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SALTSTONE VAULT NO. 2

GEOTECHNICAL INVESTIGATION REPORT (U)

Geotechnical Engineering
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SRS savannah river site

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DISCLAIMER

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1.0 INTRODUCTION

The Saltstone Production Facility (SPF) immobilizes salt solution by blending it with a dry material mixture containing of cement 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 form called saltstone. For details see Reference 1.

A new vault, Vault No. 2, will be constructed to store the saltstone. The Vault No. 2 basic design consists of a cylindrical type vault. The cylindrical vaults will be constructed of pre-cast concrete panels that will be erected on a poured in place concrete base slab (Ref. 2). The concrete vault will be encased in a High Density Polyethylene (HDPE) and Geosynthetic Clay Liner (GCL) system.

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 Vault No. 2 project site and the results are summarized 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.

2.0 SUBSURFACE EXPLORATION

Subsurface exploration was conducted at the Saltstone Vault No. 2 project site using Piezocone Penetrometer Test (CPTu) soundings, geotechnical boreholes, test-pits, and laboratory testing. Data from previously conducted subsurface explorations in the surrounding areas were also evaluated and included in the evaluation as appropriate. Groundwater monitoring wells in the surrounding area were used to determine the groundwater elevation.

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.

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

2.1 Piezocone Penetration Test Soundings

Eleven CPTu soundings, including seven seismic soundings (SCPTu), two resistivity soundings (RCPTu), and two piezocone soundings (CPTu), were performed in accordance with ASTM D5778. The coordinates, ground elevations, and depths of these CPTu soundings are provided in Table 1. CPTu logs including the tip resistances, sleeve resistances, friction ratios, and pore pressures as well as dissipation test results are provided in Appendix C, page 10 through page 22. SCPTu shear wave velocities are provided in Appendix B, page 26 to page 32.

2.2 Geotechnical Boreholes

Six geotechnical boreholes were drilled at the project site. The Standard Penetration Test (SPT) was conducted in three 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 Appendix A, page 31 through page 63.

2.2.1 Standard Penetration Test

SPT testing was performed in boreholes Z-V2-B1, Z-V2-B2, and Z-V2-B3. The boreholes utilized for SPT testing were sampled continuously in the upper 20 to 30 feet and at five-foot centers for the remaining depth of the boreholes. The boreholes were terminated at a depth of approximately 130 feet.

Tests were performed in accordance with ASTM D1586. SPT N-values 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 SPT energy measurements were taken on the same equipment and the average energy transferred was about 76 percent (Appendix A, page 63). SPT blow counts obtained from the field were converted to appropriate energy levels (Appendix A, p. 4 and Appendix C, p. 6) for evaluations.

2.2.2 Soil Samples

Undisturbed soil samples were obtained form boreholes Z-V2-B1U, Z-V2-B2U, and Z-V2-B3U for laboratory testing with direct push Shelby tubes in accordance with ASTM D1587. Disturbed soil samples were obtained form boreholes Z-V2-B1, Z-V2-B2, and Z-V2-B3 using split-spoon samplers.

2.3 Test Pits

Two test pits were excavated the project site. The coordinates, ground elevations, and depth of the test pits are provided in Table 1. Bag samples were collected from the test pits for laboratory testing.

2.4 Groundwater Monitoring Wells

No groundwater monitoring wells were specifically installed for this investigation. However, groundwater monitoring wells in the surrounding area were used to determine the groundwater elevation. The groundwater elevation at the project site is estimated to be 225 ft, MSL. A seasonal groundwater fluctuation is estimated to be ± 5 feet (Appendix C, p. 5).

2.5 Laboratory Tests

Laboratory tests were performed on undisturbed soil samples obtained form boreholes Z-V2-B1U, Z-V2-B2U, and Z-V2-B3U; disturbed samples collected from borehole Z-V2-B2; and bag samples collected from two test pits. Laboratory tests included index tests, strength tests, consolidation tests, and Proctor tests. The results of the laboratory tests are included in Appendix A, page 64 through page 127.

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. 2 will be 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. The subsurface conditions are depicted on three cross sections. Figure 1 shows the exploratory map and the locations of the cross sections as well as the notes for cross sections. Figures 2, 3, and 4 are the cross sections. CPTu soundings, geotechnical boring logs are provided in Appendix A.

Appendix D 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 CPTu tip resistance, friction ratio, pore pressure, and SCPTu wave velocities for each engineering layers. Table 5 provides the statistics of SPT N-value for each engineering layers. Subsurface conditions of each engineering layer are also described in the following sections in descending order from the ground surface.

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).

Shear wave velocity was not measured since the layer is generally too shallow.

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 the evaluation that follows, being able to distinguish between the TR and DB is unimportant. What is critical is the 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 at 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. The material generally classifies as a MH in the USCS.

3.2 Groundwater

The groundwater level was based on the results of CPTu dissipation tests at the project site (Appendix C, Section 2.2) as well as groundwater measurements available at locations of permanent well installations (Appendix C, Ref. 5). Based on the dissipation tests and the historical data, the average groundwater elevation in the vicinity of the project site is estimated at 225 feet, MSL. An estimated seasonal fluctuation is about ± 5 feet.

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, which occurred about 35 to 50 million years ago. At the location of Saltstone Vault No. 2, 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 to the surface settlement. However, the soft zone at elevation 170 feet, MSL, encountered in SCPTu Z-V2-CP15, under the center of the vault in the southern portion of the site, is approximately 14 feet thick (see Appendix A, Section 3.3). The impact of this soft zone on the surface settlement is discussed in Appendix F.

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, shear and compression wave velocities, electrical resistivity, and pore pressure. SPT's were conducted to acquire blow counts and soil samples for classification purposes. Undisturbed soil samples were obtained from 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 include sieve analysis, determination of Atterberg Limits, unit weight, strength tests, and consolidation tests. In addition to the test data from the project site, previous SRS studies were utilized, where appropriate.

Appendix A provides the details of evaluating data from field exploration and laboratory testing. In addition, theoretical relation was used to compute Poisson's ratio (Appendix A, Section 3.1) and empirical formulas were used to estimate subgrade modulus (Appendix C, Section 3.4). Engineering properties are summarized in Appendix A, Section 4.0 and Appendix C, Section 4.0.

5.0 ENGINEERING EVALUATIONS

Engineering evaluations were performed to determine bearing capacities, liquefaction potential, and slope stability. Appendix E provides the evaluation of bearing capacity, Appendix B provides the evaluation of liquefaction potential, and Appendix G provides the evaluation of slope stability. Engineering evaluations were also performed to estimate the settlement due to static load, liquefaction and partial liquefaction, as well as the compression of soft zones. Appendix E estimates the settlement due to static load, Appendix B estimates the settlement due to liquefaction and partial liquefaction, and Appendix F estimates the settlement due to the compression of the soft zones. Evaluation methodology and results are also summarized in the following sections.

5.1 Bearing Capacities

Ultimate bearing capacity q_o was computed using site-specific soil strength properties. Allowable bearing capacity q_a was computed using the ultimate bearing divided by an appropriate factor of safety. Allowable bearing capacity is used for the design of foundations when the allowable stress design method is chosen.

Design bearing capacity q_d was computed based on site-specific soil strength properties with appropriate strength reduction factors. Design bearing capacity is used for the design of foundations when the strength method is chosen.

Appendix E provides the details of the computations. Computations are also summarized in the following sections.

5.1.1 Ultimate Bearing Capacity

Ultimate bearing capacity q_o is computed using the equations originated by Terzaghi and later modified by others:

$$q_o = q_c + q_q + q_\gamma$$

where $q_c = c' \, N_c \, S_c \, D_c \, G_c$

$$q_q = q \, N_q \, S_q \, D_q \, G_q$$

$$q_\gamma = \gamma \, (B/2) \, N_\gamma \, S_\gamma \, D_\gamma \, G_\gamma$$

q is the overburden or surcharge pressure at the foundation base, γ is the soil unit weight, ϕ' is the effective friction angle, c' is the effective cohesion, and B is the foundation width. N_c, N_q, and N_γ are the bearing capacity factors; S_c, S_q, and S_γ are the shape factors; D_c, D_q, and D_γ are the depth factors; and G_c, G_q, and G_γ are the inclination factors.

Using the above equations with appropriate factors, the ultimate bearing capacity or ultimate pressure applied to foundation to cause a bearing failure is 48,000 psf. Appendix E provides the details of the computations and the references.

5.1.2 Allowable Bearing Capacity

Consider a factor of safety of 3 to the ultimate bearing capacity, the allowable bearing capacity is:

$q_a = q_o/3 = 16,000$ psf

5.1.3 Design Bearing Capacity

The design bearing capacity q_d was computed using the same equations for computing the ultimate bearing capacity. However, the strength parameters including effective cohesion and effective friction angle were reduced using appropriate reduction factors (Appendix E, Ref. 7). The effective cohesion is reduced by applying a factor f_c of 0.5:

$$c_{reduced} = f_c c' = 0.5\ c'.$$

The effective friction angle is reduced by applying a factor f_φ of 0.8 to the tangent of the effective friction angle:

$$\tan(\phi_{reduced}) = f_\phi \tan\phi' = 0.8 \tan\phi'$$

Therefore

$$\phi_{reduced} = \tan^{-1}(0.8 \tan\phi')$$

Using a reduced cohesion $c_{reduced}$ and a reduced friction angle $\phi_{reduced}$, compute the bearing capacity factors, shape factors, and depth factors as before, the design bearing capacity is:

$$q_d = 22,000 \text{ psf}$$

The strength reduction factor Φ for the bearing capacity can be computed as:

$$\Phi = q_d/q_o = 0.45$$

5.2 Static Settlement and Heave

Settlement or heave occur when the overburden changes. At the completion of excavation, the overburden of the soil was reduced between 900 and 2,400 psf. At the completion of the operation, a static load of 3,000 psf will be added to the site. After the installation of the closure cap, additional load of 1,600 to 3,000 psf will be added to the site.

Settlements of each engineering layer under various static loads were computed using appropriate methods for cohesionless or cohesive soils. Total settlements were obtained by superimposing the settlements computed for each engineering layer. Appendix E provides the details of the computations. Computations are also summarized in the following sections.

5.2.1 Settlement of Cohesionless Layers

Settlement of cohesionless layers S1/2, S3, and S4 was computed using Burland and Burbidge method utilizing the SPT results and Schmertmann's method utilizing the CPTu results.

Burland and Burbidge method estimates the immediate settlement ρ_i using the relation (Appendix E, Ref. 11, p.3-10):

$$\rho_i = f_s\, f_1\, [(\Delta P'_{ave} - 2/3\ \sigma'_p)\, B^{0.7} I_c] \text{ for } \Delta P'_{ave} > \sigma'_p$$

$$\rho_i = f_s\, f_1\, \Delta P'_{ave} I_c/3 \text{ for } \Delta P'_{ave} < \sigma'_p$$

where f_s is the shape factor, $[(1.25L/B)/(L/B + 0.25)]^2$

f_1 is the layer correction factor, $H/Z_1(2 - H/Z_1)$

$\Delta P'_{ave}$ is the average effective pressure, $q_{0ave} + \sigma'_{0ave}$

q_{0ave} is the average pressure in stratum from foundation load,

σ'_{0ave} is the effective overburden pressure

σ'_p is the maximum effective past pressure

H is the thickness of the layer

Z_1 is the depth of influence of loaded area

N_{ave} is the average SPT blow count over depth influence by loaded area, provided in Appendix A, page 10

I_c is the compressibility influence factor, $0.23/N_{ave}^{1.4}$ with coefficient of correlation 0.848

The lower bound, upper bound, and average settlement are computed as follows:

Lower bound settlement is computed using $I_{cmin} = 0.08/N_{ave}^{1.3}$

Upper bound settlement is computed using $I_{cmax} = 1.34/N_{ave}^{1.67}$

Average settlement is computed using compressibility influence factor, I_c, defined earlier.

Settlement after time t at least 3 years following construction may be estimated by:

$\rho_c = f_t \rho_i$

where $f_t = 1 + R_i + R_c \log (t/3)$

R_i and R_c are the time–dependent settlement ratios, for t = 30 years

$R_i = 0.3$ and $R_c = 0.2$

$f_t = 1 + 0.3 + 0.2 \log (30/3) = 1.5$

Therefore, primary and secondary settlement is:

$\rho_c = 1.5\, \rho_i$

The lower bound, upper bound, and average settlements were computed using I_{cmin}, I_{cmax}, and I_c as described earlier. For phases after closure cap completed, lower bound settlements were computed considering a minimum cap thickness of 13 feet while upper bound settlements were computed considering a maximum cap thickness of 23.5 feet. The average settlement is the average of four settlements computed using lower and upper bounds as well as minimum and maximum cap thicknesses. The results are:

Phase	Lower Bound Settlement (inches)	Upper Bound Settlement (inches)	Average Settlement (inches)
Immediate after operation completed	0.9	5.3	2.1
Immediate after closure cap completed	1.2	8.3	3.1
30 Years after closure cap completed	1.9	12.4	4.6

Appendix E provides the details of the computation.

Schmertmann method estimates the immediate settlement ρ_i using the equation (Appendix E, Ref. 11, p.3-6):

$$\rho_i = C_1\, C_t\, \Delta p\, \Sigma\, (\Delta z_i\, I_{zi}/\, E_{si})$$

where C_1 is the correction to account from strain relief from embedment

C_t is the correction for time-dependent increase in settlement and $C_t = 1 + 0.2 \log (t/0.1)$

t is time in years, for t = 30 years, $C_t = 1 + 0.2 \log (30/0.1) = 1.5$.

Δp is the net applied footing pressure

Δz_i is the depth increment i

I_{zi} is the influence factor of soil layer i

$E_{si} = 2.5\, q_t$ is the elastic modulus of soil layer I for axisymmetric footings L/B = 1

where q_t is the average tip stress of soil layer i in tsf.

For this evaluation, thickness of each soil layer is 2 feet. Tip stress q_t is provided in Appendix A, pages 20 through 30.

Previous analyses performed in a similar site found that the results obtained from Schmertmann method were approximately 2.3 times the actual measurements (Appendix E, Ref. 11). Realistic results can be obtained by increasing the elastic modulus E_{si}, i.e., multiply E_{si} by an adjustment factor of 2.3. Since the resulting settlement is a linear function of elastic modulus, in lieu of multiplying the elastic modulus by an adjustment factor, the resulting settlement was divided by the same adjustment factor. For this evaluation the results were divided by a conservative factor of 2.

Settlements were computed using data from each of the seven deep SCPTu's. For phases after closure cap completed, both the minimum cap thickness of 13 feet and the maximum cap thickness of 23.5 feet were considered. The average settlement is the average of 14 settlements computed using seven SCPTu's as well as minimum and maximum cap thicknesses. The results are:

Phase	Minimum Settlement (inches)	Maximum Settlement (inches)	Average Settlement (inches)
Immediate after operation completed	0.7	4.1	1.6
Immediate after closure cap completed	1.1	8.5	3.0
30 Years after closure cap completed	1.6	12.8	4.4

Appendix E provides the details of the computation.

5.2.2 Settlement of Cohesive Layer

Settlement of cohesive Layer C2 was computed using the one-dimensional consolidation equation (Appendix E, Ref. 11):

$$\rho_p = H_0 \Delta e/(1 + e_0)$$

where H_o = initial height of the layer

e_0 = 1.38, initial void ratio

Δe = change in void ratio

For a normally consolidated soil

$$\Delta e = C_c \log (P_f / P_o)$$

For an overconsolidated soil

$$\Delta e = C_r \log (P_f / P_p) + C_c \log (P_p/P_o)$$

where C_c = 0.75 is the virgin compression index

C_r = 0.090 is the re-compression index

P_o = initial effective overburden pressure

P_p = preconsolidation stress

P_f = final applied effective pressure

Layer C2 is approximately 15 feet thick and 56 feet below the foundation at the north side and 19 feet thick 41 feet below the foundation at the south side. Settlement was computed using appropriate compression indexes based on the applied stress. The average heave is the average of the heaves considering minimum and maximum depths of excavation. The average settlement is the average of the settlements considering minimum and maximum cap thicknesses. The estimated settlements of cohesive layer C2 are:

Phase	Minimum Settlement/ Heave (inches)	Maximum Settlement/ Heave (inches)	Average Settlement/ Heave (inches)
Heave after Excavation	-0.2	-1.0	-0.6
Immediate after operation completed	1.5	2.5	2.0
Immediate after closure cap completed	3.7	5.0	4.3

Note that settlements include the recompressions of heave. Appendix E provides the details of the computation.

5.2.3 Settlement Data

In some instances, historical settlement data from similar site under similar conditions may be available for estimating the settlement. Settlement data is available for the Vitrification Building located southwest of the project site, less than 4,000 feet away.

Stratigraphy at the Vitrification Building site contains the same types of engineering layers as the saltstone site. Groundwater at the Vitrification Building site is approximately 30 to 35 feet below the foundation compared to an average of 43 feet below the foundation at the saltstone site. Consolidation properties obtained from laboratory tests are very similar (Appendix E Ref. 9). The foundation of the Vitrification Building is 117 feet by 362 feet, the bottom elevation of the foundation is 270 feet, MSL, and the static building load is between 5,000 to 5,500 psf. These dimensions and load are comparable to the saltstone facility.

Based on the similarities of the stratigraphy, soil properties, and facility configurations, as well as the availability of the settlement data. Settlement data from the Vitrification Building was used to estimate the settlement of the project site.

The total settlement of the Vitrification Building over a 10-year period since excavation is between 2.148 to 3.510 inches. Since most of the layers are cohesionless, it is assumed that the settlement is linearly elastic and the subgrade modulus is computed as:

$$K = p/D$$

where p is the load and D is the settlement. Using an average pressure of 5,250 psf, the subgrade modulus can be computed as a minimum value of 10.39 pci and a maximum value of 16.97 pci. For phases after closure cap completed, minimum settlements were computed considering a minimum cap thickness of 13 feet and a maximum subgrade modulus while maximum settlements were computed considering a maximum cap thickness of 23.5 feet and a minimum subgrade modulus. The average heave is the average of four heaves computed using minimum and maximum subgrade moduli as well as minimum and maximum depths of excavation. The average settlement is the average of four settlements computed using minimum and maximum subgrade moduli as well as minimum and maximum cap thicknesses. The results are:

Phase	Minimum Settlement/ Heave (inches)	Maximum Settlement/ Heave (inches)	Average Settlement/ Heave (inches)
Heave after Excavation	-0.4	-1.6	-0.9
10 years after operation completed	1.2	2.0	1.6
10 years after closure cap completed	1.9	4.0	2.9

Note that settlements include the recompression of heave. Appendix E provides the details of the computation.

5.2.4 Total Settlements and Heave

Based on the settlement computed in the previous sections, the estimated total settlements and heaves are summarized as follows:

Phase	Minimum Settlement/ Heave (inches)	Maximum Settlement/ Heave (inches)	Average Settlement/ Heave (inches)
Heave after Excavation	- 1/2	- 1-1/2	- 1
Immediate after operation completed	2	7	4
Immediate after closure cap completed	5	13	7
30 Years after closure cap completed	6	18	9

Note that settlements include the recompression of heave. The amount of differential settlement or heave at various phases is approximately the average settlement or heave given above.

When one of the vaults is being filled with saltstone and the other vault is left empty, differential settlement will occur due to the differential loading. The portion of the empty vault adjacent to the vault being filled will experience more settlement than the portion opposite the vault being filled. This type of differential settlement was not considered in the analysis, but can be eliminated if both vaults are filled simultaneously. However, this type of differential settlement will be diminished when both vaults are filled.

Due to the large overburden pressure, settlement was predicted with large amount of deviations. In order to verify the estimated settlement, it is recommended that settlement monitoring points be installed and settlement be monitored during construction and operation phases. Settlement data will provide important information for the actual settlement and will be used to verify and calibrate the predicted settlement as well as for the design of the closure cap.

5.3 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.

Appendix B provides the evaluation of liquefaction potential and estimation of settlement due to liquefaction and partial liquefaction. Appendix F provides the computations of compression of the soft zone and estimation of settlement due to the compression of soft zone. Computations are also summarized in the following sections.

5.3.1 Liquefaction Potential

A modified version of the simplified procedure (Appendix B, Ref. 8) was used to evaluate the liquefaction potential. The modification being use of SRS site specific CRR as described in Appendix B, pages 5 and 6. 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

Appendix B provides the details of computing CRR, CSR, and FS.

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 B, Figure 8).

The liquefaction analyses were performed using shear wave velocity data and SCPTu tip resistance results. Both the methods were used to determine CRR in conjunction with the Design Basis Earthquake (DBE) having a peak ground acceleration (PGA) of 0.21g. The fines content was estimated based on the SCPTu data and determined from laboratory testing. Appendix B provides the detailed analyses.

Liquefaction analyses using shear wave velocity and CPTu tip stress suggest 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, Figures 9 and 10 show plots of the factor of safety versus the depth for one of the CPTu locations.

5.3.2 Settlement due to Liquefaction and Partial Liquefaction

Settlements were calculated using the SRS volumetric strain relationship for each of the seven SCPTu and for all magnitudes in the USGS seismic hazard for SRS. Appendix B, Figure 8 shows the SRS relationship between the volumetric strain and the factor of safety. Settlement versus depth for SCPTu location Z-V2-CP15 is also computed for several different magnitudes. Note that SCPTu location Z-V2-CP15 was selected for presentation as it has the highest calculated settlement of the seven SCPTu for the site for a magnitude 7.5 earthquake (see Appendix B, Table 7).

Settlements were not calculated using the shear wave safety factors, as the liquefaction and partial liquefaction strain are functions of CPTu tip stress. In addition the factor of safety calculated from shear wave velocity is typically higher than that calculated using the CPTu tip stress method.

Settlement due to liquefaction and partial liquefaction ranges from near zero to nearly 2-1/4 inches depending on SCPTu location and earthquake magnitude (see Appendix B, Table 7). The magnitude weighted average using the USGS PGA hazard dissagregation is less than an inch for the 2,500-year earthquake.

5.3.3 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.

5.3.4 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 F). 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 due to the compression of soft zone, at the ground surface, the maximum settlement is 1/2 inch, the maximum differential settlement is 1/2 inch, the maximum slope is 0.00075 feet/feet, and the maximum curvature of the slope of 0.000035 per foot. Appendix F provides the detailed calculation.

5.4 Post-Closure Slope Stability

The post-closure geometry at the Saltstone Vault No. 2 is assumed to be similar to the proposed post-closure geometry at Vault No. 4. However, final design of the closure system will determine slope geometry. It is anticipate that further slope stability analysis will be performed during final design and that final slopes will be designed to remain stable for the DBE. Post-closure slope stability evaluation of Vault No. 4 was performed using the Spencer method. This method was chosen because it satisfies both force and moment equilibrium of a sliding mass of soil.

Two independent computer software programs, SLOPE/W and PCSTABL, were used to compute the results. Appendix G provides the details of the analysis.

The post-closure geometry used in the analysis assumes 20 feet of compacted fill will be placed with side slopes conservatively chosen to be 4 H to 1 V. For the analysis, it is assumed that the compacted fill will be taken from onsite borrow sources and will be placed and compacted to SRS standards, i.e., 95% of ASTM D-1557 in loose lifts not exceeding 12 inches.

Comparisons of subsurface conditions at Vault No. 4 (Appendix G, Figure 1) with those at Vault No. 2 (Figures 2 through 4) show good correlation between Vault No. 2 and Vault No. 4 soils. The acceptable safety factors were chosen based on past experience at the SRS and recommendations from the literature. For the static and pseudostatic (seismic) cases, the minimum acceptable safety factor was chosen as 1.5 and 1.0, respectively. These safety factors are common in the industry, although higher values have been used for the seismic case.

The minimum required safety factor for static and dynamic conditions is 1.5 and 1.0, respectively. The results in Tables 1 and 2 of Appendix G show that the stability of the assumed post-closure condition (4 H to 1 V) meets these requirements and is therefore stable under the conditions and assumptions analyzed. However, final design will dictate final closure slopes.

6.0 SUMMARY AND CONCLUSIONS

Extensive geotechnical investigation was performed for the Saltstone Vault No. 2 project. Geotechnical exploration was conducted at the project site. Site-specific data were collected utilizing Piezocone Penetrometer Test soundings, geotechnical boreholes, test-pits, 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. Engineering evaluations were performed on bearing capacities, liquefaction potential, and slope stability. Settlement due to static load, liquefaction, and compression of soft zones was estimated.

Bearing capacities for the Saltstone Vault No. 2 are shown in two different formats;

1. For the ultimate design method, the ultimate bearing capacity is 48,000 psf, the strength reduction factor is 0.45, and the design bearing capacity is 22,000 psf.
2. For the allowable stress design method, the allowable static bearing capacity is 16,000 psf and the allowable dynamic bearing capacity is 21,000 psf.

Bearing capacities for both design methods are significantly higher then the expected loadings. Therefore, the soils will support the foundation with sufficient margin of safety to failure.

To evaluate the adequacy of the facility, in additional to the bearing capacity of the soil, settlement of the facility should also be considered. 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.

The following table summarizes settlements at various phases:

Phase	Average Static Heave or Settlement (inches)	Dynamic Settlement due to Liquefaction & Partial Liquefaction (inches)	Dynamic Settlement due to Soft Zone Compression (inches)
Heave after Excavation	-1	N/A	N/A
Immediate after operation completed	4	N/A	N/A
Immediate after closure cap completed	7	N/A	N/A
30 Years after closure cap completed	9	N/A	N/A
Due to DBE event	N/A	0 to 2-1/4	0 to 1/2

Note that settlements include the recompressions of heave. The differential heave or settlement is estimated as the same as the average static heave or settlement shown above.

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 estimated dynamic settlement due to liquefaction and partial liquefaction range from about 0 to 2-1/4 inches. This settlement is expected to be rather uniform and not contribute to differential settlement.

After the design basis earthquake, compression may occur at the soft zones. This compression will propagate to the ground surface and cause the ground to settle. The estimated maximum settlement and the estimated maximum differential settlement is 1/2 inch, the estimated maximum slope is 0.00075, and the estimated maximum curvature of the slope is 0.000035 per foot.

Both static and pseudo-static slope stability analyses show that assumed post-closure condition is stable with minimum computed safety factors well in excess of the required safety factors. In addition, the computed safety factors conservatively account for the soil strength and the application of the horizontal and vertical DBE seismic loads. However, additional stability analyses will be done once the final closure system is determined. The analyses should take into account any geosynthetics and the interface shear resistance between the geosynthetic materials and the compacted fill. Based on the slopes being considered, it is not expected that static nor dynamic slope stability will be an issue.

Settlement monitoring points shall be installed on the foundation. Settlement surveying shall be conducted as soon as the monitoring points are installed. Surveying results shall be evaluated to verify the estimated settlement.

REFERENCES

(1) WSP-SSF-2005-0023, Application for Modification, Z-Area Industrial Solid Waste Landfill Permit # 025500-1603, Engineering Report for Vault Two Construction, Rev. 0, 2006.

(2) M-TC-Z-00004, Saltstone Facility Cylindrical Vault No. 2 Project, Bldg 451-002Z (U), Rev. 1, October 2005.

Table 1 Subsurface Exploration Locations

Type of Exploration	CPTu or SCPTu I.D.	SRS North (feet)	SRS East (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
SCPTu	Z-V2-CP5	77,192	67,008	287.0	114.0
SCPTu	Z-V2-CP6	77,293	66,929	283.0	107.0
SCPTu	Z-V2-CP7	77,282	67,082	279.4	110.0
SCPTu	Z-V2-CP8	77,360	66,983	278.8	113.0
SCPTu	Z-V2-CP9	77,483	67,019	275.3	111.5
SCPTu	Z-V2-CP10	77,470	66,880	279.0	103.0
RCPTu	Z-V2-CP11	77,191	67,145	283.0	40.0
CPTu	Z-V2-CP12	77,145	66,989	289.7	40.0
CPTu	Z-V2-CP13	77,234	66,895	286.2	40.0
RCPTu	Z-V2-CP14	77,381	66,859	280.8	40.0
SCPTu	Z-V2-CP15	77,270	67,020	281.7	143.5
SPT	Z-V2-B1	77,272	67,024	281.7	132.0
SPT	Z-V2-B2	77,353	66,983	279.0	132.0
SPT	Z-V2-B3	77,422	66,957	278.1	131.5
UD	Z-V2-B1U	77,271	67,018	281.9	123.5
UD	Z-V2-B2U	77,151	66,979	289.7	36.0
UD	Z-V2-B3U	77,430	66,953	278.0	29.0
Test Pit	TP-1	77,311	66,936	281.5	12.0
Test Pit	TP-2	77,266	67,070	280.9	12.0

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	
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 SCPTu Locations

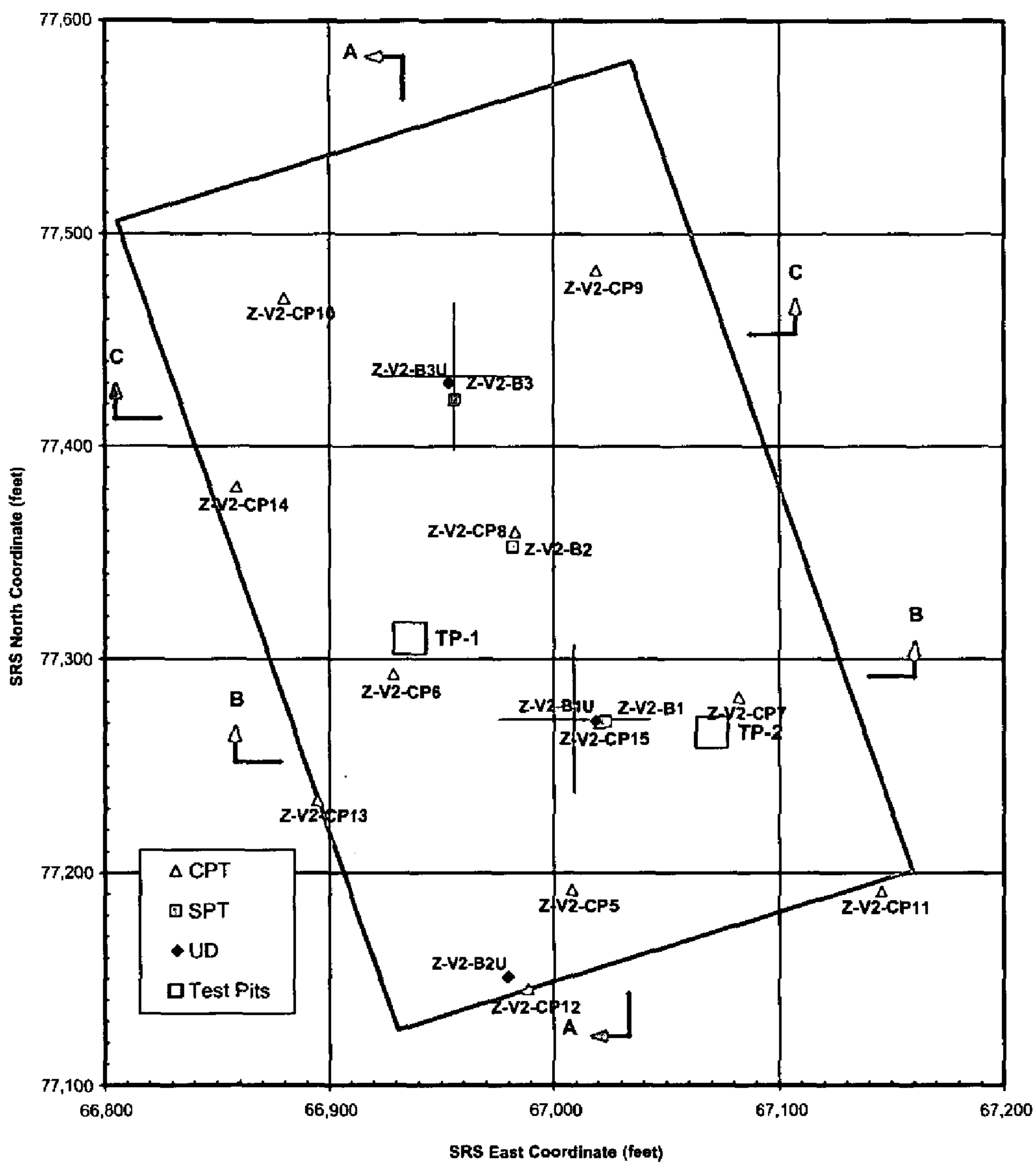
Exploratory I.D.	Ground Elevation (feet MSL)	Contact between Layers S1/2 and C2 (ft, MSL)	Contact between Layers C2 and S3 (ft, MSL)	Contact between Layers S3 and S4 (ft, MSL)
Z-V2-CP5	287.0	227	208	176
Z-V2-CP6	281.5	230	212	178
Z-V2-CP7	280.9	223	207	176
Z-V2-CP8	279.0	224	212	184
Z-V2-CP9	275.3	212	197	166
Z-V2-CP10	279.0	228	213	186
Z-V2-CP15	281.7	223	208	181

Table 4 CPTu and SCPTu Data

Layer	Description	Sleeve Friction (tsf)	Tip Resis-tance (tsf)	Friction Ratio (%)	Pore Pressure (tsf)	P Wave (fps)	S Wave (fps)	Resis-tivity (Ω-m)
S1	Average	4.0	108	4.1	1.3	2,519		2,066
	Minimum	0.8	40	0.7	-0.5	2,116		1,019
	Maximum	8.0	401	9.4	26.4	3,431		20,280
	Standard Deviation	1.5	47	1.8	3.3			2,025
	No. of Data Points	1,340	1,340	1,340	1,340	81		315
S2	Average	1.4	156	1.0	0.0	2,082	1,156	2,445
	Minimum	0.2	24	0.2	-0.4	1,501	931	-387,405
	Maximum	6.3	490	5.4	1.1	4,454	1,432	278,888
	Standard Deviation	0.7	70	0.5	0.1			28,743
	No. of Data Points	5,157	5,174	5,157	5,174	3,504	3,355	519
C2	Average	0.7	75	1.8	2.8	2,388	976	
	Minimum	0.1	9	0.2	-0.6	1,839	714	
	Maximum	2.7	362	9.1	16.1	3,663	1,307	
	Standard Deviation	0.4	82	1.2	4.1			
	No. of Data Points	1,521	1,521	1,521	1,521	304	1,159	
S3	Average	1.1	136	1.2	1.9		1,063	
	Minimum	0.1	3	0.1	-0.6		671	
	Maximum	11.9	418	12.5	33.8		1,892	
	Standard Deviation	0.9	99	1.0	3.4			
	No. of Data Points	2,926	2,926	2,926	2,926		2,478	
S4	Average	1.6	105	1.5	9.9		1,033	
	Minimum	0.1	8	0.2	-0.6		556	
	Maximum	17.3	528	8.7	288.4		1,954	
	Standard Deviation	2.5	107	1.3	17.4			
	No. of Data Points	1,417	1,455	1,417	1,455		1,144	
All	Average	1.6	130	1.6	2.1	2,111	1,081	2,302
	No. of Data Points	12,378	12,416	12,378	12,416	3,889	8,136	834

Table 5 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



Notes for cross sections:

1 Cone penetrometer tip stress abscissa shows the scale of 0 to 200 tsf. Cone penetrometer friction ratio abscissa shows the scale of 0 to 4 percent. For detailed test result, see Appendix A, pp. 20, Cone Penetrometer Test Logs.

