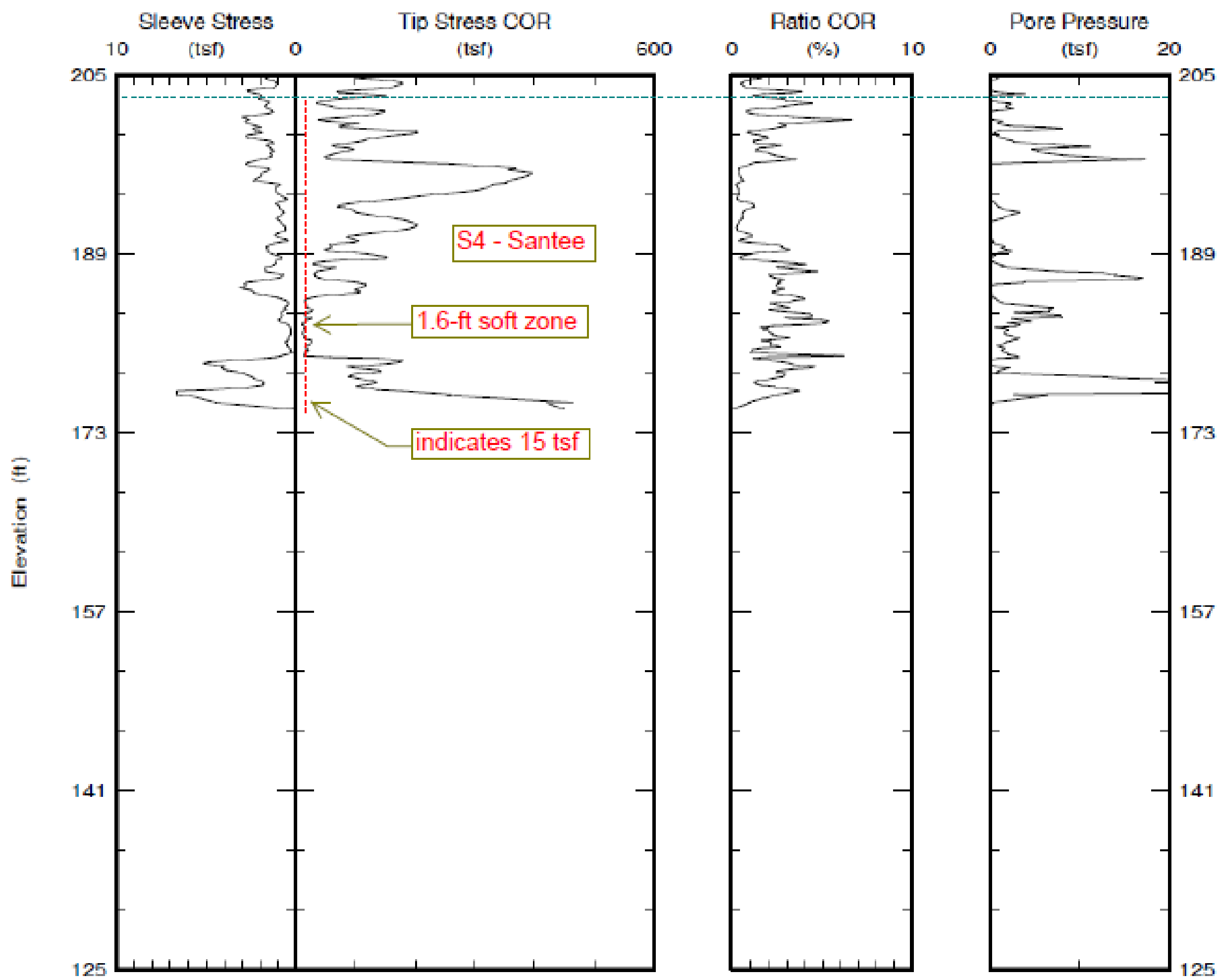
Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75619.9 Easting: 66459.9 Elevation: 285.3	Date: 23/Aug/2010 Test ID: Z-V1-C-05 Project: VAULT 1 L.P.
	Client: SRS Job Site: Z AREA	



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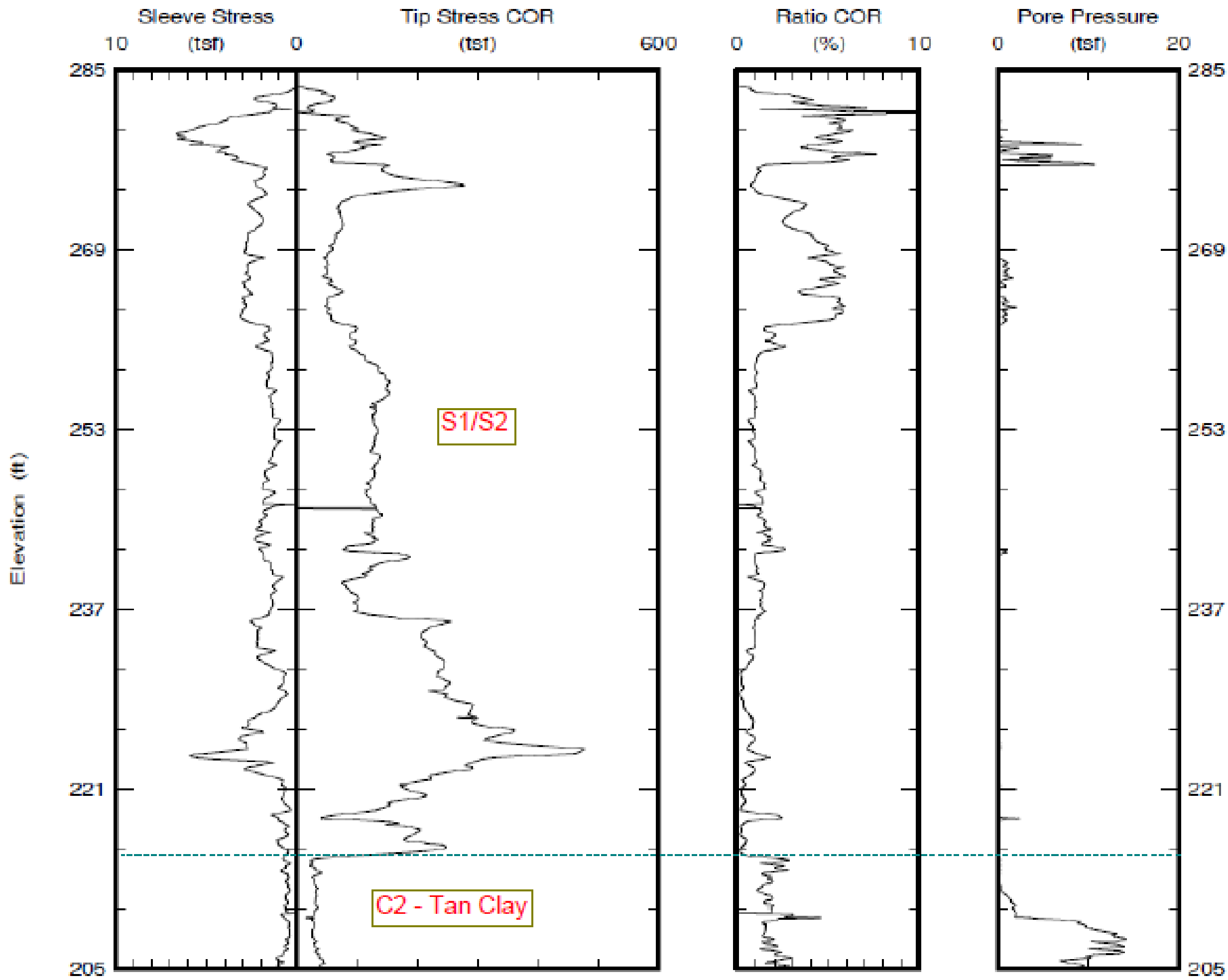
	Client: SRNS Job Site: Z AREA	



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Page 1 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75487.9 Easting: 66441.9 Elevation: 283.7	Date: 23/Aug/2010 Test ID: Z-V1-C-06 Project: VAULT 1 L.P.
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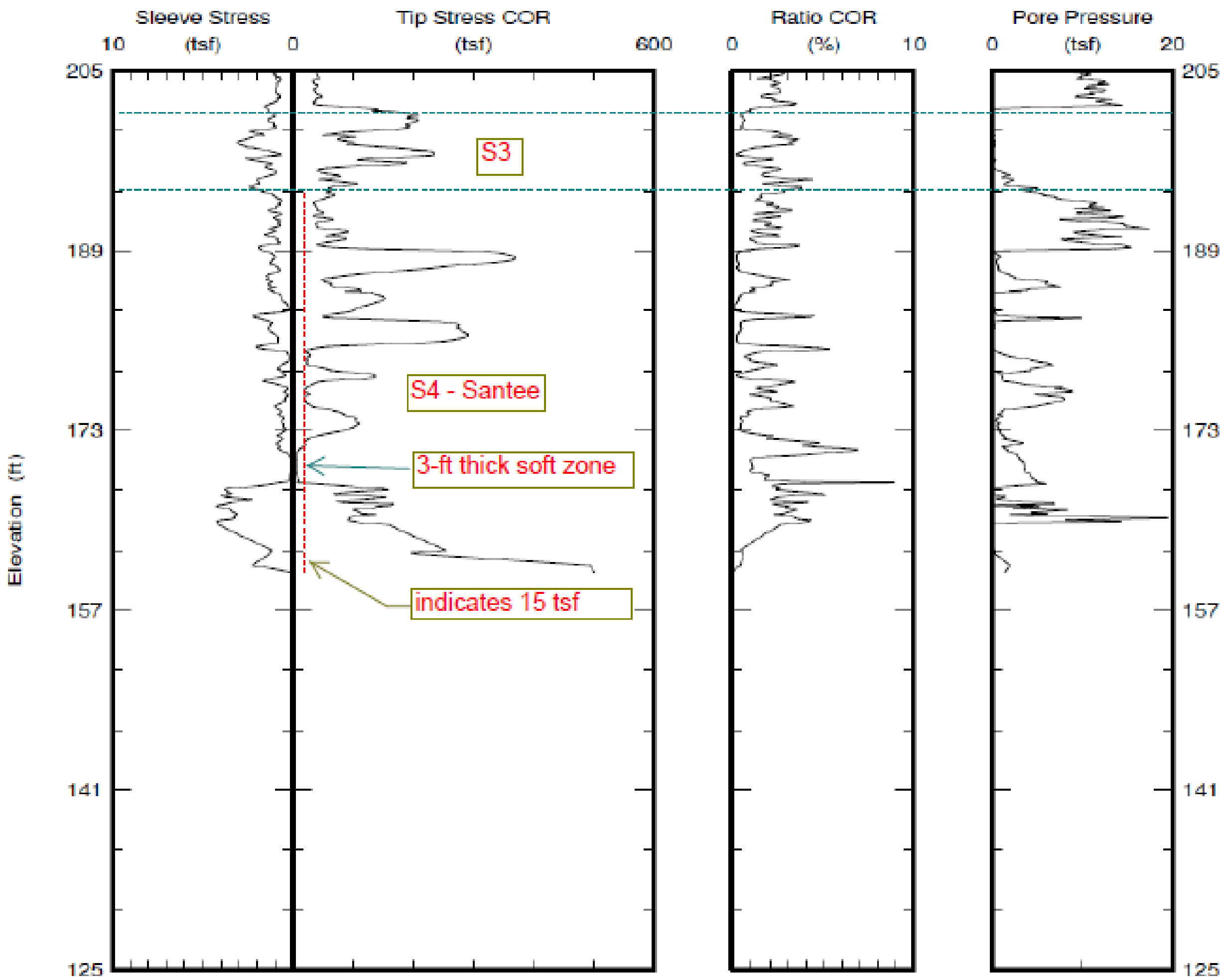

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Page 2 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75574.8 Easting: 66346.4 Elevation: 283.5	Date: 20/Aug/2010 Test ID: Z-V1-C-07 Project: VAULT 1 L.P.
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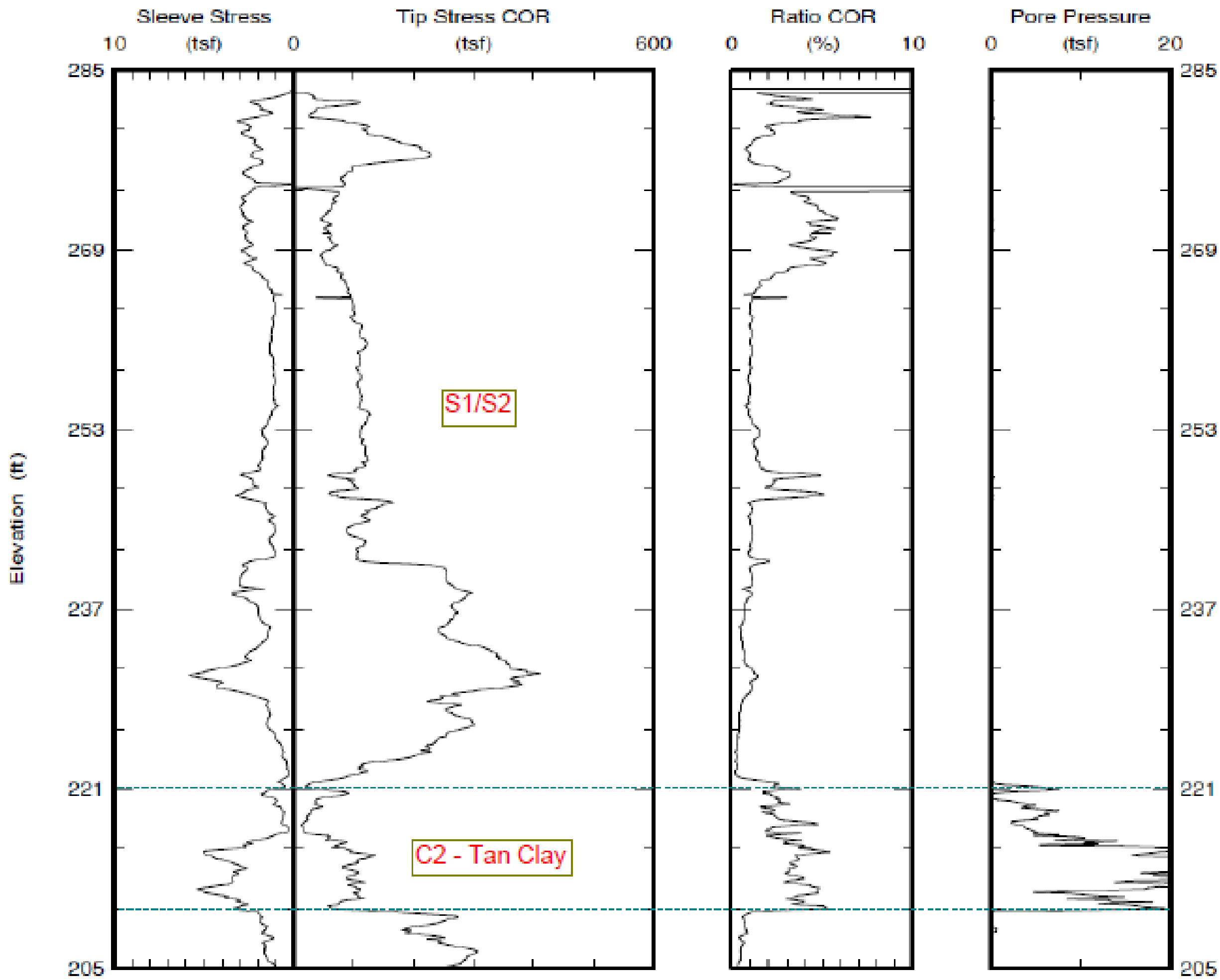

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Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75574.8 Easting: 66346.4 Elevation: 283.5	Date: 20/Aug/2010 Test ID: Z-V1-C-07 Project: VAULT 1 L.P.
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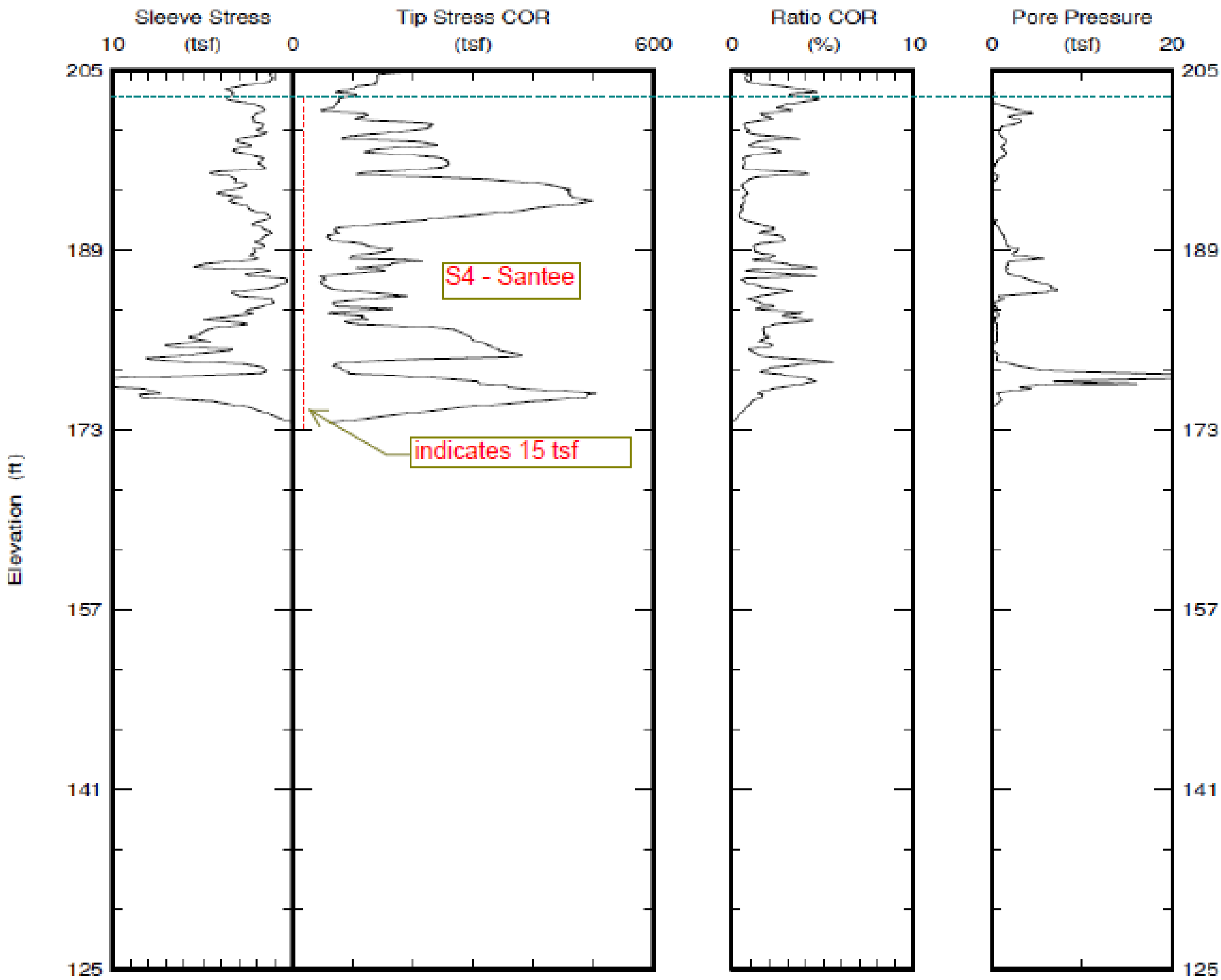

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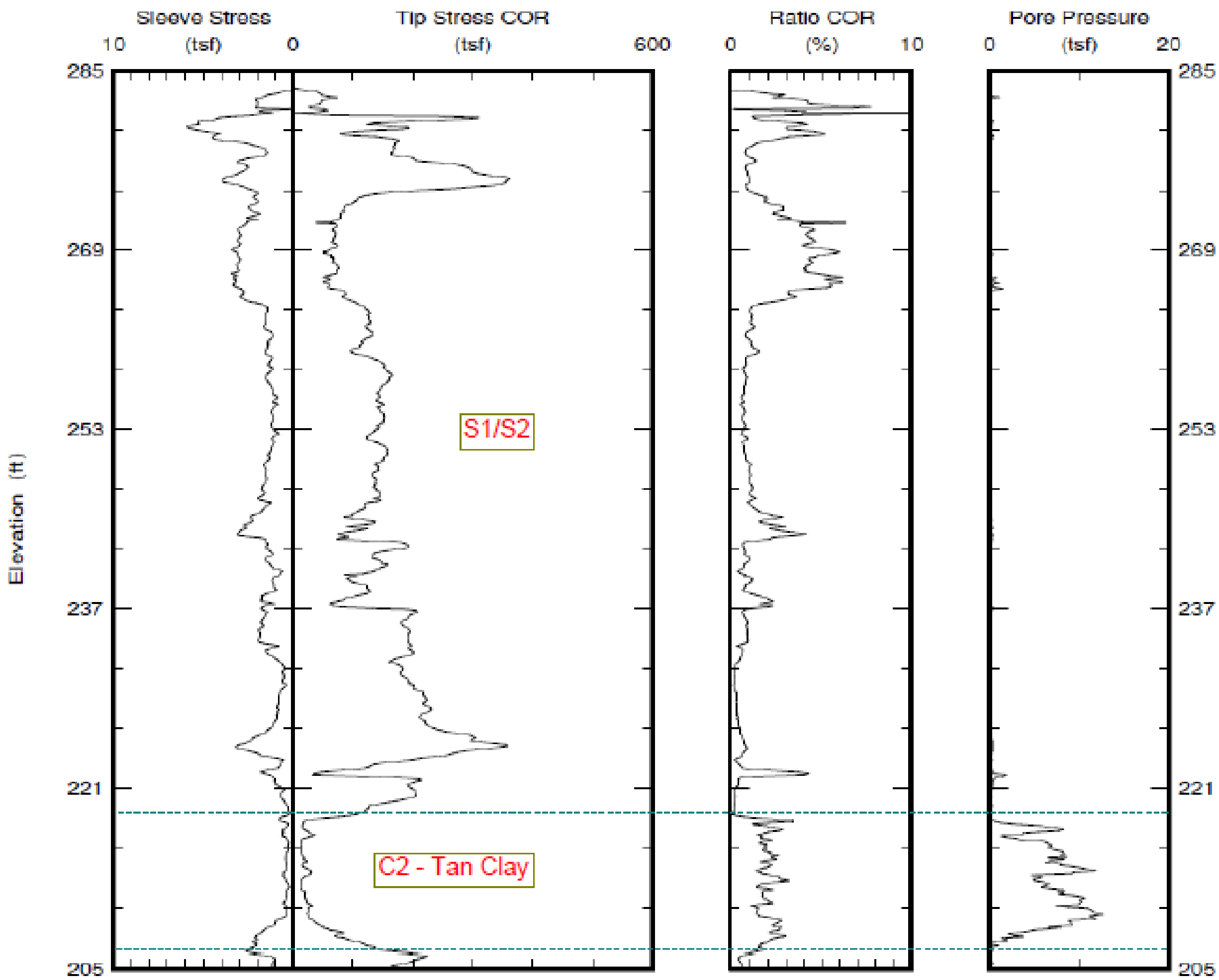

Maximum depth: 119.22 (ft)

Page 1 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75688.7 Easting: 66308.7 Elevation: 283.5	Date: 23/Aug/2010 Test ID: Z-V1-C-08 Project: VAULT 1 L.P.
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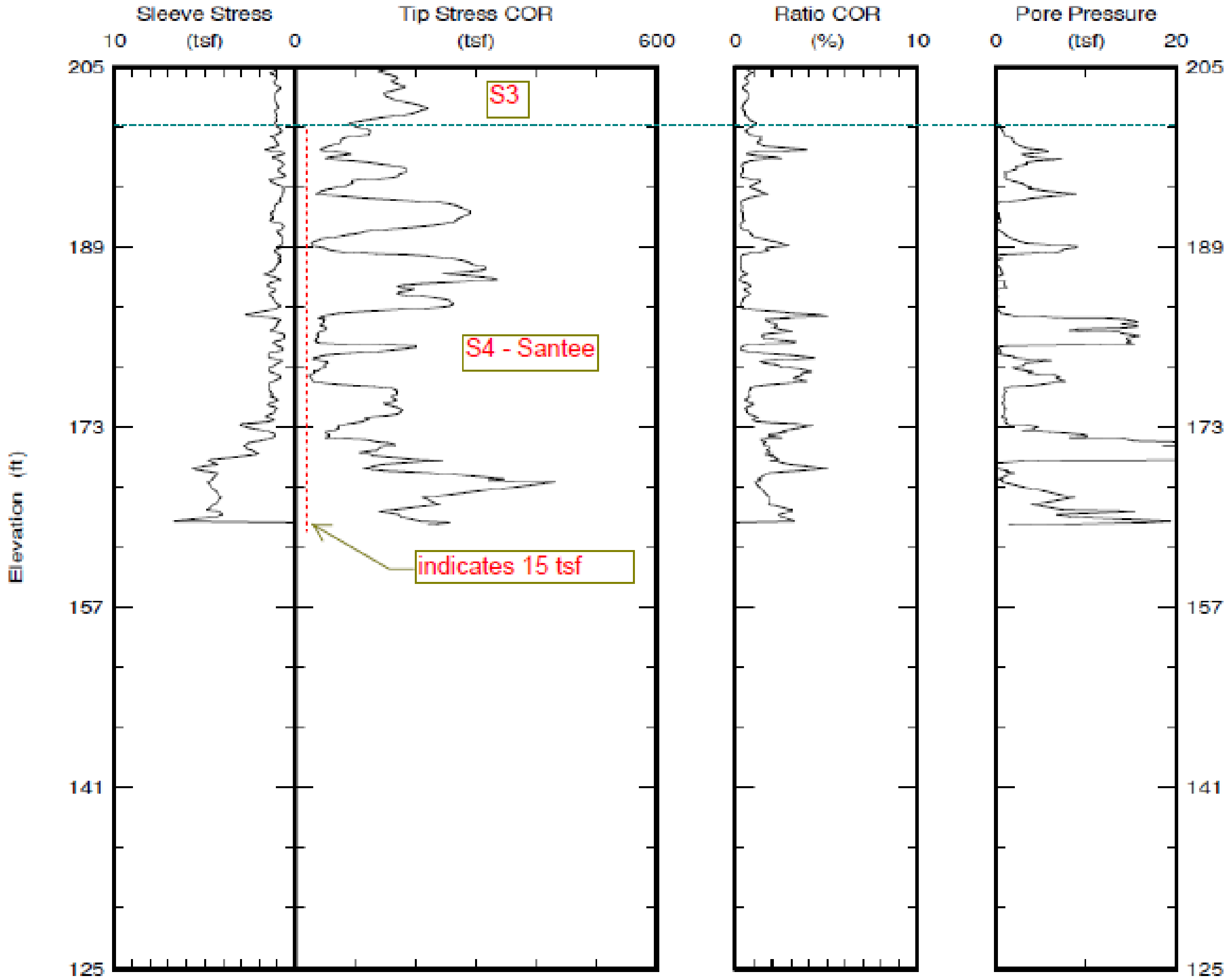

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Page 2 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75832.6 Easting: 66261.1 Elevation: 283.3	Date: 19/Aug/2010 Test ID: Z-V1-C-09 Project: VAULT 1 L.P.
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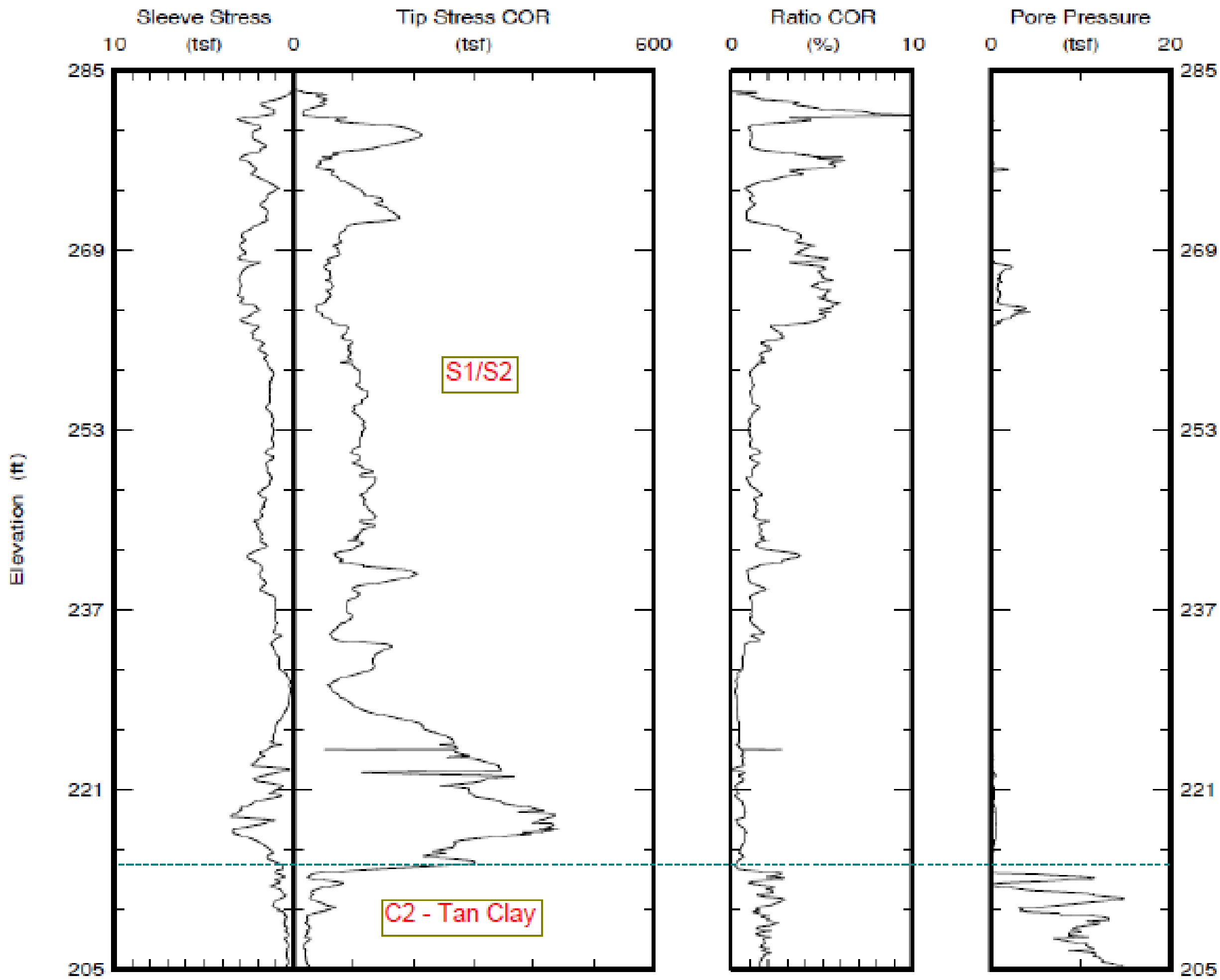

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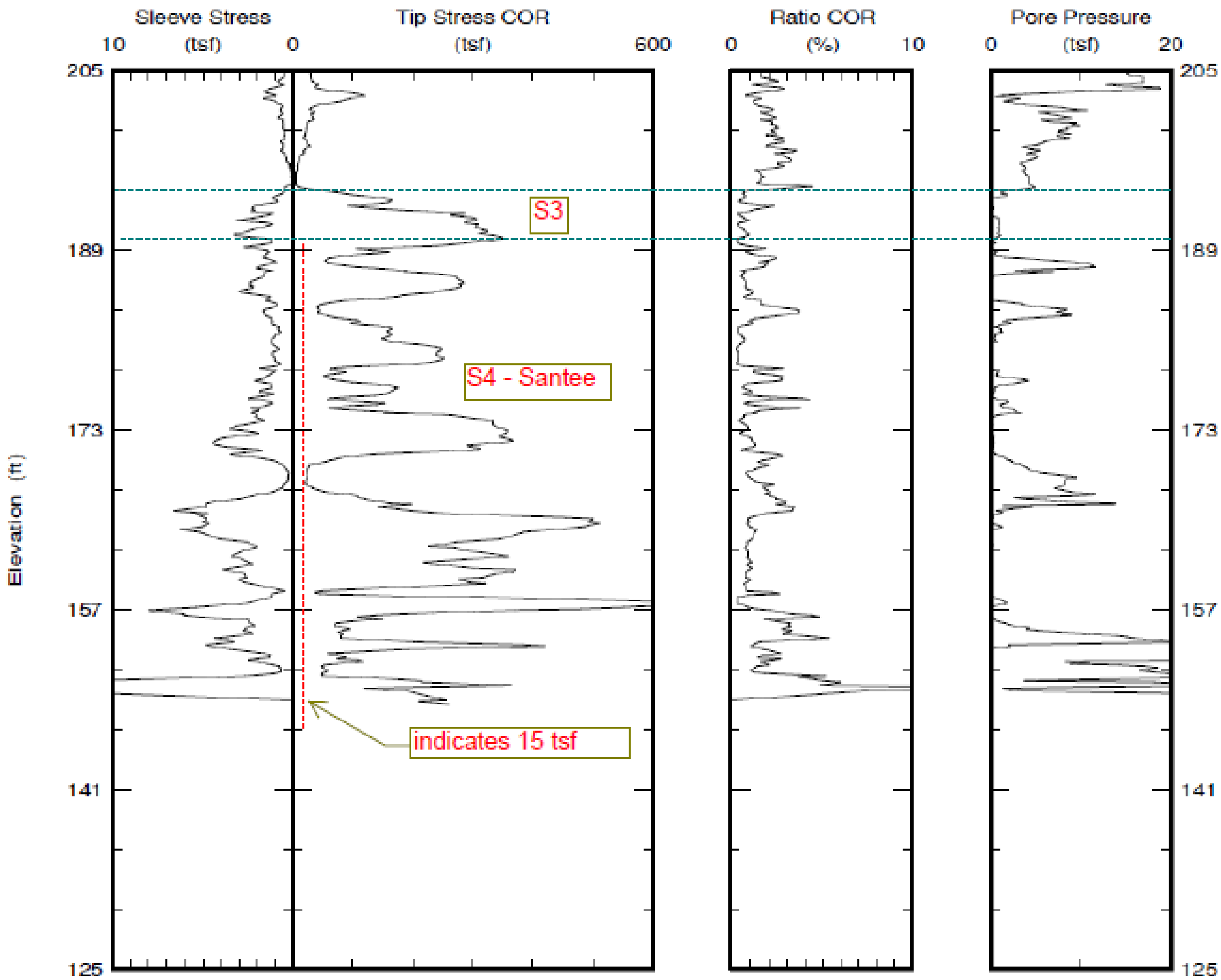
Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75832.6 Easting: 66261.1 Elevation: 283.3	Date: 19/Aug/2010 Test ID: Z-V1-C-09 Project: VAULT 1 L.P.
	Client: SRNS Job Site: Z AREA	



Maximum depth: 134.70 (ft)
Page 2 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75960.8 Easting: 66218.0 Elevation: 283.5	Date: 19/Aug/2010 Test ID: Z-V1-C-10 Project: VAULT 1 L.P.
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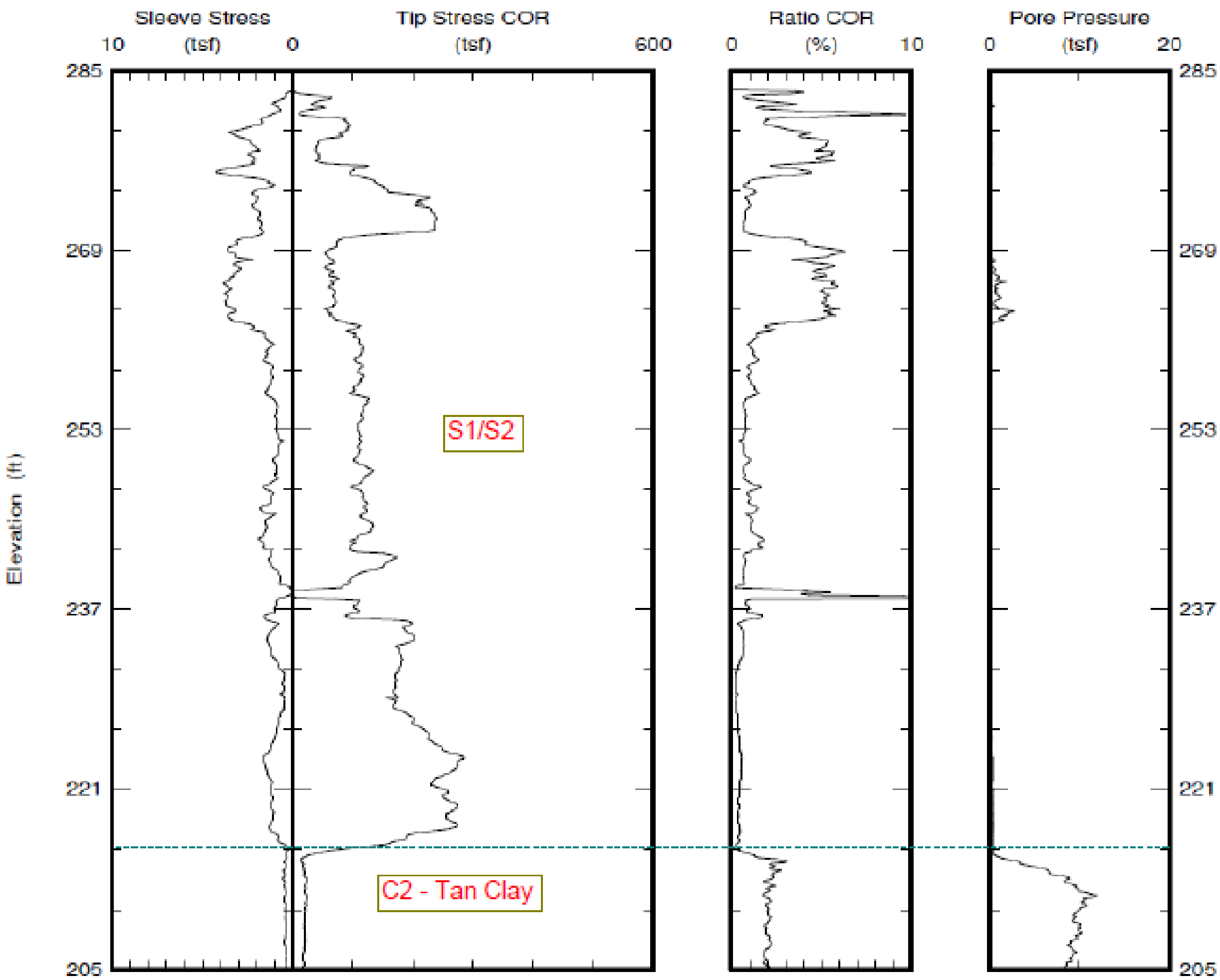
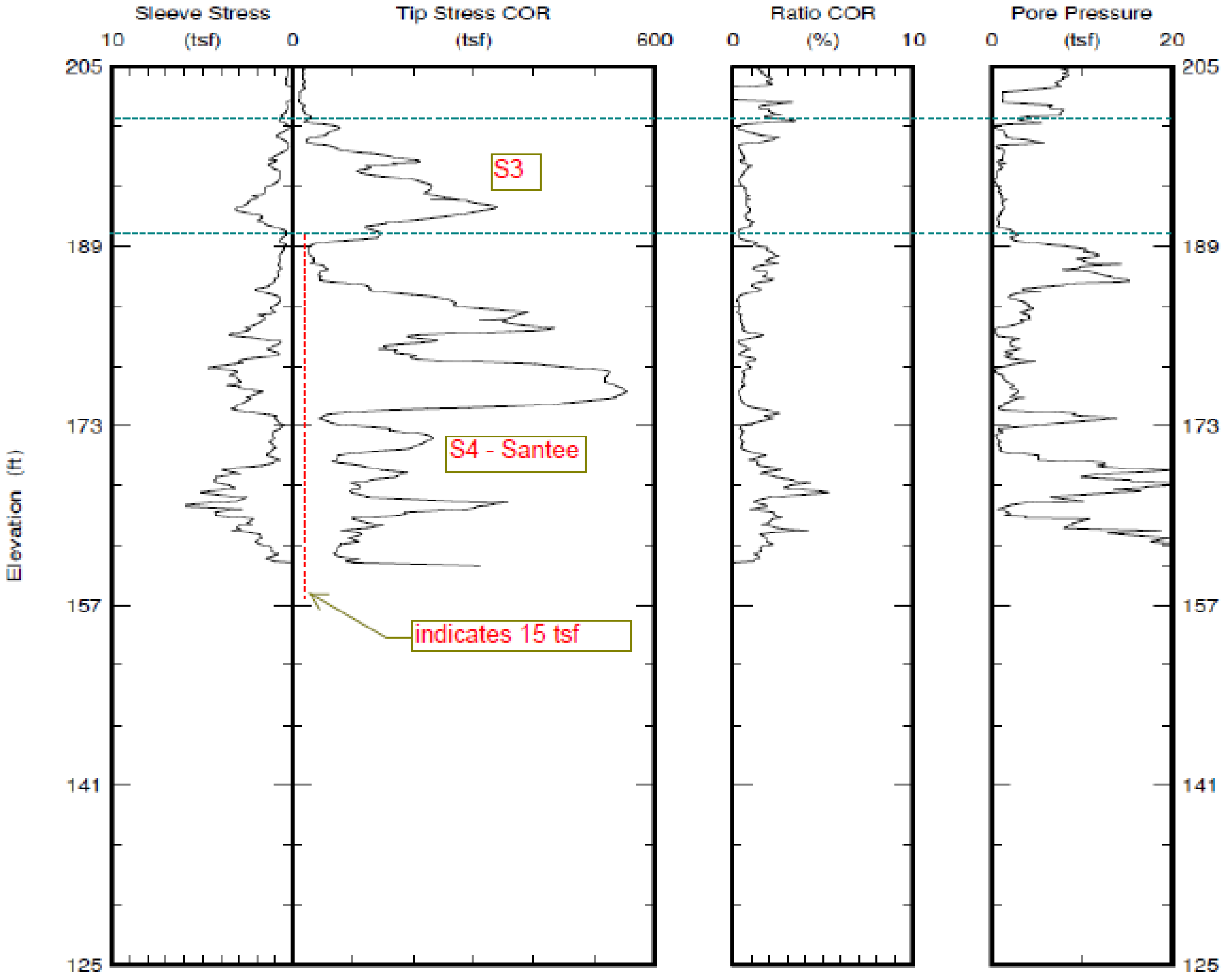