2 Standard Penetration Test N-values abscissa shows the scale of 0 to 50 blow count. for *detailed test results, see* Appendix A, *pp.* 31, *Geotechnical Borehole Logs*

Figure 1 Geotechnical exploratory map and notes for cross sections

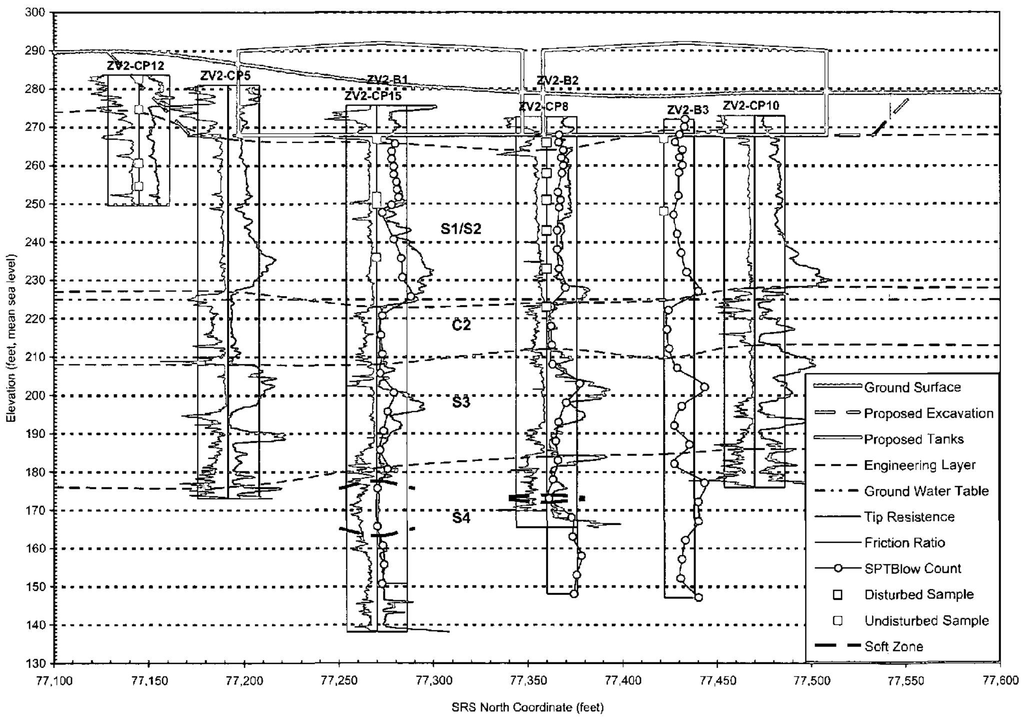


Figure 2 Geotechnical cross section A-A
(see notes in Figure 1)

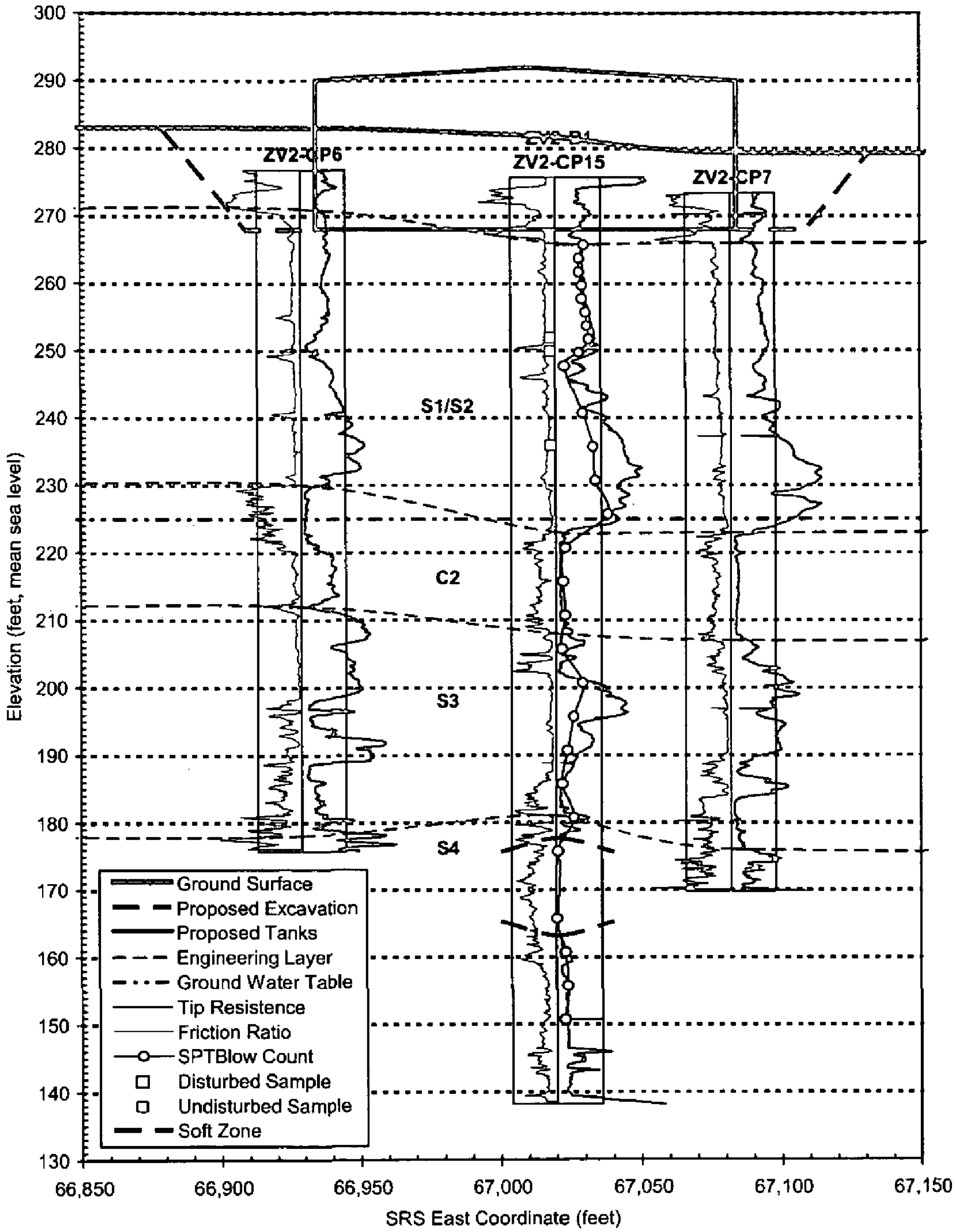


Figure 3 Geotechnical cross section B-B

(see notes in Figure 1)

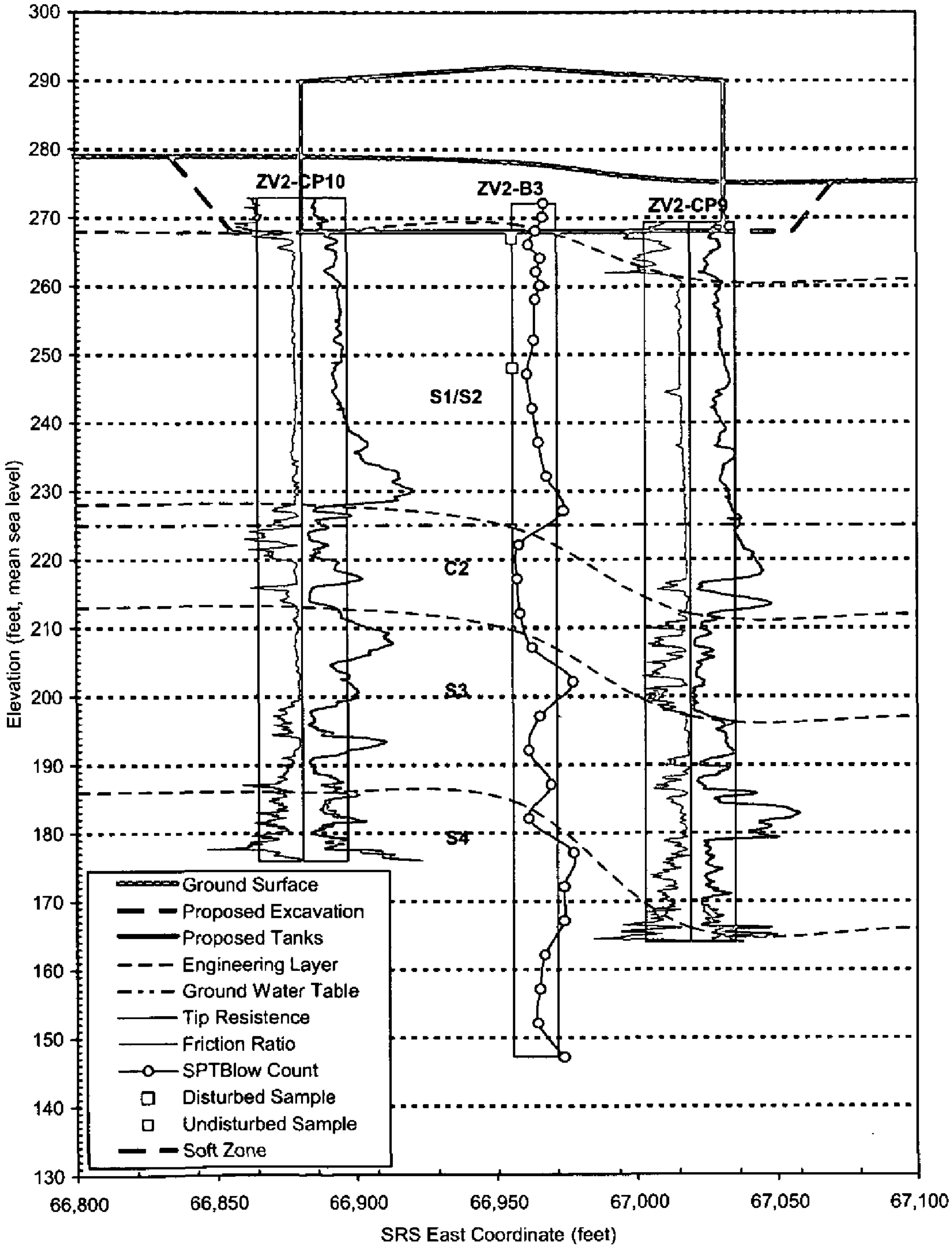


Figure 4 Geotechnical cross section C-C

(see notes in Figure 1)

Appendix A

K-CLC-Z-00008, Evaluation of Test Data

Rev. 0, February, 2006

(128 pages)

ENGINEERING DOC. CONTROL–SRS

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Calculation Cover Sheet

Project	Calculation No.	Project No.
Saltstone Vault No. 2	K-CLC-Z-00008	N/A
Title: Evaluation of Test Data	Functional Classification: PS	Sheet 1 of 127
	Discipline: Geotechnical	
Calc Level: X Type 1 Type 2	Type 1 Calc Status: Preliminary X Confirmed	
Computer Program No.: X N/A	Version/Release No.: N/A	

Purpose and Objective

This calculation provides the evaluation of test data.

Summary of Conclusion

see last section.

UNCLASSIFIED
DOES NOT CONTAIN
UNCLASSIFIED CONTROLLED
NUCLEAR INFORMATION
ADC & Reviewing Official [signature] (Name and Title)
Date 2-15-2006

Revisions

Rev No.	Revision Description
0	Initial issue

Sign Off

Rev No.	Originator (Print) Sign/Date	Verification/ Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign//Date
0	William Li [signature] 1-25-05	Document review	ALARABI ELCHOUFI [signature] 1-26-06	[signature] 2/9/06

Design Authority — (Print)	Signature	Date
Release to Outside Agency – (Print)	Signature	Date

Security Classification of the Calculation

TABLE OF CONTENTS

1. INTRODUCTION

This calculation provides the evaluation of the geotechnical test data for the project site.

2. INPUT DATA

Input data includes the field exploration and the laboratory test data as described in the following sections.

3.1 Field Exploration

Field exploration includes Piezocone Penetrometer Tests (CPTu), seismic Piezocone Penetrometer Tests (SCPTu's), geotechnical boreholes, and test pits. Tables 1, 2, and 3 show the field exploration performed at the project site. Figure 1 shows the geotechnical exploratory map. Figures 2, 3, and 4 show the subsurface cross sections. Appendix A provides the CPTu and SCPTu logs (Ref. 1). Appendix B provides the geotechnical Borehole Logs.

3.2 Laboratory Testing

Laboratory tests were performed on samples retrieved from geotechnical boreholes and test pits. These boreholes and test pits are listed in Tables 2 and 3, respectively. Table 4 shows the elevations of the engineering layers at these geotechnical boreholes and test pits where the soil samples were taken.

Table 5 shows the undisturbed soil samples used for the evaluation. Table 6 shows the disturbed soil samples used for the evaluation. Appendix C provides the laboratory test reports (Ref. 2, 3, and 4). Engineering Layer for each sample was also shown in Tables 5 and 6. These layers were identified in Reference 5.

3. COMPUTATIONS

3.1 CPTu and SCPTu Data

Statistics of CPTu and SCPTu data, including tip resistances, Sleeve frictions, friction ratios, pore pressures, compressive wave velocities, shear wave velocities, and sensitivities are show in Table 7.

Average compression and shear wave velocity for each engineering layer was computed using the following equation:

$$V_{ave} = \Sigma d_i / (\Sigma (d_i / V_i)$$

where d_i is the distance between two points at Location i where the wave velocity is measured

V_i is the measured wave velocity at Location i

Poisson's ratio is computed using the following equation:

$$v = (2V_s^2 - V_p^2) / (2V_s^2 - 2V_p^2)$$

where v is the Poisson's ratio

V_s and V_p are the shear and compression wave velocities at each data point.

As shown in Figure 5, some of the Poisson's data points provided unreasonable large deviations from average. Some of the data points are unreasonably large or unreasonably small or even negatives. These large deviations were the results of wave velocities consist of large deviations from their actual values. The deviations of Poisson's ratio data points will not be seen explicitly if the averages of the wave velocities were used to compute the Poisson's ratio rather then compute the Poisson's ratio from wave velocities at each data point then take the average of these Passion's ratios. Without introducing any bias judgment, all the wave velocities, regardless their values, were used. Consequently, Poisson's ratio data points, regardless the values, from all the wave velocity data points, were used to compute the averages and if needed, standard deviations and confidence levels.

The average Poisson's ratios are 0.236 and 0.368, respectively, above and below the groundwater table. The recommended Poisson's ratios are 0.25 and 0.40, respectively, above and below the groundwater table.

3.2 SPT Data

Table 8 shows the statistics of blow counts for various Engineering Layers. Also shown are energy-corrected blow counts N_{60}, corresponding to 60% of input energy using:

$$N_{60} = (ER/60)\ N_{MEASURED}$$

where $N_{MEASURED}$ is the measured blow counts

ER, the average energy ratio measured was 76% (last sheet of Appendix B).

Average blow count for Layer S4 is not available since some of the locations were advanced due to the weight of the rod only and other locations met refusal criteria.

3.3 Soft Zone Locations

Soft zones are indicated from SPT N values less than 5 or CPTu (or SCPTu) tip resistances less than 15 tsf over an interval of two feet or greater. Table 9 summarizes zones having tip resistance less than 15 tsf for the CPTu's (or SCPTu) or N value less than 5 for the SPT's at the project site, regardless of thickness.

The most critical soft zone is 14 feet thick at SCPTu location Z-V2-CP15, found under the center of one of the vault. This finding is consistent with the SPT results from geotechnical borehole Z-V2-B1, approximately 3 feet east from SCPTu Z-V2-CP15. No soft zone was found at similar elevation at three SCPTu's, Z-V2-CP5, Z-V2-CP6, and Z-V2-CP7 under the perimeter of the vault. Distances from Z-V2-CP15 to these three CPTu's are approximately 75 feet. Based on the limited data, the best estimated lateral extend of the soft zone is 75 feet.

3.4 Laboratory Data

Laboratory test data are evaluated in the following sections.

3.4.1 Grain size Distribution and Atterberg Limits

Results of particle size analyses and Atterberg Limits testing performed on selected samples are included in Tables 10 and 11, respectively.

3.4.2 Unit Weight and Moisture Contents

Unit weight and moisture contents tests were determined on selected undisturbed soil samples, some of the results were part of the strength testing. The results are provided in Table 12.

3.4.3 Strength

Strength tests were performed on selected samples. Test results are summarized in Table 13. Strength test results are consistent with the results obtained from corresponding engineering layers elsewhere at the SRS (Ref. 6), except that the total friction angle of Sample No. Z-V2-B2U-ST1 appears to be an anomaly and is deleted from analyses conservatively. Based on the test results, p-q and p'-q diagrams were evaluated (Ref. 7).

3.4.4 Consolidation

Consolidation tests were performed on selected soil samples. Pre-consolidation pressures were determined using Casagrande method (Ref. 8). Table 14 summarize the initial pressures, pre-consolidation pressures, over-consolidation ratios, compression Indexes, re-compression indexes, and ratio of compression to re-compression indexes. These test results are consistent with the results obtained from corresponding engineering layers elsewhere at the SRS (Ref. 6).

4. RESULTS AND CONCLUSION

Table 7 provides statistic of CPTu and SCPTu data. Table 8 provides statistics of SPT data.

The recommended Poisson's ratios are 0.25 and 0.40, respectively, above and below the groundwater table.

The most critical soft zone is 14 feet thick at CPT under the center of one of the vault. The best estimated lateral extent of this soft zone is 75 feet.

Tables 10 and 11 provide particle size analyses and Atterberg Limits, respectively.

Table 12 provides the statistics of moisture contents and unit weight. The recommended unit weight is 120 pcf.

Table 13 provides effective and total strength parameters. Note that the total friction angle of Sample No. Z-V2-B2U-ST1 appears to be an anomaly.

Table 14 provides the consolidation parameters including OCR, C_c, C_r, and C_c/C_r.

5. REFERENCES

1. Subcontract No. AC39054N, Task 3, Applied Research Associates, Inc., Cone Penetrometer Data Report, July 2005.
2. Subcontract No. AB80187N Sheet 003, Task 8, QORE Laboratory Test Report, June 2005.
3. Subcontract No. AB80188N, Sheet 014, Task 13, GeoTesting Express Laboratory Test Report, June 2005.
4. Subcontract No. AB80188N, Sheet 015, Task 17, GeoTesting Express Laboratory Test Report, July 2005.
5. K-CLC-Z-00005, Stratigraphy for the Saltstone Vault No. 2, Rev. 0, July 2005
6. WSRC-TR-96-0069, F-Area Geotechnical Report, Rev. 0, September 1996.
7. K-CLC-Z-00007, Vault No. 2 Slope Stability Analysis, Rev. 0, October 2005.
8. ASTM D 2435-04, Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.

Table 1 Piezocone Penetrometer Tests at the Project Site

CPTu or SCPTu I.D.	Type of CPTu or SCPTu	SRS North (feet)	SRS East (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
Z-V2-CP5	Seismic	77,192	67,008	287.0	114.0
Z-V2-CP6	Seismic	77,293	66,929	283.0	107.0
Z-V2-CP7	Seismic	77,282	67,082	279.4	110.0
Z-V2-CP8	Seismic	77,360	66,983	278.8	113.0
Z-V2-CP9	Seismic	77,483	67,019	275.3	111.5
Z-V2-CP10	Seismic	77,470	66,880	279.0	103.0
Z-V2-CP11	Resistivity	77,191	67,145	283.0	40.0
Z-V2-CP12	Non-seismic	77,145	66,989	289.7	40.0
Z-V2-CP13	Non-seismic	77,234	66,895	286.2	40.0
Z-V2-CP14	Resistivity	77,381	66,859	280.8	40.0
Z-V2-CP15	Seismic	77,270	67,020	281.7	143.5

Table 2 Geotechnical Boreholes at the Project Site

Borehole I.D.	Type of Borehole	SRS North (feet)	SRS East (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
Z-V2-B1	SPT	77,272	67,024	281.7	132.0
Z-V2-B2	SPT	77,353	66,983	279.0	132.0
Z-V2-B3	SPT	77,422	66,957	278.1	131.5
Z-V2-B1U	UD	77,271	67,018	281.9	123.5
Z-V2-B2U	UD	77,151	66,979	289.7	36.0
Z-V2-B3U	UD	77,430	66,953	278.0	29.0

Table 3 Test Pits at the Project Site

Test Pit I.D.	SRS North (feet)	SRS East (feet)	Ground Elevation (ft, MSL)	Total Depth (feet)
TP-1	77,311	66,936	281.5	12.0
TP-2	77,266	67,070	280.9	12.0

Table 4 Engineering Layers at Geotechnical Borehole Locations

Exploratory I.D.	Nearest CPTu or SCPYu	Ground Elevation (feet MSL)	Contact Between S1/2 and C2 (ft, MSL)	Contact Between C2 and S3 (ft, MSL)	Contact Between S3 and S4 (ft, MSL)
TP-1	Z-V2-CP6	281.5	< 269.5	-	-
TP-2	Z-V2-CP7	280.9	< 268.9	-	-
Z-V2-B2	Z-V2-CP8	279.0	224.0	212.0	184.0
Z-V2-B1U	Z-V2-CP15	281.9	223.0	208.0	181.0
Z-V2-B2U	Z-V2-CP5,12	289.7	< 253.7	-	-
Z-V2-B3U	Z-V2-CP8,9,10	278.0	< 247.0	-	-

Table 5 Undisturbed Soil Samples

Sample No.	Layer	SRS North (feet)	SRS East (feet)	Ground Elev. (ft, msl)	Depth from (feet)	Depth to (feet)	Elev. from (ft, msl)	Elev. to (ft, msl)
Z-V2-B1U-ST1	S1/2	77,271	67,018	281.9	14.0	16.0	267.9	265.9
Z-V2-B1U-ST2	S1/2	-	-	-	29.0	31.0	252.9	250.9
Z-V2-B1U-ST3	S1/2	-	-	-	31.0	33.0	250.9	248.9
Z-V2-B1U-ST4	S1/2	-	-	-	45.0	47.0	236.9	234.9
Z-V2-B1U-ST5	C2	-	-	-	62.0	64.0	219.9	217.9
Z-V2-B1U-ST7	S3	-	-	-	87.0	89.0	194.9	192.9
Z-V2-B1U-PS1	S4	-	-	-	104.25	105.2	177.7	176.7
Z-V2-B1U-PS3	S4	-	-	-	122.0	124.0	159.9	157.9
Z-V2-B2U-ST1	S1/2	77,151	66,979	289.7	6.0	8.0	283.7	281.7
Z-V2-B2U-ST2	S1/2	-	-	-	14.0	16.0	275.7	273.7
Z-V2-B2U-ST3	S1/2	-	-	-	28.0	30.0	261.7	259.7
Z-V2-B2U-ST4	S1/2	-	-	-	34.0	36.0	255.7	253.7
Z-V2-B3U-ST1	S1/2	77,430	66,953	278.0	10.0	12.0	268.0	266.0
Z-V2-B3U-ST3	S1/2	-	-	-	27.0	29.0	249.0	247.0

Table 6 Disturbed Soil Samples

Sample No.	Layer	Pro-posed Fill	SRS North (feet)	SRS East (feet)	Surface Elev. (ft, msl)	Depth from (feet)	Depth to (feet)	Elev. from (ft, msl)	Elev. to (ft, msl)
TP-1-Bag 4	S1/2	Yes	77,311	66,936	281.5	2.0	4.0	279.5	277.5
TP-1-Bag 6	S1/2	Yes	-	-	-	4.0	6.0	277.5	275.5
TP-1-Bag 8	S1/2	Yes	-	-	-	6.0	8.0	275.5	273.5
TP-1-Bag 10	S1/2	Yes	-	-	-	8.0	10.0	273.5	271.5
TP-1-Bag 12	S1/2	Yes	-	-	-	10.0	12.0	271.5	269.5
TP-1-Composite	S1/2	Yes	-	-	-	2.0	12.0	279.5	269.5
TP-2-Bag 4	S1/2	Yes	77,266	67,070	280.9	2.0	4.0	278.9	276.9
TP-2-Bag 6	S1/2	Yes	-	-	-	4.0	6.0	276.9	274.9
TP-2-Bag 8	S1/2	Yes	-	-	-	6.0	8.0	274.9	272.9
TP-2-Bag 10	S1/2	Yes	-	-	-	8.0	10.0	272.9	270.9
TP-2-Bag 12	S1/2	Yes	-	-	-	10.0	12.0	270.9	268.9
TP-2-Composite	S1/2	Yes	-	-	-	2.0	12.0	278.9	268.9
Z-V2-B2-SS2	S1/2	No	77,353	66,982	279.0	12.0	14.0	267.0	265.0
Z-V2-B2-SS6	S1/2	No	-	-	-	20.0	22.0	259.0	257.0
Z-V2-B2-SS8	S1/2	No	-	-	-	27.0	29.0	252.0	250.0
Z-V2-B2-SS10	S1/2	No	-	-	-	35.0	37.0	244.0	242.0
Z-V2-B2-SS12	S1/2	No	-	-	-	45.0	47.0	234.0	232.0
Z-V2-B2-SS14	C2	No	-	-	-	55.0	57.0	224.0	222.0
Z-V2-B2-SS15	C2	No	-	-	-	60.0	62.0	219.0	217.0
Z-V2-B2-SS16	C2	No	-	-	-	65.0	67.0	214.0	212.0
Z-V2-B2-SS17	S3	No	-	-	-	70.0	72.0	209.0	207.0
Z-V2-B2-SS18	S3	No	-	-	-	75.0	77.0	204.0	202.0
Z-V2-B2-SS19	S3	No	-	-	-	80.0	82.0	199.0	197.0
Z-V2-B2-SS20	S3	No	-	-	-	85.0	87.0	194.0	192.0
Z-V2-B2-SS21	S3	No	-	-	-	90.0	92.0	189.0	187.0
Z-V2-B2-SS22	S4	No	-	-	-	95.0	97.0	184.0	182.0
Z-V2-B2-SS23	S4	No	-	-	-	100.0	102.0	179.0	177.0
Z-V2-B2-SS24	S4	No	-	-	-	105.0	107.0	174.0	172.0
Z-V2-B2-SS25	S4	No	-	-	-	110.0	112.0	169.0	167.0

Table 7 CPTu and SCPTu Data

Layer	Description	Sleeve Friction (tsf)	Tip Resistance (tsf)	Friction Ratio (%)	Pore Pressure (tsf)	P Wave (fps)	S Wave (fps)	Resistivity (Ω-m)
S1	Average	4.0	108	4.1	1.3	2,519		2,066
	Minimum	0.8	40	0.7	-0.5	2,116		1,019
	Maximum	8.0	401	9.4	26.4	3,431		20,280
	Standard Deviation	1.5	47	1.8	3.3			2,025
	No. of Data Points	1,340	1,340	1,340	1,340	81		315
S2	Average	1.4	156	1.0	0.0	2,082	1,156	2,445
	Minimum	0.2	24	0.2	-0.4	1,501	931	-387,405
	Maximum	6.3	490	5.4	1.1	4,454	1,432	278,888
	Standard Deviation	0.7	70	0.5	0.1			28,743
	No. of Data Points	5,157	5,174	5,157	5,174	3,504	3,355	519
C2	Average	0.7	75	1.8	2.8	2,388	976	
	Minimum	0.1	9	0.2	-0.6	1,839	714	
	Maximum	2.7	362	9.1	16.1	3,663	1,307	
	Standard Deviation	0.4	82	1.2	4.1			
	No. of Data Points	1,521	1,521	1,521	1,521	304	1,159	
S3	Average	1.1	136	1.2	1.9		1,063	
	Minimum	0.1	3	0.1	-0.6		671	
	Maximum	11.9	418	12.5	33.8		1,892	
	Standard Deviation	0.9	99	1.0	3.4			
	No. of Data Points	2,926	2,926	2,926	2,926		2,478	
S4	Average	1.6	105	1.5	9.9		1,033	
	Minimum	0.1	8	0.2	-0.6		556	
	Maximum	17.3	528	8.7	288.4		1,954	
	Standard Deviation	2.5	107	1.3	17.4			
	No. of Data Points	1,417	1,455	1,417	1,455		1,144	
All	Average	1.6	130	1.6	2.1	2,111	1,081	2,302
	No. of Data Points	12,378	12,416	12,378	12,416	3,889	8,136	834

Table 8 SPT Data

Description	S1	S2	C2	S3	S4
Average	24.50 say 25	24.57 say 25	6.45 say 6	23.33 say 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 9 Soft Zone Locations

ID No.	Top Elevation (feet MSL)	Bottom Elevation (feet MSL)	Average q_c or Blow Counts	Total Interval (feet)	Soft Interval Thickness (feet)
Z-V2-CP5	194.8	192.2	6.7 tsf	2.6	2.6
Z-V2-CP6	224.6	222.8	13.6 tsf	1.7	0.9
Z-V2-CP7	182.8	181.2	13.8 tsf	1.6	1.6
Z-V2-CP8	219.5	218.0	11.9 tsf	1.5	1.5
Z-V2-CP8	216.7	213.9	13.1 tsf	2.8	2.6
Z-V2-CP9	200.5	200.1	12.3 tsf	0.4	0.4
Z-V2-CP9	198.7	197.9	12.6 tsf	0.8	0.8
Z-V2-CP15	221.3	220.8	13.9 tsf	0.4	0.4
Z-V2-CP15	202.7	202.2	12.6 tsf	0.5	0.5
Z-V2-CP15	177.6	163.2	11.1 tsf	14.4	14.0
Z-V2-B1	179.7	176.7	N/A	3.0	12.5 to 19.0
Z-V2-B1	176.7	174.7	0-0-1-2	2.0	
Z-V2-B1	174.7	174.2	N/A	0.5	
Z-V2-B1	174.2	167.7	Weight of rod	7.5	
Z-V2-B1	167.7	166.7	N/A	1.0	
Z-V2-B1	166.7	164.7	0-0-0-0	2.0	
Z-V2-B1	164.7	161.7	N/A	3.0	
Z-V2-B1	161.7	159.7	1-3-5-8	2.0	
Z-V2-B2	177.0	174.0	N/A	3.0	2.5 to 8.5
Z-V2-B2	174.0	172.0	2-0-3-2	2.0	
Z-V2-B2	172.0	169.0	N/A	3.0	
Z-V2-B2	169.0	167.0	0-8-28-50R	2.0	
Z-V2-B3	211.1	218.1	N/A	3.0	3.0 to 9.0
Z-V2-B3	218.1	216.1	2-2-2-2	2.0	
Z-V2-B3	216.1	213.1	N/A	3.0	
Z-V2-B3	213.1	211.1	2-3-4-6	2.0	

Table 10 Distribution of Particle Sizes

Sample No.	Layer	% passing U.S. standard sieve sizes / (opening in mm)								
		3/8 (9.50)	4 (4.75)	10 (2.00)	20 (0.85)	40 (0.425)	60 (0.250)	100 (0.150)	140 (0.106)	200 (0.075)
TP-1-Bag 4	S1/2	-	100.0	98.9	91.9	69.0	46.8	-	22.8	17.2
TP-1-Bag 6	S1/2	-	100.0	99.1	91.1	75.3	66.1	-	54.2	49.4
TP-1-Bag 8	S1/2	-	100.0	99.0	87.8	66.7	57.7	-	36.8	30.0
TP-1-Bag 10	S1/2	-	100.0	99.3	91.4	76.0	69.8	-	40.0	29.0
TP-1-Bag 12	S1/2	-	100.0	99.4	93.7	80.3	73.8	-	46.2	32.3
TP-2-Bag 4	S1/2	-	100.0	99.8	97.4	78.8	56.3	-	34.5	30.8
TP-2-Bag 6	S1/2	-	100.0	99.6	97.1	80.1	59.7	-	44.4	41.4
TP-2-Bag 8	S1/2	-	100.0	98.3	85.8	56.1	44.7	-	33.8	31.2
TP-2-Bag 10	S1/2	-	100.0	98.8	85.0	63.0	53.6	-	30.9	25.6
TP-2-Bag 12	S1/2	-	100.0	98.6	83.9	59.6	51.8	-	28.1	20.8
Z-V2-B2-SS2	S1/2	-	100.0	99.6	91.3	78.2	71.5	64.5	-	31.9
Z-V2-B2-SS6	S1/2	-	100.0	100.0	99.0	98.4	97.5	85.5	-	30.6
Z-V2-B2-SS8	S1/2	-	100.0	99.9	97.8	87.6	64.4	20.1	-	13.7
Z-V2-B2-SS10	S1/2	-	100.0	100.0	97.0	87.5	52.0	19.4	-	15.0
Z-V2-B2-SS12	S1/2	-	100.0	99.4	94.9	57.6	34.7	20.3	-	12.0
Z-V2-B2-SS14	C2	-	100.0	93.5	69.0	60.7	33.1	22.4	-	19.3
Z-V2-B2-SS15	C2	-	100.0	99.5	90.0	70.4	49.0	30.8	-	19.4
Z-V2-B2-SS16	C2	-	100.0	99.8	98.9	80.7	33.4	19.7	-	17.0
Z-V2-B2-SS17	S3	-	-	100.0	97.5	87.8	38.2	19.9	-	15.4
Z-V2-B2-SS18	S3	-	-	100.0	96.7	70.4	23.1	9.1	-	7.0
Z-V2-B2-SS19	S3	-	100.0	98.7	91.6	67.3	41.1	13.3	-	8.2
Z-V2-B2-SS20	S3	100.0	99.9	97.0	76.3	37.2	25.1	17.6	-	9.7
Z-V2-B2-SS21	S3	100.0	99.8	98.9	95.4	81.8	52.1	34.7	-	32.2
Z-V2-B2-SS22	S4	-	100.0	99.2	91.9	74.6	56.3	34.1	-	12.6
Z-V2-B2-SS23	S4	-	-	100.0	99.0	98.4	97.5	85.5	-	30.6
Z-V2-B2-SS24	S4	-	100.0	99.3	99.0	98.2	97.2	91.6	-	43.6
Z-V2-B2-SS25	S4	100.0	98.9	93.1	80.9	74.9	69.5	60.3	-	44.3
Z-V2-B1U-ST1	S1/2	-	100.0	99.6	89.4	71.0	62.5	-	37.8	25.1
Z-V2-B1U-ST2	S1/2	-	100.0	99.8	95.7	92.4	61.7	19.1	-	13.9
Z-V2-B1U-ST3	S1/2	-	100.0	99.9	97.4	76.5	51.5	-	18.3	17.6
Z-V2-B1U-ST4	S1/2	-	100.0	99.9	93.6	81.7	73.1	30.6	-	18.3
Z-V2-B1U-ST5	C2	-	-	100.0	99.5	96.7	92.1	62.5	-	40.6
Z-V2-B1U-ST7	S3	-	100.0	98.2	75.3	40.5	28.1	7.7	-	5.2
Z-V2-B1U-PS1T	S4	-	100.0	97.4	74.1	57.3	53.2	-	22.4	13.0
Z-V2-B1U-PS1B	S4	-	100.0	99.1	85.9	77.5	70.2	-	27.5	15.4
Z-V2-B1U-PS3	S4	100.0	99.6	96.4	89.7	78.5	64.0	43.9	-	27.0

Table 10 Distribution of Particle Sizes (continue)

Sample No.	Layer	% passing U.S. standard sieve sizes / (opening in mm)								
		3/8 (9.50)	4 (4.75)	10 (2.00)	20 (0.85)	40 (0.425)	60 (0.250)	100 (0.150)	140 (0.106)	200 (0.075)
Z-V2-B2U-ST1	S1/2	-	100.0	99.9	98.5	81.7	51.2	43.2	-	40.4
Z-V2-B2U-ST2	S1/2	-	100.0	99.4	89.2	75.6	68.7	59.0	-	39.2
Z-V2-B2U-ST3	S1/2	-	100.0	98.9	81.5	52.7	38.8	27.5	-	16.2
Z-V2-B2U-ST4	S1/2	-	100.0	99.9	97.3	87.0	65.1	24.0	-	16.1
Z-V2-B3U-ST1	S1/2	-	100.0	99.3	92.5	80.7	63.5	-	43.8	29.8
Z-V2-B3U-ST3	S1/2	-	100.0	99.8	95.6	75.7	38.8	-	7.8	7.2

Table 11 Atterberg Limits and USCS

Sample No.	Layer	LL (%)	PL (%)	PI (%)	USCS
TP-1-Composite	S1/2	38	17	21	SC
TP-2-Composite	S1/2	39	20	19	SC
Z-V2-B2-SS2	S1/2	44	18	26	SC
Z-V2-B2-SS15	C2	33	14	19	SC
Z-V2-B2-SS16	C2	NP	NP	NP	SM
Z-V2-B2-SS21	S3	51	12	39	SC
Z-V2-B2-SS25	S4	NP	NP	NP	SM
Z-V2-B1U-ST1	S1/2	46	23	23	SC
Z-V2-B1U-ST2	S1/2	NP	NP	NP	SP-SM
Z-V2-B1U-ST3	S1/2	52	26	26	SC
Z-V2-B1U-ST4	S1/2	NP	NP	NP	SM
Z-V2-B1U-ST5	C2	53	17	36	SC
Z-V2-B1U-ST7	S3	NP	NP	NP	SP
Z-V2-B1U-PS1	S4	NP	NP	NP	SM
Z-V2-B1U-PS3	S4	NP	NP	NP	SM
Z-V2-B2U-ST1	S1/2	43	19	24	SC
Z-V2-B2U-ST2	S1/2	42	19	23	SC
Z-V2-B2U-ST3	S1/2	38	18	20	SC
Z-V2-B2U-ST4	S1/2	NP	NP	NP	SM
Z-V2-B3U-ST1	S1/2	44	19	25	SC
Z-V2-B3U-ST3	S1/2	NP	NP	NP	SP-SM

Table 12 Unit Weights and Water Contents

Sample No.	Layer	Type of Test	Dry Unit Weight (pcf)	Moisture contents (%)	Wet Unit Weight (pcf)
Z-V2-B2-SS2	S1/2	Moisture	-	19.8	-
Z-V2-B2-SS15	C2	Moisture	-	30.2	-
Z-V2-B2-SS16	C2	Moisture	-	26.3	-
Z-V2-B2-SS21	S3	Moisture	-	33.6	-
Z-V2-B2-SS25	S4	Moisture	-	61.9	-
Z-V2-B1U-ST1	S1/2	Unit weight	105.9	13.6	120.3
Z-V2-B1U-ST2	S1/2	Consolidation	100.1	17.2	117.2
Z-V2-B1U-ST2	S1/2	Consolidation	101.9	16.7	118.
Z-V2-B1U-ST3	S1/2	Unit weight	105.6	17.0	123.6
Z-V2-B1U-ST4	S1/2	Consolidation	102.8	15.4	118.6
Z-V2-B1U-ST4	S1/2	Consolidation	101.6	17.7	119.5
Z-V2-B1U-ST5	C2	Consolidation	69.5	45.3	101.0
Z-V2-B1U-ST5	C2	Consolidation	69.2	48.7	102.8
Z-V2-B1U-ST7	S3	Consolidation	105.3	17.7	123.9
Z-V2-B1U-PS1	S4	Triaxial 1 of 2	92.6	32.4	122.5
Z-V2-B1U-PS1	S4	Triaxial 2 of 2	87.6	34.9	118.2
Z-V2-B1U-PS1	S4	Consolidation	86.7	35.5	117.4
Z-V2-B1U-PS1	S4	Consolidation	97.1	25.7	122.1
Z-V2-B1U-PS3	S4	Consolidation	86.1	32.6	114.2
Z-V2-B2U-ST1	S1/2	Triaxial 1 of 3	104.4	20.2	125.5
Z-V2-B2U-ST1	S1/2	Triaxial 2 of 3	105.8	19.8	126.7
Z-V2-B2U-ST1	S1/2	Triaxial 3 of 3	111.6	16.1	129.6
Z-V2-B2U-ST2	S1/2	Triaxial 1 of 2	111.3	15.7	128.8
Z-V2-B2U-ST2	S1/2	Triaxial 2 of 2	116.1	14.1	132.5
Z-V2-B2U-ST3	S1/2	Triaxial 1 of 3	107.8	16.4	125.5
Z-V2-B2U-ST3	S1/2	Triaxial 2 of 3	108.3	14.6	124.1
Z-V2-B2U-ST3	S1/2	Triaxial 3 of 3	109.7	14.6	125.7
Z-V2-B2U-ST4	S1/2	Triaxial 1 of 3	99.0	17.8	116.6
Z-V2-B2U-ST4	S1/2	Triaxial 2 of 3	101.6	19.3	121.2
Z-V2-B2U-ST4	S1/2	Triaxial 3 of 3	99.8	19.8	119.6
Z-V2-B3U-ST1	S1/2	Unit weight	110.9	14.9	127.4
Z-V2-B3U-ST3	S1/2	Unit weight	104.1	11.6	116.2