Maximum depth: 122.94 (ft)
Page 1 of 2

Lankelma, Inc. New Ellenton, South Carolina (832) 364-1025 umeshbachu@lankelma.com www.lankelmainc.com	Northing: 75960.8 Easting: 66218.0 Elevation: 283.5	Date: 19/Aug/2010 Test ID: Z-V1-C-10 Project: VAULT 1 L.P.
	Client: SRNS Job Site: Z AREA	



Maximum depth: 122.94 (ft)

Page 2 of 2

Appendix C

K-CLC-Z-00019, CPT Liquefaction Analysis for Saltstone Vault No. 1, Rev. 0, December 7, 2010

(32 pages)

OSR 45-24 (Rev 1-20-2009)

Calculation Cover Sheet

Project/Task	Calculation No.	Project/Task No.
Saltstone Disposal Unit 1	K-CLC-Z-00019	
Title: CPT-Based Liquefaction Analysis for Saltstone Vault 1	Functional Classification: PS	Sheet 1 of 32
	Discipline: Geotechnical	
Calculation Type: ☒ Type 1 ☐ Type 2	Type 1 Calc Status: ☐ Preliminary ☒ Confirmed	
Computer Program No. ☒ N/A	Version/Release No.	

Purpose and Objective	DC/RO	Date
Evaluate the liquefaction potential of the soil beneath Saltstone Vault #1 based on information obtained through cone penetration tests.	UNCLASSIFIED DOES NOT CONTAIN UNCLASSIFIED CONTROLLED NUCLEAR INFORMATION ADC & Reviewing Official [signature] A.A. Siddall for ACP (Name) Date: 12/10/2010	

Summary of Conclusion

See the Results and Conclusions Section.

Revisions

Rev #	Revision Description
0	Original

Sign Off

Rev #	Originator (Print) Sign/Date	Verification/Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign/Date
0	Aaron J. Geiger [signature] 11/29/10	☐ Design Check (GS/PS only) ☒ Document Review ☐ Qualification Testing ☐ Alternative Calculation ☐ Operational Testing	D. Bruce Nothdurft, PE [signature] 11/29/2010	William T. Li [signature] 11/30/2010
		☐ Design Check (GS/PS only) ☐ Document Review ☐ Qualification Testing ☐ Alternative Calculation ☐ Operational Testing		

Additional Reviewer (Print)	Signature	Date
N/A		
Design Authority (Print)	Signature	Date
N/A		
Release to Outside Agency (Print)	Signature	Date
N/A		

Security Classification of the Calculation

Unclassified

TABLE OF CONTENTS

1. Introduction

This calculation provides the evaluation of liquefaction potential for the Z-Area Saltstone Disposal Unit 1 based upon data from 10 new cone penetration tests (CPTs).

2. Input

2.1 Piezocone (Seismic and Non-Seismic) Penetration Tests.

Figure 1 shows the project site along with geotechnical exploratory locations. Four seismic cone piezocone penetration tests (SCPT) and six CPTs were performed at the project site. At the time of this calculation, the seismic data is still being evaluated and will be presented at a later date. Table 1 provides the list of the test locations (Ref. 1). For convenience, all tests will be referred to as CPTs.

Table 1-CPT Test Locations and Surface Elevations

CPT	Elev	Northing	Easting
Z-V1-C-01	284.1	76084.96	66243.05
Z-V1-C-02	285.7	76005.02	66333.05
Z-V1-C-03	285.7	75895.52	66369.19
Z-V1-C-04	285.7	75750.12	66417.30
Z-V1-C-05	285.4	75619.95	66459.92
Z-V1-C-06	283.7	75487.96	66441.91
Z-V1-C-07	283.5	75574.82	66346.47
Z-V1-C-08	283.5	75688.76	66308.74
Z-V1-C-09	283.4	75832.62	66261.18
Z-V1-C-10	283.5	75960.85	66218.03

2.2 Soil Properties and Stratigraphy

The stratigraphy at the project site was evaluated by K-CLC-Z-00020 (Ref. 1). Engineering layers are identified as Layers S1/2, C2, S3, S4, and M1. Layer locations are presented in Table 2. Layer S1/2 consists of Upland Formation, Tobacco Road Formation, and upper portion of the Dry Branch Formation; Layer C2 is the Tan Clay Unit within the Dry Branch Formation; Layer S3 is the lower portion of the Dry Branch Formation; Layer S4 is the Santee/Tinker Formation; and Layer M1 is the Warley Hill Formation. Table 4 provides the elevations of the layers at each CPT location. For this calculation liquefaction analysis is performed for the saturated portions of Tobacco Road and Dry Branch Formations. The Santee/Tinker Formation is assumed too deep for liquefaction. Liquefaction analyses in this calculation were terminated at the top of the Santee/Tinker Formation (i.e., the top of the S4 layer). Dynamic settlement in the Santee/Tinker Formation is due to “soft zones” and is covered in another calculation.

Table 2-Engineering Stratigraphy Based on CPTs (Elevations are at top of layer)

CPT ID	Ground Elevation (ft, msl)	C2	S3	S4	M1	SRS_N	SRS_E
Z-V1-C-01	284.1	210	201	194	134	76084.0	66243.0
Z-V1-C-02	285.7	217	205	201	NDE	76005.0	66333.1
Z-V1-C-03	285.7	214	NP	195	142	75895.5	66369.2
Z-V1-C-04	285.7	220	208	205	140	75750.1	66417.3
Z-V1-C-05	285.4	222	212	204	NDE	75620.0	66459.9
Z-V1-C-06	283.7	216	202	194	NDE	75488.0	66441.9
Z-V1-C-07	283.5	221	210	204	NDE	75574.8	66346.5
Z-V1-C-08	283.5	219	206	200	NDE	75688.8	66308.7
Z-V1-C-09	283.4	215	194	190	NDE	75832.6	66261.2
Z-V1-C-10	283.5	216	199	190	NDE	75960.9	66218.0

NP=Not Present

NDE=Not Deep Enough

Unit weights for the Tobacco Road and Dry Branch formations vary between 115 and 130 pcf, as determined from previous geotechnical investigations. 115 pcf is assumed for this calculation as was done in the SPT liquefaction calculation (Ref 1).

2.3 Groundwater Elevation

The current groundwater elevation at the project site is 230 feet, MSL with estimated seasonal fluctuation of + 5 feet (Ref. 2). For this calculation a conservative groundwater elevation of 240 feet, MSL is used. Only saturated soils are analyzed for potential liquefaction.

2.4 Design Basis Earthquake

SRS Standard 1060 Rev. 10 (Ref. 3) recommended a design response spectrum for analysis. The acceleration recommended by this standard for a SDC-1 structure is 0.16g. For a SDC-2 structure, 0.24g is recommended. Previous studies for saltstone vaults in Z-Area have used 0.2g, therefore 0.2g was used in this calculation.

3. Evaluation and Computation

3.1 Evaluation of Liquefaction Potential

A modified version of the Simplified Procedure for Evaluating Soil Liquefaction Potential (Refs. 5, 6, and 7) was used to evaluate the liquefaction potential. This procedure computes the factor of safety:

Factor of Safety = CRR/CSR

where CRR is the cyclic resistance ratio and CSR is the cyclic stress ratio,

$CRR = \tau_{ave} / \sigma'_{vo}$

where τ_{ave} is average shear stress required to induce liquefaction and
σ'_{vo} is the effective vertical overburden stress,

$CSR = \tau_{ave} / \sigma'_{vo}$

where τ_{ave} is the average shear stress induced by the earthquake and
σ'_{vo} is the effective vertical overburden stress.

3.2 Computation of CSR

The simplified method for calculating CSR is:

$$CSR = \tau_{ave} / \sigma'_{vo} = 0.65(a_{max} / g)(\sigma_{vo} / \sigma'_{vo})\, r_d$$

where a_{max} is the maximum horizontal acceleration at ground surface, g is the gravitation acceleration, σ_{vo} is the total vertical overburden stress, σ'_{vo} is the effective vertical overburden stress , r_d is the stress reduction factor and is calculated as

$r_d = 1.000 - 0.00765\, z$ for $z \leq 9.15$ m
$= 1.174 - 0.0267\, z$ for $9.15 \text{ m} < z \leq 23$ m
$= 0.744 - 0.0080\, z$ for $23 \text{ m} < z \leq 30$ m
$= 0.500$ for $z > 30$ m
where z is the depth in meters.

$$\text{Factor of Safety} = CRR_{7.5}\, K_\sigma\, K_{age}\, K_\alpha\, MSF/CSR$$

where K_α is the static driving shear correction factor. For this calculation, no K_α correction was used (i.e., $K_\alpha = 1.0$). K_{age} is the age factor. Because aging is incorporated in the site specific equations used to compute the CRR, no additional correction is made ($K_{age} = 1$). K_σ is the static effective overburden correction factor. For this calculation a site specific K_σ relationship is used, $K_\sigma = 1.009376 - 0.18326 \log(\sigma'_{vo}) - 0.08340 \log(\sigma'_{vo})^2$ (Ref. 8), where σ'_{vo} is the effective vertical overburden pressure in tsf. Figure 2 shows the equation. The NCEER recommended K_σ curves (Ref. 7) are also shown for comparison. MSF is the earthquake magnitude scaling factor. Values of the MSF are provided in Figure 3. The equation derived from the various relationships is

$$MSF = 27.3e^{-0.44 \times Mw}$$

3.3 Computation of CRR using CPTu Tip Resistance

Laboratory testing and development of the most recent SRS CRR curves are discussed in SRS (2007a) (Ref. 9) and by Lewis and Arango (Ref. 10). The SRS CRR relationship has adopted the shape of the Idriss and Boulanger (Ref. 11) CRR relationship. The SRS clean curve is the Idriss and Boulanger relationship multiplied by a factor of 1.3 to account for strength gain due to aging. The Idriss and Boulanger CRR clean curve (i.e., fines content ≤ 5%) is given below.

$$CRR = \exp\left[\left(\frac{(q_t)_1}{540}\right) + \left(\frac{(q_t)_1}{67}\right)^2 - \left(\frac{(q_t)_1}{80}\right)^3 + \left(\frac{(q_t)_1}{114}\right)^4 - 3\right]$$

where q_{t1} is the tip stress normalized for overburden stress at the time of data collection. This same curve is multiplied by increasing factors to account for aging and increased fines content. The multipliers for various fines contents are provided in Table 3.

Table 3-Multipliers for CRR based on fines content.

Fines Content (%)	SRS Multiplier for CRR Curve
≤ 5	1.3
10	1.6
15	2.1
20	2.6
25	3.1
≥30	3.4

The tip stress (qt) is normalized using the following equation (Ref. 8):

$q_{t1} = C_Q\, q_t$
where C_Q is the CPTu overburden normalization factor, computed from

$C_Q = (P_a / \sigma'_{vo})^n$ with $C_Q \le 1.7$
where P_a is the atmospheric pressure
σ'_{vo} is the effective vertical overburden pressure at time of testing
n is an exponent ranges between 0.5 for clean sand and 1.0 for clays

For this calculation the n was varied linearly, based on percent fines, between 0.5 for clean sand (i.e., fines ≤ 5%) and 1.0 for clays (i.e., fines ≥ 50%).

To use the SRS CRR equations it is necessary to determine percent fines. Percent fines were estimated using the following equation (Ref. 12):

Percent Fines = $29.47\ I_c^{\ 1.21} - 0.09$

where $I_c = [(1.60 - \log Q_t)^2 + (\log F_r + 0.41)^2]^{0.5}$

Q_t is a normalized tip stress, $Q_t = (q_t - \sigma_{vo}) / \sigma'_{vo}$

F_r is stress normalized friction ratio, $F_r = [f_s / (q_t - \sigma_{vo}) \times 100]$

q_t is the CPTu tip stress corrected for unequal area effects

f_s is the CPTu sleeve friction

σ_{vo} is the total vertical overburden stress

σ'_{vo} is the effective vertical overburden stress

3.4 Dynamic Settlement above the Santee Formation

Settlement due to liquefaction and partial liquefaction were calculated based on SRS site specific testing. It was assumed that all liquefiable and partially liquefiable zones within the profile will settle and the resulting settlement will be cumulative at the surface. Total cumulative settlement resulting from liquefaction and partial liquefaction is estimated for the profile by summing the liquefaction settlement (i.e., FS ≤ 1.15) and partial liquefaction settlement (i.e., 1.15 < FS ≤ 2.2) for each increment:

$$S_{Total} = \Sigma\ S_{Liq} + \Sigma\ S_{P\ Liq}$$

where S_{Total} is the cumulative settlement,

S_{Liq} is the settlement of the increment due to liquefaction, and

$S_{P\ Liq}$ is the settlement of the increment due to partial liquefaction.

Where S_{Liq} is the volumetric strain due to liquefaction multiply by dz

$S_{P\ Liq}$ is the volumetric strain due to partial liquefaction multiply by dz

dz is the thickness of the increment.

Soils with a factor of safety > 2.2 are considered to be non-liquefiable. No settlement is expected for Factors of Safety greater than 2.2:

3.4.1 Volumetric Strain Curves

The volumetric strain curves developed for SRS (Refs. 13 and 14) using H-Area data are used for computing settlement. They can be applied to Z-Area and are presented below and on the next page:

for $qt_1 = 160$, $0.4 < FS < 1.15$ strain(%) = 0.65

for $qt_1 = 130$, $0.4 < FS < 1.15$

strain(%) = $2.9883 + 10.354(FS)^4 - 30.258(FS)^3 + 30.7(FS)^2 - 13.064(FS)$

for $qt_1 = 100$, $0.4 < FS < 1.15$

$$strain(\%) = 2.0308 + 8.3929(FS)^4 - 21.111(FS)^3 + 16.12(FS)^2 - 4.5756(FS)$$

for $qt_1 = 50$, $0.4 < FS < 0.65$

$$strain(\%) = - 41.6495 - 756.666(FS)^4 + 1505.222(FS)^3 - 1123.65(FS)^2 + 371.2387(FS)$$

for $qt_1 = 50$, $0.65 < FS < 1.15$

$$\log strain(\%) = 1.256225 - 0.21100(FS)^2 - 1.01242(FS)$$

for $qt_1 = 30$, $0.4 < FS < 0.65$

$$strain(\%) = - 45.4815 - 830.0000(FS)^4 + 1651.074(FS)^3 - 1231.64(FS)^2 + 406.5062(FS)$$

for $qt_1 = 30$, $0.65 < FS < 1.15$

$$\log strain(\%) = 1.181442 - 0.47909(FS)^2 - 0.63184(FS)$$

for $qt_1 = 20$, $0.4 < FS < 0.65$

$$strain(\%) = -45.2315 - 830.0000(FS)^4 + 1651.074(FS)^3 -1231.64(FS)^2 + 406.5062(FS)$$

for $qt_1 = 20$, $0.65 < FS < 1.15$

$$\log strain(\%) = 0.679601 - 1.17026(FS)^2 + 0.616392(FS)$$

for $qt_1 = 10$, $0.4 < FS < 0.65$

$$strain(\%) = - 29.6577 - 560.0000(FS)^4 + 1114.074(FS)^3 - 836.066(FS)^2 + 278.6576(FS)$$

for $qt_1 = 10$, $0.65 < FS < 1.15$

$$\log strain(\%) = 0.454166 - 1.56185(FS)^2 + 1.272068(FS)$$

for $qt_1 = 5$, $0.4 < FS < 0.65$

$$strain(\%) = - 29.7775 - 566.666(FS)^4 + 1127.333(FS)^3 - 845.883(FS)^2 + 281.8638(FS)$$

for $qt_1 = 5$, $0.65 < FS < 1.15$

$$\log strain(\%) = 0.367762 - 1.73636(FS)^2 + 1.555255(FS)$$

for partial liquefaction $1.15 < FS < 1.6$ and all qt_1 values

$$\log strain(\%) = 1.256225 - 0.21100(FS)^2 - 1.01242(FS)$$

for partial liquefaction $1.6 < FS < 2.2$ and all qt_1 values

$$strain(\%) = 0.728794 + 0.100221(FS)^2 - 0.54090(FS)$$

Figure 4 shows the relation between the volumetric strain and the Factors of Safety for various

qt_1.

3.4.2 Liquefaction Hazard Weighting

Table 4 summarizes the PGA hazard for the 2,500 year earthquake (Ref. 4). The weight for a given magnitude range is the sum of the weights for the various distances for the given magnitude. Settlement due to liquefaction and partial liquefaction is calculated different magnitudes. The magnitudes selected are the midpoints of the magnitude ranges given in Table 4 (i.e. 4.75, 5.25, 5.75, 6.25, 6.75, 7.5) and in Figure 5. The results are then weighted according to the PGA hazard disaggregation.

Table 4-USGS Seismic Hazard Disaggregation Weighting

USGS Seismic Hazard for Savannah River Site
Peak Ground Acceleration Return Period 2,500 years Rock PGA = 0.15g

Distance	Mw = 4.5 to 5.0	Mw = 5.0 to 5.5	Mw = 5.5 to 6.0	Mw = 6.0 to 6.5	Mw = 6.5 to 7.0	Mw = 7.0 to 8.0
7.5 km	7.13	3.56	1.73	0.78	0.42	0.00
20 km	4.93	3.64	2.35	1.28	0.78	0.00
37.5 km	3.11	3.90	4.04	3.24	2.72	0.00
75 km	0.33	0.93	2.05	3.24	0.00	16.45
150 km	0.03	0.14	0.59	1.68	0.00	29.25
250 km	0.00	0.00	0.00	0.04	0.00	1.63
550 km	0.00	0.00	0.00	0.00	0.00	0.08
Mw Bin Sum =	15.52	12.16	10.76	10.25	3.92	47.40

4. Results and Conclusion

As seen in the results in Appendix B for a magnitude 7.5 earthquake, the majority of the soil column above the Santee/Tinker formation could either experience partial liquefaction or no liquefaction. There are only a few locations which could completely liquefy. The CPT-based liquefaction potential results compare well with those found in the SPT-based analysis (Ref. 15).

Table 5 summarizes the liquefaction induced settlement based on the SRS CRR curves. The settlements were weighted according to the PSHA hazard disaggregation. The average weighted liquefaction settlement above the Santee/Tinker layer is 0.32 inches whereas the average for a magnitude of 7.5 is 0.65 inches. Settlements at individual CPT locations for a magnitude of 7.5 vary from 0.37 inches to 1.39 inches and vary between 0.17 inches to 0.68 inches using the hazard disaggregation. A conservative maximum liquefaction settlement is 1.5 inches. Assuming that the differential settlement is the difference between the minimum and maximum settlement for a given magnitude, the maximum differential settlement that can be expected is approximately 1 inch.