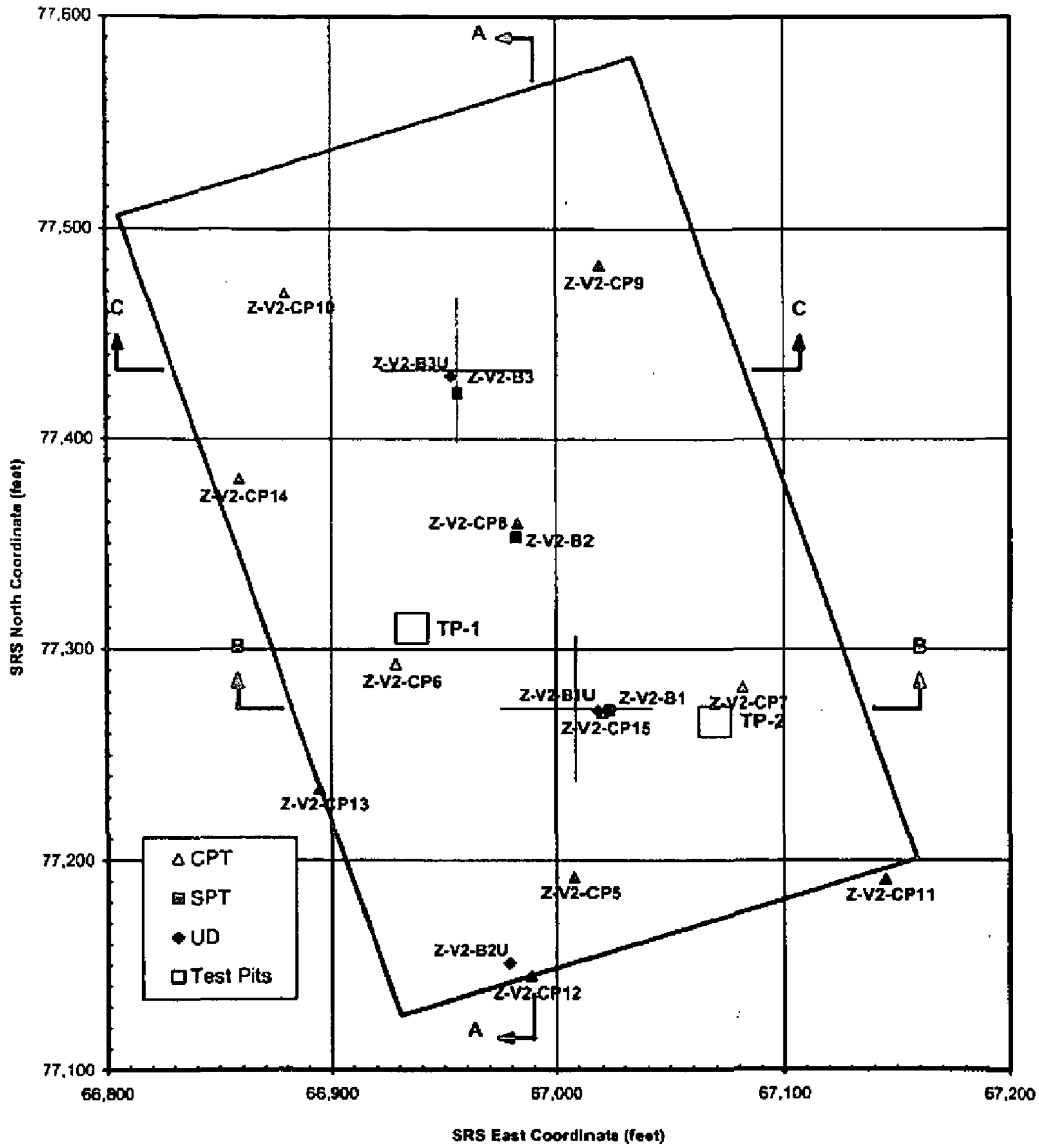
Average	S1/2	20 tests	105.7	16.6	123.3
Average	C2	4 tests	69.4	37.6	101.9
Average	S3	2 tests	105.3	25.7	123.9
Average	S4	6 tests	90.0	37.2	118.9
Average	All	32 tests	100.1	23.7	120.8

Table 13 CU/pp Tests

Sample No.	Layer	Total Friction Angle ϕ (degrees)	Total Cohesion C (psf)	Effective Friction Angle ϕ' (degrees)	Effective Cohesion c' (psf)
Z-V2-B1U-PS1	S4	11.0	1,526	26.5	566
Z-V2-B2U-ST1	S1/2	deleted	300	33.4	250
Z-V2-B2U-ST2	S1/2	35.0	1,700	30.0	380
Z-V2-B2U-ST3	S1/2	36.8	250	33.3	50
Z-V2-B2U-ST4	S1/2	26.6	250	32.0	260

Table 14 Consolidation Tests

Sample No.	Layer	P_O (ksf)	P_C (ksf)	OCR	C_c	C_r	C_c/C_r
Z-V2-B1U-ST2	S1/2	3.6	7.3	2.0	0.071	0.007	9.7
Z-V2-B1U-ST2a	S1/2	3.6	3.0	0.8	0.056	0.009	6.6
Z-V2-B1U-ST4	S1/2	5.5	7.2	1.3	0.101	0.008	12.7
Z-V2-B1U-ST4a	S1/2	5.5	4.5	0.8	0.069	0.008	8.5
Z-V2-B1U-ST5	C2	7.6	8.4	1.1	0.781	0.102	7.6
Z-V2-B1U-ST5a	C2	7.6	7.8	1.0	0.725	0.083	8.7
Z-V2-B1U-ST7	S3	10.6	7.2	0.7	0.071	0.012	5.8
Z-V2-B1U-PS1	S4	12.6	11.6	0.9	0.196	0.023	8.4
Z-V2-B1U-PS3	S4	14.8	15.9	1.1	0.361	0.016	23.3



Notes for cross sections:

1. Cone penetrometer tip stress abscissa shows the scale of 0 to 200 tsf. Cone penetrometer friction ratio abscissa shows the scale of 0 to 4 percent. For detailed test result, see Appendix A, Cone Penetrometer Test Logs.
2. Standard Penetration Test N-values abscissa shows the scale of 0 to 50 blow count. for detailed test results, see Appendix B, Geotechnical Borehole Logs

Figure 1 Geotechnical exploratory map and notes for cross sections

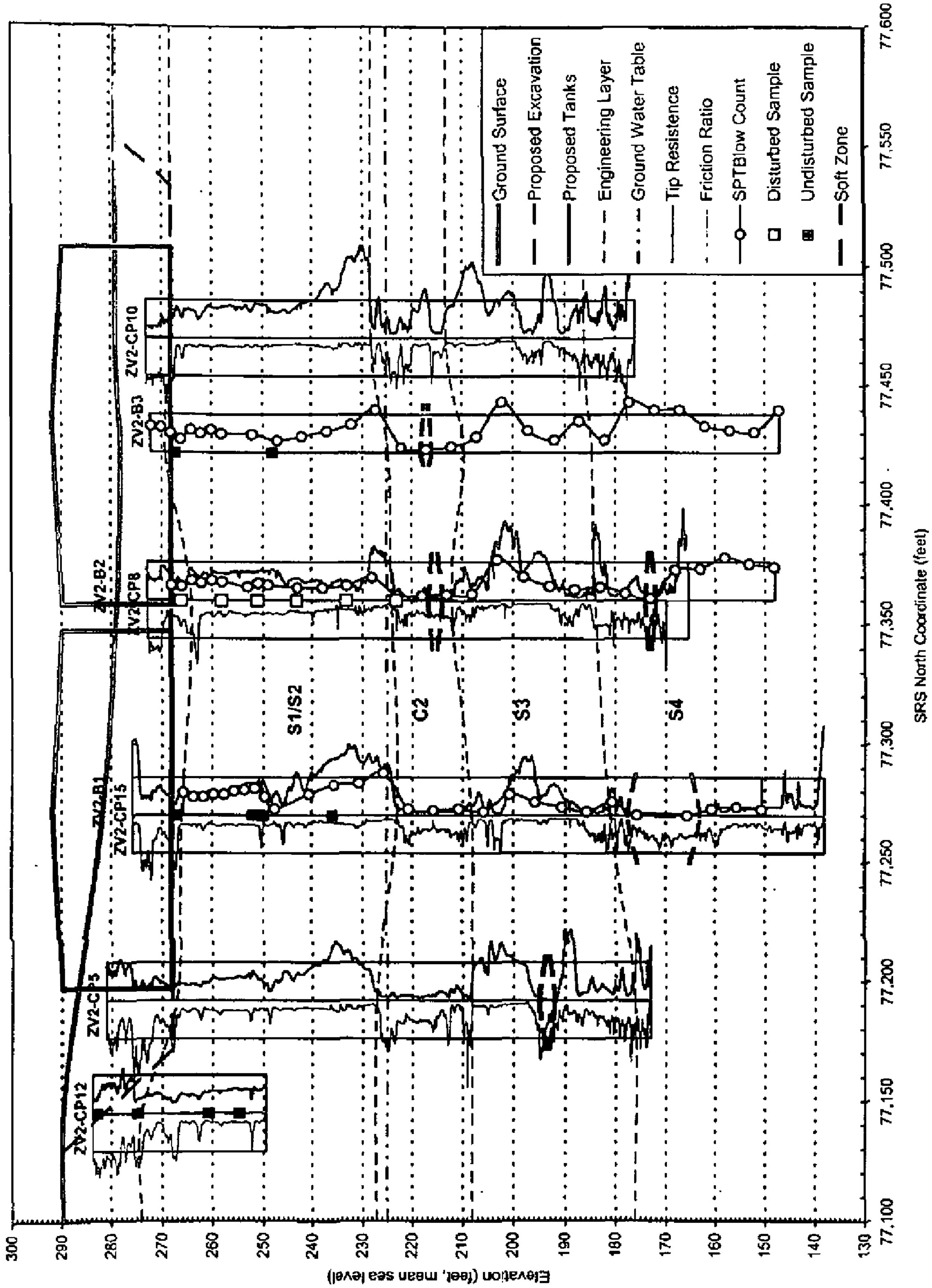


Figure 2 Geotechnical cross section A-A
(see notes in Figure 1)

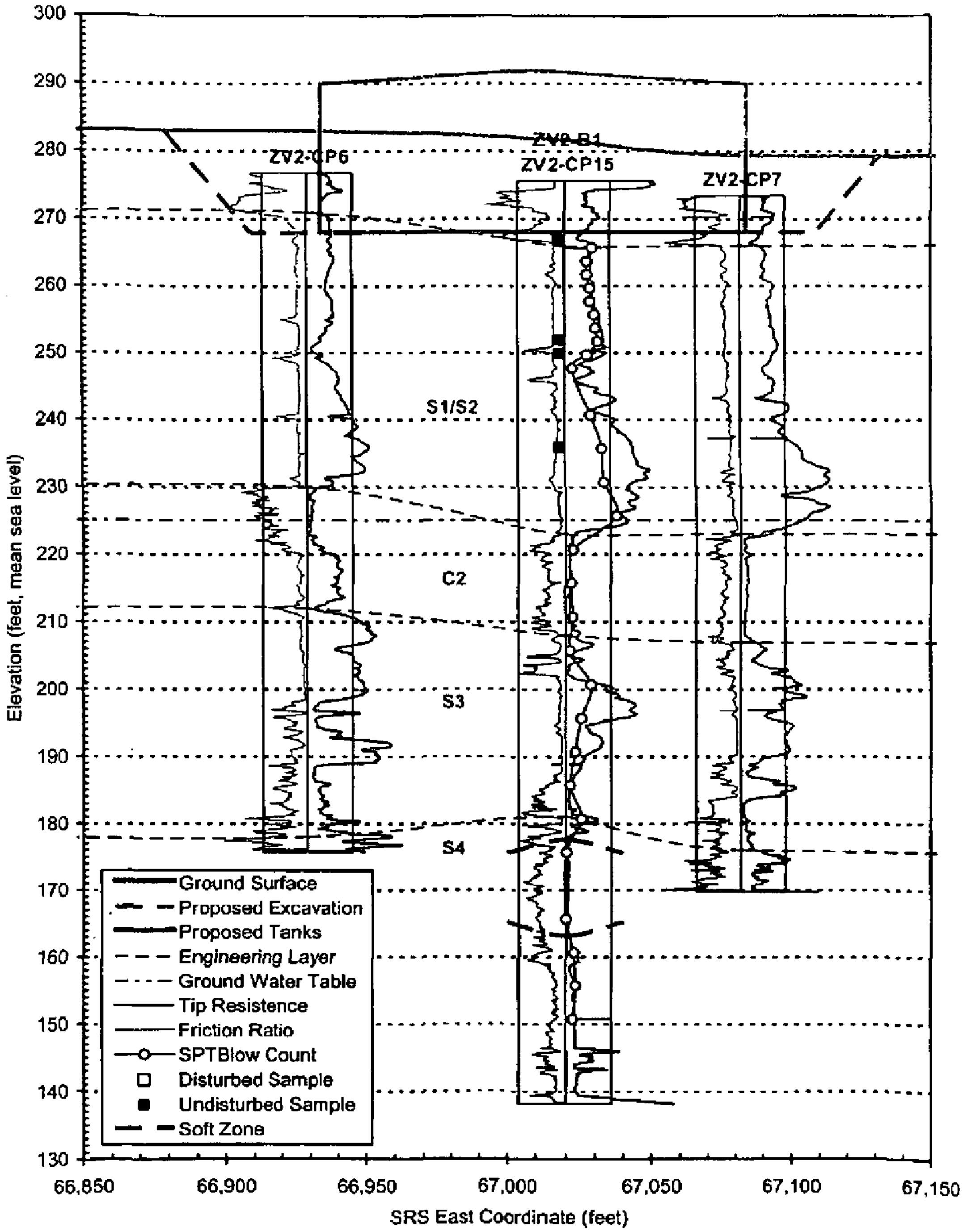


Figure 3 Geotechnical cross section B-B

(see notes in Figure 1)

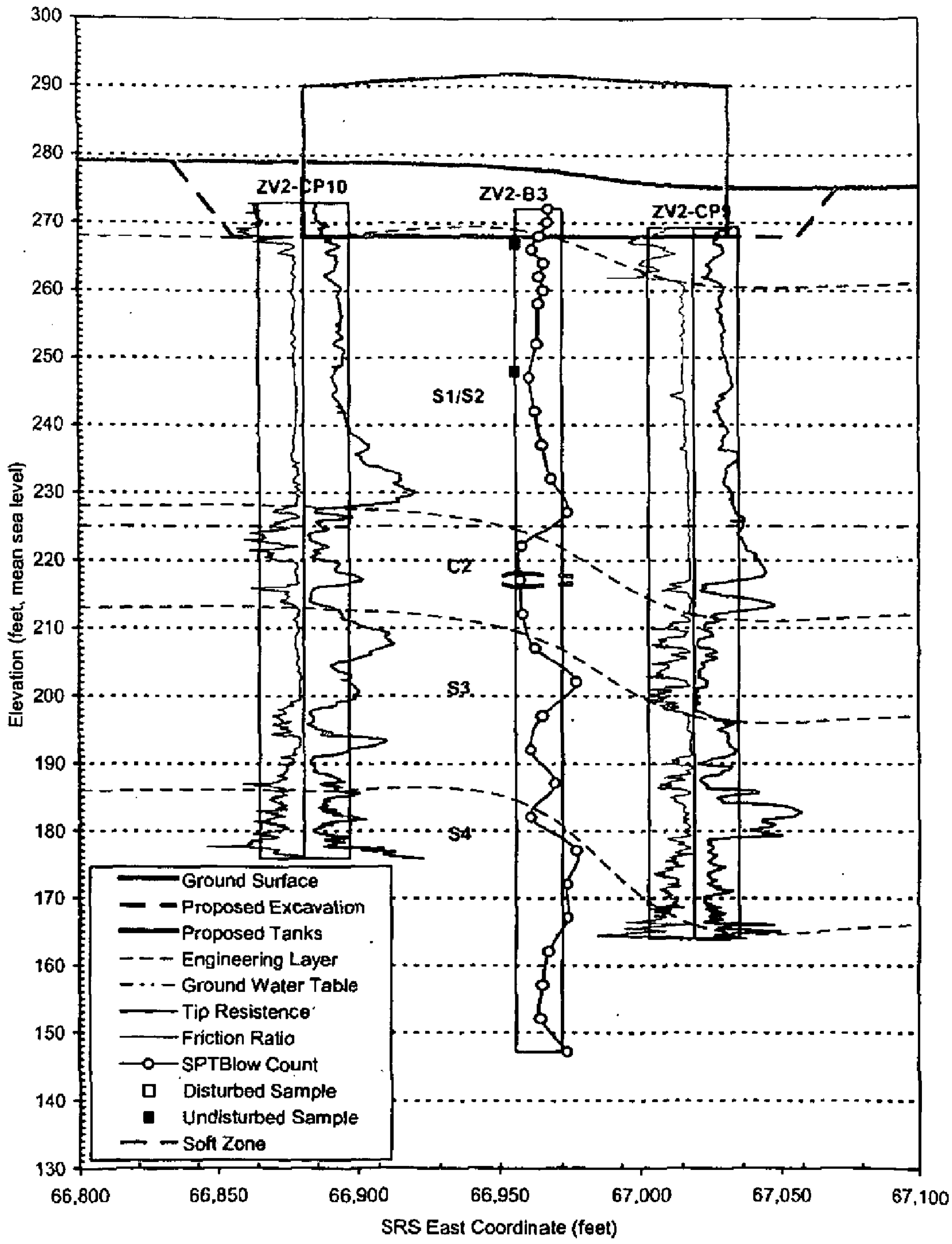


Figure 4 Geotechnical cross section C-C

(see notes in Figure 1)

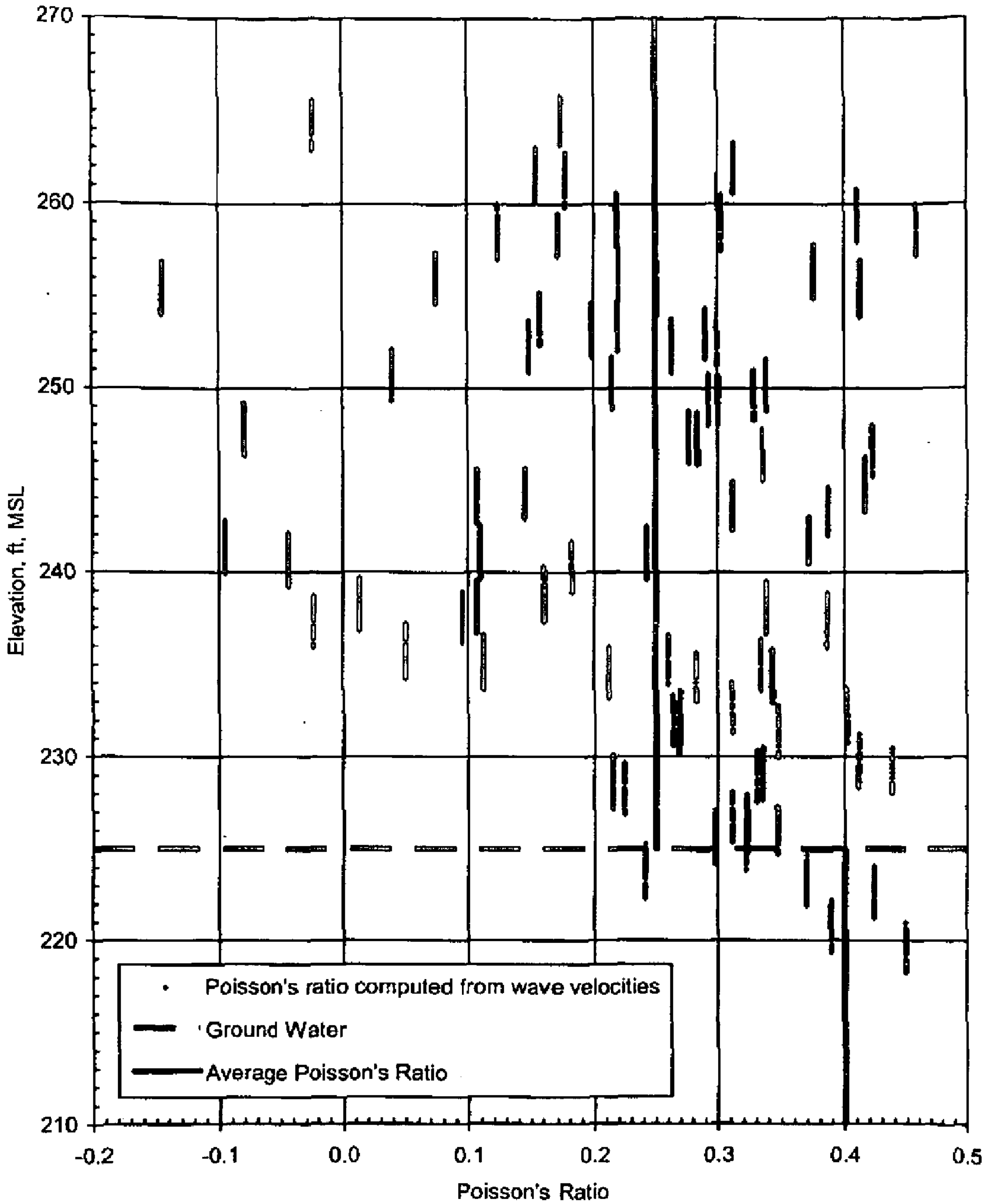
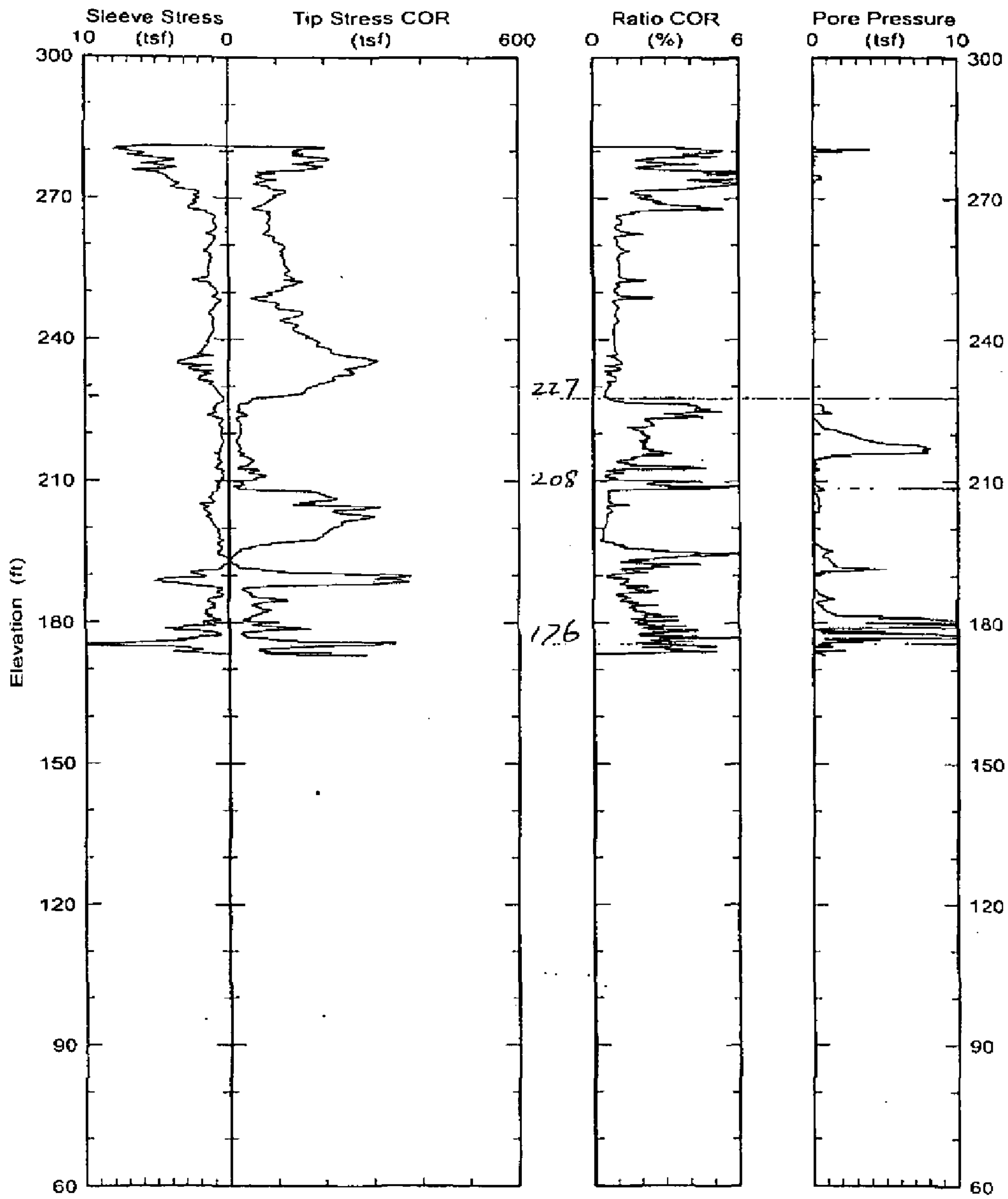


Figure 5 Poisson's ratio

Appendix A Cone Penetrometer Test Logs

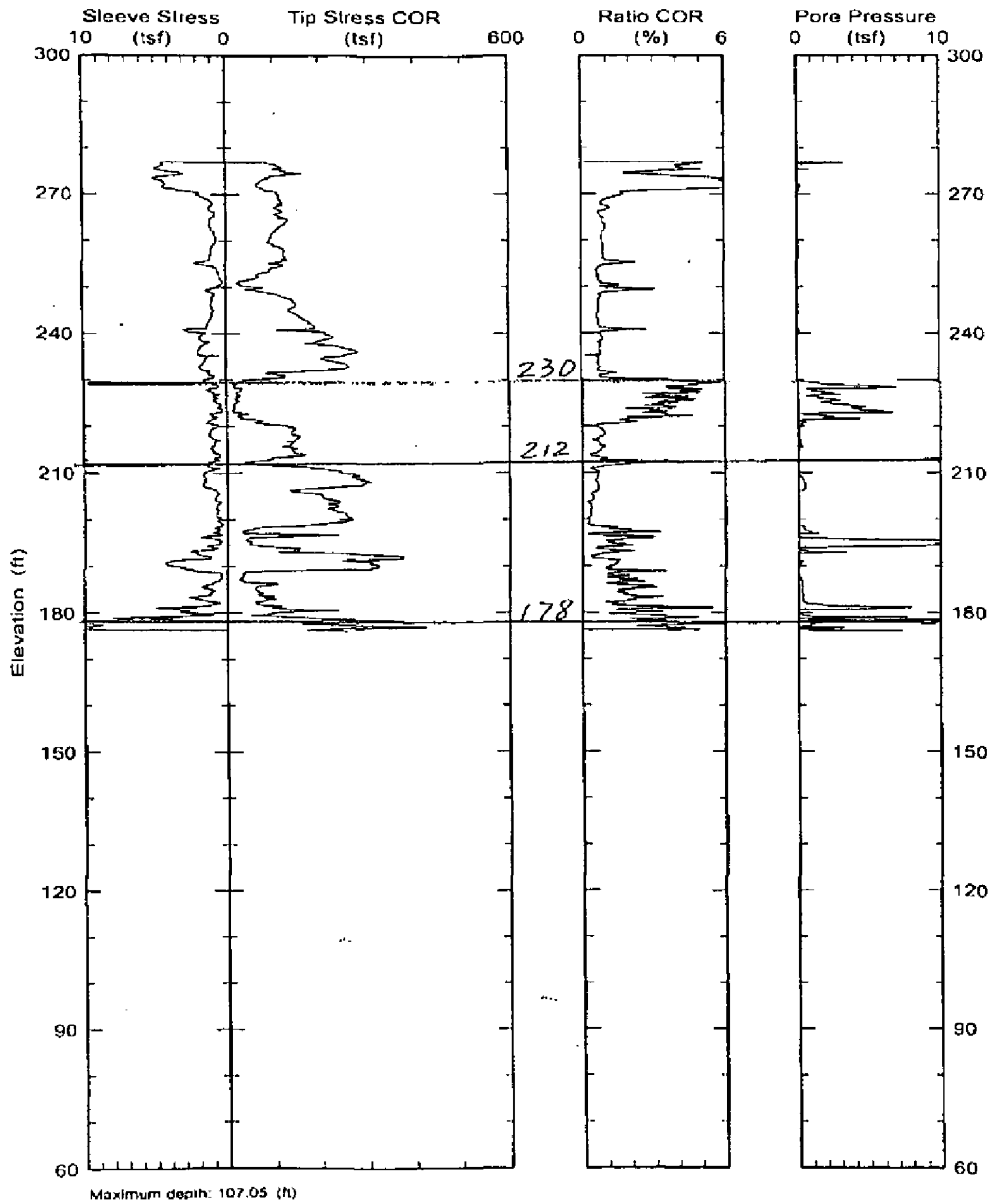
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 12/Apr/2005 Test ID: ZV2-CP5 Northing: 77192.0 Easting: 67008.0 Elevation: 287.0



Maximum depth: 114.03 (ft)

Appendix A Cone Penetrometer Test Logs

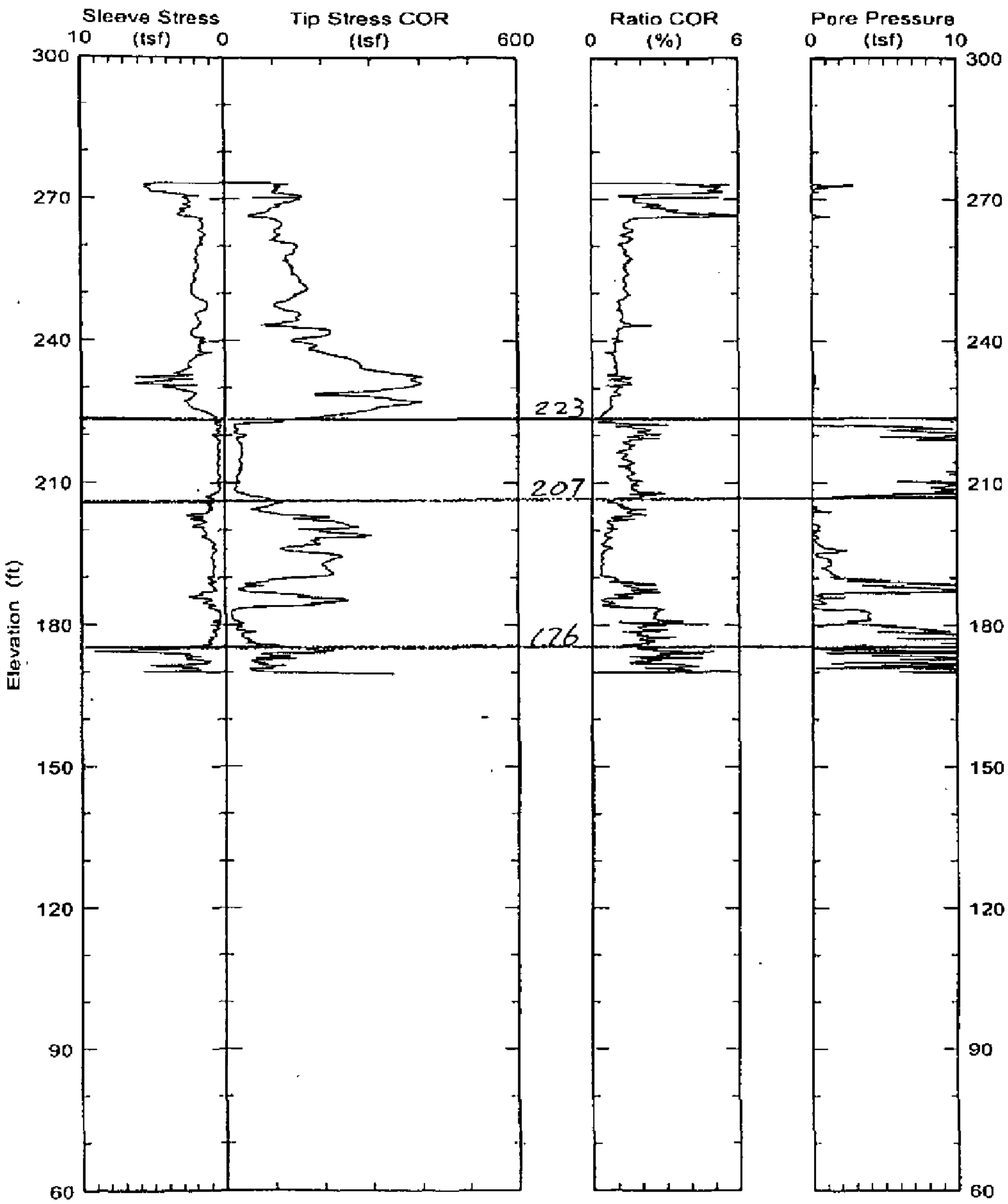
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 17/May/2005 Test ID: ZV2-CP6 Northing: 77293.0 Easting: 66929.0 Elevation: 283.0



Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc.
South Royalton, VT 05068
802-763-8348
cpt@ned.ara.com
www.ara.com

Date: 16/May/2005
Test ID: ZV2-CP7
Northing: 77282.0
Easting: 67082.0
Elevation: 279.4

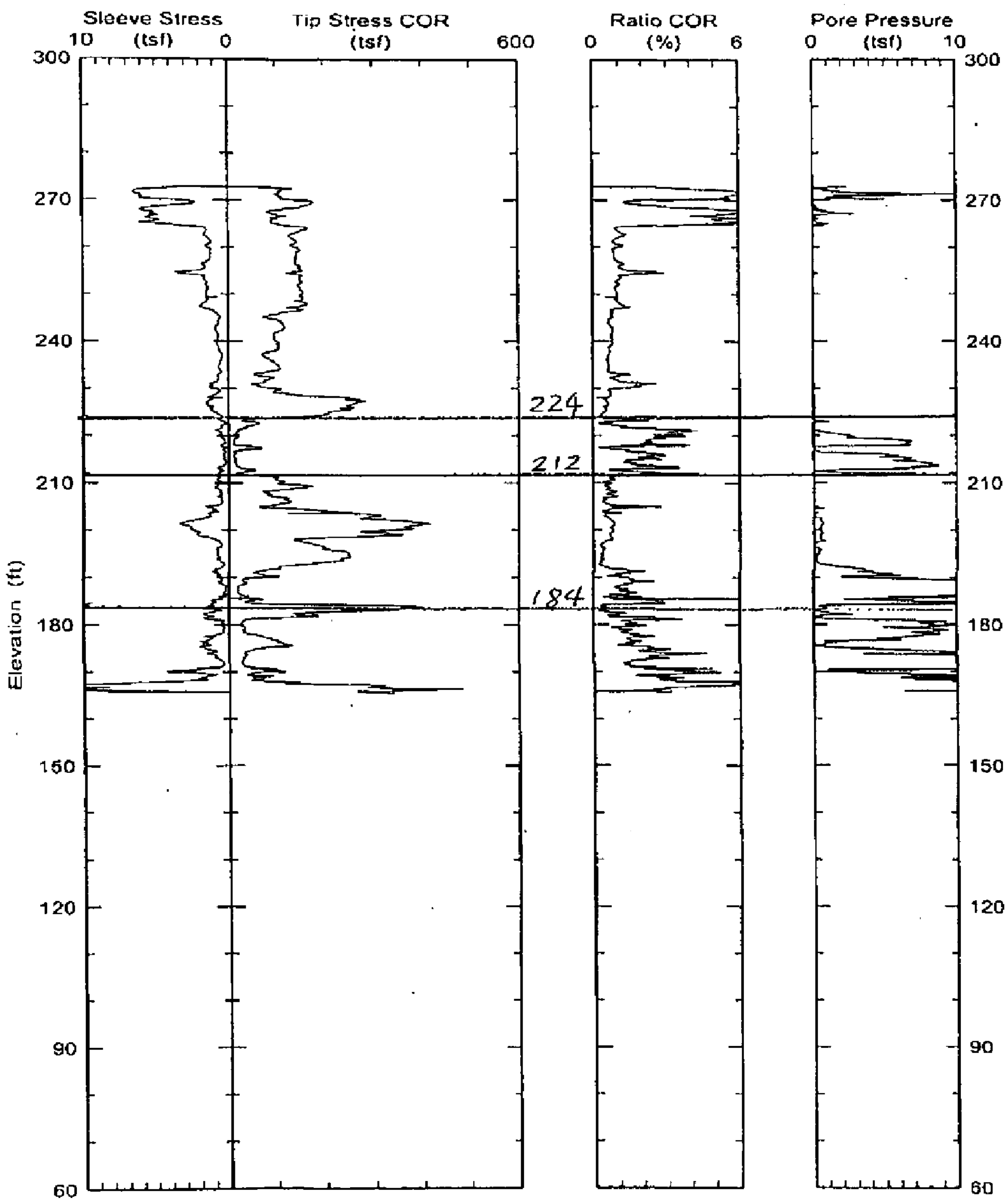


Maximum depth: 109.63 (ft)

Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc.
South Royalton, VT 05068
802-763-8348
cpt@ned.ara.com
www.ara.com

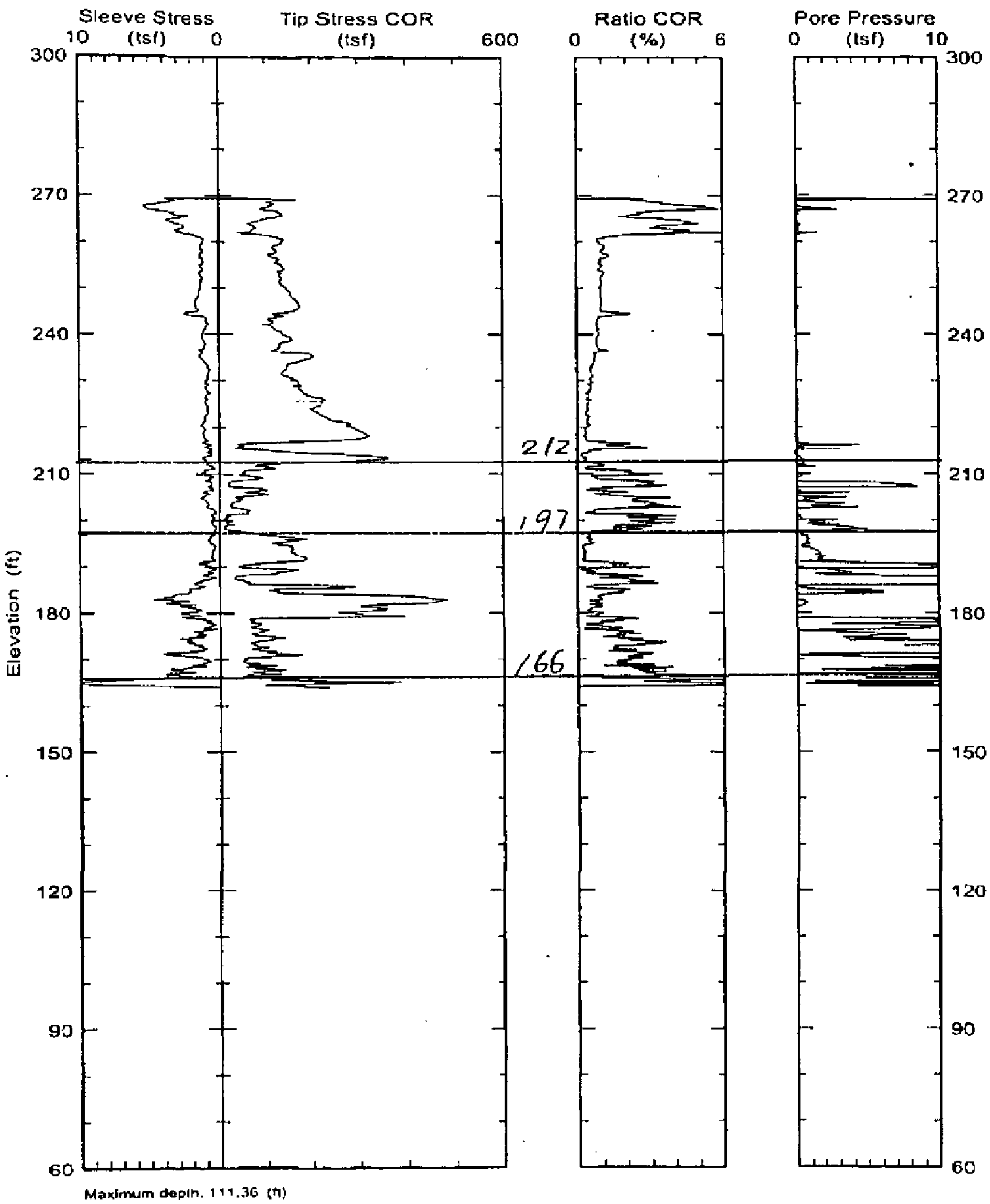
Date: 16/May/2005
Test ID: ZV2-CP8
Northing: 77359.9
Easting: 66983.0
Elevation: 278.8



Maximum depth: 113.33 (ft)

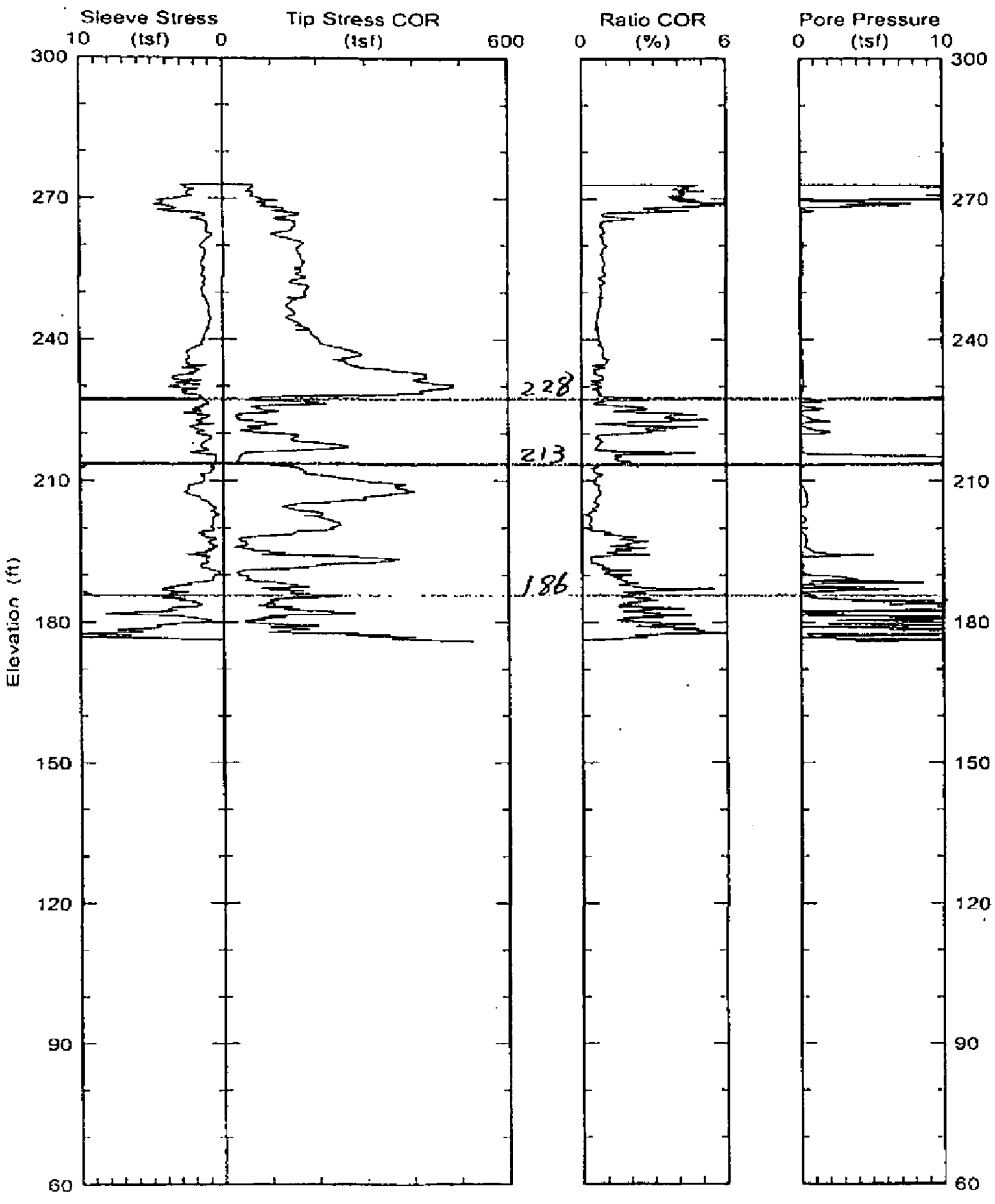
Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 11/May/2005 Test ID: ZV2-CP9 Northing: 77483.0 Easting: 67019.0 Elevation: 275.3



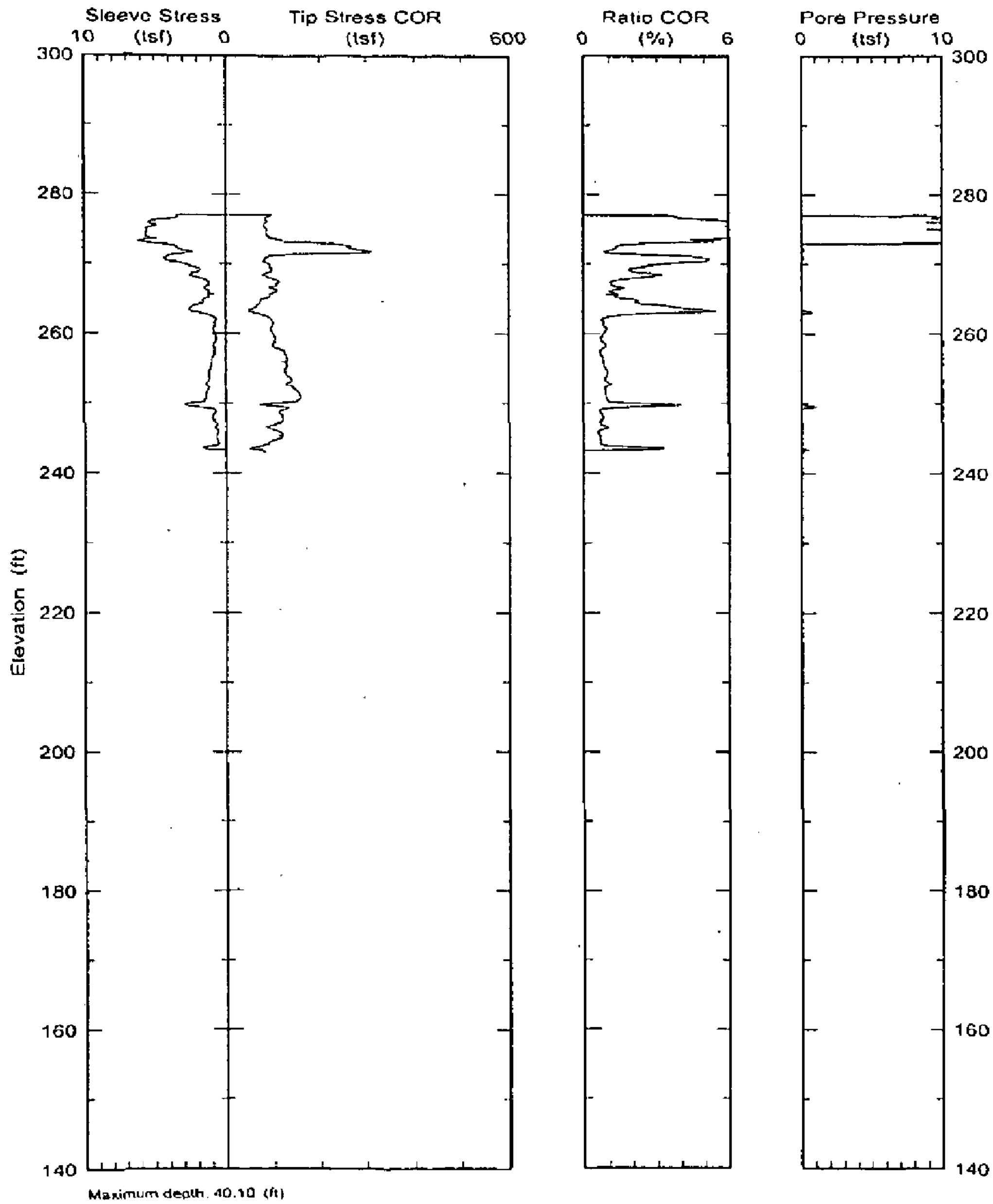
Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 13/May/2005 Test ID: ZV2-CP10 Northing: 77470.0 Easting: 66880.0 Elevation: 279.0



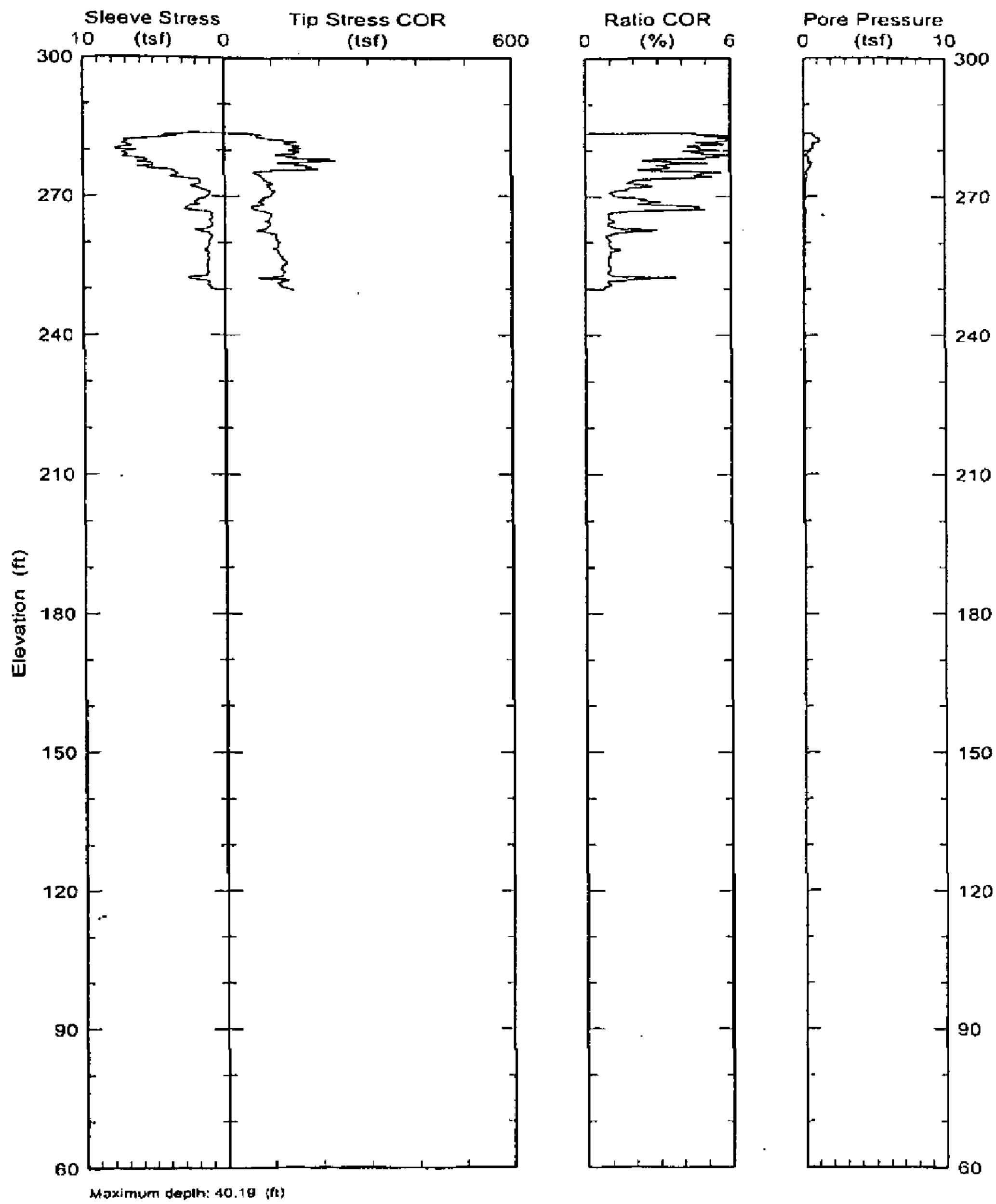
Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 18/May/2005 Test ID: ZV2-CP11 Northing: 77191.0 Easting: 67145.0 Elevation: 283.0



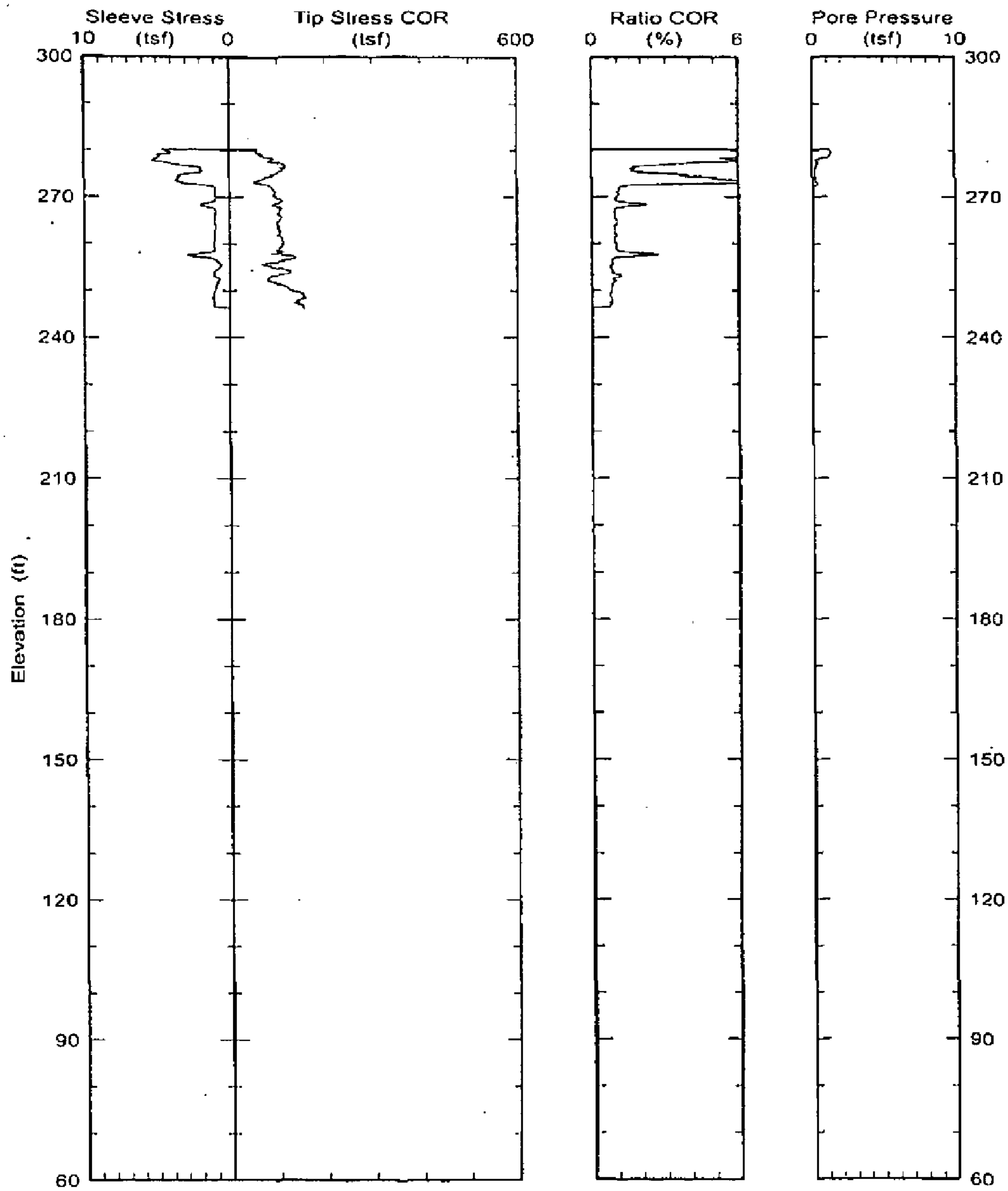
Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 12/Apr/2005 Test ID: ZV2-CP12 Northing: 77145.0 Easting: 66989.0 Elevation: 289.7



Appendix A Cone Penetrometer Test Logs

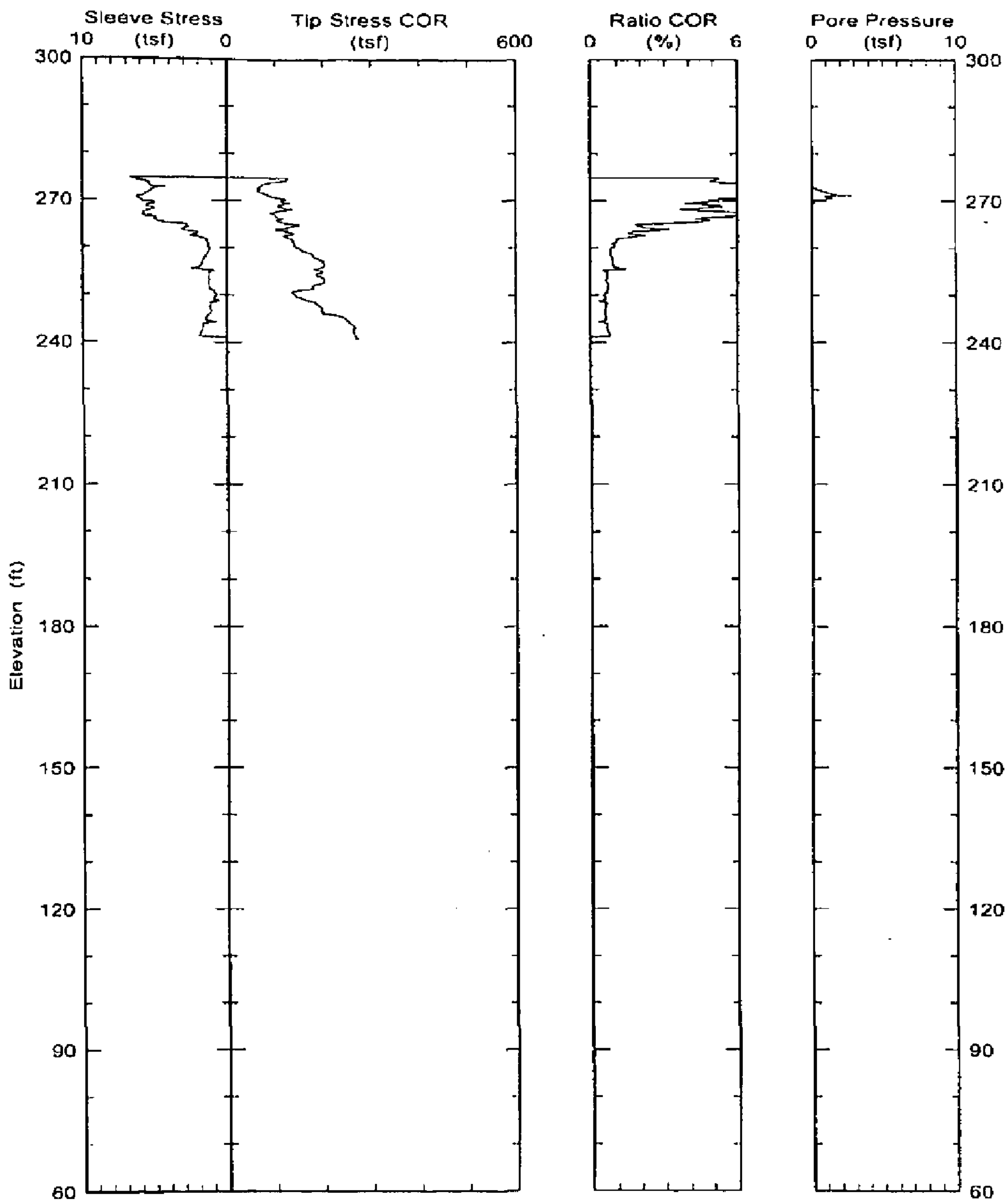
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 12/Apr/2005 Test ID: ZV2-CP13 Northing: 77324.0 Easting: 66895.0 Elevation: 286.2



Maximum depth: 40.07 (ft)

Appendix A Cone Penetrometer Test Logs

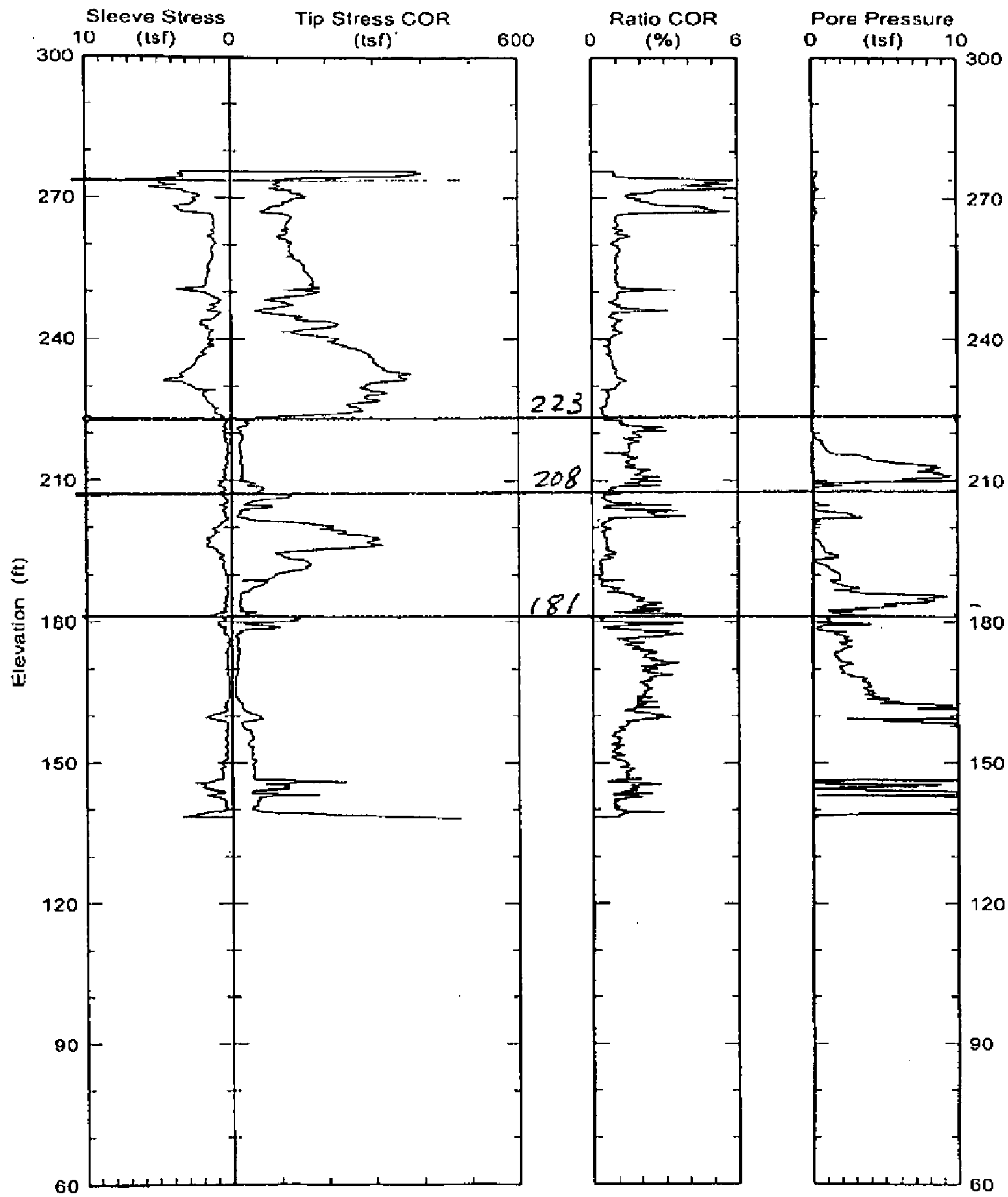
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 18/May/2005 Test ID: ZV2-CP14 Northing: 77381.0 Easting: 66859.0 Elevation: 280.8



Maximum depth: 40.02 (ft)

Appendix A Cone Penetrometer Test Logs

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 16/May/2005 Test ID: ZV2-CP15 Northing: 77270.0 Easting: 67020.0 Elevation: 281.7



Maximum depth: 143.58 (ft)

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		1 of 7	ZV2B1

SITE	COORDINATES	WEATHER CONDITIONS
Z AREA VAULT 2	N 77271.5 E 57[illegible].98	CLOUDY 70°F

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH
5-5-05		GREGG DRILLING	CME 75	4"	CME AUTOMATIC HAMMER	132

GROUND EL.	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:
281.70		F H SYMS	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						5-5-05 THUR
						CLOUDY 70°F
						0-15' DRILL
						OUT
	5					
	10					
	15					
SS-1		6	0"/24"		RD BRN CLAYEY SAND (SC) POORLY SORTED F-C	
		12			MED DENSE, FINES ELASTIC, MOIST WITH WHT	
		15			KAOLIN LENSES	
		17				
		7				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		2 OF 7	ZV2B1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-2		10	18"/24"		SAA	
		12				
		13				
SS-3		7	24"/24"		SAA BECOMING SLIGHTLY MRG BRN	
	20	10				
		12				
		13				
SS-4		9	24/24"		SAA CLAY CONTENT INCREASING + 40%	
		11				
		14				
		15				
SS-5		7	18"/24"		LT BRN SILTY SAND (SM) WELL SORTED F.S. IN	10:30 AM
		12			SILTY MATRIX ~35% FINES 10% CLAY, MOIST	
		13			LOW PLASTICITY, MED DENSE	
	25	15				
SS-6		16	20"/24"		RD BRN SILTY SAND w/ CLAY (SM-SC) WELL SORTED	
		14			M.S.	
		15				
		19				
SS-7		11	18"/24"		SAA	
		14				
		16				
		16				
SS-8		10	16"/24"		BECOMING POORLY SORTED	11:00 AM
	30	16				
		16				
		14				
SS-9		12	15"/24"		RD BRN CLAYEY SAND (SC) WELL SORTED M.S.	
		14			WET ~20% FINES	
		8				
		6				
SS-10		3	18"/24"		BRN CLAYEY SAND (SC, POORLY SORTED F-C	11:20 RAIN
		3			SD, WET	
		5				
	35	5				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

FIELD BORING LOG	PROJECT: SALTSTONE VAULT 2	JOB NO.	SHEET NO. 3 OF 7	HOLE NO. ZV2B1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	40					
SS-11		6	20"/24		RD BRN CLAYEY SAND (SC) POORLY [illegible] F-C	1:15 PM LIGHT
		11			25% FINES WET	RAIN
		14				
		14				
	45					
SS-12		11	21"/24"		BRN SAND W/ SILT (SW-SM) WIDELY GRADED	2:00 PM
		17			F-C SAND ~10% FINES WET	
		19				RE-CONDITIONING
		21				MUD
	50					
SS-13		13	20"/24"		RD BRN SAND W/SILT (SW-SM) WIDELY GRADED	2:30 PM
		18			F-C WET, DENSE	
		20				
		23				
	55					
SS-14		17	10"/24"		RD BRN SAND (SW) WIDELY GRADED F-C, WET	3:15 PM
		22			DENSE	RAIN OUT
		28				
		27				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 4 OF 7 | HOLE NO.: ZV2B1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-15	60	WR, 4, 4, 6	23/24"		TOP 6" SAND (SM) [illegible], WET. LT BRN SILTY SAND (SM) V.F.S. TR MAG OXIDE. WET. SOFT.	
SS-16	65	2, 3, 3, 4	24/24"		LT. BRN CLAY W/SAND (CL), MED. PLASTIC, WET MAG OXIDE V.F.S.	
SS-17	70	WR, 4, 4, 4	20/24"		LT. BRN SILTY SAND (SM), SOFT, MOIST LAMINATED TR. MAG OXIDE	MONDAY SUNNY 85°F 9:30 TRUCK OUT OF WATER
SS-18	75	WR, 2, 3, 4	23"/24"		LT. BRN SAND W/SILT (SP-SM), SOFT, WET V.F.S.	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		5 of 7	ZV281

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-19	80	7 11 14 27	24"/24"		LT BRN SAND (SW) WELL SORTED F.S., WET	11:30
SS-20	85	6 8 8 10	20"/24"		LT BRN SAND (SP) POORLY SORTED, M-C, WET	12:00
SS-21	90	3 4 6 8	18"/24"		LT BRN SAND w/SILT (~~SP~~ SW-SM) WELL SORTED M.S. WET, LOOSE	1:30
SS-22	95	WR 3 2 4	20"/24"		BRN SILT w/ SAND (ML) V.F.S., LOW PLASTICITY, MOIST, MED STIFF	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV281

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		6 of 7	ZV2B1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	100					
SS-23		8	15"/24"		BRN SAND (SP) P... GRADED F.S.	3:30
		8			FINES WET, - HCl	
		8				
		9				
	105					
SS-24		WH	23/24"		TOP 8" SAND W/CLAY (SW-SC) F-C VERY	TUESDAY 8:30
		WH			SOFT. - HCl	
		1			BOT 15" SILTY SAND (SM) VFS VERY SOFT	LOST CIRCULATION
		2			LAMINATED, - HCl	
						REGAINED PARTIAL
					ROD DROP TO 114'	CIRC.
						LOSING ~ 100 g/
						PER MIN
	110					
						KEEPING PUMP
						PRESS AS LOW
						AS POSSIBLE AND
						MUD AS THICK
						A POSSIBLE
						DRILL BIT ADVANCED
	115					TO 115' FOR SAMPLE
SS-26		WH	24"/24"		LT GRN SILTY SAND (SM) V.F.S LOW PLASTICITY	10:00
		WH			WET SOFT, - HCl	
		WH				OUT OF WATER
		WH				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
		7 of 7	2VZB1

SITE	COORDINATES N E	WEATHER CONDITIONS

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH

GROUND EL.	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	120	1			SAA w/ SHELL FRAGS ~10% +HCl	CIRCULATION
		3	24"			POSSIBLE WHILE
SS-26		5	24"			DRILLING BUT
		8				LEVEL DROPS
						RAPIDLY IN HOLE
						ONCE PUMP
						SHUT OFF
						2:30
	125					MUD DARK GRN
		3			DRK GRN SILTY SAND TO SILT w/ SAND (SM-MH) +HCl	GREEN CLAY
SS-27		4	24"		VFS.	
		6	24"			
		7				
	130	2			SAA	4:00 PM
		3	24"			TD
SS-28		4	24"			
		8				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. 2VZB1

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		1 of 7	ZV2B2

SITE	COORDINATES	WEATHER CONDITIONS
Z-AREA VAULT 2	N 77353.7 E 66982.6	SUNNY 75°F

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH
4-28-05	5-04-05	GREGG IN-SITU	CME 75	4"	CME AUTOMATIC HAMMER	[illegible]

GROUND EL.	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:
279.00		FRANK H. SYMS	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						4-28-05 THUR
						SUNNY 75°F.
						0-10' DRILL OUT
	5					
	10	6	18"/24"		RDW BRN CLAYEY SAND (SC) WIDELY GRADED	
SS-1		9			LAMINATED, DRY, MED DENSE	
		9				
		11				
		6	20"/24"		SAA	11:05
SS-2		8				
		9				
		12				
		7	18"/24"		RD BRN, LT BRN, P/R CLAYEY SAND (SC) WIDELY	
SS-3	15	11			GRADED, MOD DENSE. C.S. MOTTLED	
		13				
		17				
		9	18"/24"		RD BRN LT BRN, WHT SAND (SW-SC) W/ CLAY.	
SS-4		9			WIDELY GRADED M-C MOD DENSE	
		12				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG | PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 2 of 7 | HOLE NO.: ZV2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
		12				
SS-5		8	17"/24"		LT. BRN CLAYEY SAND (SC) WIDELY GRADED, M-C	11:45 AM
		12			MOD DENSE, MOIST, HIGHLY PLASTIC FINES	
		12				
		14				
SS-6	20	7	20"/24"		SAA SLIGHTLY MORE SANDY	
		11				
		11				
		12				
						OVERDRILLED 1½' TO 25' RESUMED CONT. SAMPLING @ 25'
SS-7	25	6	18"/24"		LT. BRN SAND W/ SILT (SP-SM) NARROWLY GRADED	
		8			M.S. NON PLASTIC, WET, MOD DENSE TO	
		8			LOOSE	
		10				2:00 PM
SS-8		8	20"/24"		SAA BECOMING SLIGHTLY FINER GRAINED SAND	
		10			AND RED BRN, WET	
		10				
		12				
SS-9		8	20"/24"		SAA, WET	2:15 PM
	30	9				
		9				
		9				
SS-10	35	4	20"/24"		SAA FINES CONTENT INCREASING SLIGHTLY, WET	2:55 PM
		7				
		8				
		8				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO. 3 of 7 | HOLE NO. 2V2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-11	40	5, 8, 7, 7	20"/24"		RD BRN, LT. BRN CLAYEY SAND, SAND w/CLAY (SC, SW-SC) WIDELY GRADED, WET	
SS-12	45	5, 8, 10, 15	10"/24"		BRN, RD BRN, SAND w/SILT (SP-SM) POORLY GRADED, LAMINATED, MOD DENSE WET	FRIDAY
SS13	50	9, 12, 15, 18	12"/24"		BRN SAND (SW) WIDELY GRADED F-C, DENSE, WET	10:00 AM
SS14	55	4, 3, 3, 4	14"/24"		BRN, RD. BRN CLAYEY SAND (SC) WIDELY GRADED F-C, SOFT, WET, LAMINATED BEDS OF CLAY AND SAND	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. 2V2B2

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO. 4 OF 7 | HOLE NO. ZV2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						CLEANED OUT HOLE
						CONDITIONED MUD
SS-15	60	2, 3, 3, 2	24"/24"		LT BRN CLAYEY SAND (SC) WIDELY GRADED F-C SOFT, WET	11:10
SS-16	65	2, 3, 4, 5	24"/24"		SAA	11:35
SS-17	70	3, 3, 5, 6	20"/24"		SAA	11:55
						MONDAY 5-2-05
SS-18	75	7, 18, 30, 42	11"/24"		LT BRN SAND (SP) POORLY GRADED M.S. DENSE TR. Mg OXIDE, WET	9:30