Table 5-Summary of Liquefaction Settlement Calculations

	Settlement (inches)						
	Mw= 4.75	Mw= 5.25	Mw= 5.75	Mw= 6.25	Mw= 6.75	Mw= 7.5	2,500 Yr Weighting
Z-V1-C-01	0.00	0.00	0.00	0.02	0.07	0.59	0.28
Z-V1-C-02	0.00	0.00	0.00	0.01	0.07	0.56	0.27
Z-V1-C-03	0.00	0.00	0.00	0.02	0.07	0.58	0.28
Z-V1-C-04	0.00	0.00	0.00	0.01	0.04	0.37	0.18
Z-V1-C-05	0.00	0.00	0.00	0.01	0.05	0.40	0.19
Z-V1-C-06	0.00	0.00	0.00	0.01	0.05	0.50	0.24
Z-V1-C-07	0.00	0.00	0.00	0.02	0.06	0.34	0.17
Z-V1-C-08	0.00	0.00	0.00	0.01	0.08	0.58	0.28
Z-V1-C-09	0.00	0.00	0.02	0.09	0.27	1.39	0.68
Z-V1-C-10	0.00	0.00	0.01	0.07	0.20	1.18	0.58
				Average:		0.65	0.32

5. References

1. SRS Calculation No. K-CLC-Z-00020, Stratigraphy for Saltstone Disposal Unit 1, Rev. 0, September 2010

2. WSRC-2003-00250, Rev. 0, “An Updated Regional Water Table of the Savannah River Site and Related Coverages,” December 2003.

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Ratio Curves (U), July 2007.

11. Idriss, I.M. and Boulanger, R.W., 2004. "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," Proceedings of the Joint 11th International Conference on Soil Dynamics and Earthquake Engineering and the 3rd International Conference on Earthquake Geotechnical Engineering, Berkeley, CA 32-56.

12. SRS calculation No. K-CLC-G-00065, CPTu Fines Content Determination, Rev. 0, August 2001.

13. SRS calculation No. K-CLC-H-00064, Volumetric Strain, Nov. 1994.

14. SRS calculation No. C-CLC-H-00815, Probabilistic Liquefaction Settlement Evaluation for ITP (U), Rev. 0, Oct. 2006.

15. SRS calculation No. K-CLC-Z-00018, Updated SPT Liquefaction Analysis for Saltstone Disposal Unit 1, Rev. 0, July 2010.

Figures

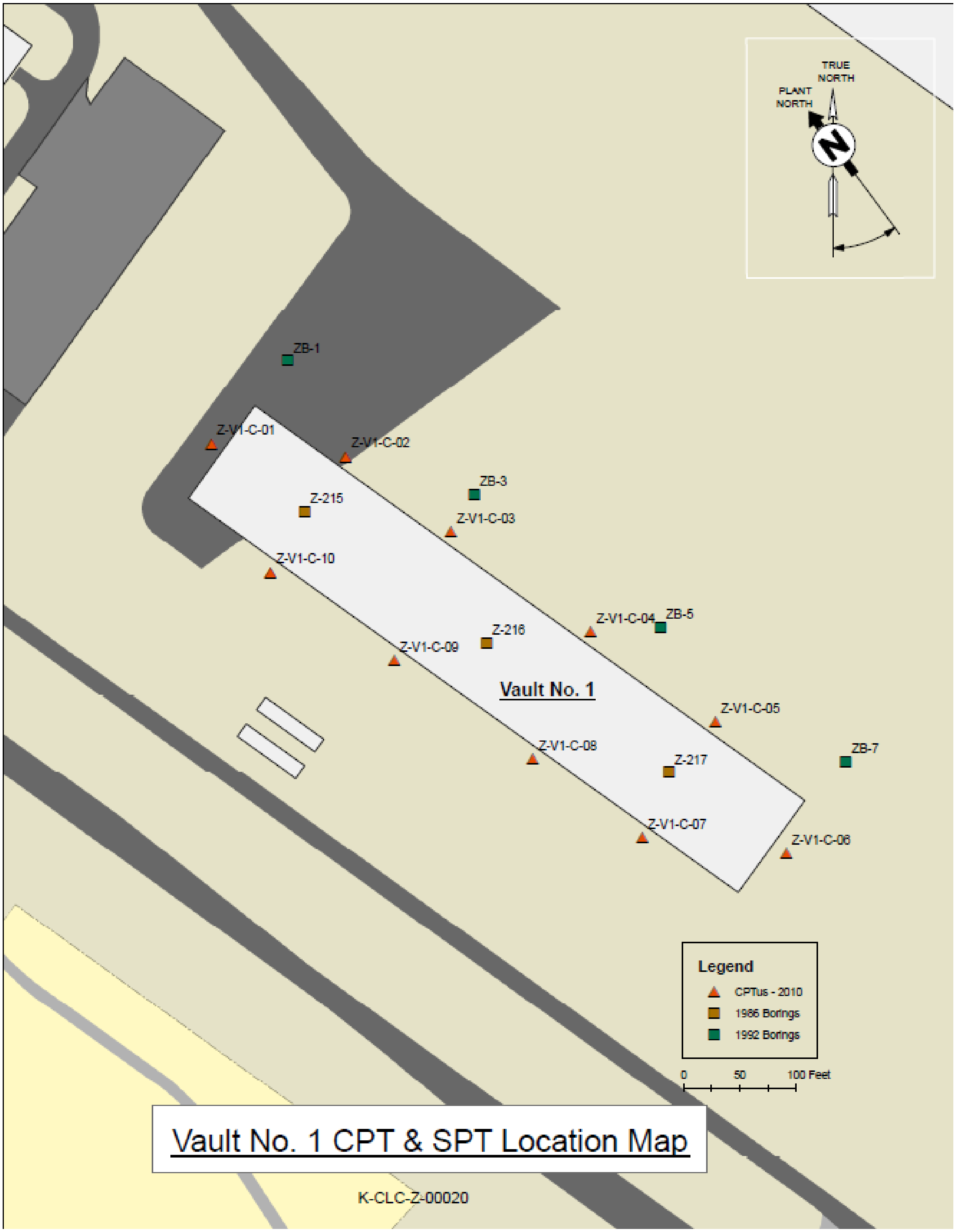

Figure 1 Map of CPT Locations Surrounding Vault No. 1 (Oriented with SRS North)

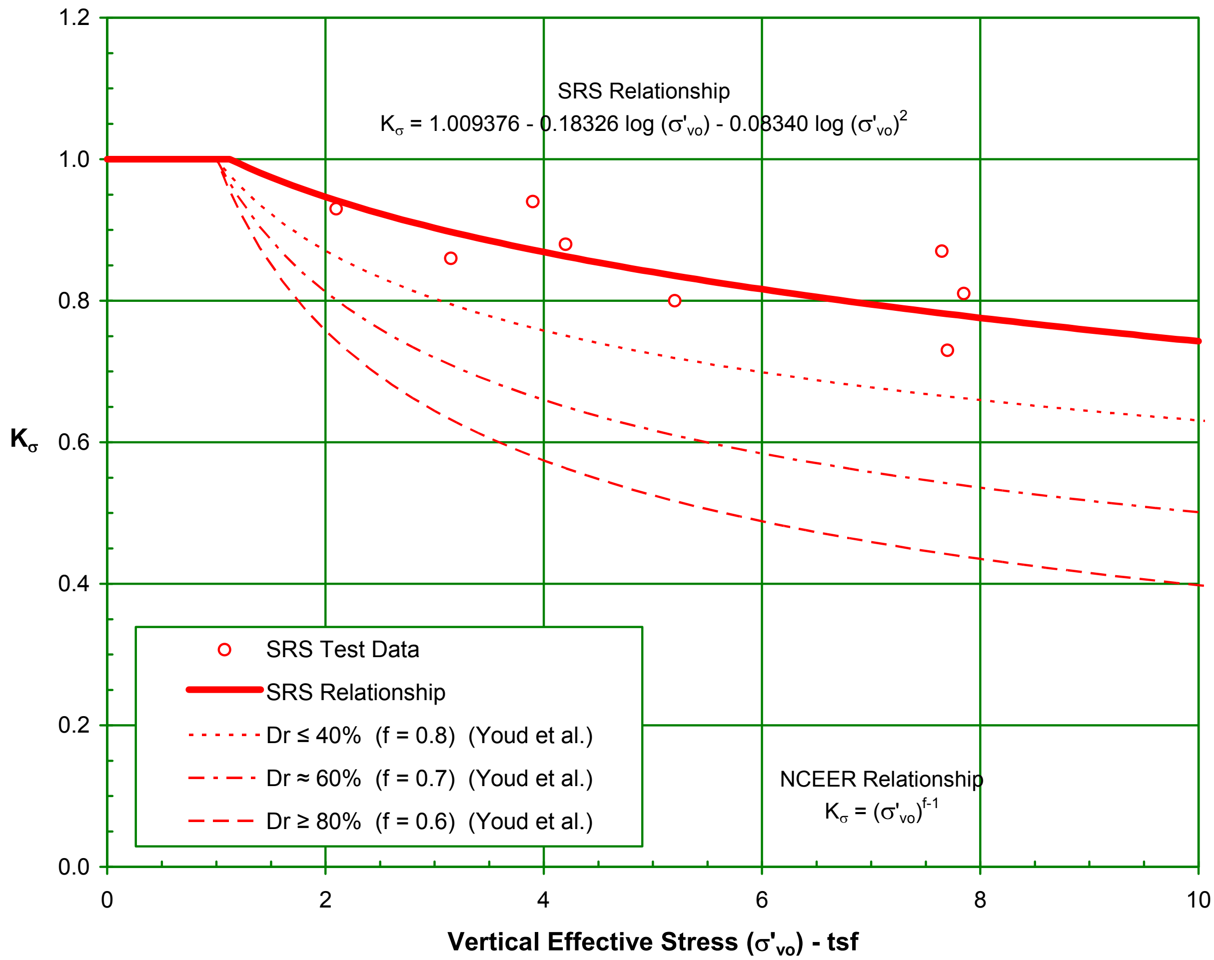

Figure 2 -Comparison of effective overburden correction factors (Kσ)

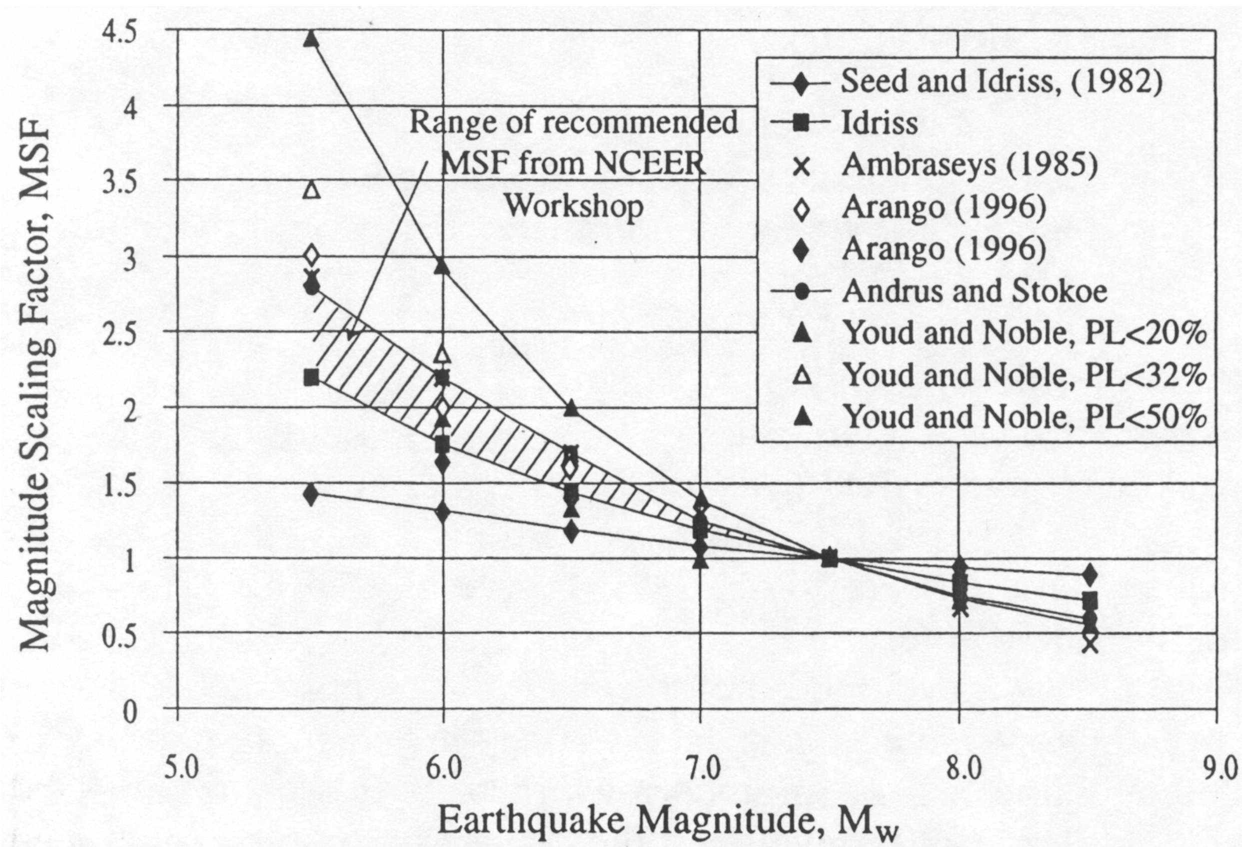

Figure 3 -Magnitude scaling factors proposed by various Investigators with range recommended by NCEER workshop

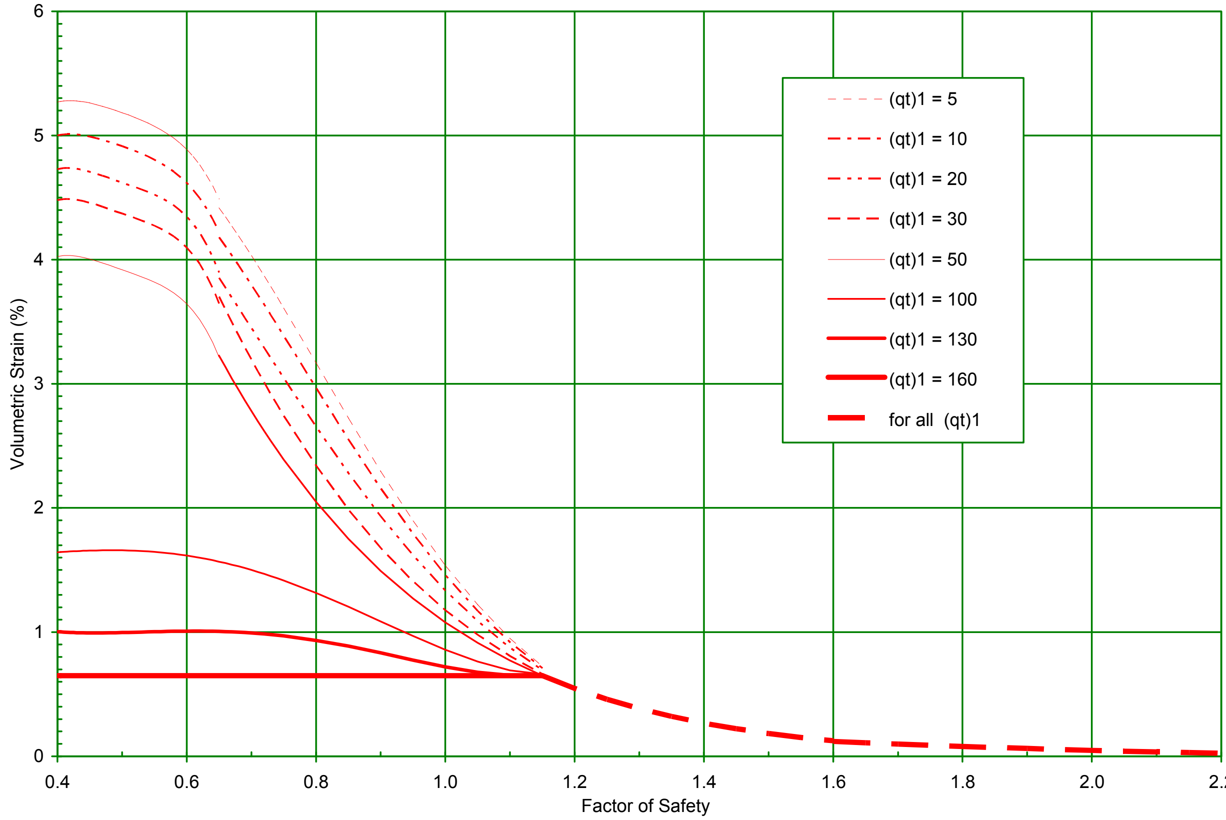

Figure 4-SRS Relationship between volumetric strain, normalized CPTu tip resistance (qt_1), and factor of safety against liquefaction

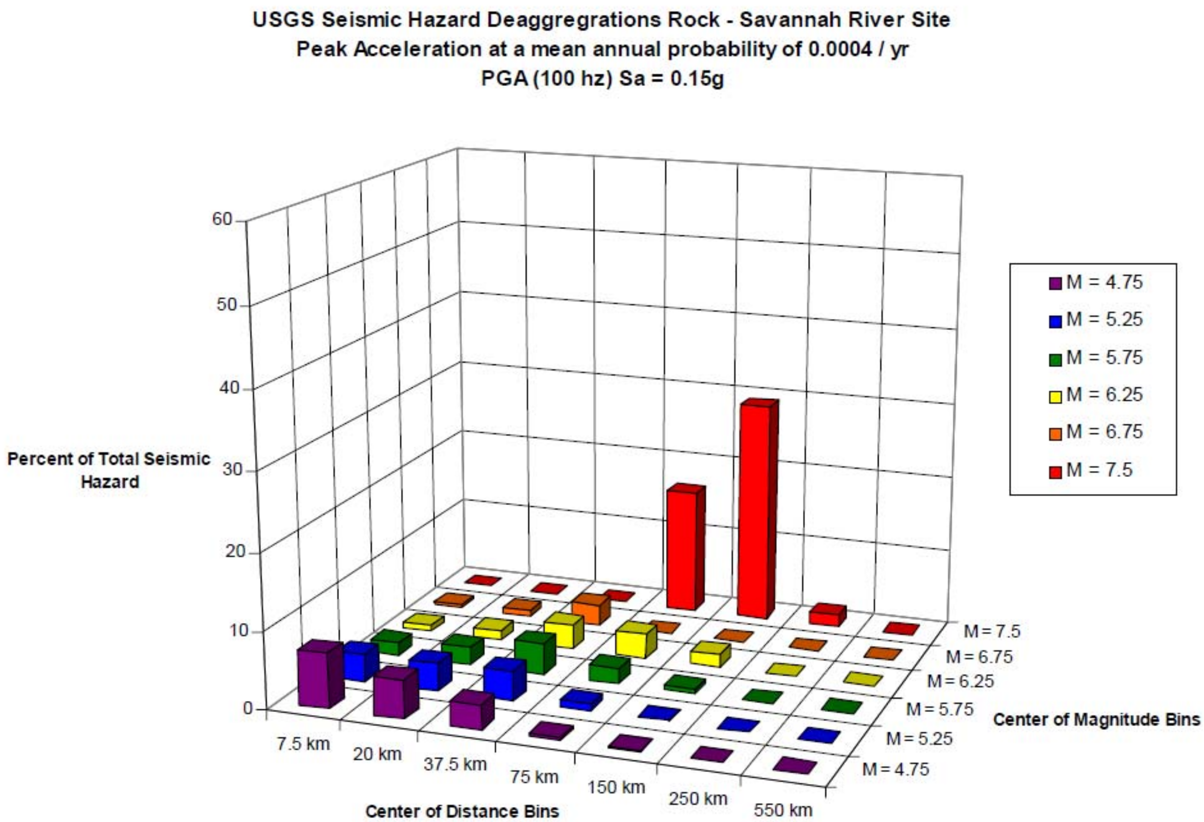

Figure 5-USGS Rock Seismic Hazard Deaggregation: 100 Hz, 2,500 Year Return, S_a=0.15g

Appendix A-PSHA Letter from A. Frankel

United States Department of the Interior

U.S. GEOLOGICAL SURVEY

Arthur Frankel
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303-273-8556, fax 303-273-8600
afrankel@usgs.gov
Mar. 1, 1999

Richard Lee
1092 Sizemore Rd.
Aiken, SC 29803

Dear Rich,

Enclosed is a Zip disk with the de-aggregation tables for the Savannah River Site. See the Srs directory on the disk. I have also printed out the contents of the Readme.txt file on the disk. If you want this in some other format, let me know. If we have missed some particular rates of exceedance that you need, let me know. We'll be happy to answer any questions you have about these results.