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		5 of 7	ZV2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-19	80	11	10"/24"		LT. BRN SAND w/ SILT (SP-SM) DENSE F-M WET	10:10 AM
		11			TR. FE OXIDE	
		17				
		21				RECONDITIONED MUD
						GOOD CIRCULATION
SS-20	85	6	20"/24"		DRK BRN SAND w/ SILT (SW-SM) MED DENSE	11:15
		8			F-C, WET	
		9				
		12				
SS-21	90	WR	24"/24"		BRN/LT. BRN SILTY SAND (SM) LOW PLASTICITY, F.S.	
		6			OCCASSIONAL C.S.	
		7				
		13				
SS-22	95	8	23"/24"		SAA	1:30
		9				
		7				
		8				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 6 OF 7 | HOLE NO.: 2V2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-2[illegible]	100	3	24"/24"		LT. BRN SILTY SAND (SM) V.F.S. ALMOST SILT SIZE, LOW PLASTICITY, WET	+ HCl
		4			FEW SHELL FRAG.S <1%, MASSIVE, NO OBVIOUS	
		5			BEDDING, NO [illegible] [illegible]. FINES MAY BE	
		9			CARB.WHITE MUD. HCl REACTION NOT LIMITED TO	
					SHELL FRAGS	
SS-24	105	2	24"/24"		SAA	+ HCl
		WR				
		3				
		2				
SS-25	110	WR	22"/20"		TOP 6" SAA	REFUSED LAST 4"
		8			BOT 16" SAA EXCEPT CEMENTED	OF 4TH 6 IN INT
		28				+ HCl
		50R				
						HARD DRILLING
SS-26	115	10	24"/24"		SAA w/ CEMENTED NODULES.	+ HCl
		17				
		20				
		21				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. 2V2B2

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 7 OF 7 | HOLE NO.: ZV2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-27	120	50 e	5"/5"		LT. GRN SHELL HASH SILTY SAND (SM) ~ 75 % SHELL FRAGS.	5" IN 1ST +HCl
SS-28	125	7, 12, 31, 30	24"/24"		LT GRN TO DRK GRN SILTY SHELL HASH LARGE SHELL FRAG'S IN SILT MATRIX. BTT 6" BECOMING MOSTLY SHELL FRAG.	+ HCl
SS-29	130	19, 19, 20, 23	6"/24"		DRK GRN SILTY SAND (SM) TO (MH) CEMENTED NODULE STOPPED UP SPOON	4:00 PM TD 132
SS-30	135					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2				JOB NO.	SHEET NO. 1 OF	HOLE NO. ZVZB3	
SITE: VAULT 2	COORDINATES: N 77433 (7744.88) E 66956.75			WEATHER CONDITIONS: SUNNY 75°F			
BEGUN: 4-21-05	COMPLETED	DRILLING CO./DRILLER: GREGG DRILLING	DRILL MAKE AND MODEL: MOBILE	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL: 140 lb/30" ATD	TOTAL DEPTH: 132	
GROUND EL.: 278.1	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY: FH SYMS		REVIEWED BY:			

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						4-21-05 SUNNY 75°F
					AUTOMATIC HAMMER	0-5 DEEP DUG OUT
	5					
SS-1		10, 15, 16, 17	18"/24"		RED/BRN/PURPLE CLAYEY SAND (SC) WIDELY GRADED M-C. DENSE. MOIST	
SS-2	7	8, 14, 16, 21	18"/24"		RED/BRN/WHITE CLAYEY SAND (SC) SAA WIDELY GRADED C-M DENSE MOIST MOD. PLASTIC BOT 6" PUR/WHITE SILTY SAND (SM)	
SS-3	9–10	6, 11, 12, 16	15"/24"		RED/BRN/WHT CLAYEY SAND (SC) WIDELY GRADED C-M DENSE MOIST. LAMINATED W/ WHT KAOLIN. MOD. PLASTIC	
SS-4	11	5, 6, 10, 10	16"/24"		SAA BECOMING V. MOIST ALMOST WET	
SS-5	13	7, 13, 14, 14	13"/24"		SAA V MOIST	
SS-6	15	8, 11, 12, 12	14"/24"		LT. BRN/WHT CLAYEY SAND W/SILT (SM) WIDELY GRADED MED DENSE MOIST	
	17	6				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB3

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE		2 OF	ZV283

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-7	18	14	16"/24"		SAA	
		13				
	19	13				
SS-8		9	13"/24"		SAA	
	20	10				
		12				
	21	13				
SS-9	25	8	18"/24"		LT. PUR SAND w/SILT (SP-SM) POORLY GRADED	2:00 PM
		10			M-F SAND MOIST MED DENSE NON PLASTIC	
		11				
	27	12				
SS-10	30	6	17"/24"		LT BRN SAND w/SILT (SP-SM/SP) POORLY GRADED	
		6			F SAND, MOIST MED DENSE, NON PLASTIC	
		8				
	32	12				
SS-11	35	8	18"/24"		RED BRN SAND w/CLAY (SP-SC) POORLY GRADED M-F	3:15 PM
		9			SAND. 15-20% FINES, WET, MED DENSE, NON	
		10			PLASTIC	
		10				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV283

FIELD BORING LOG	PROJECT: SALTSTONE VAULT 2	JOB NO.	SHEET NO. 3 OF	HOLE NO. 2V2B3

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	38					
	40					
SS-12		10	20/24		RED BRN CLAYEY SAND (SC) POSSIBLE (SP-SC)	3:50 PM
		11			M-C SAND, WET, MED DENSE NON PLAST.	
		14				
		15				
	45					
SS-13		12	18/24"		RED BRN CLAYEY SAND (SC) M-F SAND, WET,	4:30 END OF DAY
		16			DENSE, MOD PLASTIC	
		17				
	47	23				
	50					
SS-14		15	12"/24"		LT. BRN, SAND (SP) POORLY GRADED M. SAND	4-22-05 8:00 AM
		25			WET, DENSE	
		25				
	52	30				HYDRAULIC MAINT
	55					
SS-15		4	22"/24"		LT, BRN, RED BRN, CLAYEY SAND (SC) WELL GRADED	10:00 AM
		3			POORLY SORTED M-C SAND, WET, SOFT	
		3				
	57	7				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. 2V2B3

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 4 OF | HOLE NO.: ZV2B3

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-16	60	2	24"/24"		LT BRN, RED BRN CLAYEY SAND (SC) WIDELY GRADED	10:45 PM
		2			POORLY SORTED QTZ SAND SIMILAR TO ABOVE W/	
		2			CLAY CONTENT INCREASING. CLAY SEAMS UP TO 1"	
	62	2			PRESENT, WET, HIGHLY PLASTIC, SOFT LIGNITE	NO FLUID LOSS
						LUNCH
SS-17	65	2	20"/24"		SAA BECOMING MORE SANDY ~25 % FINES	12:40
		3			LIGNITE (Mn OXIDE)	
		4				
	67	6				
SS-18	70	3	18"/24"		LT. BRN SAND (SP). NARROWLY GRADED, WELL	
		8			SORTED M QTZ MED DENSE, WET, TR. LIGNITE	
		11				
	72	15				
SS-19	75	15	16"/24"		LT BRN SAND (SP) NARROWLY GRADED WELL	POSSIBLE TRANSITION
		27			SORTED F QTZ SD. WET, DENSE, TR.	TO TINKER?
		32			LIGNITE FLECKS.	
	77	29				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		5 OF	ZV2B3

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	73					
						MONDAY 4-
SS-20	80	8	15"/24"		LT. BRN SAND (SP) NARROWLY GRADED M.S.	10:00 AM
		12			WET MOD DENSE	
		14				HOLE CASED TO
	82	15				30'
SS-21	85	5	24"/24"		LT BRN CLAYEY SAND (SC) NARROWLY GRADED F.S.	10:15 AM
		7			MOIST, PLASTIC, MOD. STIFF ~35% FINES	
		8				
	87	12				
SS-22	90	13	20"/24"		LT. BRN SAND (SP) NARROWLY GRADED M.S. WET	
		18			DENSE	
		19				
	92	13				
						?
						TOP OF SANTEE ?
SS-23	95	4	24"/24"		LT. GRN SAND (SP) NARROWLY GRADED V.F.S	
		7			WET, FAIRLY DENSE OCCASIONAL C.S. TO	
		8			PEBBLE SIZE SUB-ANGULAR QTZ SAND IN	
	97	10			SMALL BEDS OR SUSPENDED IN V.F.S. OCCASIONAL	
					SHELL FRAG	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 6 OF | HOLE NO.: ZV2B3

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-24	100 – 102	16, 41, 18, 15	20"/24"		LT. GRN CALCAREOUS SAND. V.F S AS ABOVE WITH >50% SHELL FRAGS < 1/4" WET NON CEMENTED	
						104' RIG CHATTER
SS-25	105 – 107	19, 50R	10"/10"		LT. GRN SAND (SP) CEMENTED V.F.S.	4" IN 2nd 6" INT. HARD DRILLING GOOD FLUID RETRN
						TUESDAY
SS-26	110 – 112	15, 50R	8"/8"		SAA	2" IN 2nd 6" INT. RAIN-OUT 10:20AM
SS-27	115 – 117	11, 16, 15, 17	18"/24"		LT GRN SILTY SAND (SM) V.F.S. CEMENTED NODULES FEW SHELL FRAGS IN THIN LAYERS	WEDNESDAY

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 7 OF | HOLE NO.: ZV2B3

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-28	120	6, 7, 19, 15	24"/24"		LT GRN SILTY SAND TO SILT (SM)-(ML) V.F.S. FEW CONSTS. FRAGS: WEATHERED SHELLS	9:30 AM
SS-29	125	7, 10, 14, 18	24"/24"		DRK GRN SILTY SAND (SM) POORLY SORTED M-C FEW SHELL FRAGS LAMINATED	11:00 AM GREEN CLAY
SS-30	130	38, 28, 50R	15"/18"		DRK GRN SAND (SP) WELL SORTED C-S	CONGAREE
SS-31	135					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3

FIELD BORING LOG

PROJECT	SALTSTONE [illegible] Z
JOB NO.	
SHEET NO.	1 of 7
HOLE NO.	ZVZ8IU
SITE	Z-AREA
COORDINATES	N E
WEATHER CONDITIONS	SUNNY 80 F
BEGUN	[illegible]
COMPLETED	
DRILLING CO./DRILLER	[illegible]
DRILL MAKE AND MODEL	CME 75
HOLE SIZE	4"
SAMPLE HAMMER WEIGHT/FALL	N/A
TOTAL DEPTH	
GROUND EL.	291.9
GROUND WATER DEPTH/DATE	
TECHNICAL OVERSIGHT BY:	F H [illegible]
REVIEWED BY:	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	5					
	10					
ST1	15		21"/24"			SEE UD LOG PA SAMPLE

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2 8IU

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 2 of 7 | HOLE NO.: ZVZBIU

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	20					
	25					
	29					SEE UD LOG
ST2	30		19½"/22"			
	31					SEE UD LOG. PA SAMPLE
ST3			13"/24"			
	33					
	35					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZBIU

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		3 of 7	ZVZB1U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	40					
	45					SEE UD LOG
ST4			18/24			
	50					
	55					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB1U

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		4 of 7	ZV2B1U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	60					
	62					SEE JO LOG
ST5			24"/24"			
	64					
	65					
	70					
	75					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1U

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		5 of 7	ZV2B1U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	80					
ST6	85		0"/24"			SEE JC LOG
ST7	87		10"/24"		ATTEMPT NO. 2	SEE JC LOG
ST8	90		0"/24"			SEE JC LOG
	95					97' LOST CIRCULATION CONTINUING TO TOTAL DEPTH

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1U

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		6 of 7	ZV2B1U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	103					
	104					SEE UD
PS1	105		24			PISTON LOG
	107				OCHRE YELLOW CLAYEY SILT	
PS2			18.5 / 24			
	109					
	110					
	115					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1U

FIELD BORING LOG

PROJECT: SALT[illegible] | JOB NO.: | SHEET NO.: 7 of 7 | HOLE NO.: ZV281U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	20'		[illegible]"		[illegible] STA [illegible]	
[illegible]					DRILLER [illegible] [illegible] SOFT	
			24"		[illegible] WARLEY [illegible] ~12'	
			24"		TOP OF PS3 IS LT GREY-GREEN SILTY CLAY	
PS3					W/ SHELL HASH, BOTTOM OF PS3 IS SAME	
			24"		BUT DARKER GREEN. (WARLEY HILL FM)	
	[illegible]					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
VAULT 2		1 of	ZV2B2U

SITE	COORDINATES	WEATHER CONDITIONS
Z AREA	N E	SUNNY 95 F

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH
[illegible]	6 [illegible]	[illegible]	CME 75	4"	N/A	

GROUND EL.	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:
28[illegible]		F H SIMS	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	5					
	6					SEE UD LOG
ST-1			/24"			
	8					
	10					
	14					SEE UD LOG
ST-2	15		17"/24"			
	16					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2U

FIELD BORING LOG

PROJECT: [illegible] JOB NO.: SHEET NO.: 2 of 2 HOLE NO.: ZVZB2U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	20					
	25					
	28					SEE UD LOG
ST-3			6"/24"			
	30					
	34					SEE UD LOG
ST-4	35		21"/24"			
	36					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB2U

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		1 of 1	ZV2B3U

SITE	COORDINATES		WEATHER CONDITIONS
Z AREA	N	E	SUNNY 98 °F

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH
6-7-05	6-7-05	GREGG [illegible]	CME 75	4	N/A	

GROUND EL.	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:
278.		F H SIMS	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	5					
	10					SEE UD LOG
ST-1			19/24			PA SAMPLE
	12					
	15					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3U

FIELD BORING LOG

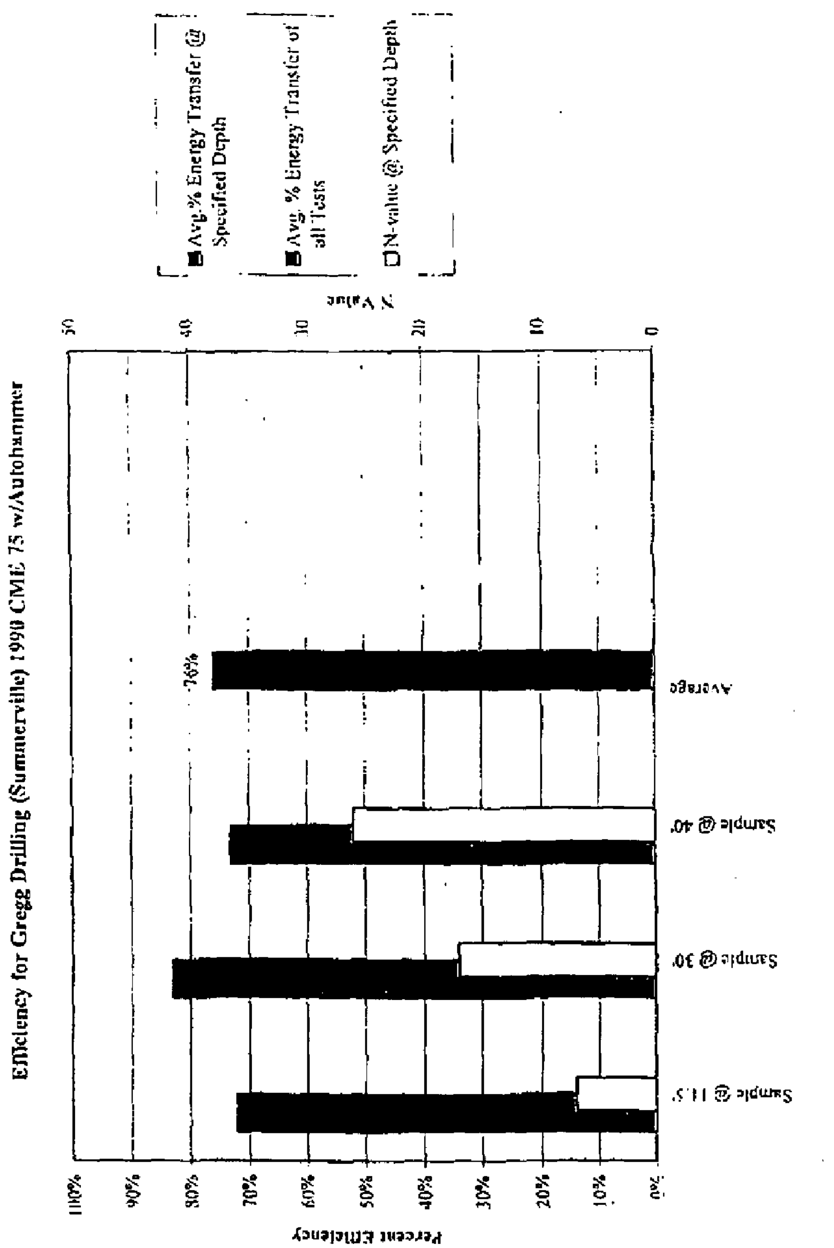
PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SAUS ...		2 of 2	ZVZB3U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	20					
	25					
	27					SEE UO LOG
ST-2						PA SAMPLE

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB3U



Summary of Results
SPT Energy Transfer Measurements
1990 CME-75 w/Autohammer
Summerville, South Carolina

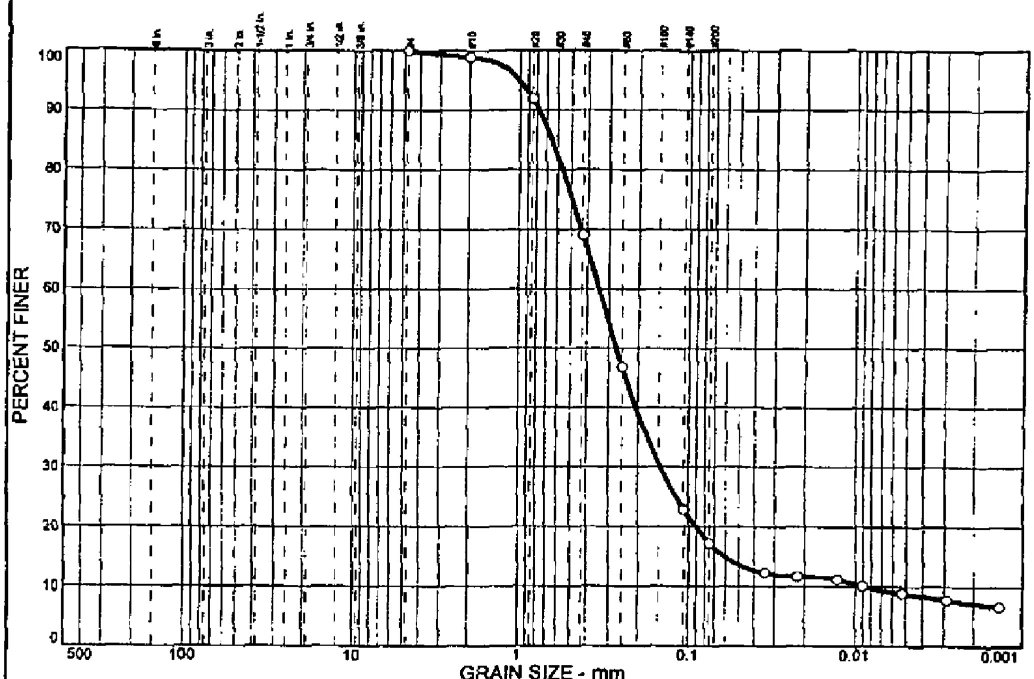
S&ME

JOB NO. 113-05-561

June 22, 2005

Figure: 1

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	82.8	8.4	8.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.9		
#20	91.9		
#40	69.0		
#60	46.8		
#140	22.8		
#200	17.2		

* (no specification provided)

Soil Description

Tan Silty Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.654 D_{60}= 0.344 D_{50}= 0.271
D_{30}= 0.147 D_{15}= 0.0608 D_{10}= 0.0088
C_u= 39.32 C_c= 7.13

Classification

USCS= AASHTO= A-2-4(0)

Remarks

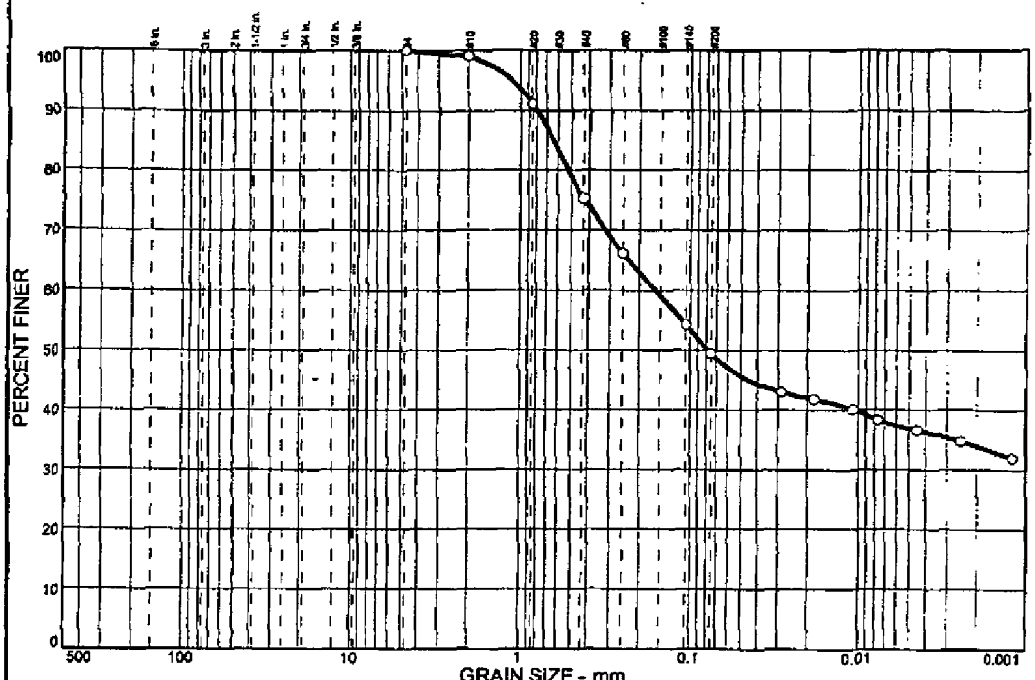
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 2-4 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	50.6	12.5	36.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.1		
#20	91.1		
#40	75.3		
#60	66.1		
#140	54.2		
#200	49.4		

* (no specification provided)

Soil Description

Light Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.640 D_{60}= 0.161 D_{50}= 0.0787
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-4(0)

Remarks

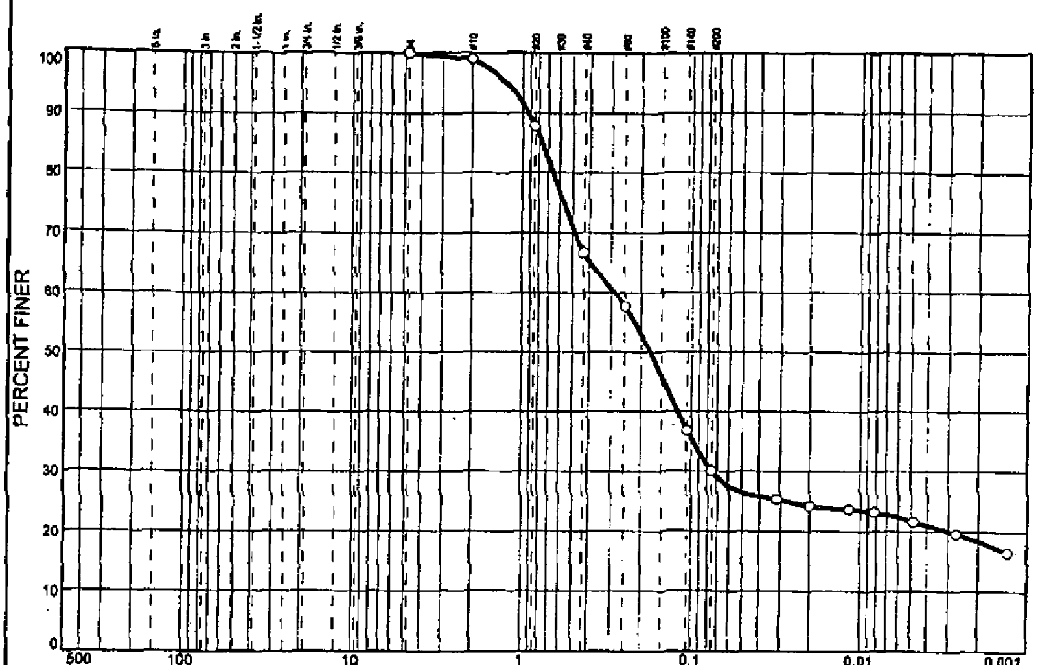
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 4-6 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	70.0	8.3	21.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.0		
#20	87.8		
#40	66.7		
#60	57.7		
#140	36.8		
#200	30.0		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.769 D_{60}= 0.288 D_{50}= 0.177
D_{30}= 0.0750 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

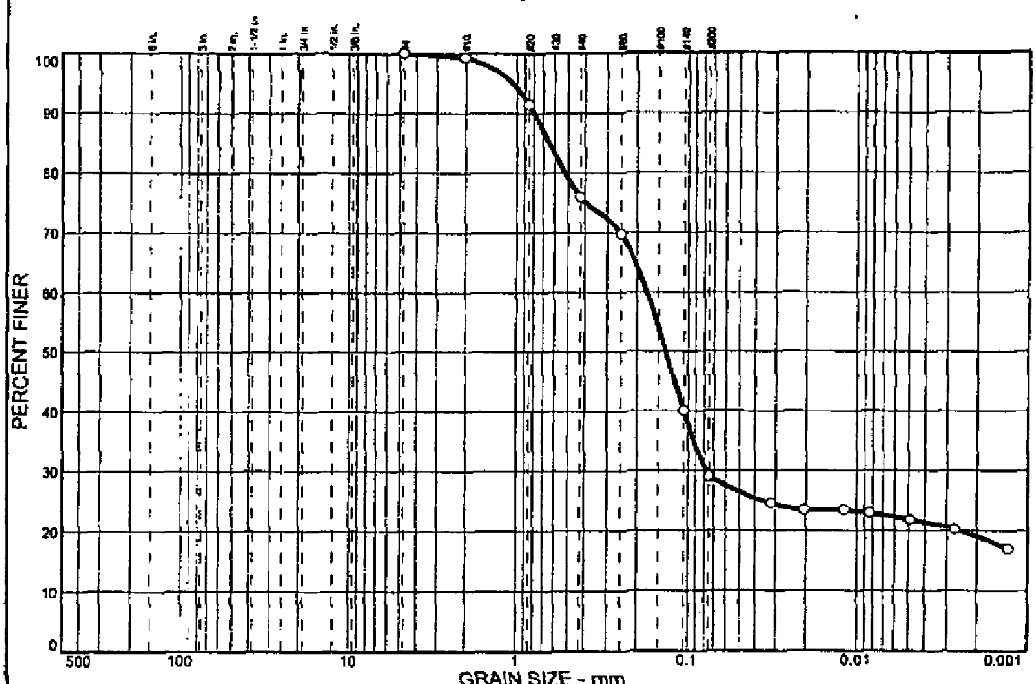
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 8 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	71.0	7.1	21.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.3		
#20	91.4		
#40	76.0		
#60	69.8		
#140	40.0		
#200	29.0		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.644 D_{60}= 0.177 D_{50}= 0.136
D_{30}= 0.0782 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

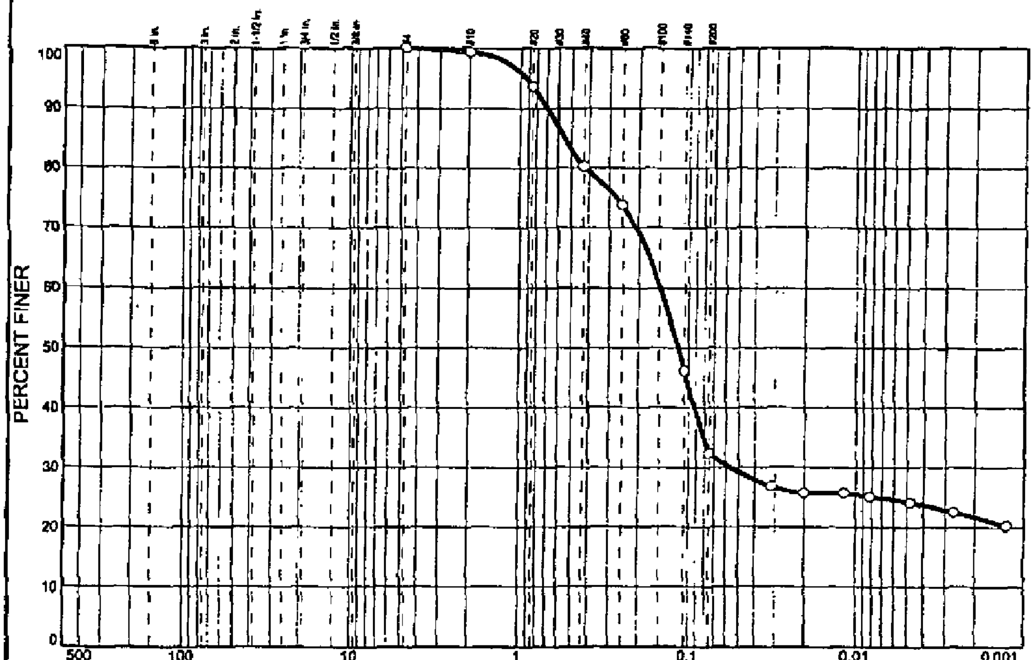
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 10 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	67.7	8.1	24.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.4		
#20	93.7		
#40	80.3		
#60	73.8		
#140	46.2		
#200	32.3		

* (no specification provided)

Soil Description

Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.550 D_{60}= 0.150 D_{50}= 0.116
D_{30}= 0.0564 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

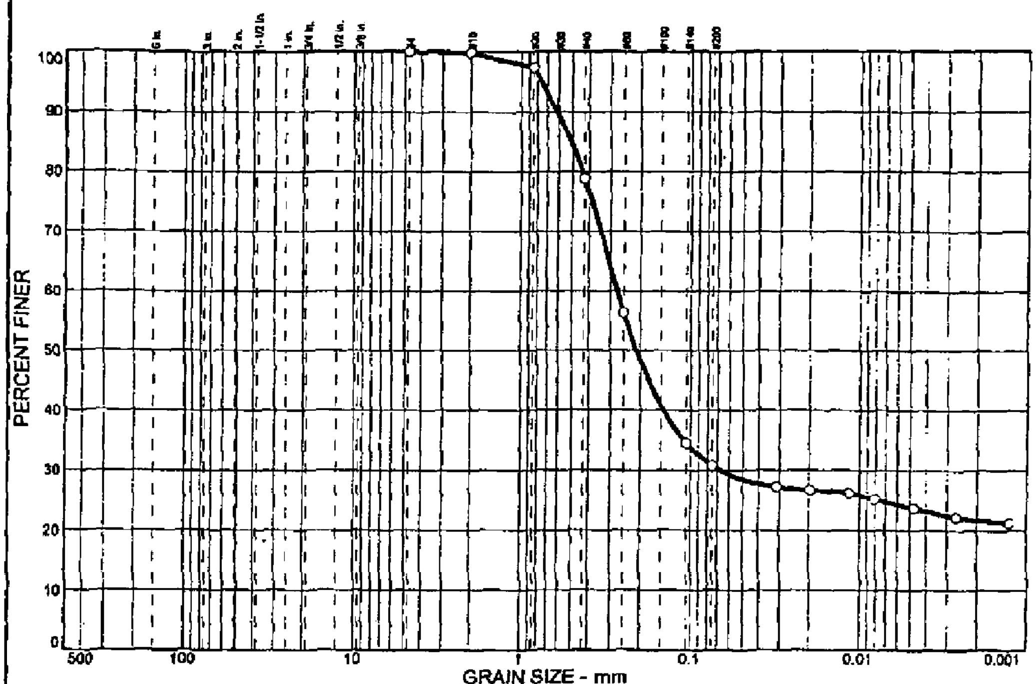
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 12 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	69.2	7.0	23.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#20	97.4		
#40	78.8		
#60	56.3		
#140	34.5		
#200	30.8		

* (no specification provided)

Soil Description

Tan Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.511 D_{60}= 0.274 D_{50}= 0.210
D_{30}= 0.0673 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

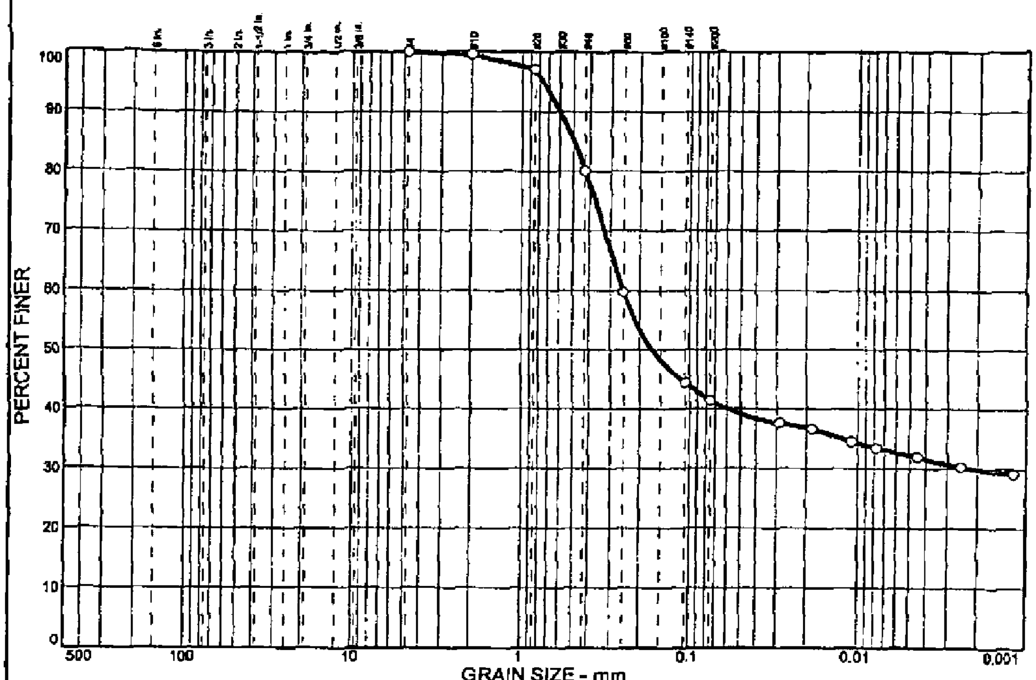
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 4 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	58.6	9.2	32.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.6		
#20	97.1		
#40	80.1		
#60	59.7		
#140	44.4		
#200	41.4		

* (no specification provided)

Soil Description

Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.496 D_{60}= 0.252 D_{50}= 0.168
D_{30}= 0.0022 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-4(0)

Remarks

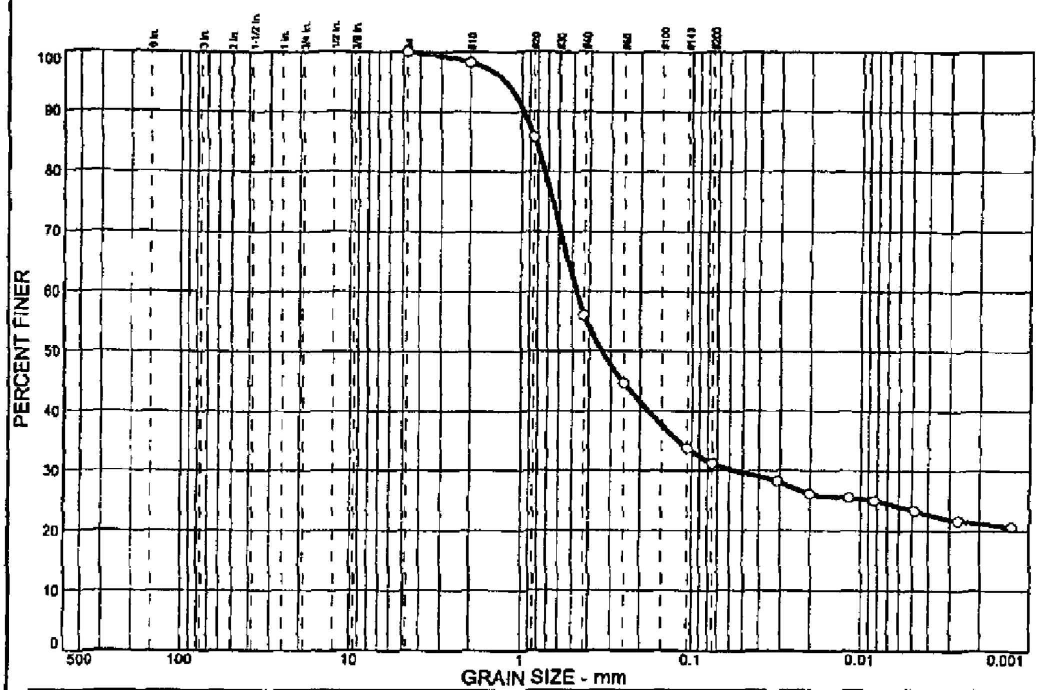
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 6 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	68.8	7.8	23.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.3		
#20	85.8		
#40	56.1		
#60	44.7		
#140	33.8		
#200	31.2		

* (no specification provided)

Soil Description

Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.831 D_{60}= 0.471 D_{50}= 0.340
D_{30}= 0.0562 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