Sincerely,

Art Frankel

Arthur Frankel

UNCLASSIFIED
DOES NOT CONTAIN
UNCLASSIFIED CONTROLLED
NUCLEAR INFORMATION
~~ADC &~~ Reviewing Official C.Q. Reeves, RO
(Name and Title)
Date: 7/24/01

Readme.txt

```
Notes on De-aggregations for Savannah River Site

The output files give relative contributions in percent (to 1 decimal
place), and include all rows with no data (no sources). There are 3 header
lines per file and 42 data (or dataless) lines per file.

The output files' names give a clue about the contents. The only
information about the attenuation model used is in the file name.

The file names start with psavrivab (for AB95 attenuation),
psavrivfr (for Frankel et al attenuation),
psavrivto (for Toro et al attenuation), and
psavrivtfa (for 1/3 wt Frankel, Toro and AB95 attenuation models combined).

The internal part of the name gives a clue about the return time, eg,
1Meg means 1,000,000 years, 33Me means 33,333,333 years, etc.
The final part (suffix) of name gives the PSA frequency (eg, 10hz) or is
pga for pga.(see below)


De-aggregations are calculated based on annual frequency of exceedance
for the case of three attenuation relations with equal weight. De-aggregations
at any given freq. of exceed. and ground motion frequency is based
on the same ground motion value.

Annual Rates of Exceedance and 4 letter code embedded in filename:
1e-2   100y
5e-3   200y
3e-3   333y
2e-3   500y
1e-3   1000
5e-4   2000
4e-4   2500
3e-4   3333
2e-4   5000
1e-4   10ky
5e-5   20ky
3e-5   33ky
2e-5   50ky
1e-5   100k
5e-6   200k
4e-6   250k
3e-6   333k
2e-6   500k
1e-6   1meg
5e-7   2meg
3e-7   3meg
2e-7   5meg
1e-7   10me
5e-7   20me
3e-7   33me
2e-7   50me
1e-8   100m

The second header line tells the approx. return time.
See table above for exact annual frequency of exceedance for that filename.
The middle of the second line shows the ground motion value
used in the de-aggrgeation. This value was derived from using the
mean hazard curve from the 3 attenuation relations.
The end of the second line shows the annual frequency of exceedance for
that attenuation relation for the given ground motion value.
When the de-aggregation is for the 3 atten reln. mean, this value
equals the annual freq. of exceedance.
```

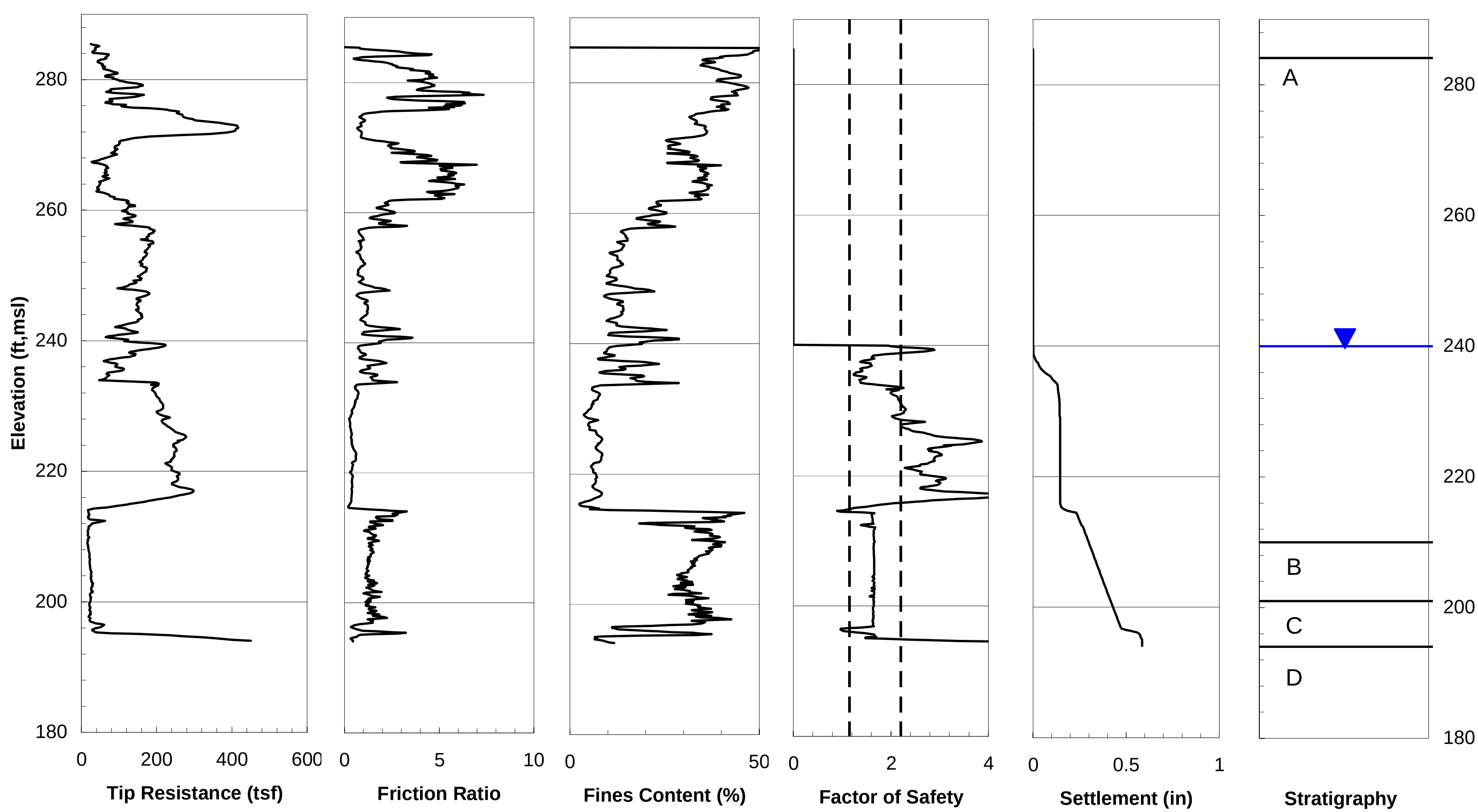
Readme.txt


```
Steve Harmsen   harmsen@usgs.gov
Art Frankel     afrankel@usgs.gov
```

Appendix B-Liquefaction Analysis Output

Project ID: Vault 1

CPT ID: Z-V1-C-01



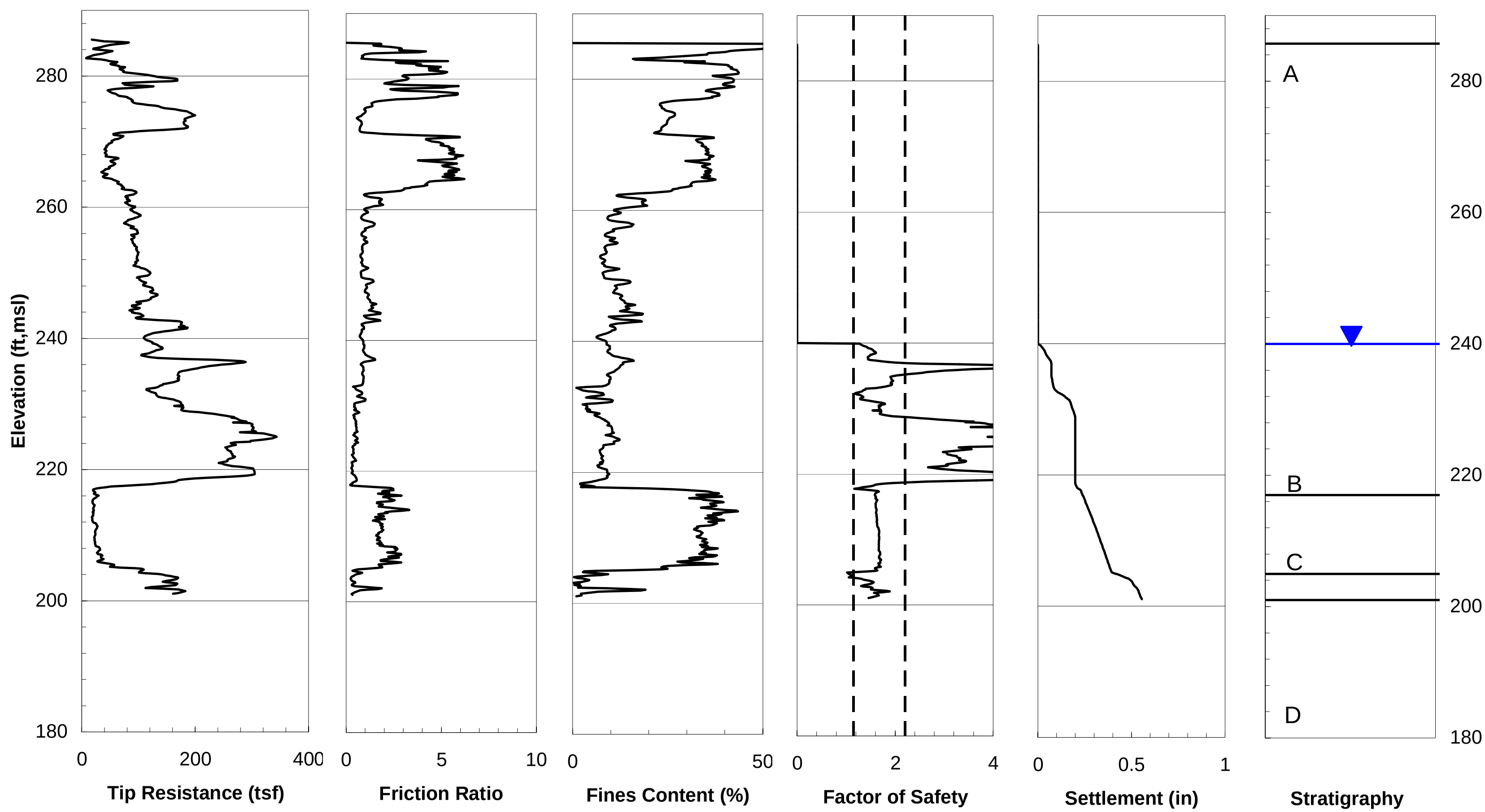
Magnitude	*4.75*	*5.25*	*5.75*	*6.25*	*6.75*	*7.5*
Minimum Factor of Safety	3.05	2.42	1.97	1.53	1.26	0.90
Average Factor of Safety	7.07	5.62	4.58	3.54	2.91	2.08
Maximum Settlement (in)	0.00	0.00	0.00	0.02	0.07	0.59

2,500 Year Weighted Average Settlement	0.28 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID:
Vault 1

CPT ID:
Z-V1-C-02



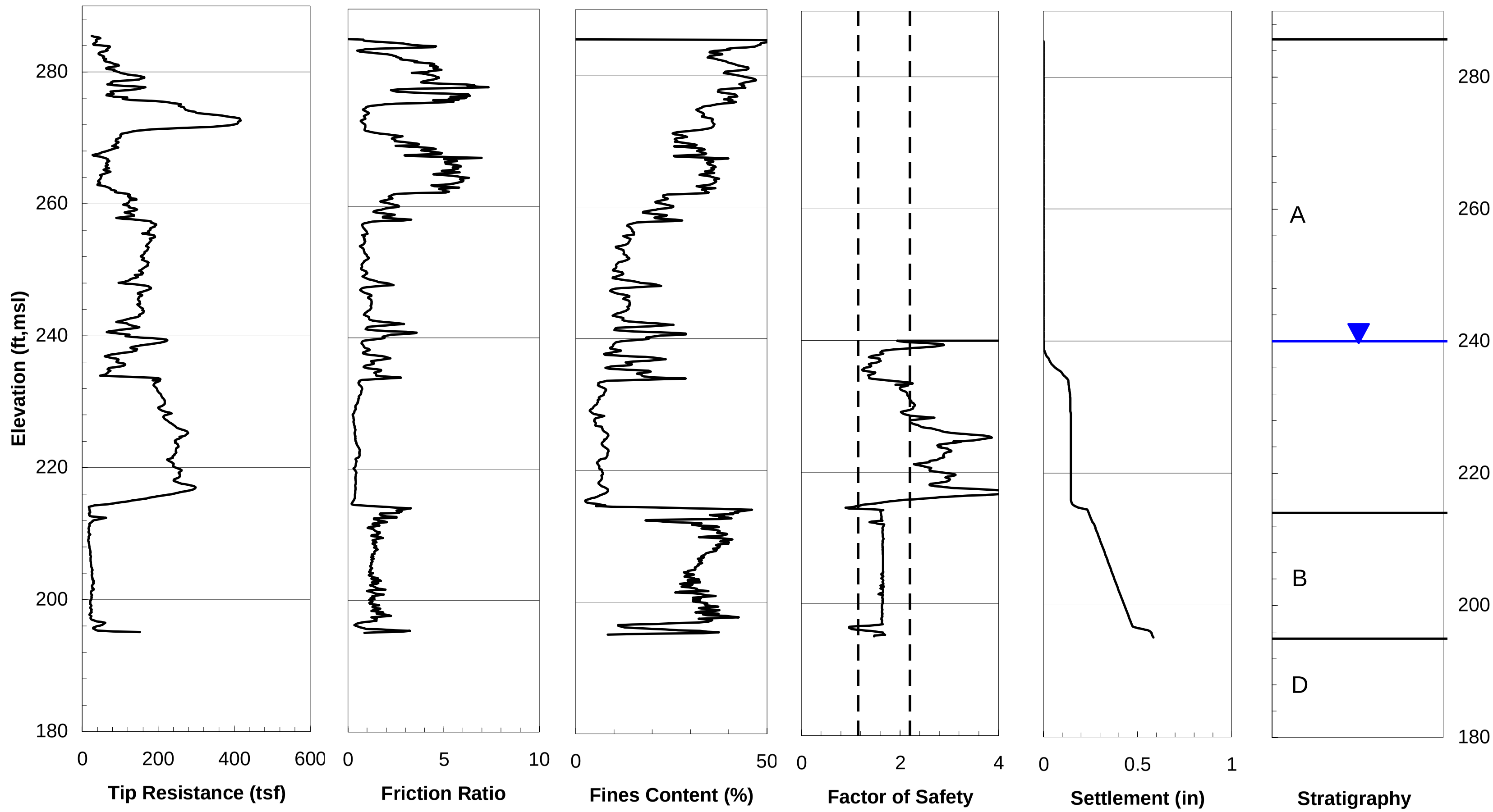
Magnitude	***4.75***	***5.25***	***5.75***	***6.25***	***6.75***	***7.5***
Minimum Factor of Safety	3.47	2.76	2.25	1.74	1.43	1.02
Average Factor of Safety	7.48	5.94	4.84	3.74	3.08	2.20
Maximum Settlement (in)	0.00	0.00	0.00	0.01	0.07	0.56

2,500 Year Weighted Average Settlement	0.27 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID: Vault 1

CPT ID: Z-V1-C-03



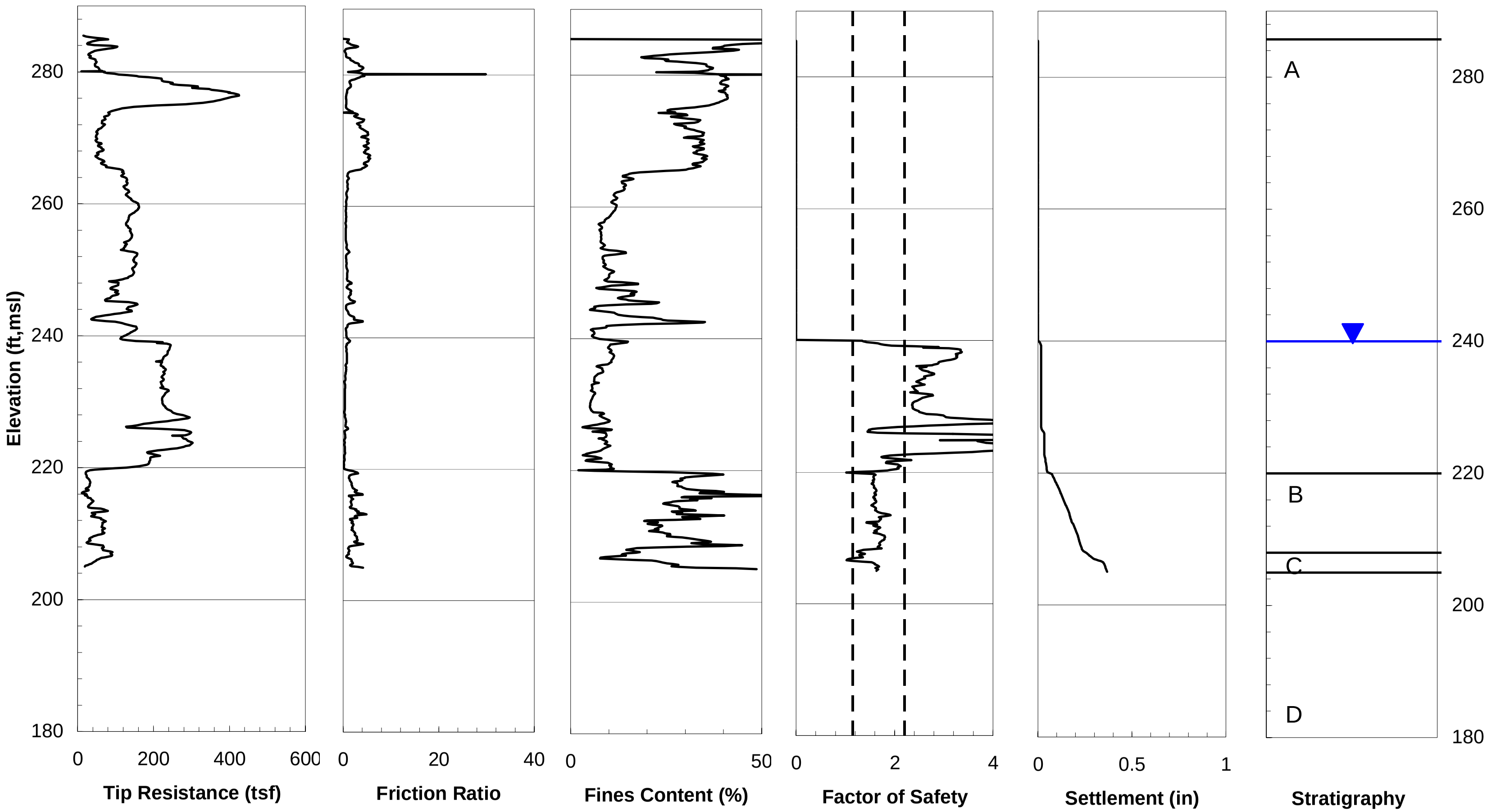
Magnitude	*4.75*	*5.25*	*5.75*	*6.25*	*6.75*	*7.5*
Minimum Factor of Safety	3.05	2.42	1.97	1.53	1.26	0.90
Average Factor of Safety	6.96	5.53	4.51	3.48	2.87	2.05
Maximum Settlement (in)	0.00	0.00	0.00	0.02	0.07	0.58

2,500 Year Weighted Average Settlement	0.28 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	Not Present
D	S4	