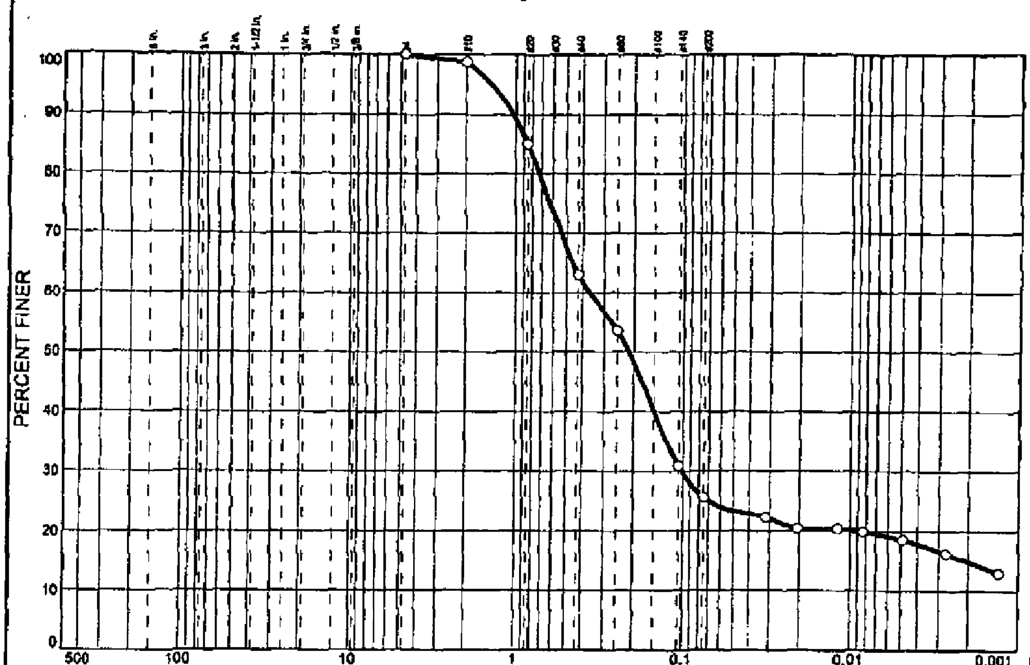
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 8 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	74.4	7.0	18.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.8		
#20	85.0		
#40	63.0		
#60	53.6		
#140	30.9		
#200	25.6		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.850 D_{60}= 0.367 D_{50}= 0.214
D_{30}= 0.101 D_{15}= 0.0021 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

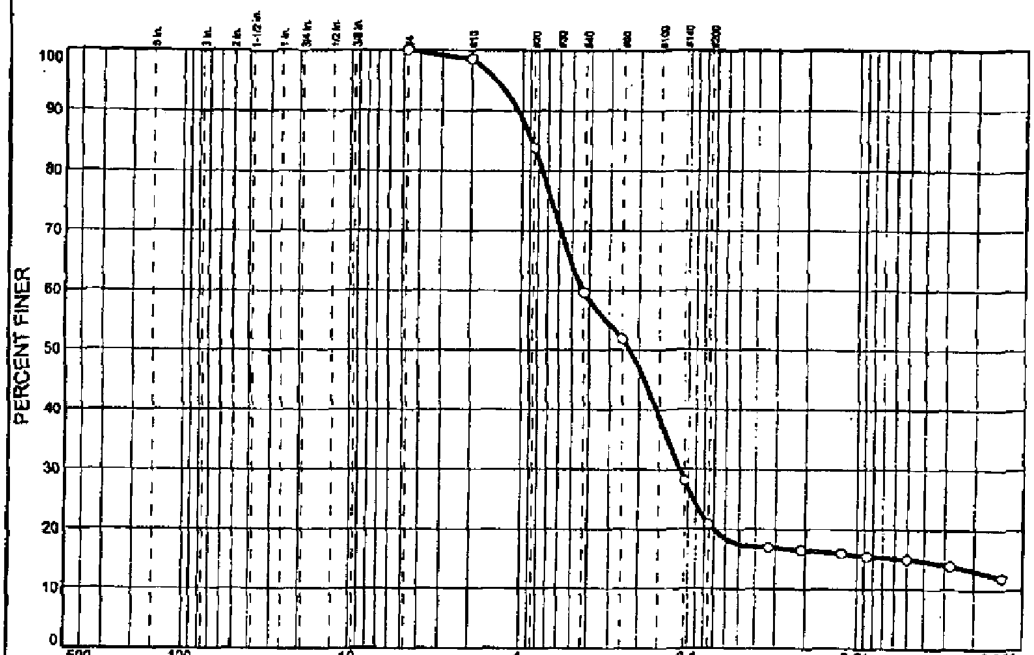
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 10 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	79.2	5.8	15.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.6		
#20	83.9		
#40	59.6		
#60	51.8		
#140	28.1		
#200	20.8		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.881 D_{60}= 0.432 D_{50}= 0.228
D_{30}= 0.114 D_{15}= 0.0050 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 12 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

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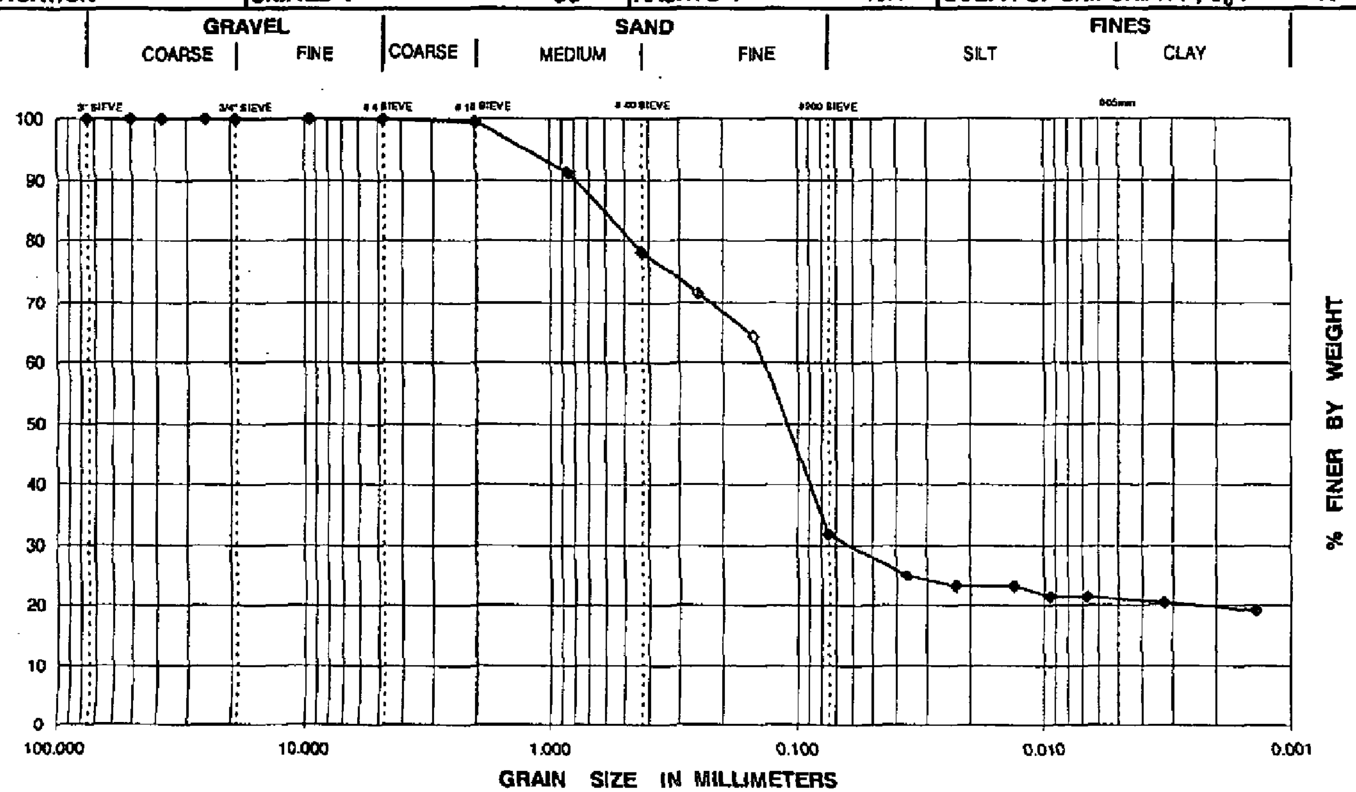
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1 8/17/03

JOB NAME :	Saltstone Vault # 2			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/28/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	12'-14'	SAMPLE NO. :	2	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Reddish brown clayey sand.					SP. GRAVITY, Gs :	2.65
LIQUID LIMIT, % :	44	PLASTICITY INDEX , % :	26	MOISTURE , % :	-	FINES , % :	32
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_C :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_U :	N/A



QORE PROPERTY SCIENCES

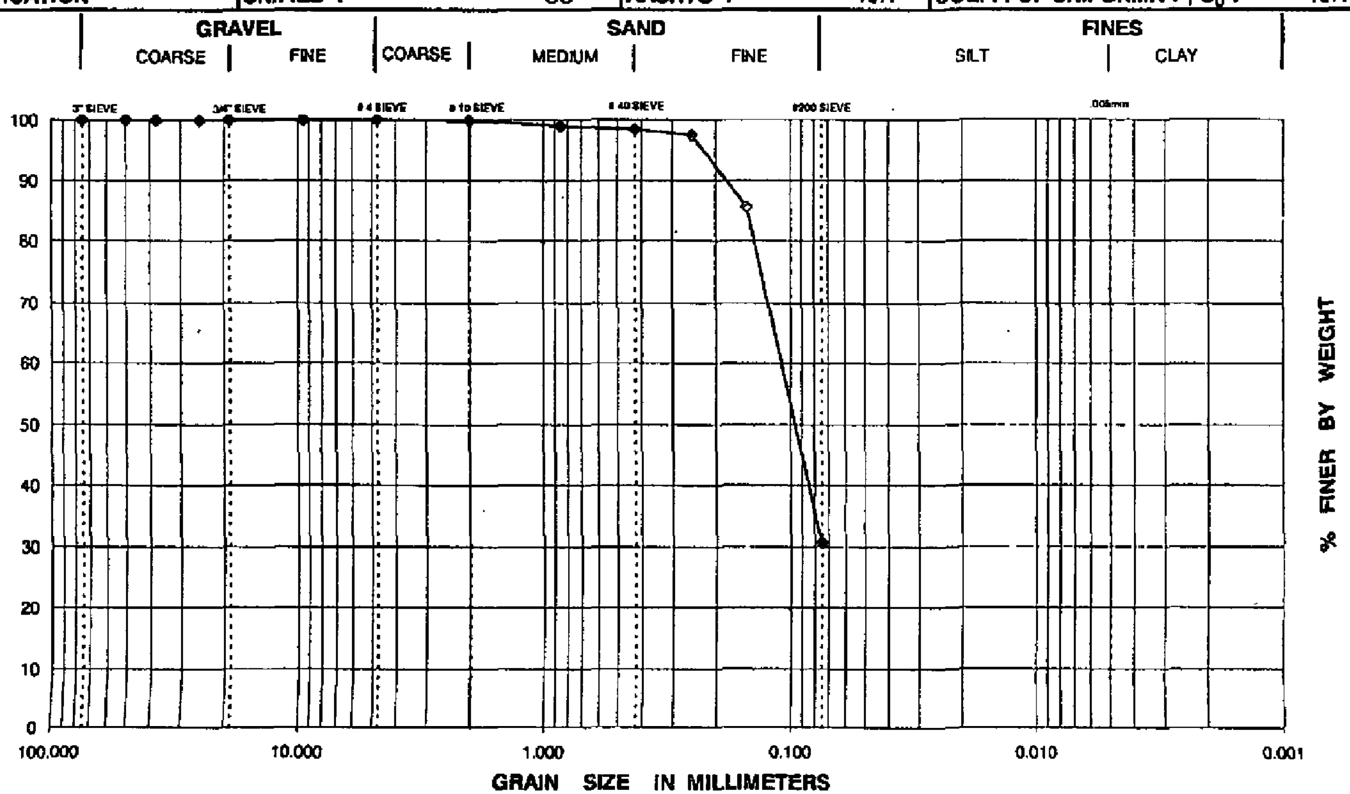
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV 1 9/17/03

AASHTO R18

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	20'-22'	SAMPLE NO. :	6	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	31
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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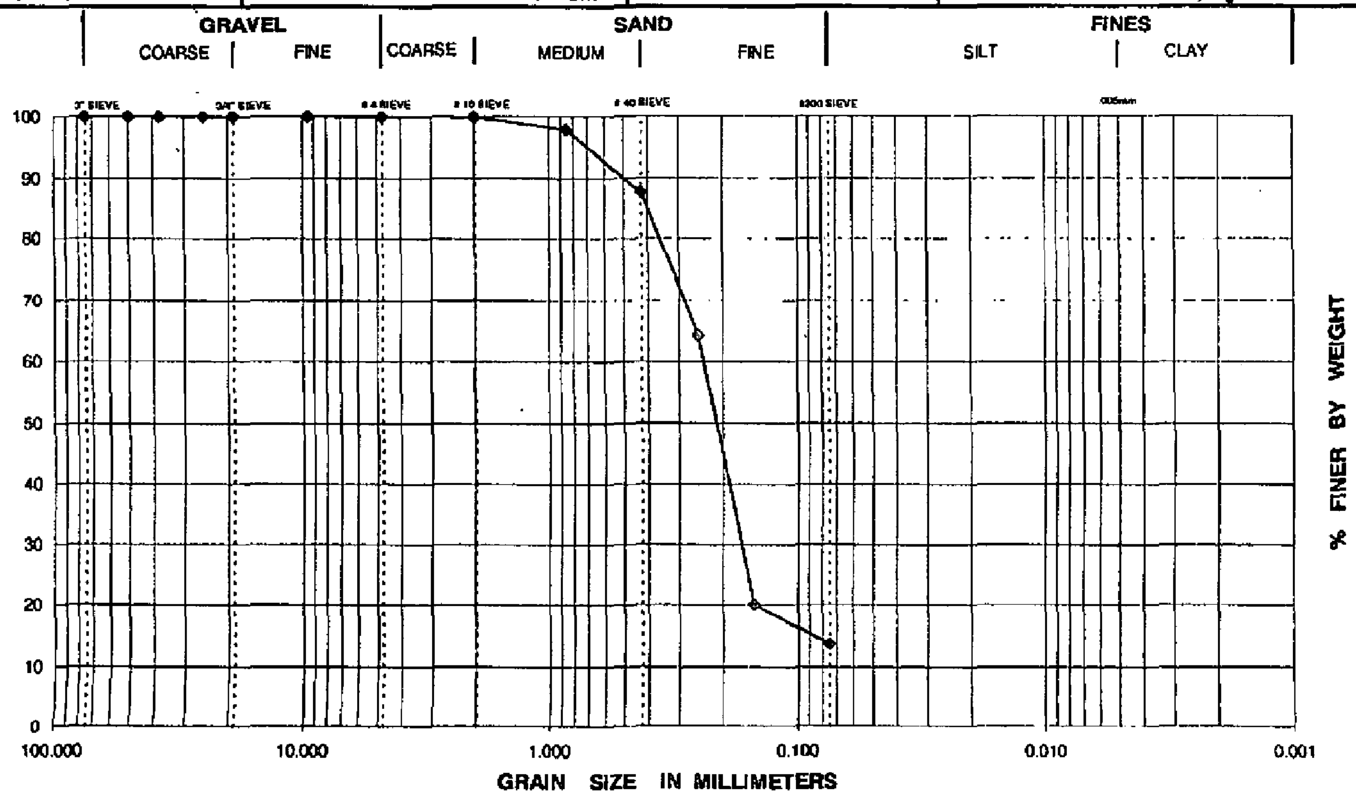
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,8/1/03

JOB NAME :	*Saltstone Vault # 2, Task 8*			*CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8*			
JOB NO. :	*24150-H*	REPORT NO. :	-	DATE :	*6/23/05*	REVIEWED BY :	
BORING / PIT NO. :	*ZV2B2*	DEPTH / ELEV. :	*27'-29'*	SAMPLE NO. :	*8*	SAMPLE TYPE :	*SS*
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	*Brown sand with silt.*					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	*14*
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	*N/A*
CLASSIFICATION		UNIFIED :	*SP-SM*	AASHTO :	*N/A*	COEFF. OF UNIFORMITY , C_u :	*N/A*



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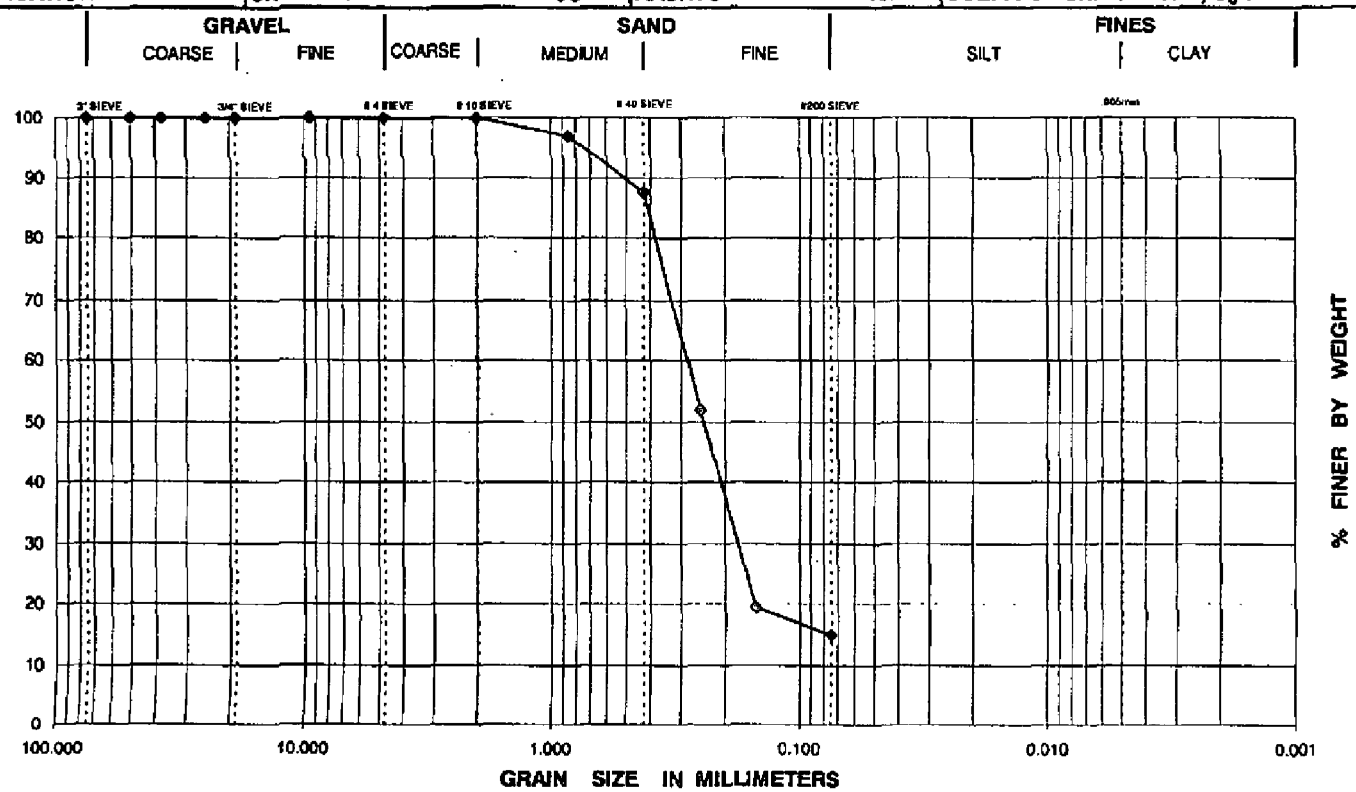
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1 9/17/03

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	35'-37'	SAMPLE NO. :	10	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	15
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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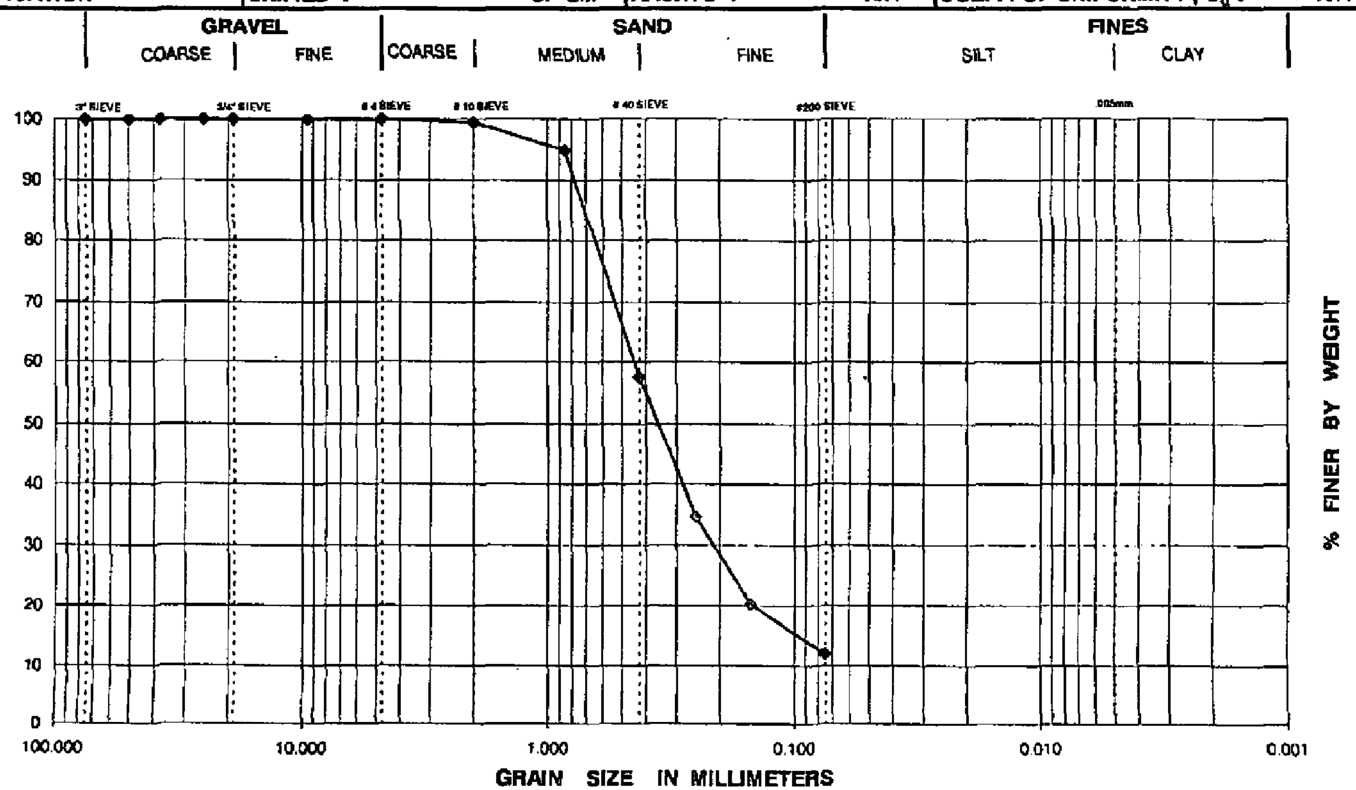
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1.9/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	45'-47'	SAMPLE NO. :	12	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown sand with silt.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	12
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SP-SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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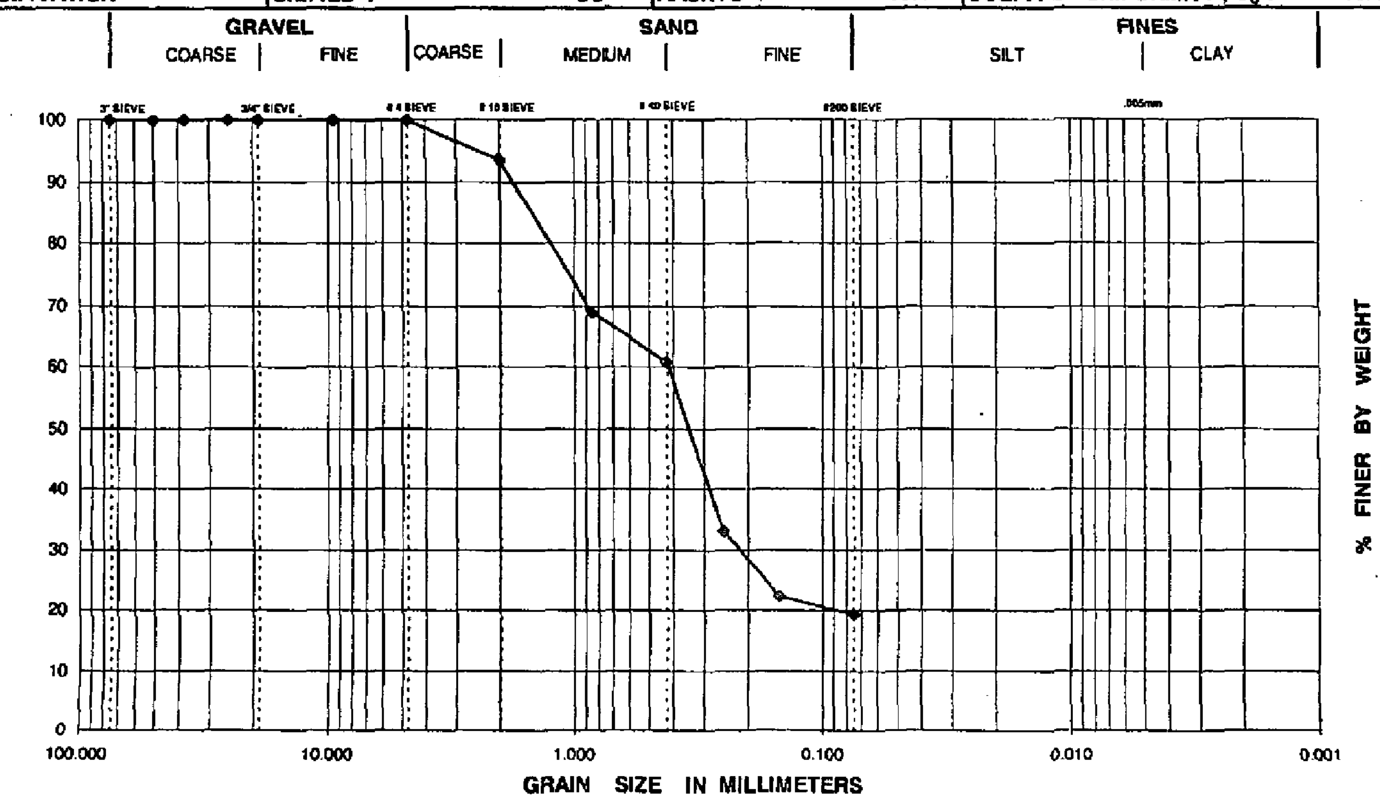
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

JOB NAME :	Saltstone Vault # 2, Task 8				CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	55'-57'	SAMPLE NO. :	14	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	19
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



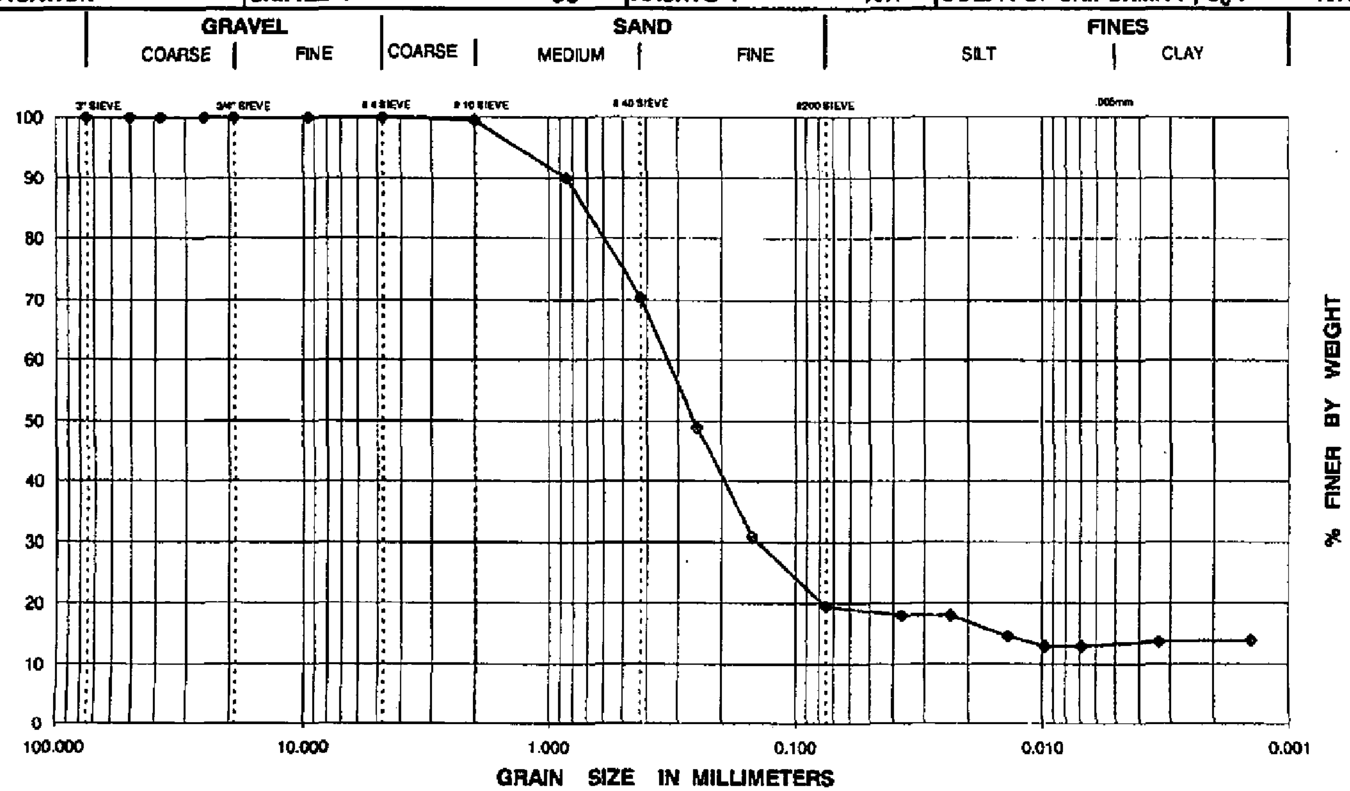
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

QORE PROPERTY SCIENCES

REV1:9/17/03

JOB NAME :	Saltstone Vault # 2				CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/28/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	60'-62'	SAMPLE NO. :	15	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown clayey sand.					SP. GRAVITY, Gs :	2.65
LIQUID LIMIT, % :	33	PLASTICITY INDEX , % :	19	MOISTURE , % :	-	FINES , % :	19
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



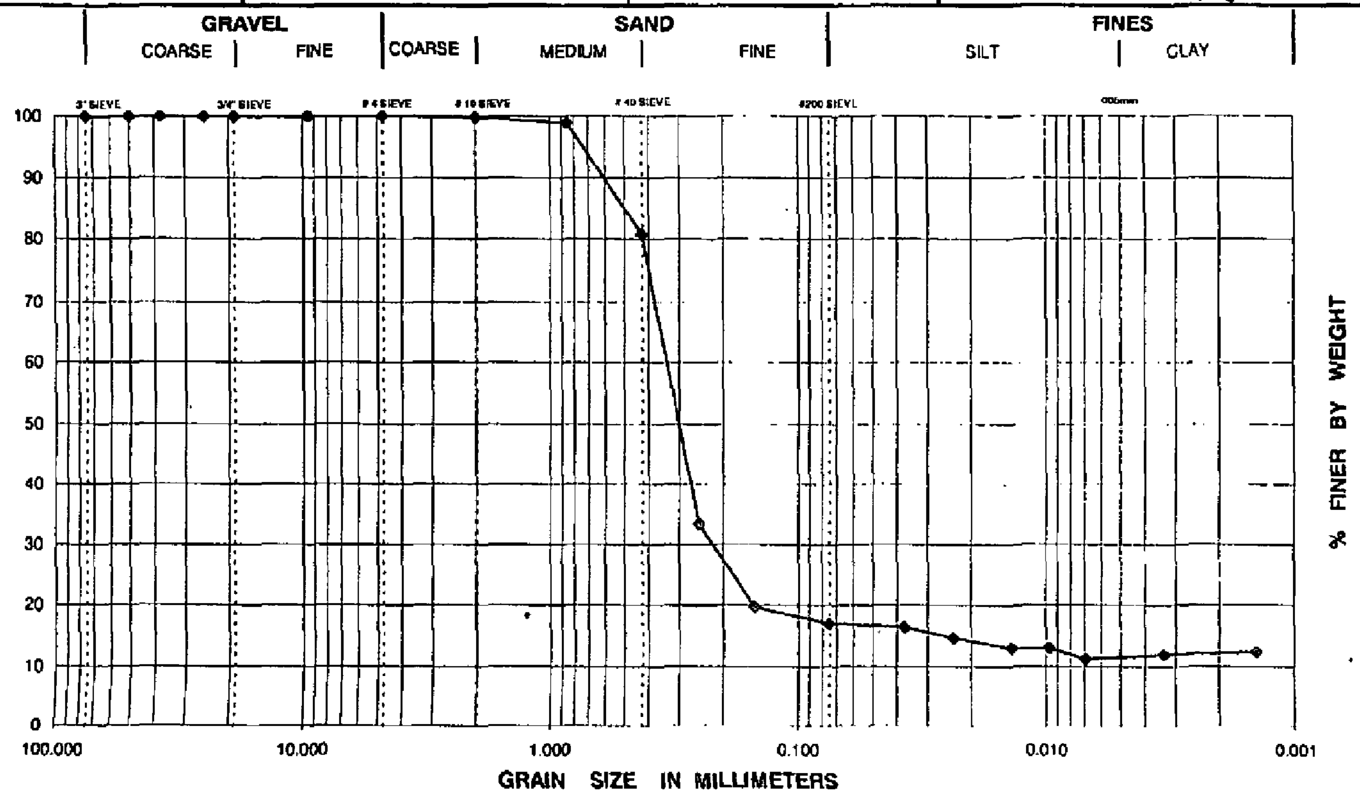
QORE

PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1.9/17/03

JOB NAME :	Saltstone Vault # 2			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/28/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	65'-67'	SAMPLE NO. :	16	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown silty sand.					SP. GRAVITY, Gs :	2.65
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX , % :	N.P.	MOISTURE , % :	-	FINES , % :	17
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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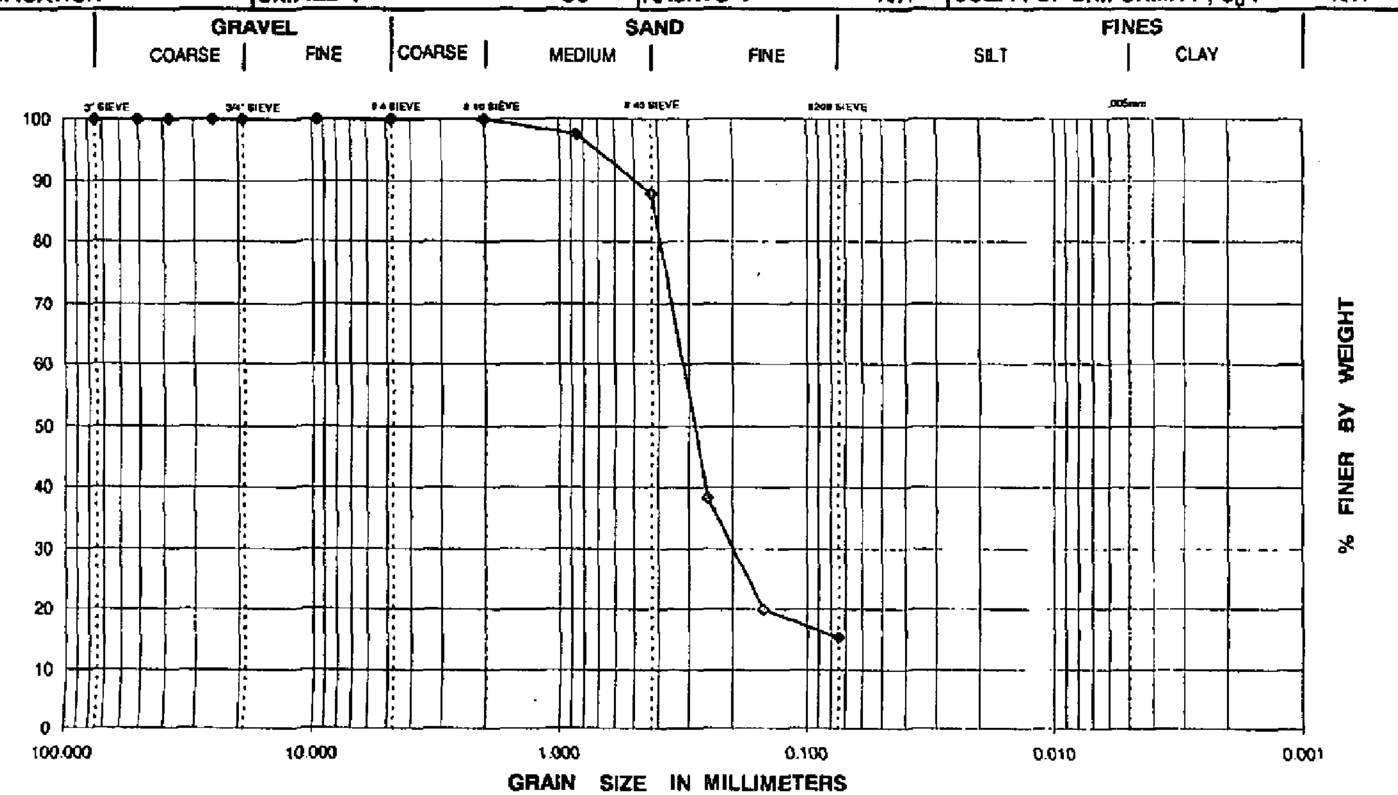
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV 1 8/17/03

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	70'-72'	SAMPLE NO. :	17	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	15
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