Project ID:
Vault 1

CPT ID:
Z-V1-C-04



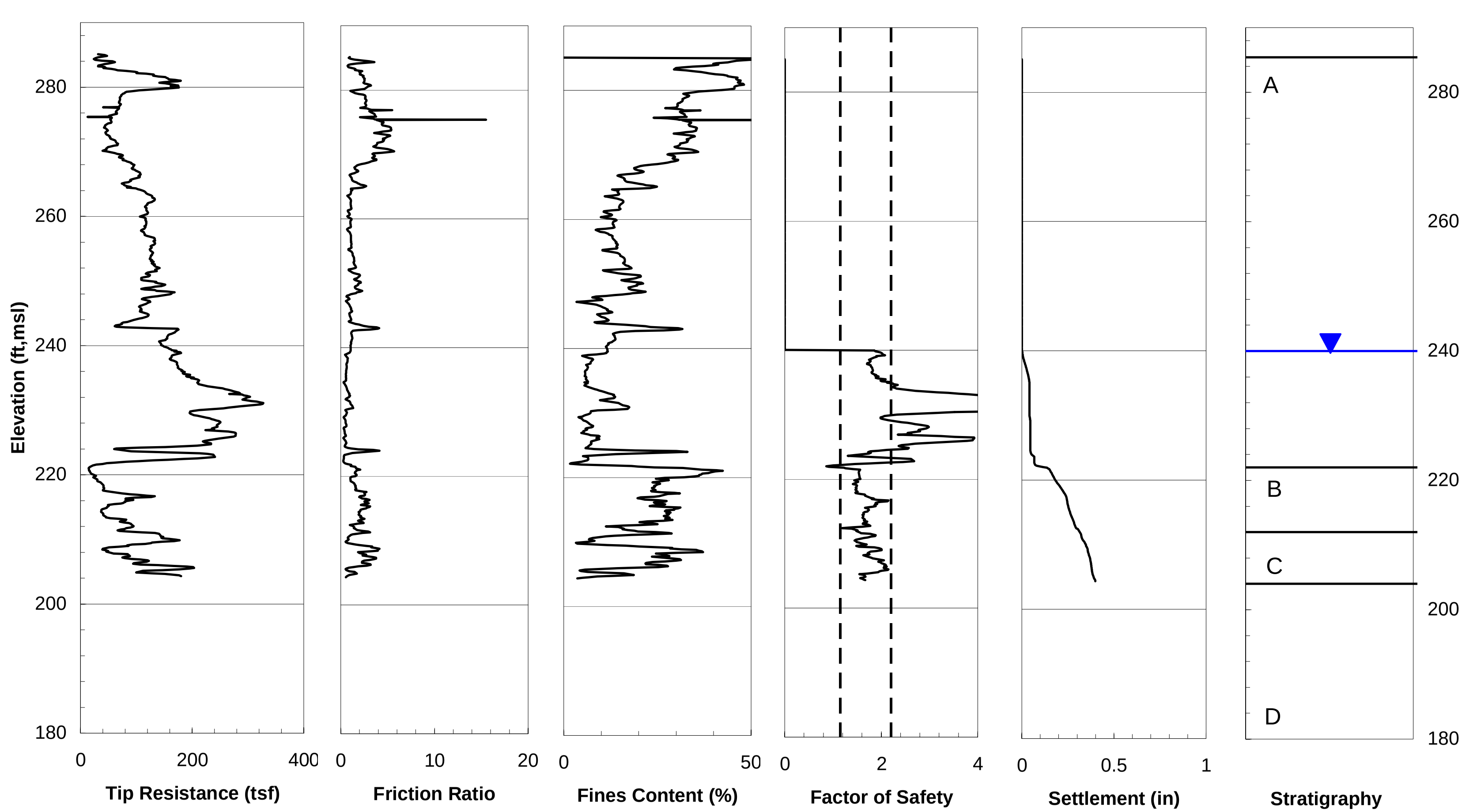
Magnitude	4.75	5.25	5.75	6.25	6.75	7.5
Minimum Factor of Safety	3.46	2.75	2.24	1.73	1.43	1.02
Average Factor of Safety	7.49	5.95	4.85	3.75	3.09	2.20
Maximum Settlement (in)	0.00	0.00	0.00	0.01	0.04	0.37

2,500 Year Weighted Average Settlement	0.18 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID:
Vault 1

CPT ID:
Z-V1-C-05



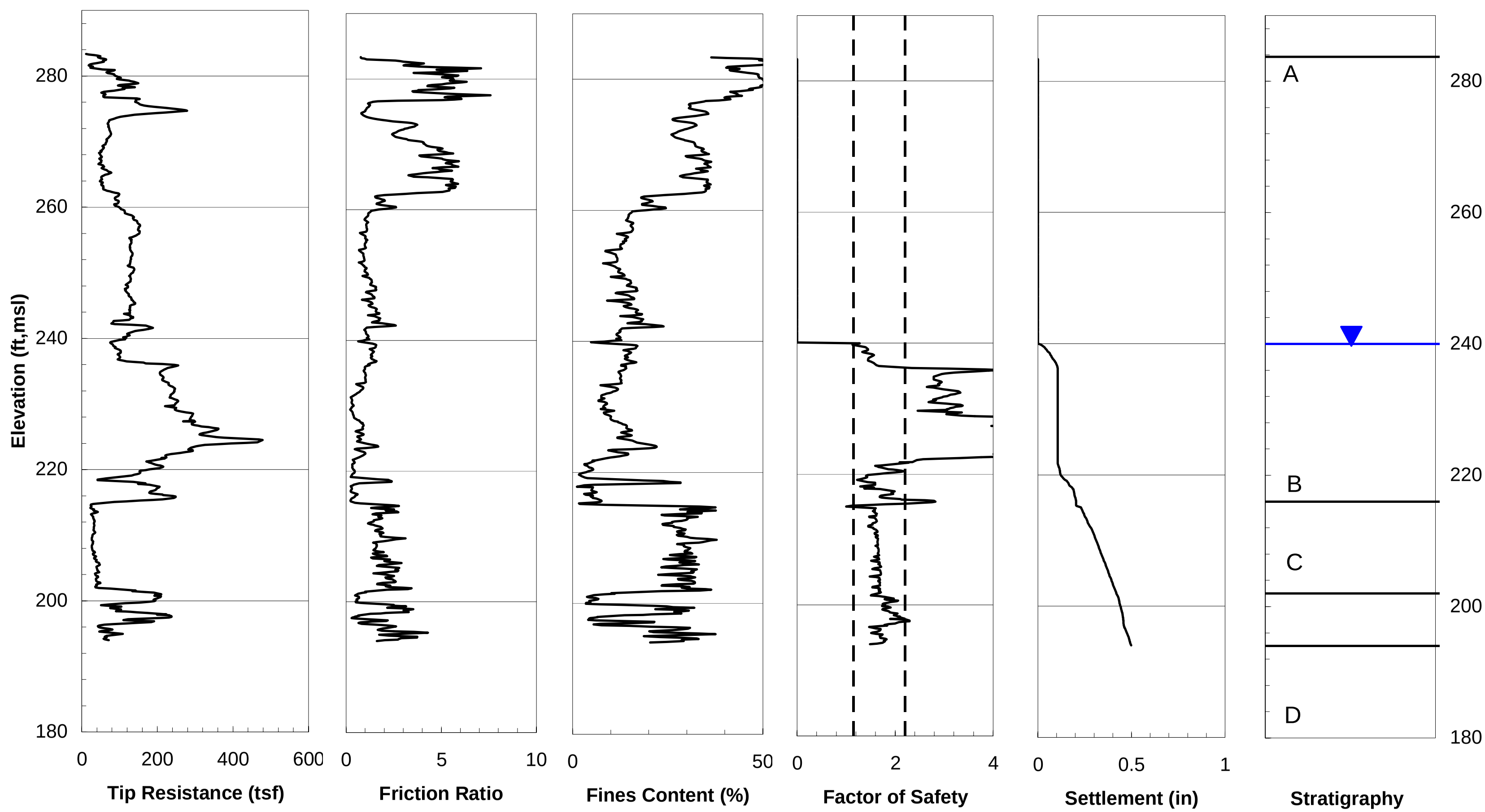
Magnitude	***4.75***	***5.25***	***5.75***	***6.25***	***6.75***	***7.5***
Minimum Factor of Safety	2.92	2.32	1.89	1.46	1.20	0.86
Average Factor of Safety	7.30	5.80	4.72	3.65	3.01	2.15
Maximum Settlement (in)	0.00	0.00	0.00	0.01	0.05	0.40

2,500 Year Weighted Average Settlement	0.19 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID: Vault 1

CPT ID: Z-V1-C-06



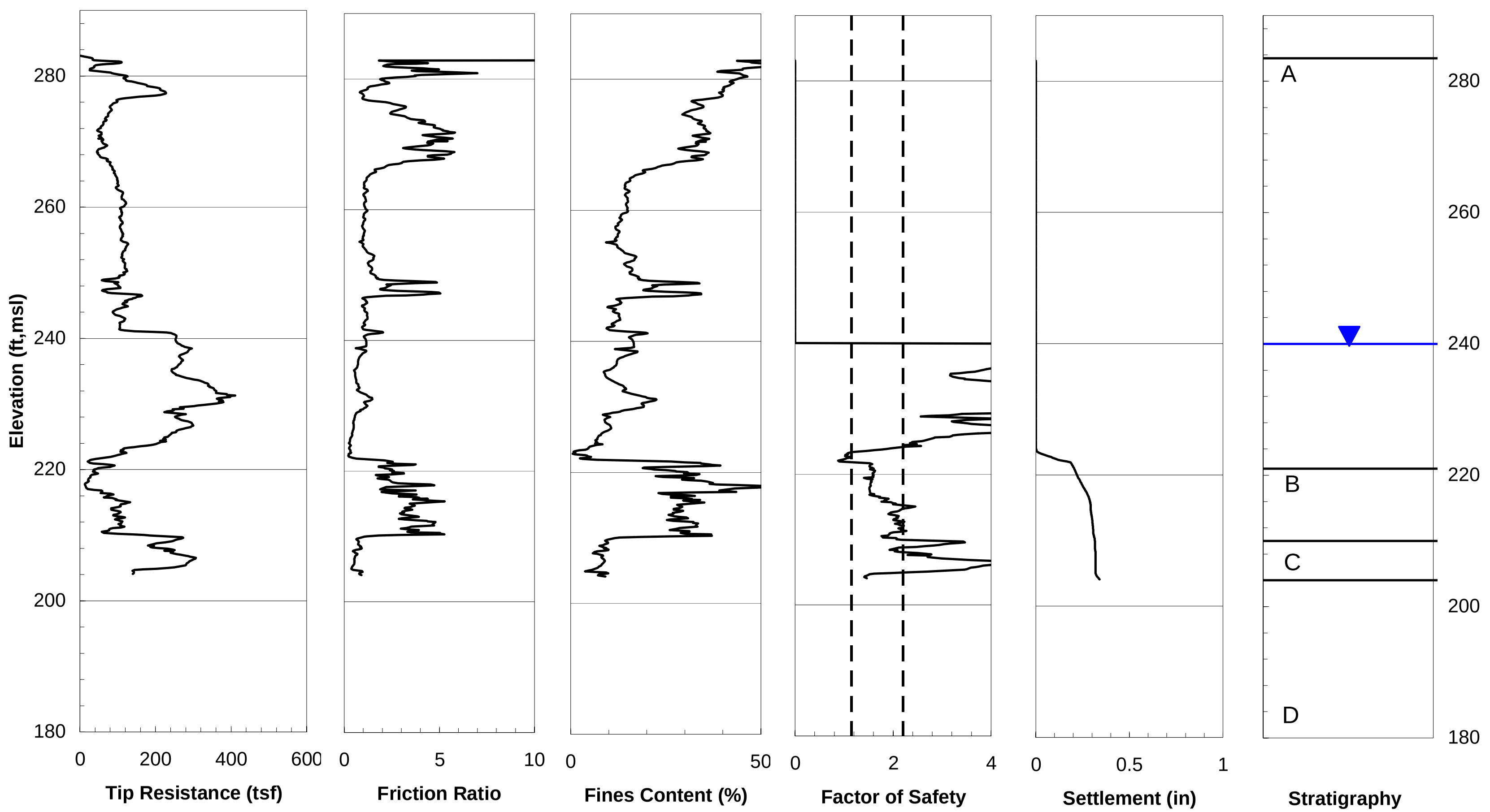
Magnitude	4.75	5.25	5.75	6.25	6.75	7.5
Minimum Factor of Safety	3.40	2.70	2.20	1.70	1.40	1.00
Average Factor of Safety	7.76	6.16	5.02	3.88	3.20	2.28
Maximum Settlement (in)	0.00	0.00	0.00	0.01	0.05	0.50

2,500 Year Weighted Average Settlement	0.24 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID:
Vault 1

CPT ID:
Z-V1-C-07



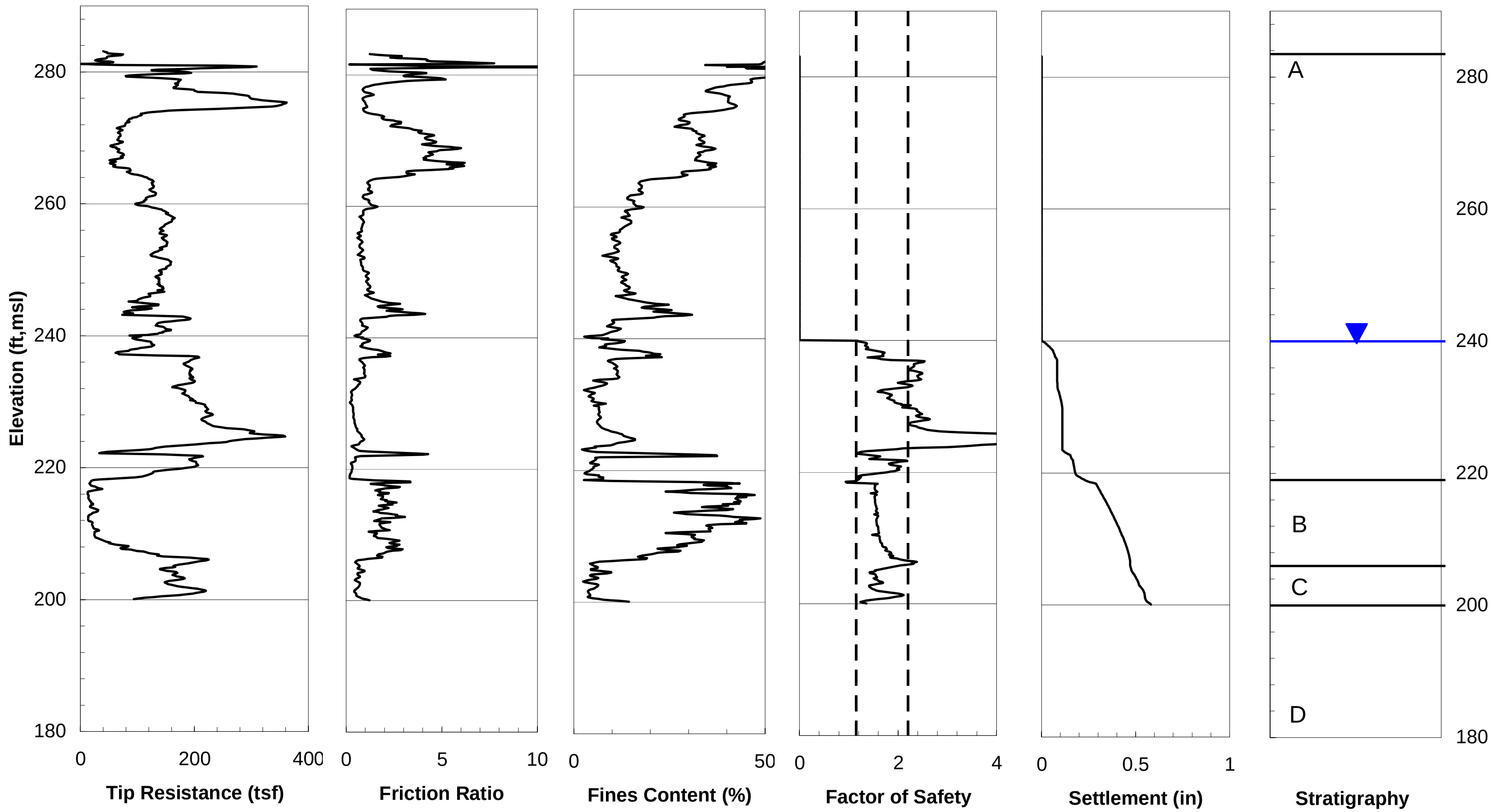
Magnitude	*4.75*	*5.25*	*5.75*	*6.25*	*6.75*	*7.5*
Minimum Factor of Safety	2.99	2.38	1.94	1.50	1.23	0.88
Average Factor of Safety	9.78	7.77	6.33	4.89	4.03	2.88
Maximum Settlement (in)	0.00	0.00	0.00	0.02	0.06	0.34

2,500 Year Weighted Average Settlement	0.17 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID:
Vault 1

CPT ID:
Z-V1-C-08



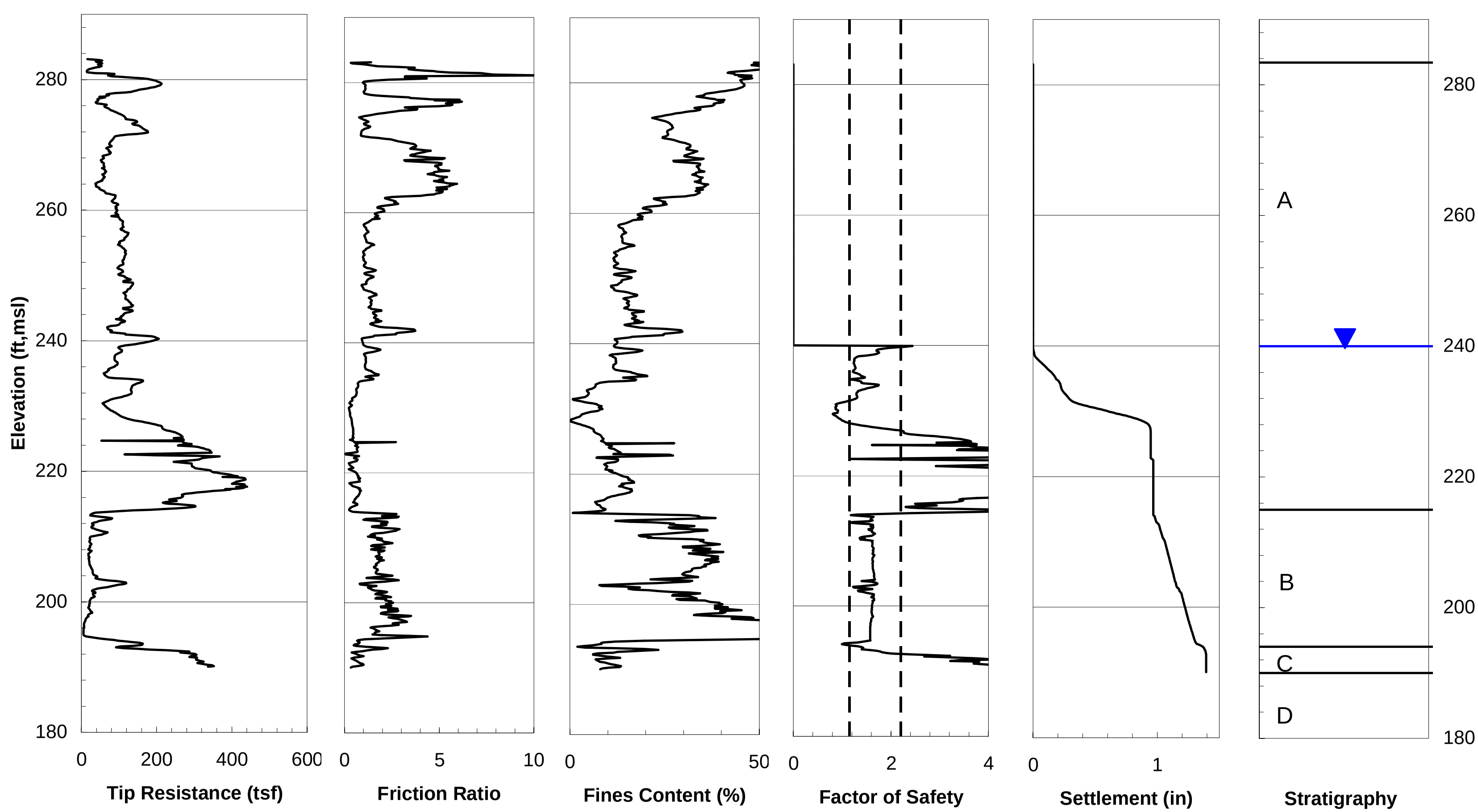
Magnitude	4.75	5.25	5.75	6.25	6.75	7.5
Minimum Factor of Safety	3.19	2.53	2.06	1.59	1.31	0.94
Average Factor of Safety	6.61	5.25	4.27	3.30	2.72	1.94
Maximum Settlement (in)	0.00	0.00	0.00	0.01	0.08	0.58

2,500 Year Weighted Average Settlement	0.28 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID:
Vault 1