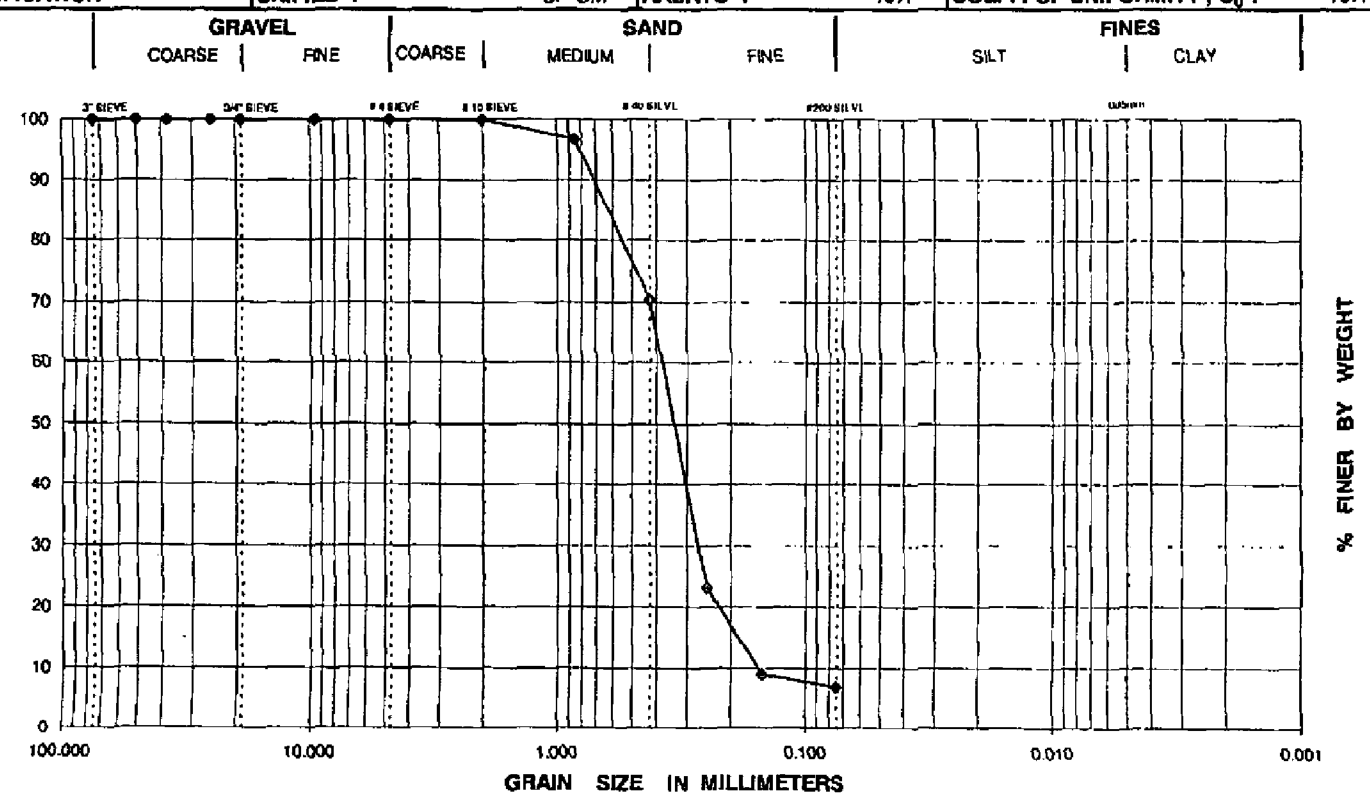
QORE PROPERTY SCIENCES

PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1.8/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N, SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	75'-77'	SAMPLE NO. :	18	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown sand with silt.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	7
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SP-SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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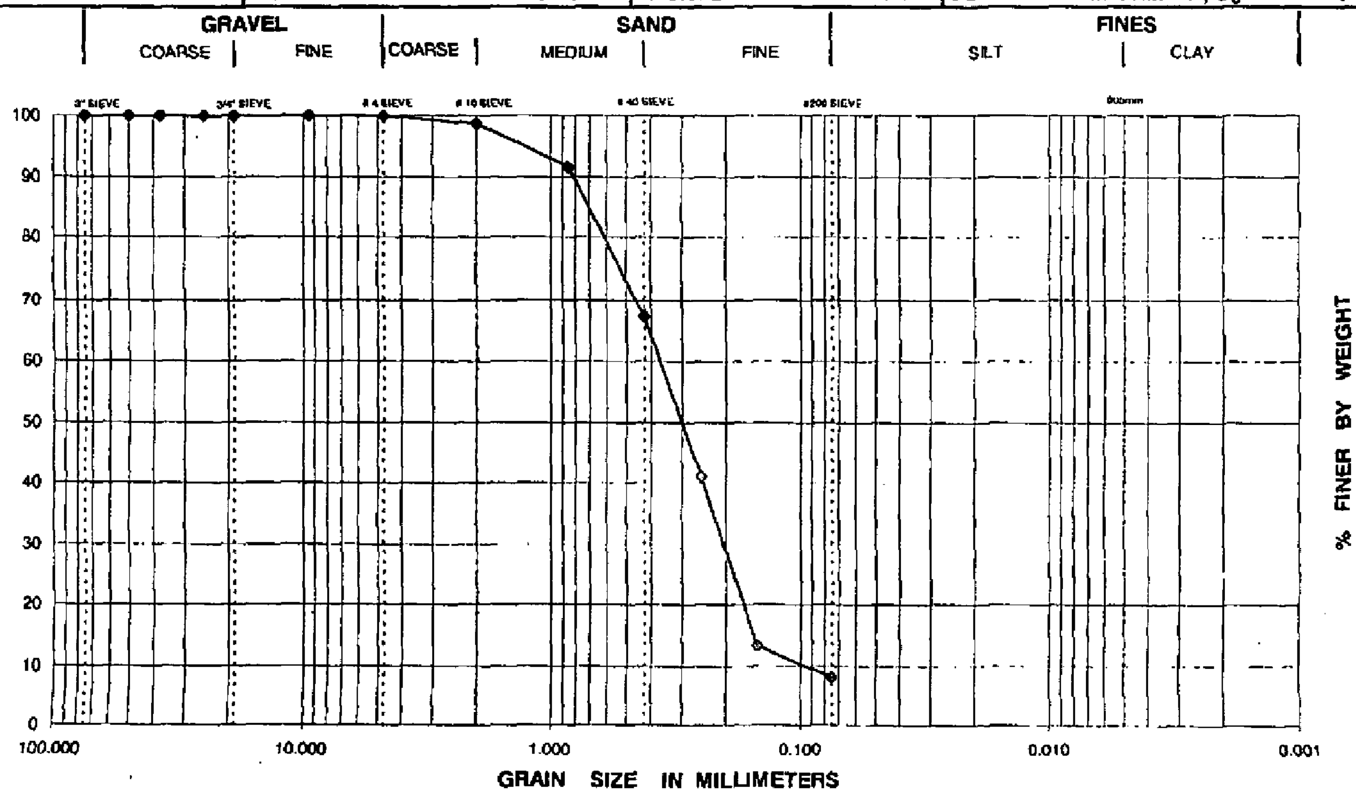
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,9/17/03

JOB NAME :	*Saltstone Vault # 2,Task 8*				CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO. :	*24150-H*	REPORT NO. :	-	DATE :	*6/23/05*	REVIEWED BY :	
BORING / PIT NO. :	*ZV2B2*	DEPTH / ELEV. :	*80'-82'*	SAMPLE NO. :	*19*	SAMPLE TYPE :	*SS*
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	*Yellowish brown sand with silt.*					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	*8*
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	*SP-SM*	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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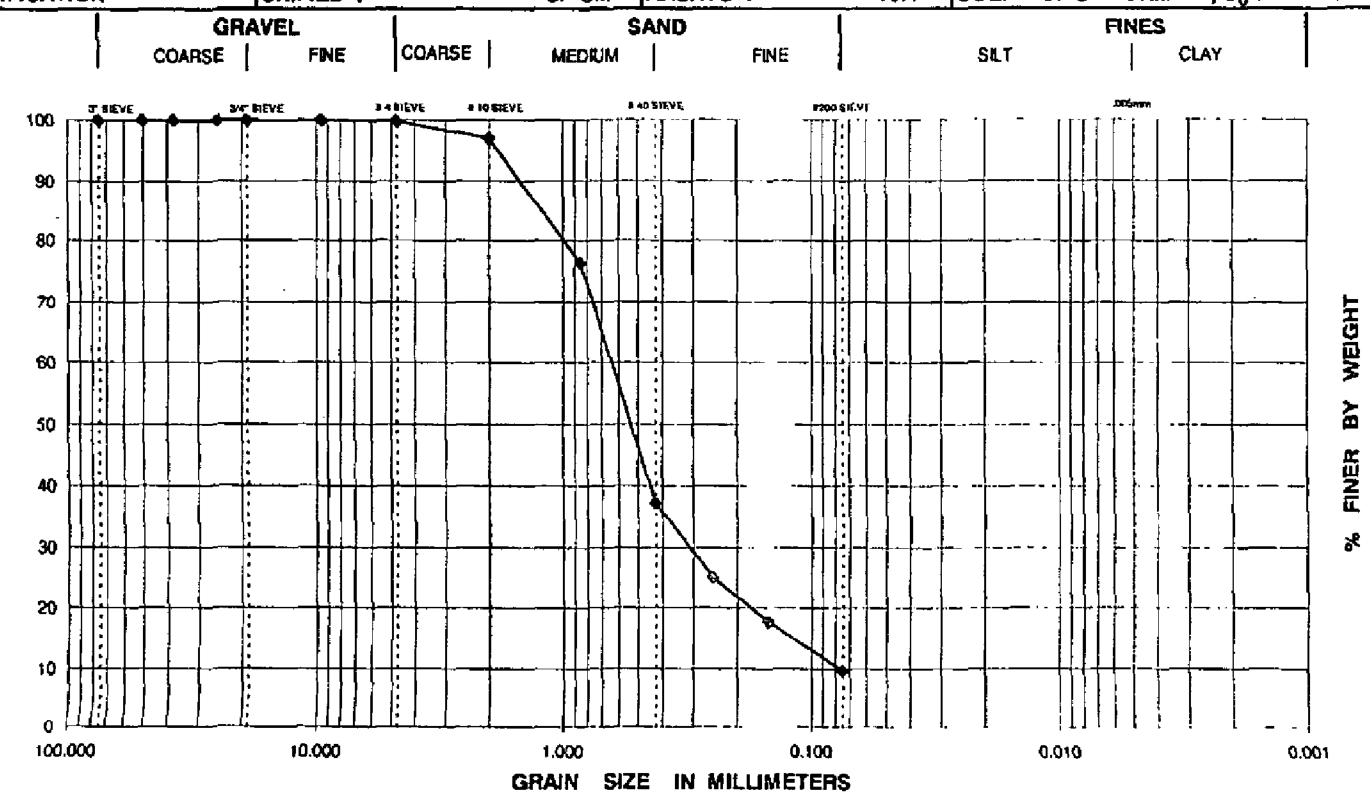
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER **ASTM D422 -63**

REV1.9/17/03

AAR AASHTO R18

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	85'-87'	SAMPLE NO. :	20	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown sand with silt.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	10
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SP-SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



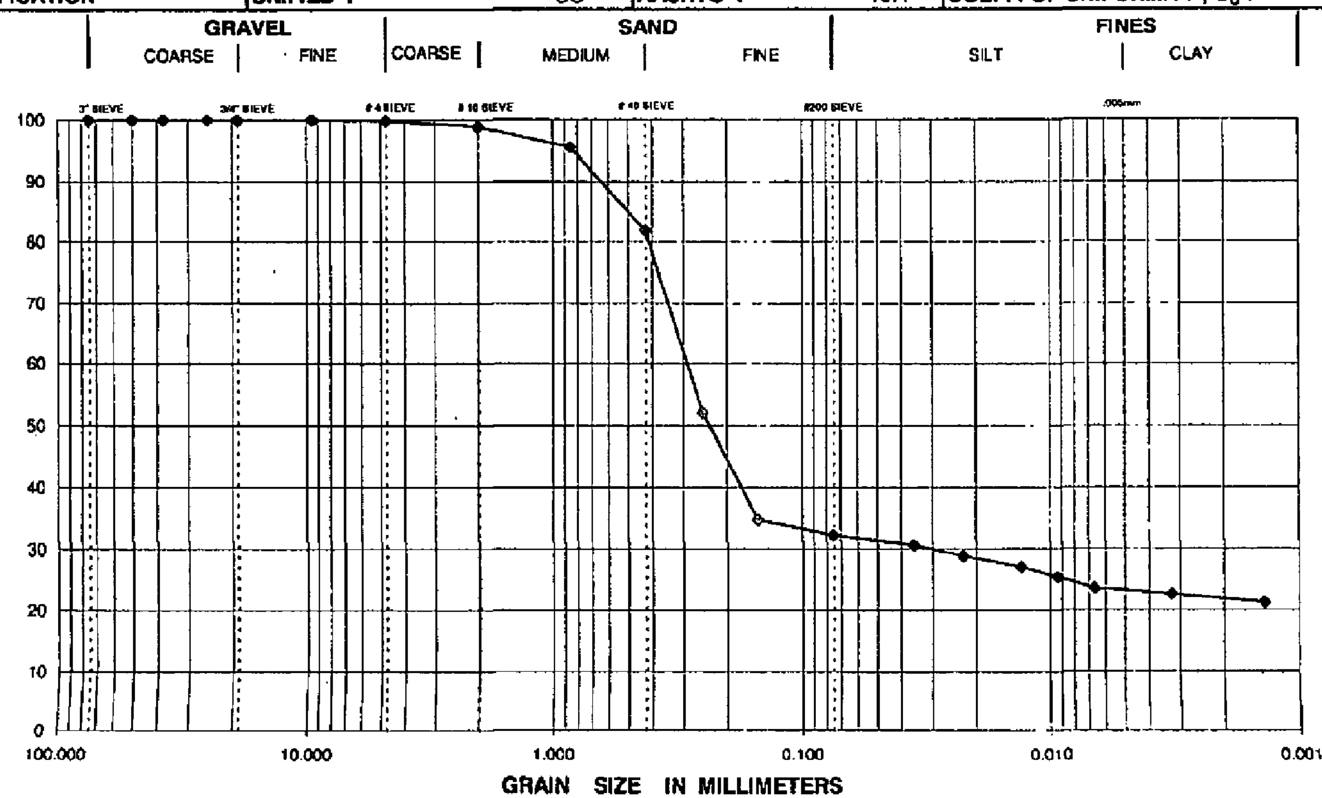
QORE PROPERTY SCIENCES

PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV 1,01/7/03

JOB NAME :	*Saltstone Vault # 2*			*CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8*			
JOB NO. :	*24150-H*	REPORT NO. :	-	DATE :	*6/28/05*	REVIEWED BY :	
BORING / PIT NO. :	*ZV2B2*	DEPTH / ELEV. :	*90'-92'*	SAMPLE NO. :	*21*	SAMPLE TYPE :	*SS*
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	*Yellowish gray clayey sand.*					SP. GRAVITY, Gs :	*2.65*
LIQUID LIMIT, % :	*51*	PLASTICITY INDEX , % :	*39*	MOISTURE , % :	-	FINES , % :	*32*
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	*SC*	AASHTO :	*N/A*	COEFF. OF UNIFORMITY , C_u :	*N/A*



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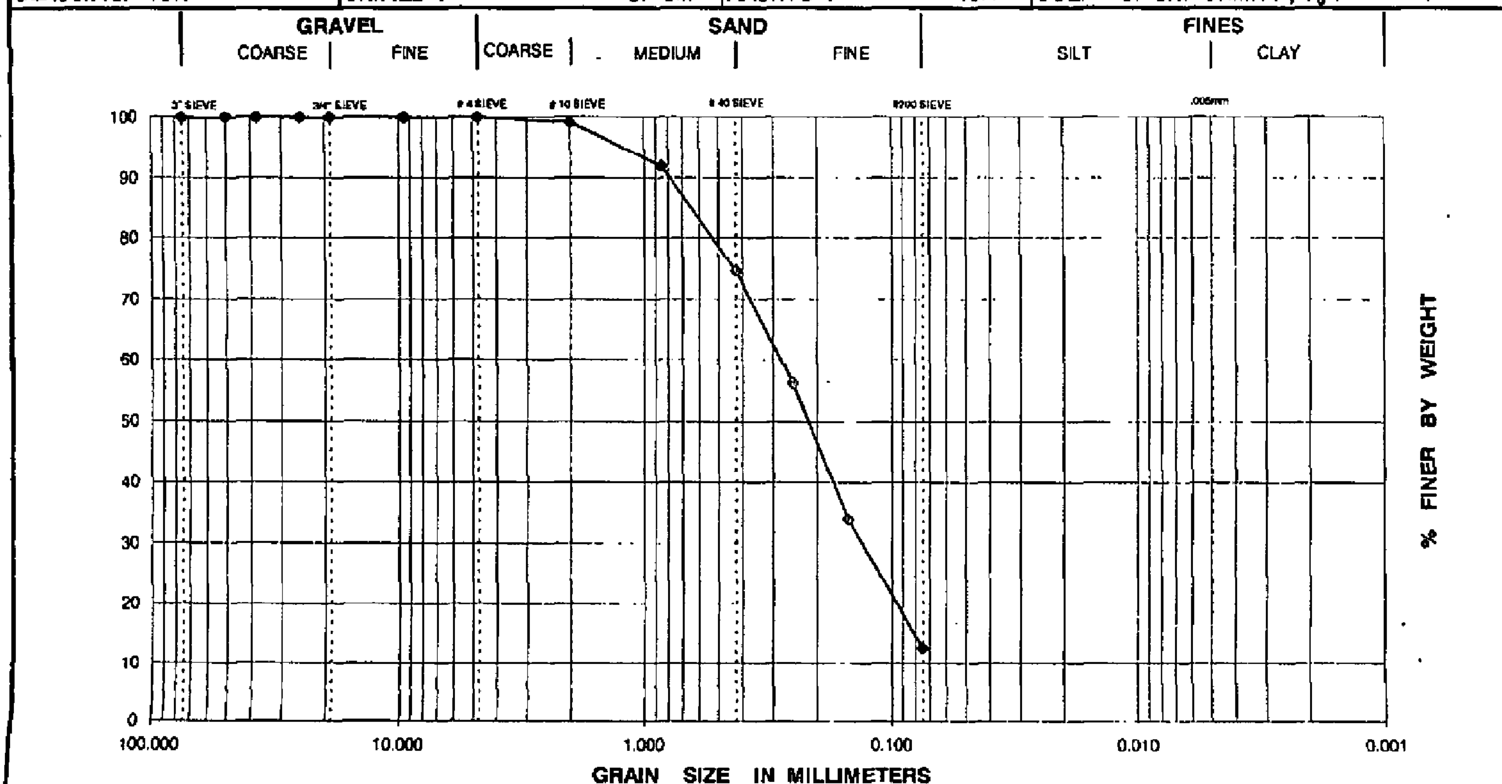
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,9/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	95'-97'	SAMPLE NO. :	22	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Yellowish brown sand with silt.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	13
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SP-SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



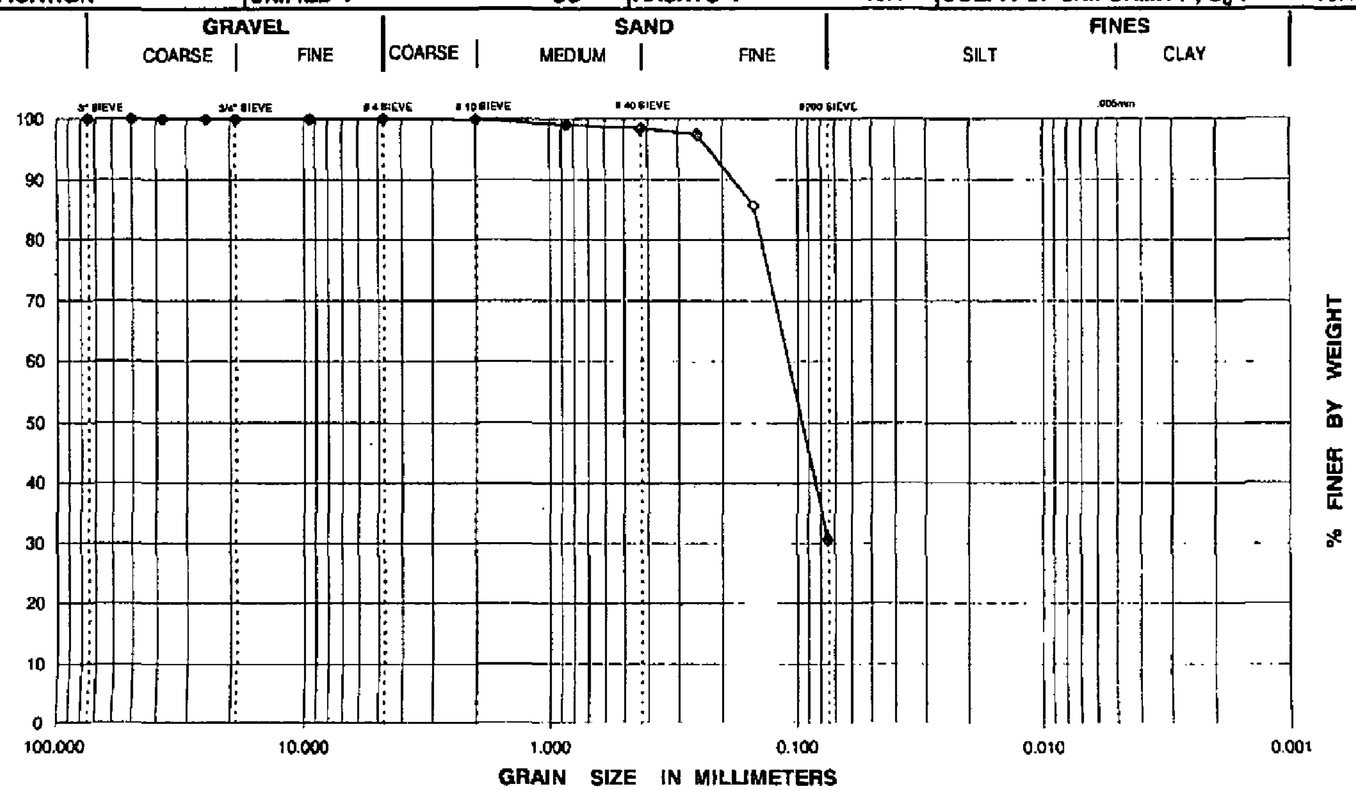
QORE PROPERTY SCIENCES

PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,9/17/03

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	100'-102'	SAMPLE NO. :	23	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Olive gray clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	31
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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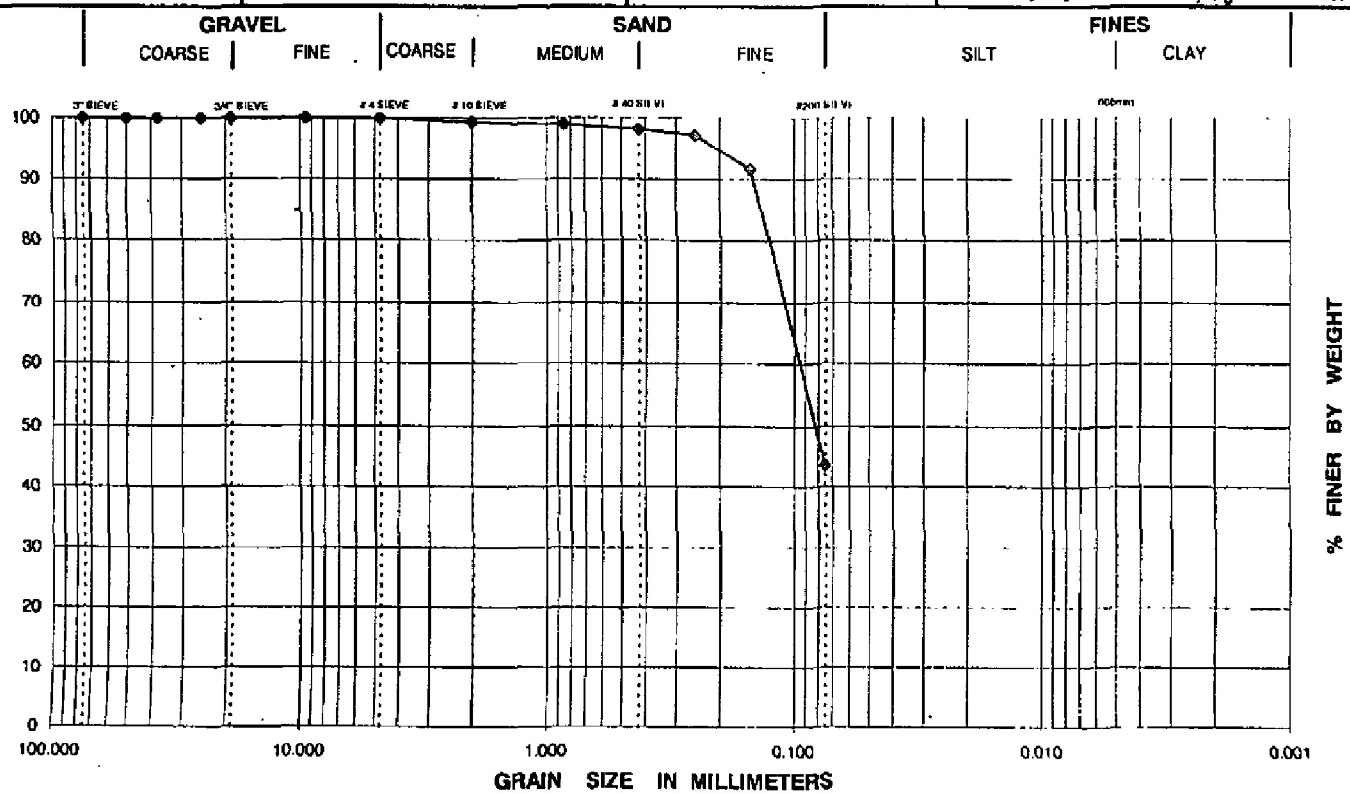
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,8/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	105'-107'	SAMPLE NO. :	24	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Olive gray clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	-	PLASTICITY INDEX , % :	-	MOISTURE , % :	-	FINES , % :	44
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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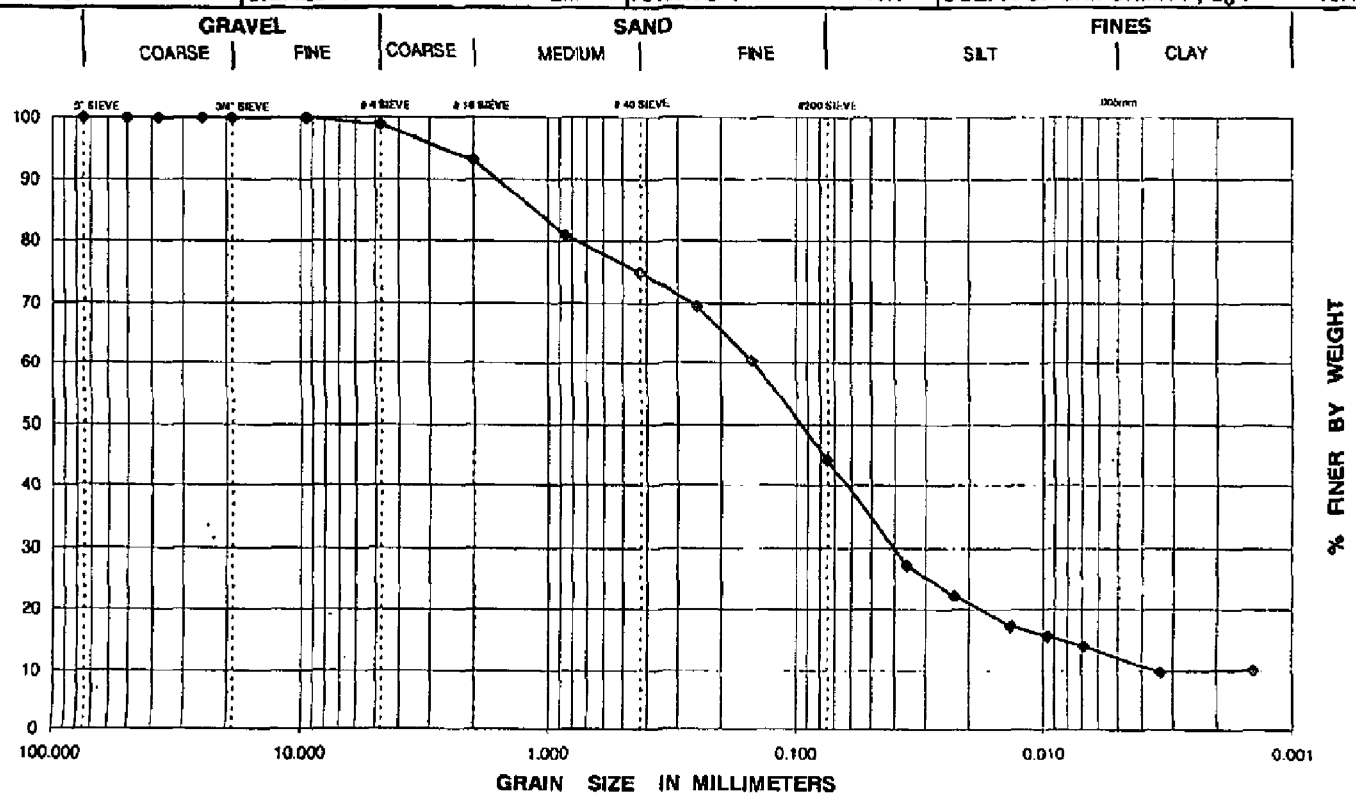
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

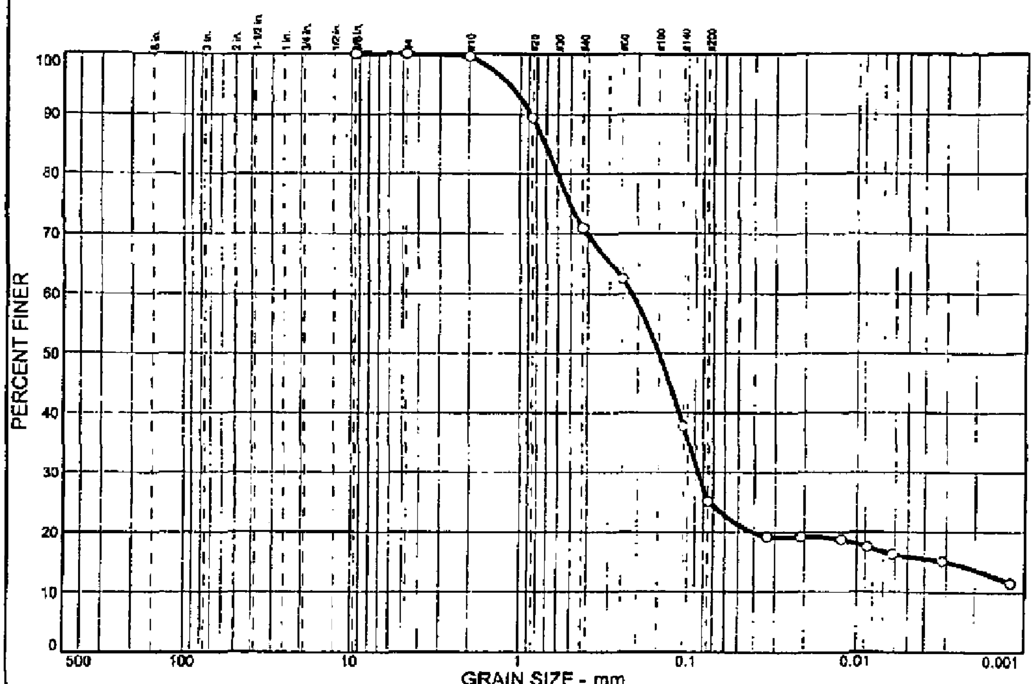
AAP AASHTO R18

REV1,9/17/03

JOB NAME :	Saltstone Vault # 2			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REVB			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/28/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2	DEPTH / ELEV. :	110'-112'	SAMPLE NO. :	25	SAMPLE TYPE :	SS
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Olive gray silty sand.					SP. GRAVITY, Gs :	2.65
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX , % :	N.P.	MOISTURE , % :	-	FINES , % :	44
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	74.9	9.3	15.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	99.6		
#20	89.4		
#40	71.0		
#60	62.5		
#140	37.8		
#200	25.1		

* (no specification provided)

Soil Description

Reddish Brown Clayey sand

Atterberg Limits

PL= 23 LL= 46 PI= 23

Coefficients

D_{85}= 0.715 D_{60}= 0.220 D_{50}= 0.150

D_{30}= 0.0869 D_{15}= 0.0030 D_{10}=

C_u= C_c=

Classification

USCS= SC AASHTO= A-2-7(1)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: ST-1 **Source of Sample:** ZV2B1U **Date:** 6/27/05

Location: ZV2B1U **Elev./Depth:** 6.0 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company

Project: Saltston Vault # 2Performance Assessment

Contract No. AB80188N Task Order No. 17

Project No: GTX G0847 **Figure**

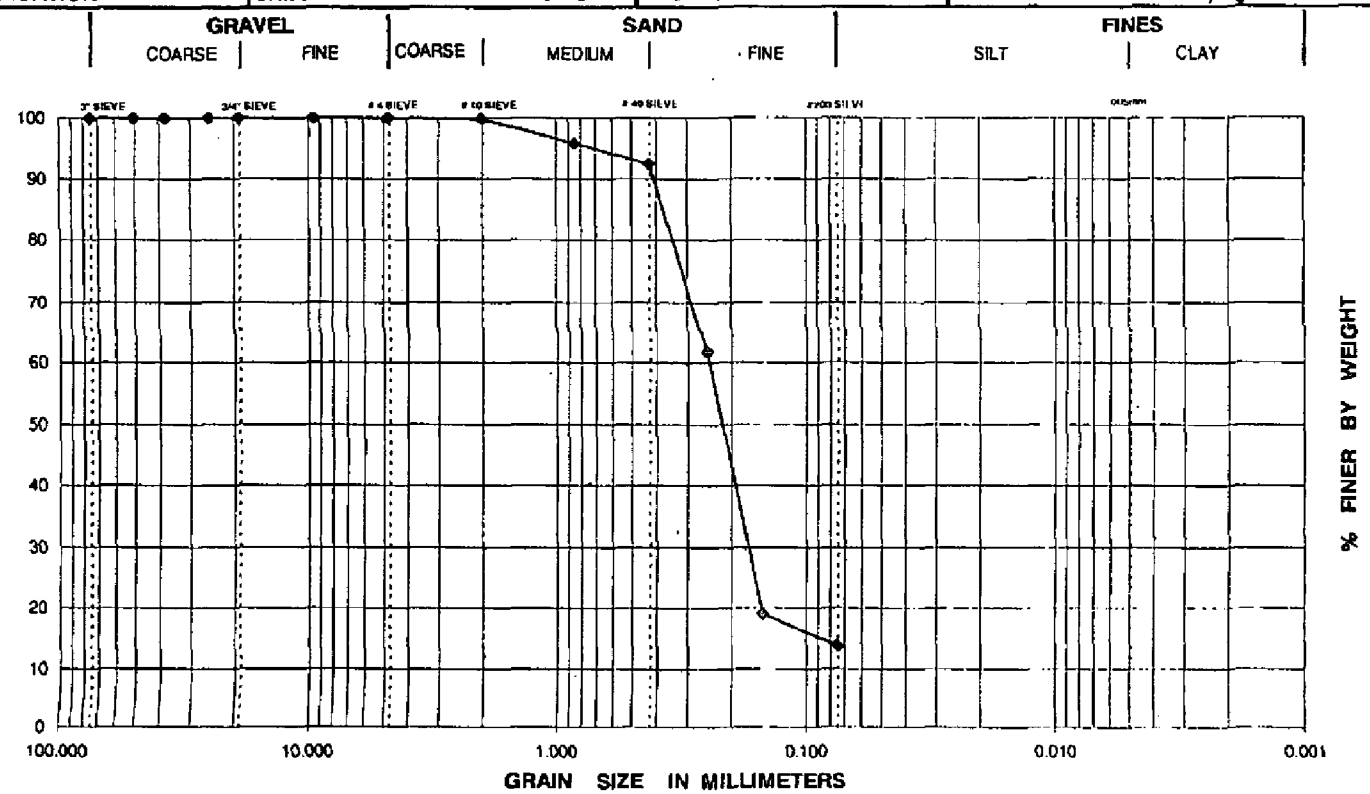
QORE

PARTICLE- SIZE DISTRIBUTION TEST REPORT

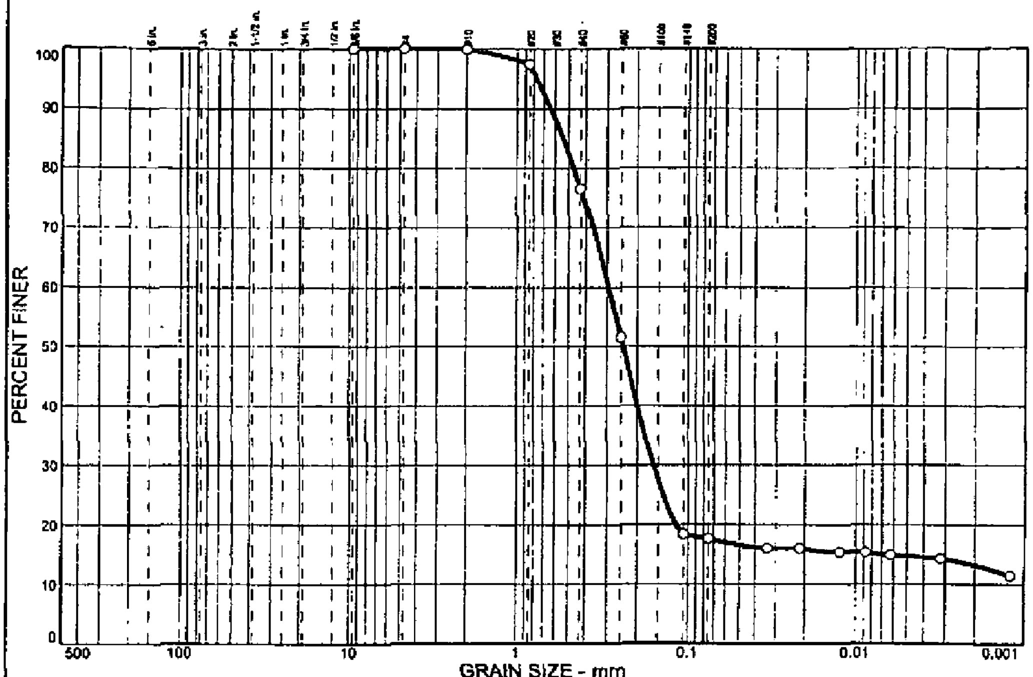
SIEVE AND HYDROMETER ASTM D422 -63

REV1.9/1/03

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	29'-31'	SAMPLE NO. :	ST2	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Reddish brown sand with silt.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX , % :	N.P.	MOISTURE , % :	-	FINES , % :	14
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SP-SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	82.4	2.9	14.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	99.9		
#20	97.4		
#40	76.5		
#60	51.5		
#140	18.3		
#200	17.6		