CPT ID:
Z-V1-C-09



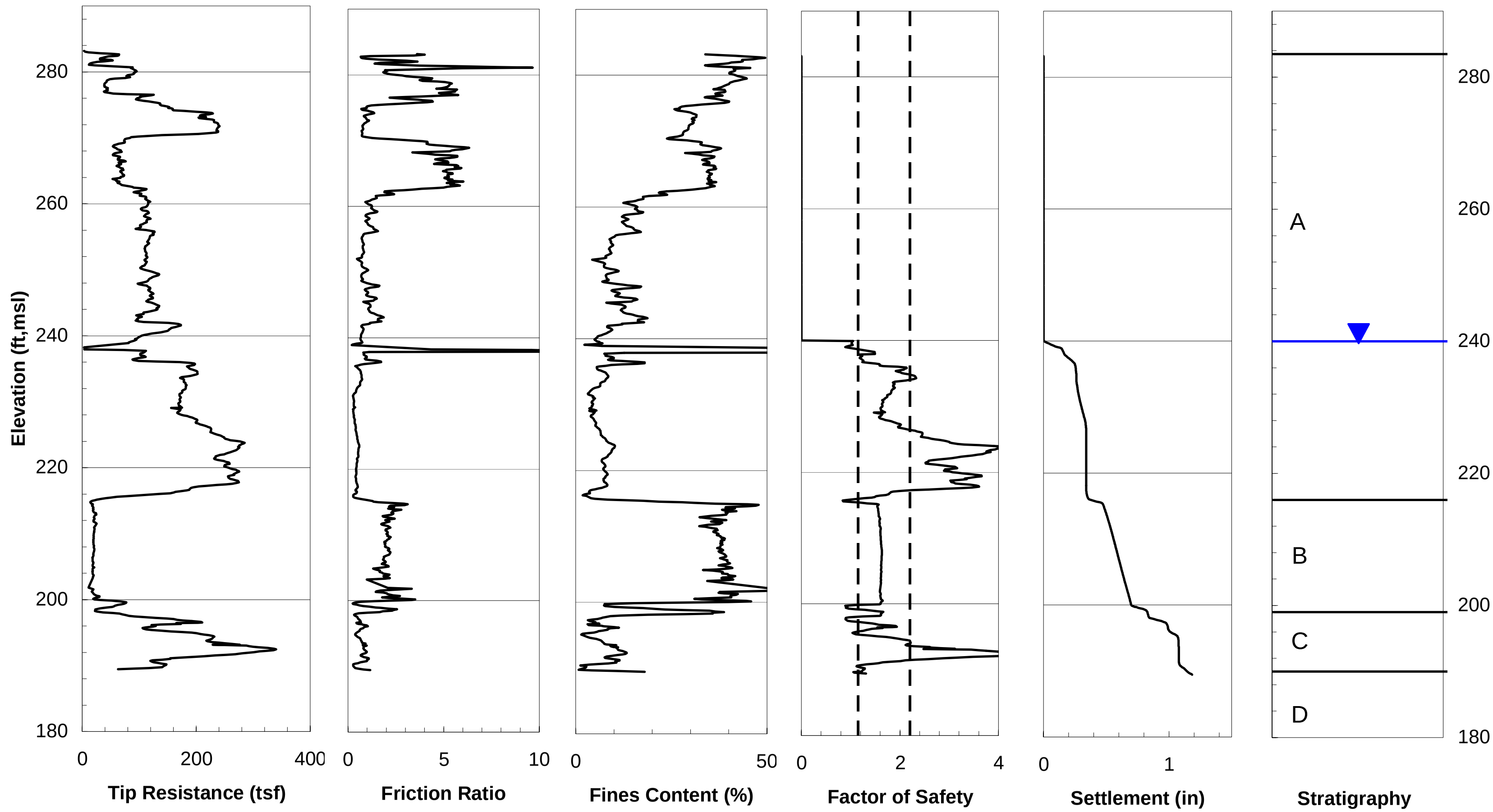
Magnitude	*4.75*	*5.25*	*5.75*	*6.25*	*6.75*	*7.5*
Minimum Factor of Safety	2.75	2.18	1.78	1.37	1.13	0.81
Average Factor of Safety	7.56	6.00	4.89	3.78	3.11	2.22
Maximum Settlement (in)	0.00	0.00	0.02	0.09	0.27	1.39

2,500 Year Weighted Average Settlement	0.68 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Project ID: Vault 1

CPT ID: Z-V1-C-10



Magnitude	4.75	5.25	5.75	6.25	6.75	7.5
Minimum Factor of Safety	2.84	2.26	1.84	1.42	1.17	0.84
Average Factor of Safety	6.67	5.30	4.32	3.34	2.75	1.96
Maximum Settlement (in)	0.00	0.00	0.01	0.07	0.20	1.18

2,500 Year Weighted Average Settlement 0.58 in.

Stratigraphy / Layer Description		
A	S1/S2	
B	C2	
C	S3	
D	S4	

Appendix D

K-CLC-Z-00021

Soft Zone Compression-Induced Settlements for Saltstone Vault No. 1, Rev. 0, December 30, 2010

(15 pages)

OSR 45-24 (Rev 1-20-2009)

Calculation Cover Sheet

Project/Task	Calculation No.	Project/Task No.
Saltstone Vault No. 1	K-CLC-Z-00021	
Title: Soft Zone Induced Settlements for Saltstone Vault No. 1	Functional Classification: PS	Sheet 1 of 15
	Discipline: Geotechnical	
Calculation Type: ■ Type 1 ☐ Type 2	Type 1 Calc Status: ☐ Preliminary ■ Confirmed	
Computer Program No.: N/A ■ N/A	Version/Release No.	
Purpose and Objective: To estimate soft zone settlement beneath Saltstone Vault No. 1	DC/RO / Date: UNCLASSIFIED DOES NOT CONTAIN UNCLASSIFIED CONTROLLED NUCLEAR INFORMATION ADC & Reviewing Official A.A. Siddall For ACP (Name) Date: 12/10/2010	

Summary of Conclusion

See the Results and Conclusions Section.

Revisions

Rev #	Revision Description
0	Original

Sign Off

Rev #	Originator (Print) Sign/Date	Verification/Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign/Date
0	Aaron J. Geiger 12/8/10	☐ Design Check (GS/PS only) ■ Document Review ☐ Qualification Testing ☐ Alternative Calculation ☐ Operational Testing	D. Bruce Nothdurft 12/8/2010	William T. Li 12/9/2010
		☐ Design Check (GS/PS only) ☐ Document Review ☐ Qualification Testing ☐ Alternative Calculation ☐ Operational Testing		

	Signature	Date
Additional Reviewer (Print): N/A		
Design Authority (Print): N/A		
Release to Outside Agency (Print): N/A		

Security Classification of the Calculation

Unclassified

TABLE OF CONTENTS

1. Introduction

The purpose of this calculation is to provide an estimate of the settlement resulting from the compression of soft zones beneath Saltstone Vault No. 1.

2. Input

2.1 Soft Zone Input

Ten (10) CPTs and and SCPTs were pushed around the Vault 1 site, see Figure 1. Stratigraphic interpretations of the CPT and SCPTs are summarized below (Ref. 1). These 10 CPTs and SCPTs were evaluated for the presence of soft zones.

Table 1-CPT Stratigraphy Around Vault 1

CPT ID	Ground Elevation (ft, msl)	C2	S3	S4	M1	SRS_N	SRS_E
Z-V1-C-01	284.1	210	201	194	134	76084.0	66243.0
Z-V1-C-02	285.7	217	205	201	NDE	76005.0	66333.1
Z-V1-C-03	285.7	214	NP	195	142	75895.5	66369.2
Z-V1-C-04	285.7	220	208	205	140	75750.1	66417.3
Z-V1-C-05	285.4	222	212	204	NDE	75620.0	66459.9
Z-V1-C-06	283.7	216	202	194	NDE	75488.0	66441.9
Z-V1-C-07	283.5	221	210	204	NDE	75574.8	66346.5
Z-V1-C-08	283.5	219	206	200	NDE	75688.8	66308.7
Z-V1-C-09	283.4	215	194	190	NDE	75832.6	66261.2
Z-V1-C-10	283.5	216	199	190	NDE	75960.9	66218.0

3. Computation

Soft zones are layers of underconsolidated soils within the Santee Formation (S4 layer) or in the Lower Dry Branch formation (S3) which are identified by a corrected tip stress value less than or equal to 15 tsf. Layers of soft zone soils (corrected tip stress less than or equal to 15 tsf) are considered to behave as a single soft zone if the interval of harder material (corrected tip stress greater than 15 tsf) between the soft zone soils is less than or equal to 2 feet in thickness. The thickness of the soft zone is the summation of the less than 15 tsf material. The top elevation of the soft zone is the top of the first layer of soil with a corrected tip stress less than 15 tsf within the Santee Formation Santee Formation (S4 layer).

If the interval of harder soil (corrected tip stress greater than 15 tsf) between soft zone soils is greater than 2 feet, the layers of soft zones soils are considered to behave as two separate soft zones (See Figure 3).

The thickest soft zone encountered in this investigation was in Z-V1-C-09 at an elevation of 194.7-197.5 ft, msl. This soft zone is just above the Santee formation boundary of 190 ft, msl.

3.1 Soft Zone Settlement

Since there have been no samples of soft zone material obtained from the immediate vicinity of Vault 1, the soft zone properties from samples taken beneath Saltstone Vault No. 2 will be used to provide an estimate of surface settlement. A factor of safety of two for soft zone thickness was used for conservatism. Therefore, the thickness of soft zone to be analyzed for surface settlement will be twice the maximum thickness. The thickest soft zone observed in the CPT soundings around Vault 1 was 2.8 ft.; therefore a thickness of 5.6 ft was assumed

The following properties are considered for the soft zone geometry and properties for Vault 1 (Ref. 2).

Table 2-Soft Zone Properties

Thickness	5.6ft
Compression Index, C_c	0.196
Initial Void Ratio, e_0	0.72
Overconsolidation Ratio, OCR	0.9
Compression Ratio, $C_c/(1+e_0)$	0.114

The bottom elevation of the Vault 1 footings is 281.5 ft, msl.

The compression of the soft zone s_S at depth was estimated assuming full overburden pressure:

$$s_S = H \{C_c/(1 + e_o)\} \log \{(P_o + \Delta P)/P_o\}$$

Where s_S is the compression of the soft zone and H is the thickness of the soft zone. C_c and e_0 were described in the previous section. When the arch above the soft zone is weakened, the $P_o + \Delta P$ term is (assumed) equal to the overburden pressure and the P_o term in the denominator is the soft zone preconsolidation pressure. In this instance the equation becomes:

$$s_S = H \{C_c/(1 + e_o)\} \log (1/OCR)$$

where OCR is the overconsolidation ratio of the soft zone.

Using the soft zone properties from Vault 2, the compression of the soft zone is:

$$s_S = H \{C_c/(1 + e_o)\} \log (1/OCR) = 14 \times 0.196/(1 + 0.72) \times \log (1/0.9) = 0.029 \text{ feet, or } s_S = 0.350 \text{ inches.}$$

3.2 Methodology for Computing Surface Settlement

A vertical slice of subsurface with unit thickness or perpendicular to the longitudinal direction of the soft zone was considered. Ground settlement, which is propagated from the subsurface deformation, was computed considering the surface settlement profile resembles the shape of an inverted normal distribution curve (Ref. 3). The surface settlement s(x) at any point x is:

$$s(x) = s(0) \text{ Exp.}\{-x^2/(2i^2)\} \qquad \text{(Eq. 1)}$$

Where i is the distance from the center of the normal probability curve to the point of inflection:

$$i = W/(2\pi)^{1/2} \qquad \text{(Eq. 2)}$$

and W is the half width of the normal probability curve and may be estimated as (Ref. 3):

$W = z \tan \beta + W_{SZ}/2.$ (Eq. 3)

where:

z is the soft zone depth and

β is based on soil type

The volume lost at-depth due to compression of soft zone can be computed as:

$V_L = s_S W_{SZ.}$ (Eq. 4)

Where s_S is the compression of the soft zone computed in the previous section and W_{SZ} is the width of the soft zone.

As the soft zone collapses, the volume of the soil above the soft zone will increase as a result of dilation and loosening as the soil stresses redistribute. For granular soils, appreciable volume changes can occur in the soil as a result of disturbances and displacement (Ref. 3).

The volume of the surface settlement is:

$V_S = R_{S/L}V_L$ (Eq. 5)

where $R_{S/L}$ is the ratio of the volume of the surface settlement to the volume lost at-depth due to compression of the soft zone. Substituting Equation (4) into Equation (5):

$V_S = R_{S/L}\ s_S\ W_{SZ.}$ (Eq. 6)

Surface settlement at the center of the normal probably curve is:

$s(0) = V_S/W$ (Eq. 7)

Substituting Equation (6) into Equation (7)

$s(0) = R_{S/L}\ s_S\ W_{SZ}\ /W$ (Eq. 8)

Substituting Equation (8) into Equation (1), settlement at any point x can then be expressed as

$s(x) = R_{S/L}\ s_S\ W_{SZ}\ /W\ \mathrm{Exp}[-x^2/(2i^2)]$ (Eq. 9)

Figure 3 illustrates the properties of a normal probability curve settlement trough.

3.3 Surface Settlement due to the Compression of a Narrow Soft Zone

The assumption of normal probability is for underground disturbance over a short width. Assume the width of the soft zone is:

$W_{SZ} = 5$ feet

For the project site, at elevation of the foundation, 281.5 feet, MSL, the distance to the average depth of the soft zone is:

$z = 281.5 - 197.7 = 83.8$ feet

For SRS soil conditions β falls between 33 and 50 degrees. For sands below groundwater level, β is generally greater than 50 degrees. A smaller β will provide conservative values of maximum slope and maximum change of slope (Ref. 3) at the project site, consider $\beta = 33$ degrees:

$W = z \tan \beta + W_{SZ}/2 = 83.8 \tan(33^o) + 5/2 = 56.920$ feet

$i = W/(2\pi)^{1/2} = 56.920/(2\pi)^{1/2} = 22.708$ feet

The volume of the surface settlement is generally one third to two thirds less than the volume of lost ground (Ref. 1). In this calculation, $R_{S/L}$ is considered to be 2/3.

$s(0) = R_{S/L}\ s_S\ W_{SZ}\ /W = (2/3) \times 0.350 \times 5/56.920 = 0.0205$ inches

Equation 1 becomes:

$s(x) = 0.0205\ \mathrm{Exp}\{-x^2/(2 \times 22.708^2)\}$ inches

or $s(x) = 0.0205\ \mathrm{Exp}(-x^2/1031.31)$ inches

3.4 Surface Settlement

Wide soft zones may be represented as a series of adjacent narrow soft zones. Surface settlements due to the wide soft zone are computed by superimposing the settlement troughs for each of the narrow soft zones. For this calculation, a series of 5 foot wide soft zones were utilized to represent soft zones ranging in width from 25 feet to 150 feet.

Figure 4 presents the surface settlement profiles for each of the soft zone widths considered. Figures 5 to 7 illustrate the maximum surface settlement, maximum slope, and maximum curvature as functions of soft zone width. The values for these parameters are summarized in the table below.

Table 3-Soft Zone Settlement Results

Soft Zone Width (ft)	Maximum Settlement (in)	Maximum Differential Settlement (in)	Maximum Slope (ft/ft)	Maximum Curvature (ft/ft per ft)	Ratio of Maximum Settlement to Soft Zone Compression
25	0.098	0.098	0.00021	0.00001	28%
50	0.170	0.170	0.00032	0.00002	48%
75	0.211	0.210	0.00034	0.00001	60%
100	0.227	0.223	0.00034	0.00001	65%
125	0.232	0.221	0.00034	0.00001	66%
150	0.233	0.211	0.00034	0.00001	67%
Maximum	0.233	0.223	0.00034	0.00002	67%

3.5 Reaction under a Rigid Foundation

Foundations can be designed as either flexible foundations or as rigid foundations. For a flexible foundation, the foundation is considered to deform in the same shape and magnitude as the settlement profile provided in this calculation.

For a rigid foundation, the soil under the slab will settle uniformly, but the reaction on the foundation will not be uniform. The recommended vertical subgrade reaction k_1 is 200 pci (Ref. 4). Adjusting this for a foundation that is 100 ft wide using the following equation (Ref. 5);

$k_s=[(B+1)/(2B)]^2*k_1$

The recommended subgrade modulus, k_{100}, is 51 pci. Based on this and a differential settlement of 0.223 inches, the reaction at the center of the trough could be less than the reaction away from the center by an amount of

$p = k \times y = 51$ pci $\times$ 0.223 inches = 11.373 psi $\approx$ 1,640 psf.

4. Results and Conclusions

The summary of the worst case soft zone settlement for Vault 1 is presented in Table 4. Soft zone settlement is not expected to be very significant at Vault 1.

Table 4-Soft Zone Settlement Summary

Parameter	Recommended Value
Maximum soft zone surface settlement:	¼ inch
Maximum soft zone differential surface settlement:	¼ inch
Maximum Slope:	0.0003 ft/ft
Maximum Curvature:	0.00002 ft/ft per ft

5. References

1. K-CLC-Z-00020, Rev. 0, Stratigraphy for Saltstone Vault No. 1, October 2010.
2. K-CLC-Z-00009, Rev. 0, Settlement due to Compression of Soft Zone, March 2006.
3. Cording, E.J., et al., "Displacements Around Tunnels in Soils," Report No. 76T-22, U.S. Department of Transportation, Washington D.C.
4. Z-SDF-2, Saltstone Disposal, Z-Area, October 1986, Mueser Rutledge Consulting Engineers.
5. Concrete Institute, *Design of Slabs on Grade*, ACI 360R-92, Reapproved 1997.

Figures

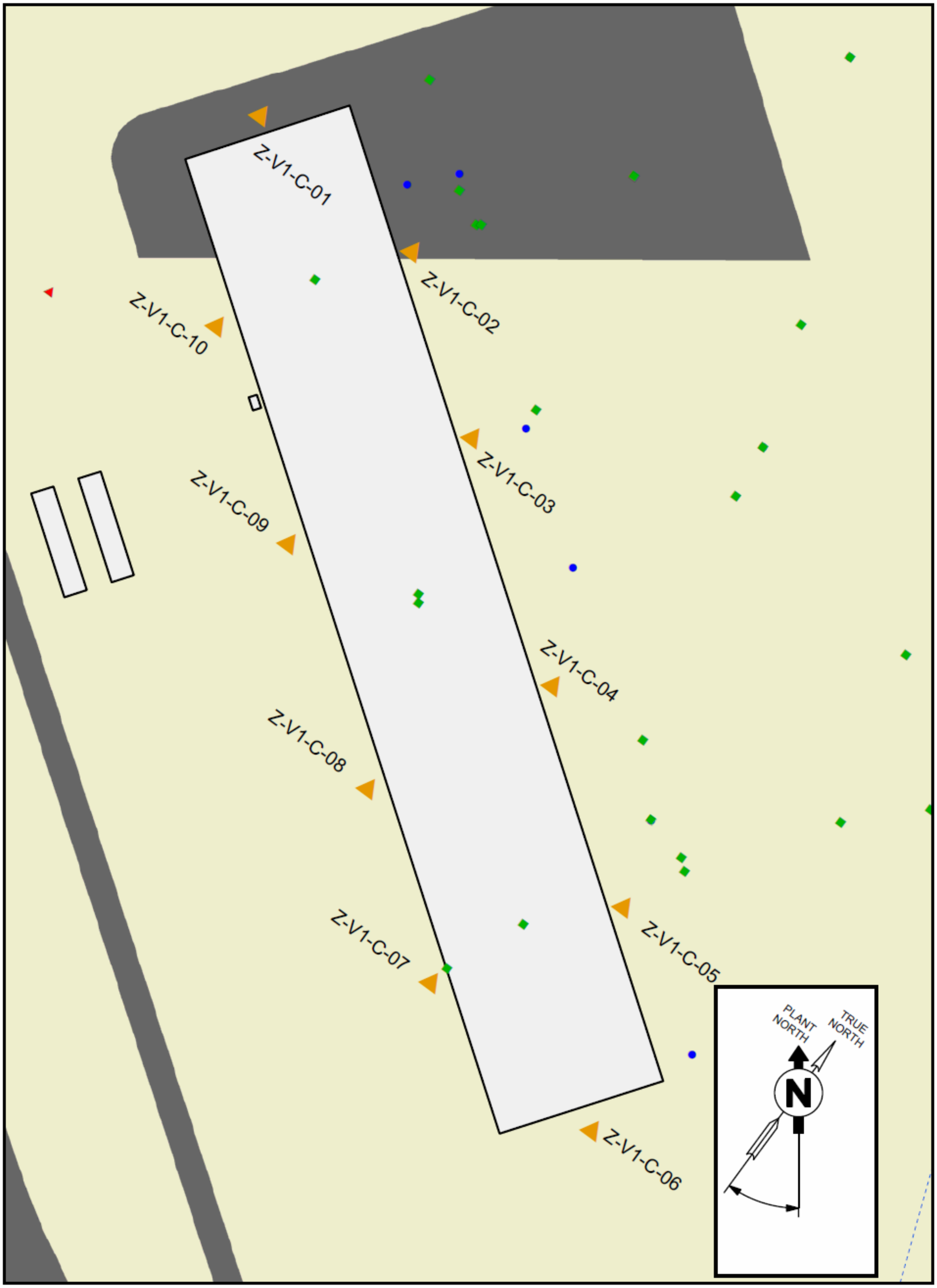


Figure 1: Saltstone Vault 1 Site Map

Thickness		Depth
1.5 feet thick	q_t = 15 tsf	100.0 ft 101.5 ft
0.5 feet thick	q_t > 15 tsf	
1.0 feet thick	q_t = 15 tsf	102.0 ft 103.0 ft

One Soft Zone
- 2.5 feet thick
  Top depth - 100 feet

1.5 feet thick	q_t = 15 tsf	100.0 ft 101.5 ft
2.5 feet thick	q_t > 15 tsf	
1.0 feet thick	q_t = 15 tsf	104.0 ft 105.0 ft

Two Soft Zones
- 1.5 feet thick
  Top depth - 100 feet
- 1.0 foot thick
  Top depth - 104 feet

Figure 2: Graphical Depiction of Soft Zone Determination

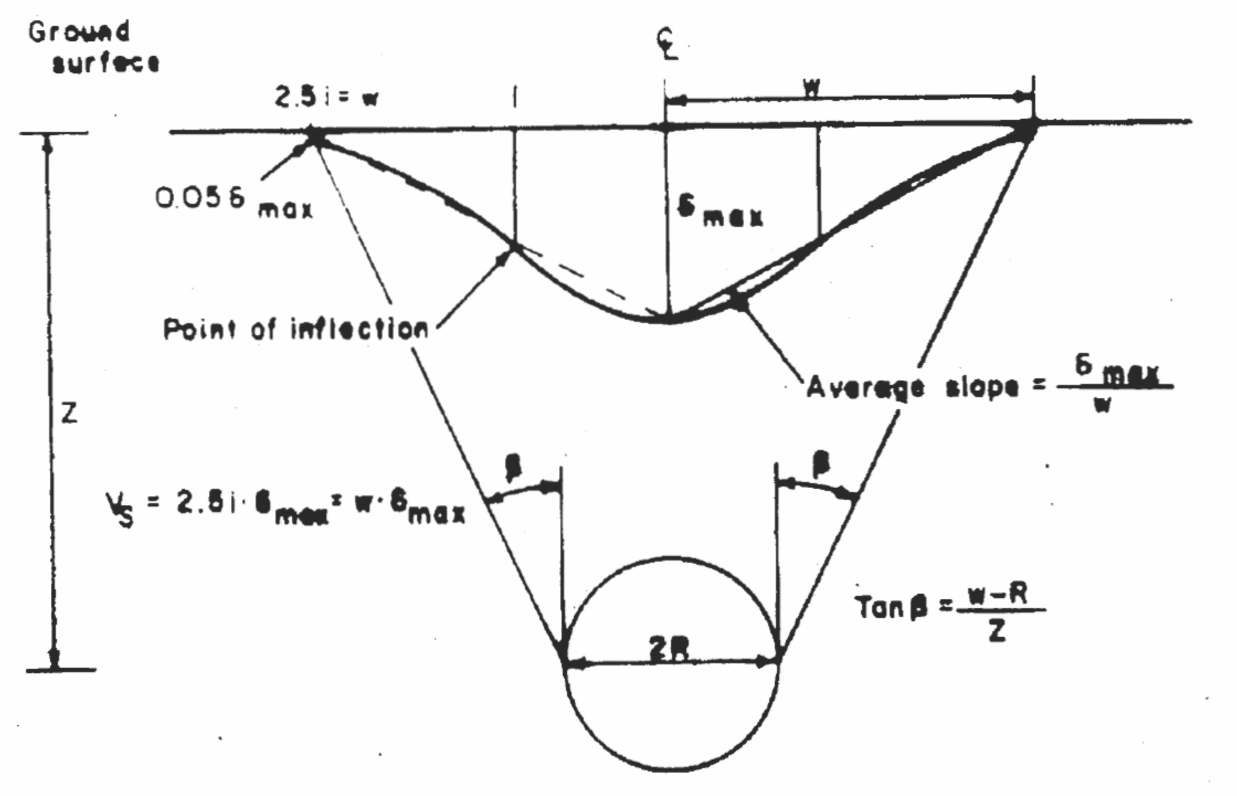


Figure 3: Geometry of Surface Settlement Trough

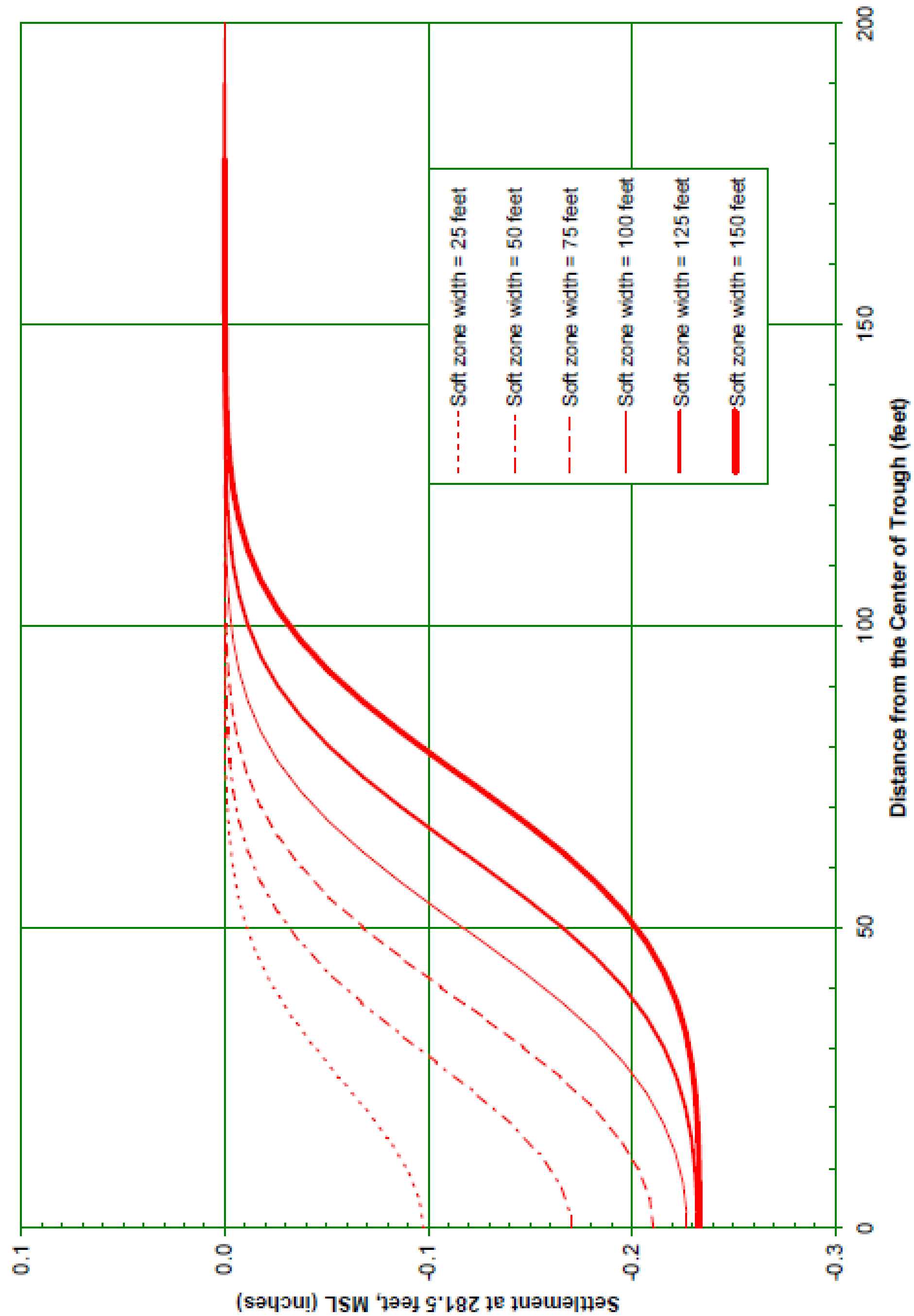


Figure 4: Surface Settlement Profile for Soft Zones of Various Widths

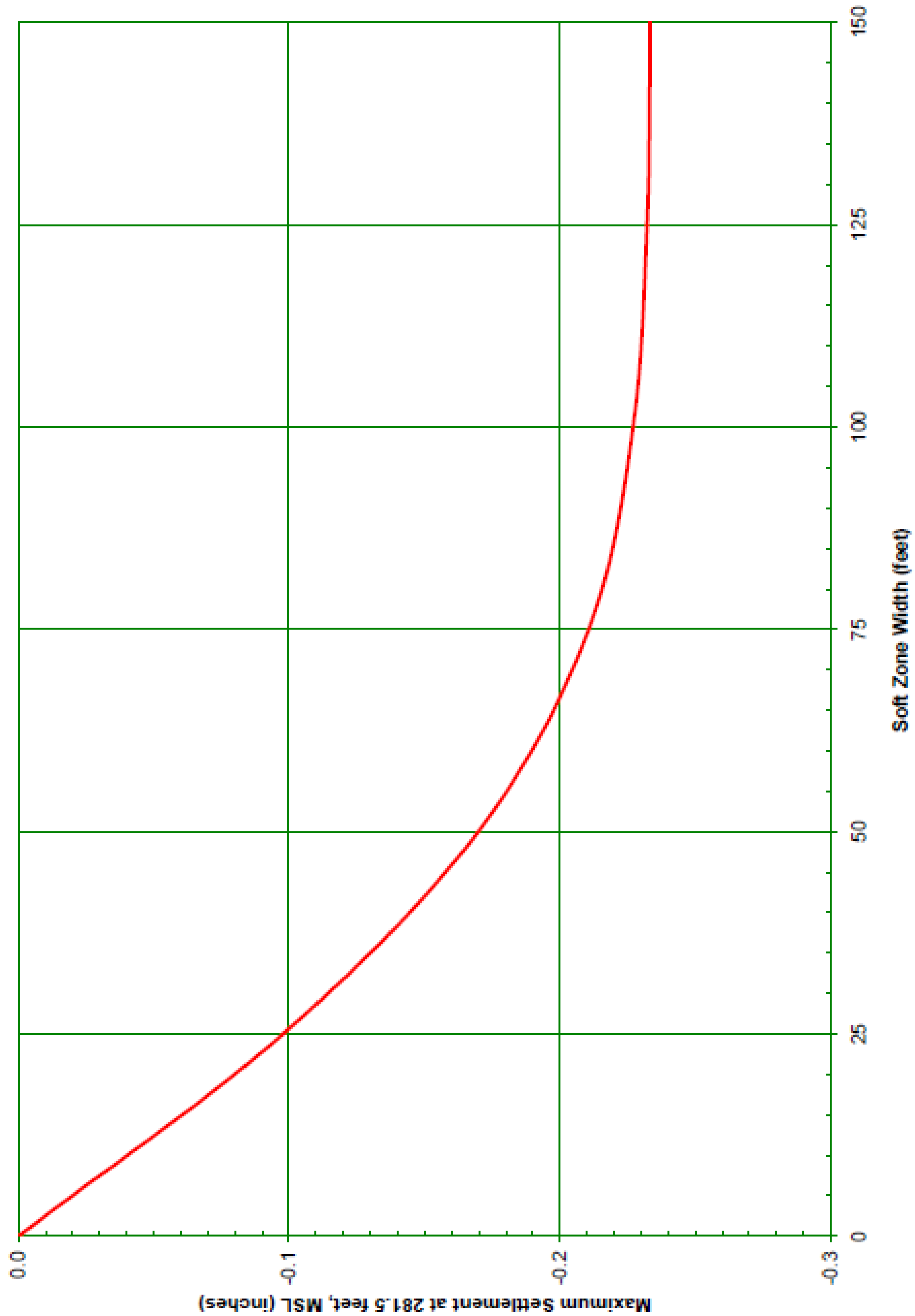


Figure 5: Maximum Surface Settlement for Soft Zones of Various Widths

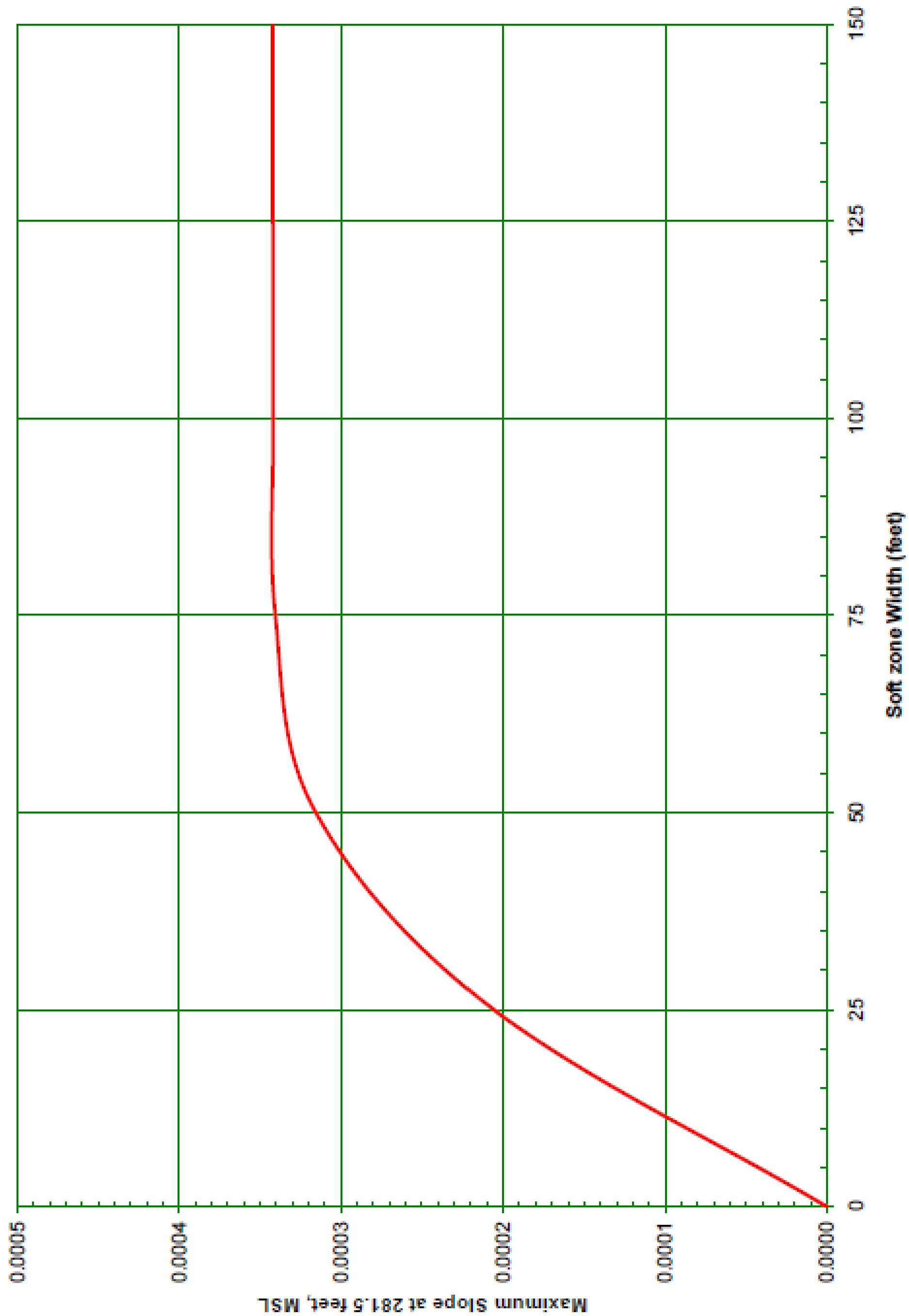


Figure 6: Maximum Surface Slope for Soft Zones of Various Widths

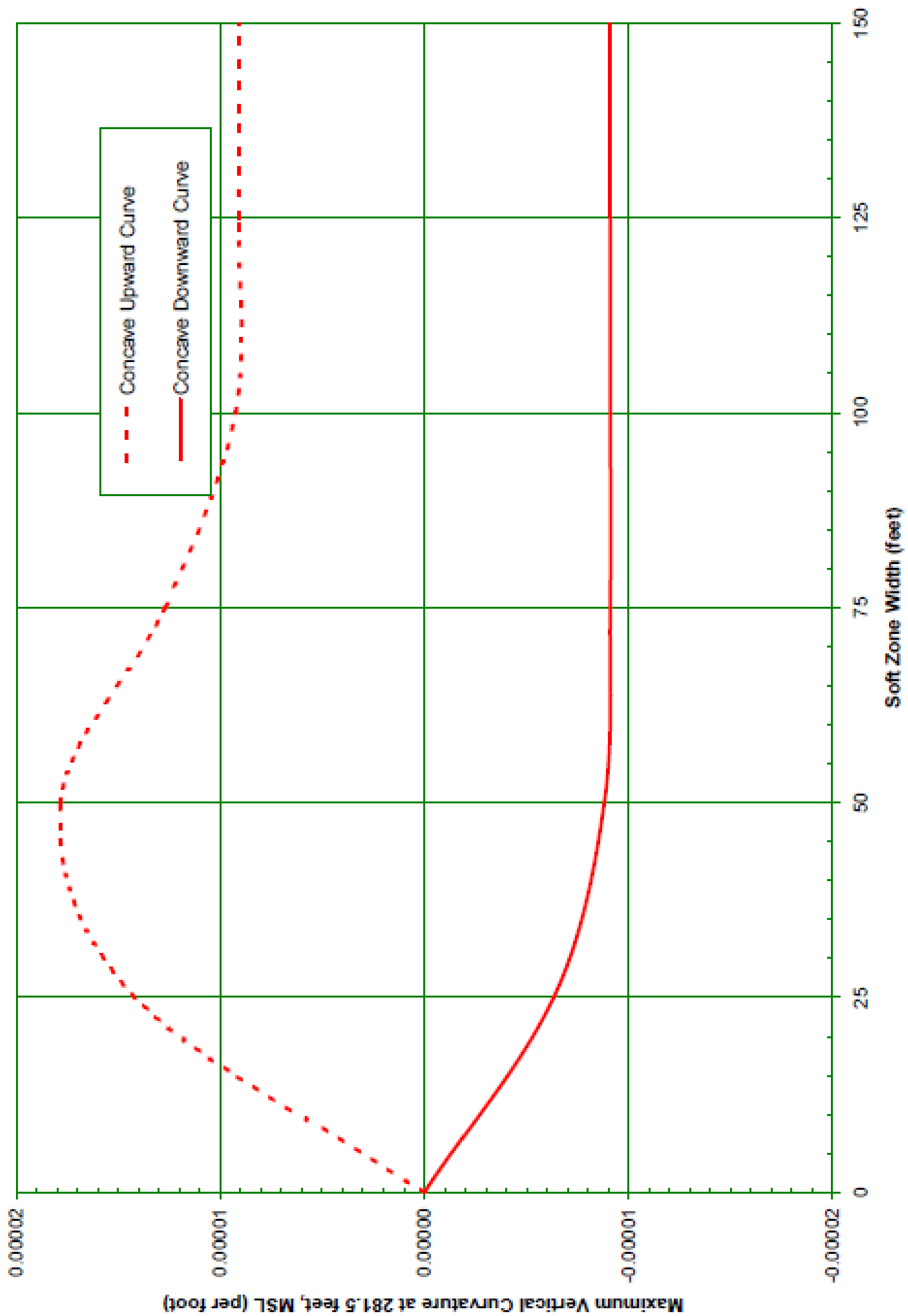


Figure 7: Maximum Surface Settlement Curvature for Soft Zones of Various Widths