* (no specification provided)

Soil Description

Red Brown Clayey sand

Atterberg Limits

PL= 26 LL= 52 PI= 26

Coefficients

D_{85}= 0.540 D_{60}= 0.296 D_{50}= 0.243
D_{30}= 0.160 D_{15}= 0.0067 D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-7(0)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: ST-3 **Source of Sample:** ZV2B1U **Date:** 6/27/05
Location: ZV2B1U **Elev./Depth:** 31.0 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltston Vault # 2Performance Assessment
Contract No. AB80188N Task Order No. 17
Project No: GTX G0847 **Figure**

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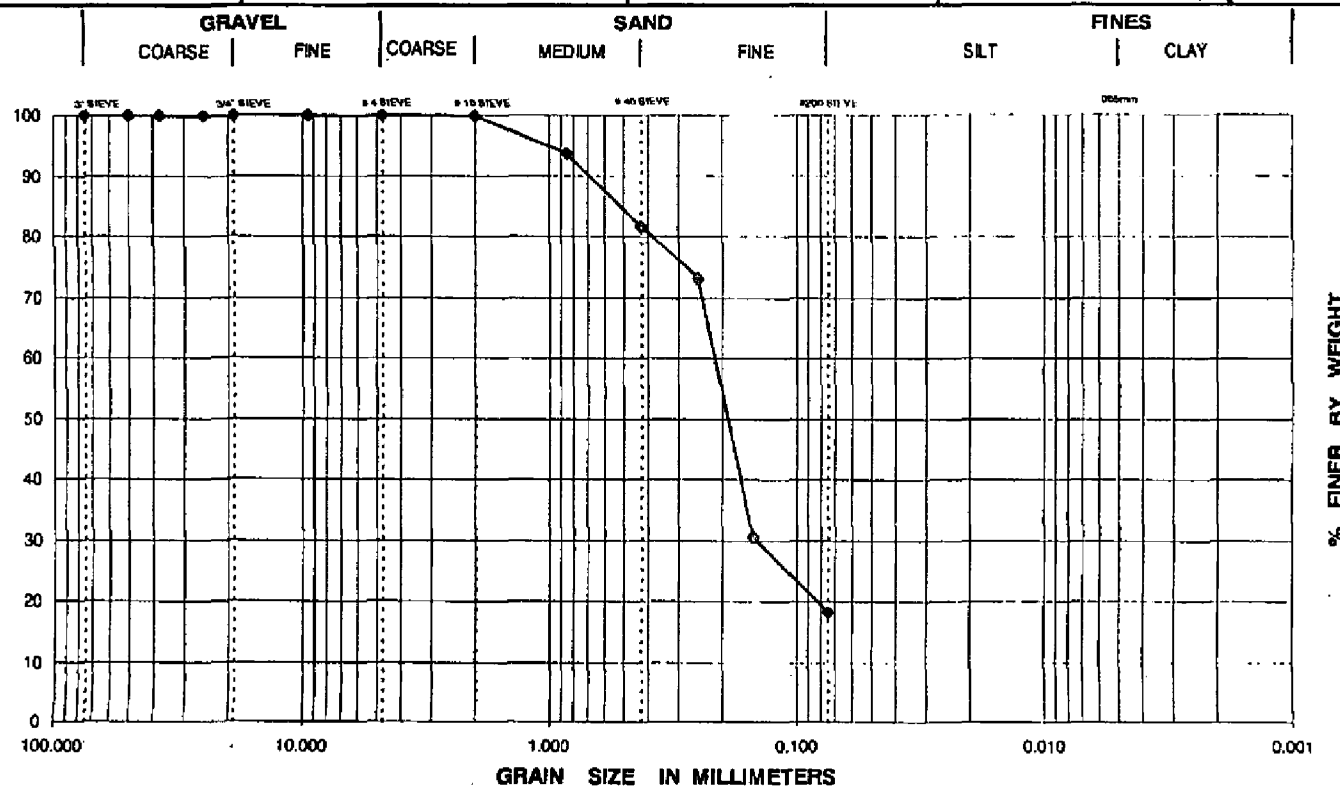
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

AAR AASHTO R18

REV1,9/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N, SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	45'-47'	SAMPLE NO. :	ST4	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Reddish brown silty sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX, % :	N.P.	MOISTURE , % :	-	FINES , % :	18
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



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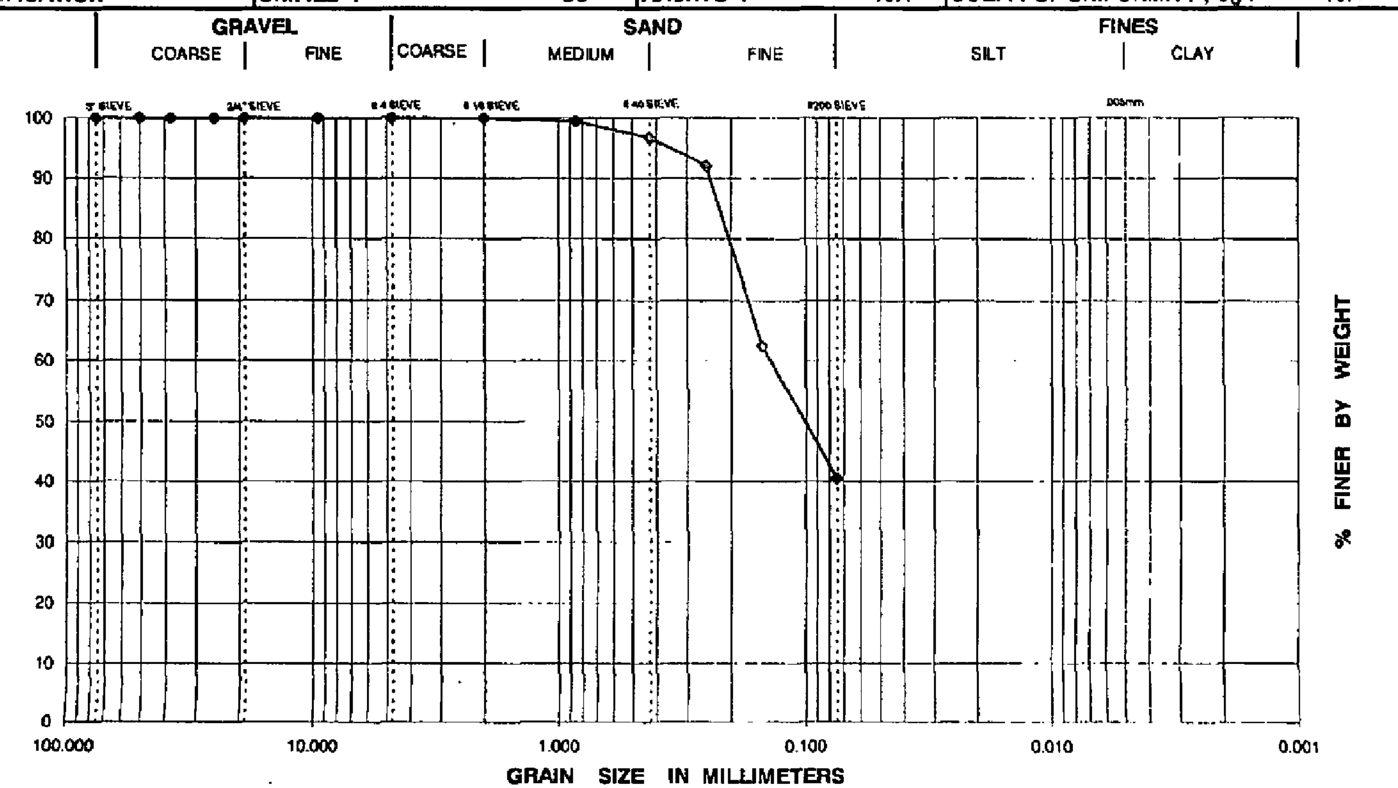
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,8/1/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N, SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	62'-64'	SAMPLE NO. :	ST5	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Tan gray clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	53	PLASTICITY INDEX , % :	36	MOISTURE , % :	-	FINES , % :	41
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



QORE PROPERTY SCIENCES

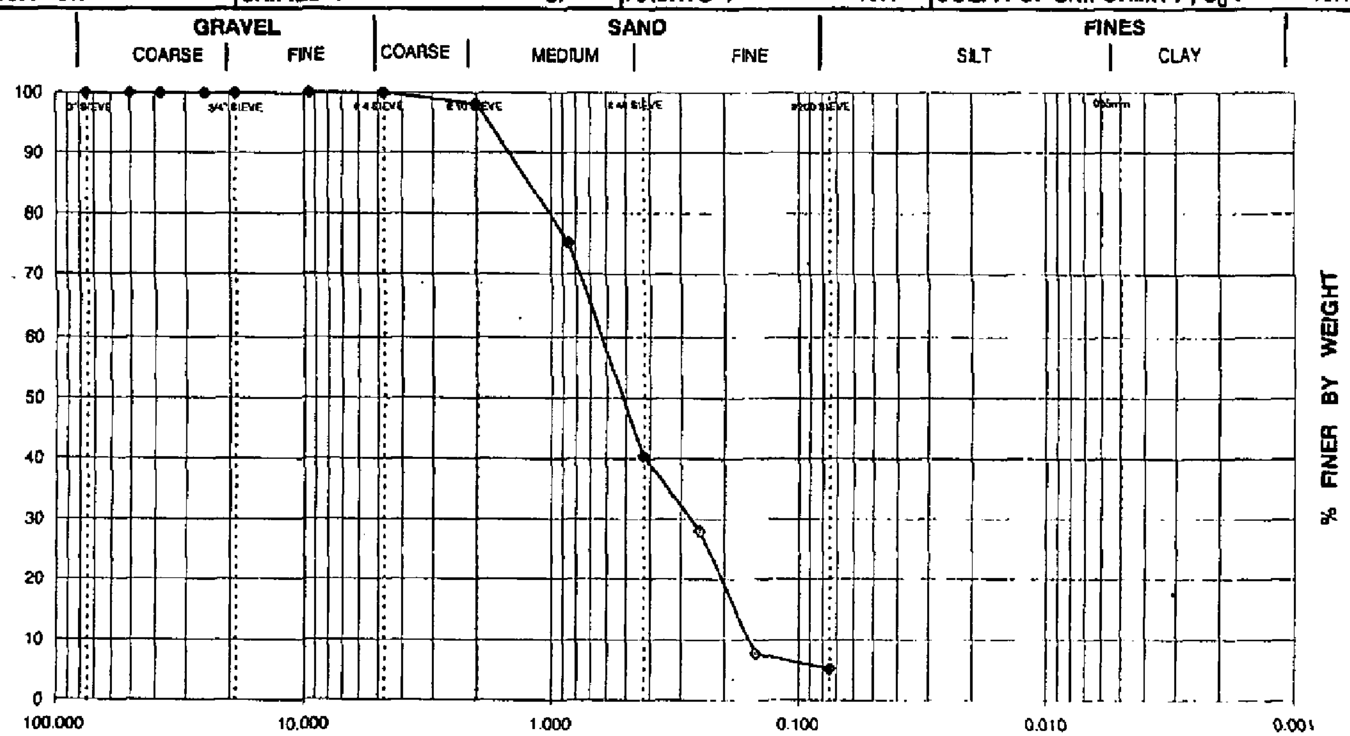
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

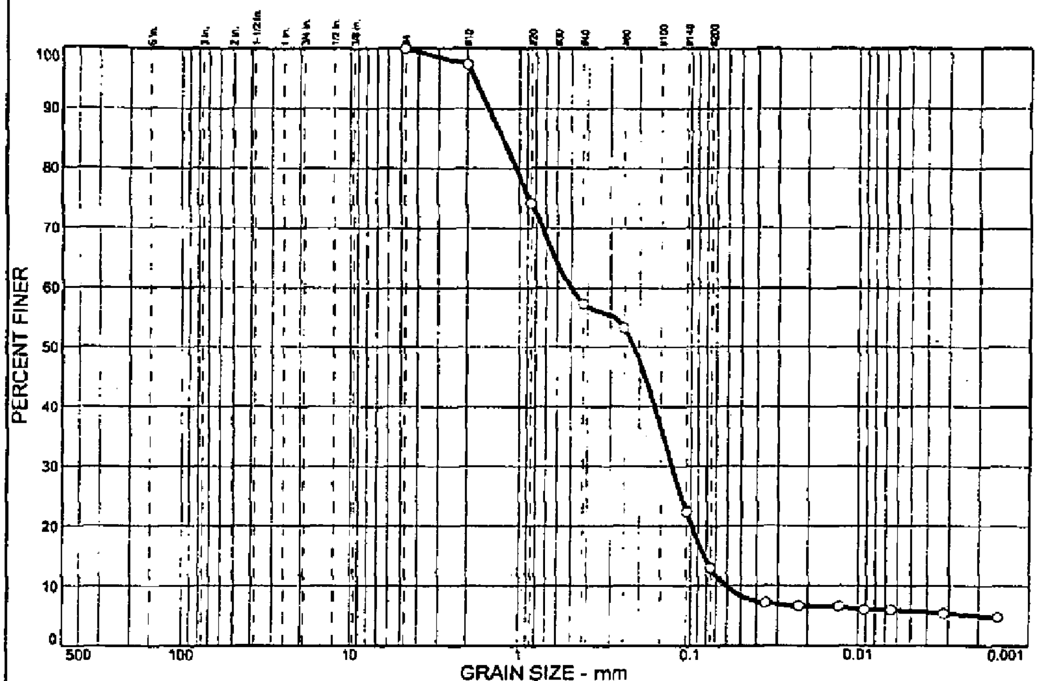
REV1,8/17/03

AASHTO R18

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	87'-89'	SAMPLE NO. :	ST7	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Gray sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX , % :	N.P.	MOISTURE , % :	-	FINES , % :	5
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_C :	N/A
CLASSIFICATION		UNIFIED :	SP	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_U :	N/A



Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	87.0	7.2	5.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	97.4		
#20	74.1		
#40	57.3		
#60	53.2		
#140	22.4		
#200	13.0		

* (no specification provided)

Soil Description

Yellow Brown Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 1.25 D_{60}= 0.505 D_{50}= 0.218
D_{30}= 0.129 D_{15}= 0.0824 D_{10}= 0.0615
C_u= 8.21 C_c= 0.54

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

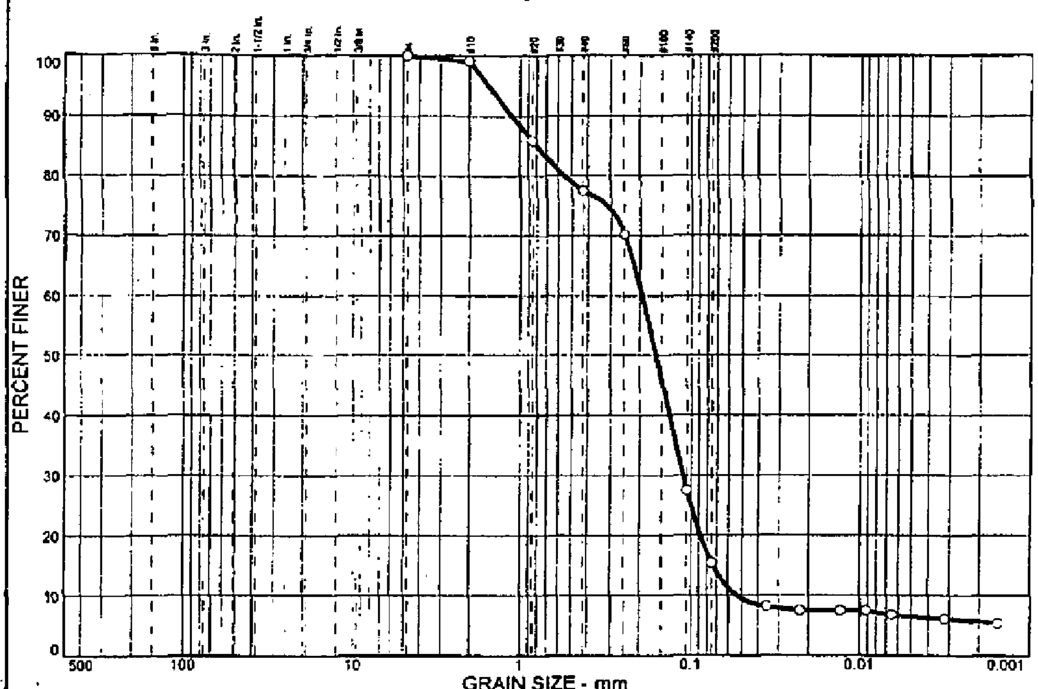
Tested by: HJ Reveiwed by: JW

Sample No.: PS-1 **Source of Sample:** ZB2B1U **Date:** 6/3/05
Location: ZB2B1U **Elev./Depth:** 104-104.5 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault # 2
Task Order Number 13 Contract Number AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	84.6	8.9	6.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.1		
#20	85.9		
#40	77.5		
#60	70.2		
#140	27.5		
#200	15.4		

* (no specification provided)

Soil Description

Yellow Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.799 D_{60}= 0.195 D_{50}= 0.161
D_{30}= 0.112 D_{15}= 0.0738 D_{10}= 0.0535
C_u= 3.64 C_c= 1.20

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: PS-1 **Source of Sample:** ZB2B1U **Date:** 6/3/05
Location: ZB2B1U **Elev./Depth:** 104.5-106 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault # 2
Task Order Number 13 Contract Number AB80188N
Project No: GTX G0821 **Figure**

Appendix C Laboratory Test Reports

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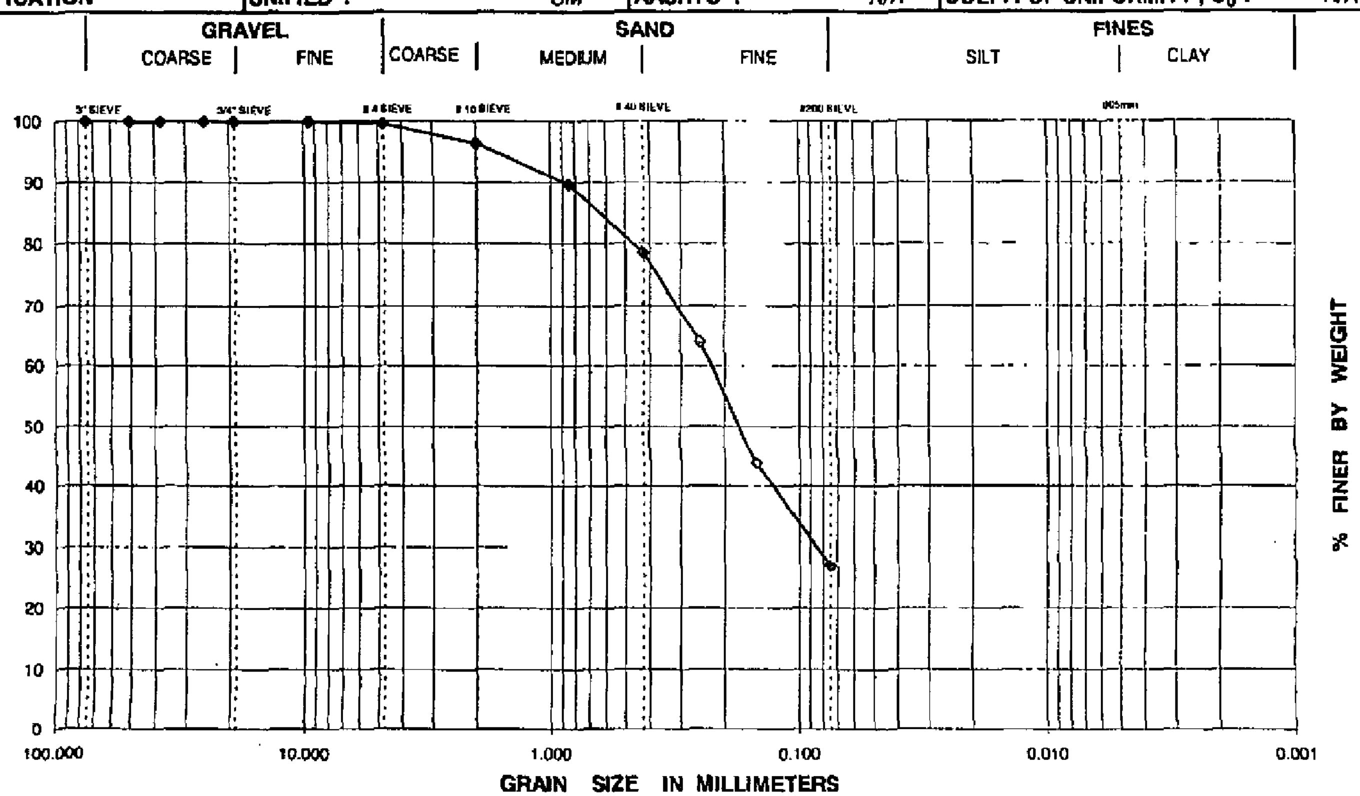
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,8/17/00

JOB NAME :	Saltstone Vault # 2, Task 8				CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	122'-124'	SAMPLE NO. :	PS3	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Gray silty sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX , % :	N.P.	MOISTURE , % :	-	FINES , % :	27
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_C :	N/A
CLASSIFICATION		UNIFIED :	SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_U :	N/A



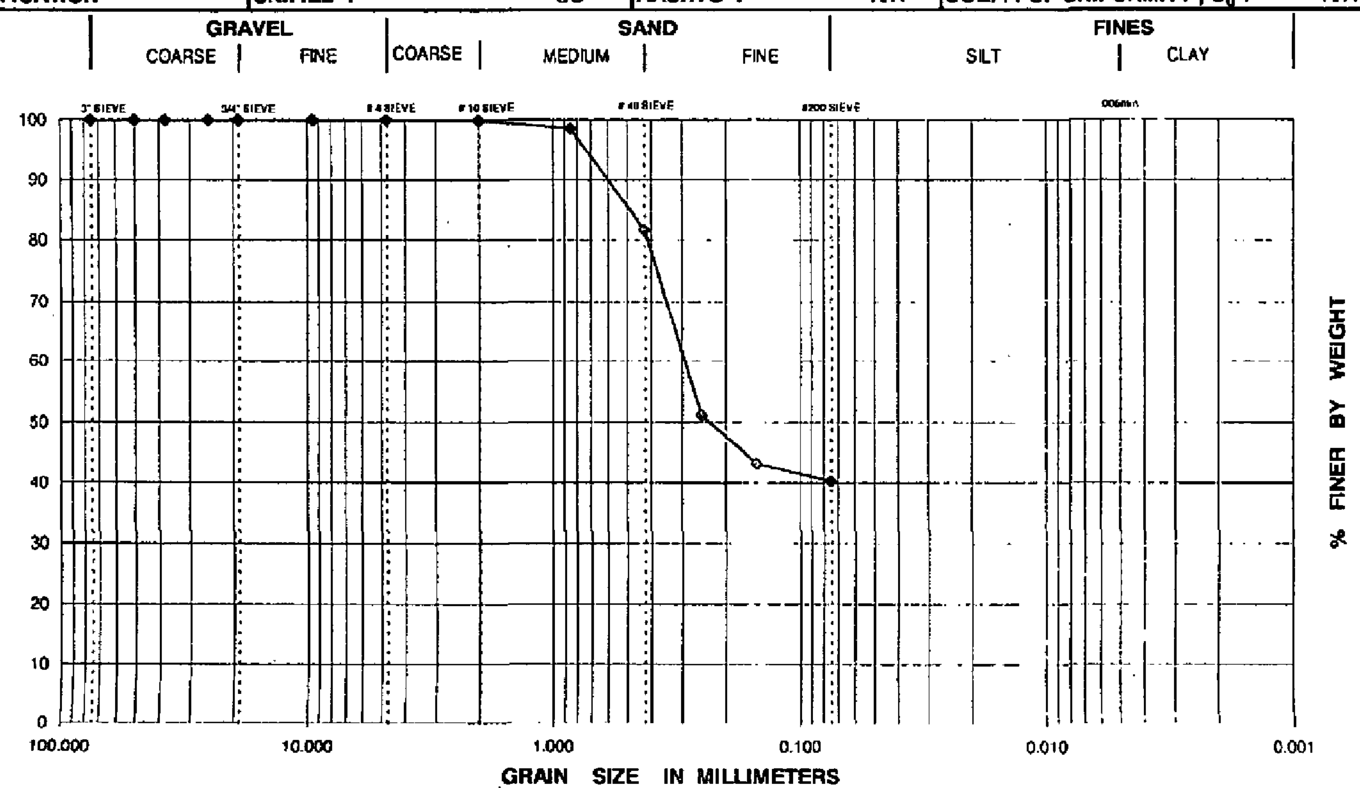
QORE

PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

REV1,9/17/03

JOB NAME :	Saltstone Vault # 2,Task 8	CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8					
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	6'-8'	SAMPLE NO. :	ST1	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Orange brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	43	PLASTICITY INDEX , % :	24	MOISTURE , % :	-	FINES , % :	40
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



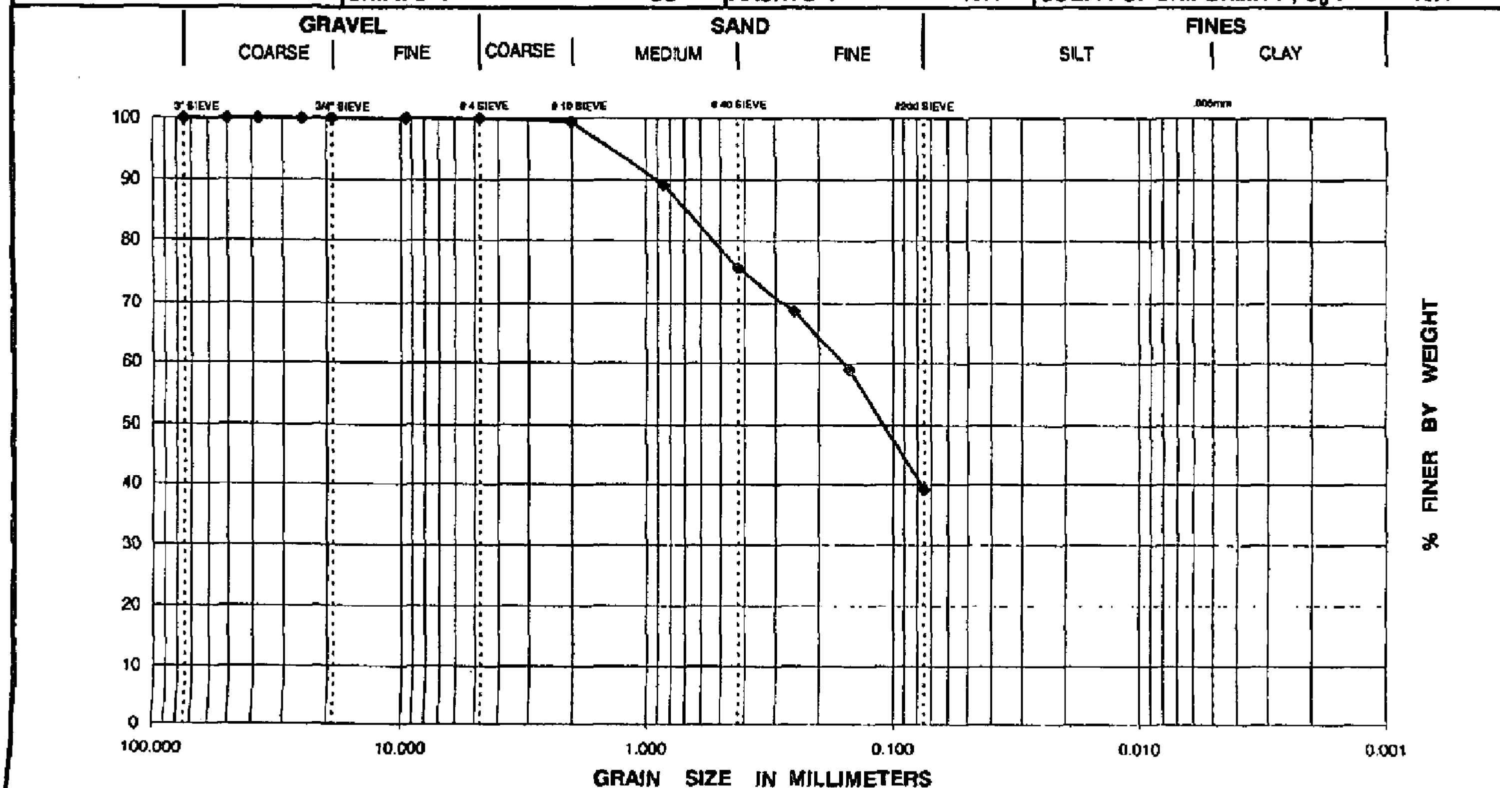
QORE PROPERTY SCIENCES

PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER **ASTM D422 -63**

REV1,8/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N, SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/28/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	14'-16'	SAMPLE NO. :	ST2	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Red brown and purple clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	42	PLASTICITY INDEX, % :	23	MOISTURE, % :	-	FINES, % :	39
D10, MM :	N/A	D30, MM :	N/A	D60, MM :	N/A	COEFF. OF CURVATURE, C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY, C_u :	N/A



QORE

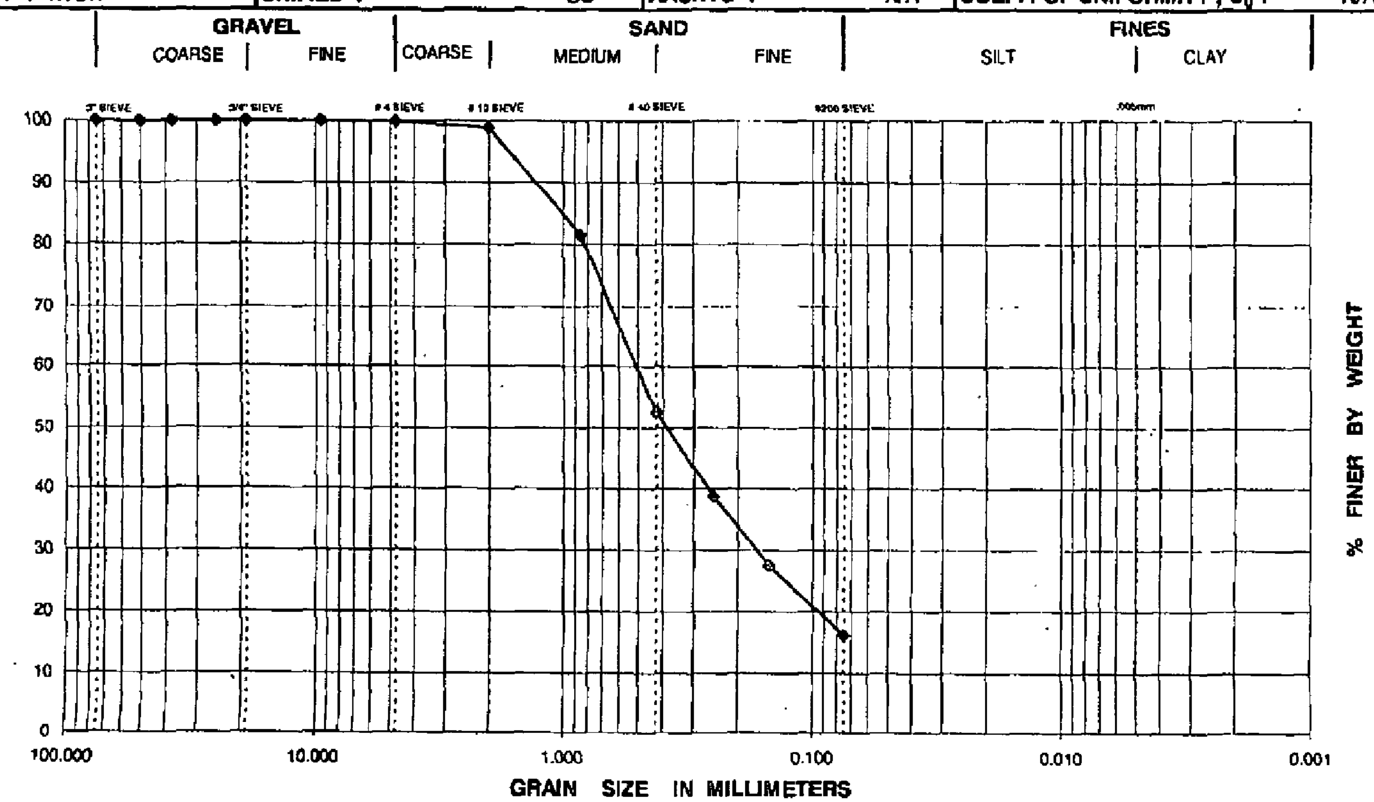
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

AAP

REV1 8/17/03

JOB NAME :	Saltstone Vault # 2, Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	28'-30'	SAMPLE NO. :	ST3	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Orange brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	38	PLASTICITY INDEX , % :	20	MOISTURE , % :	-	FINES , % :	16
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



QORE PROPERTY SCIENCES

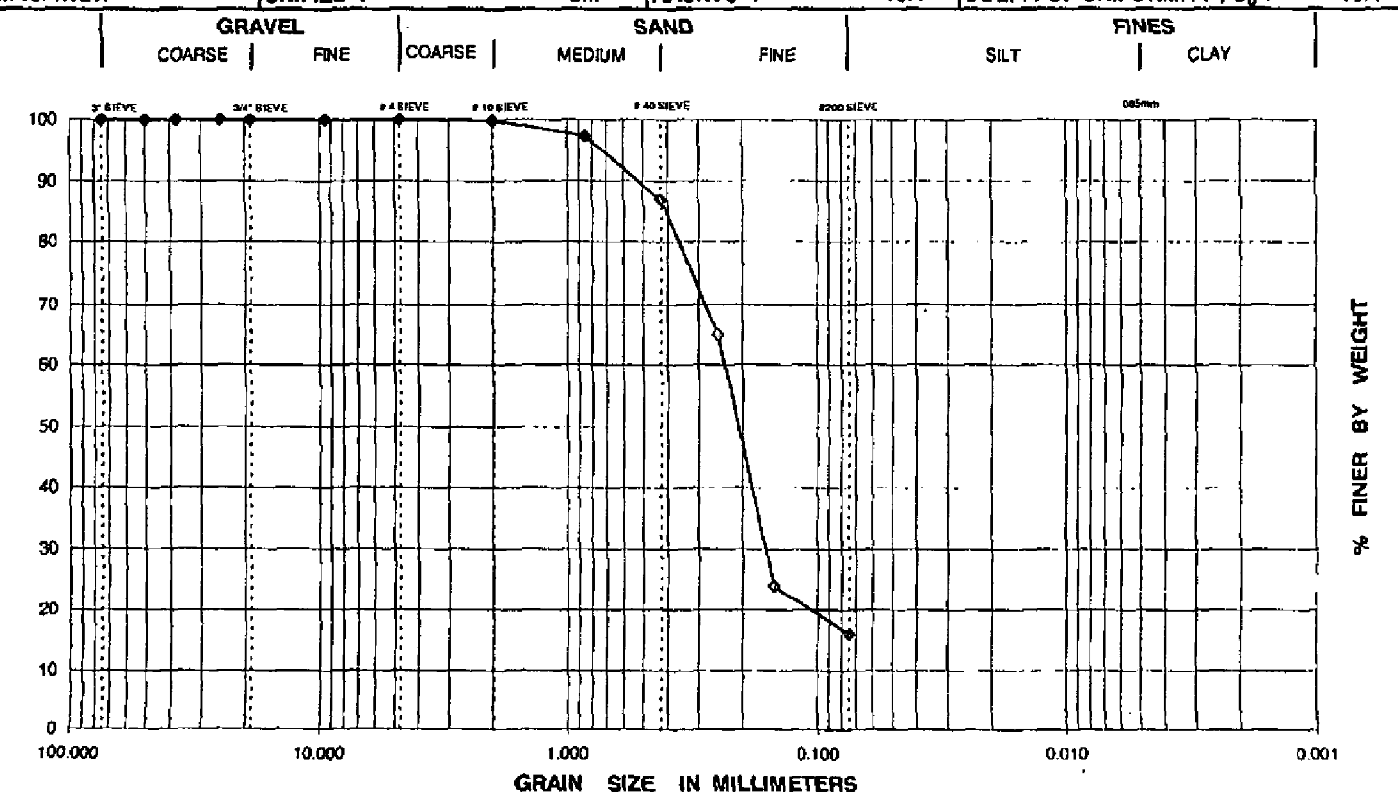
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

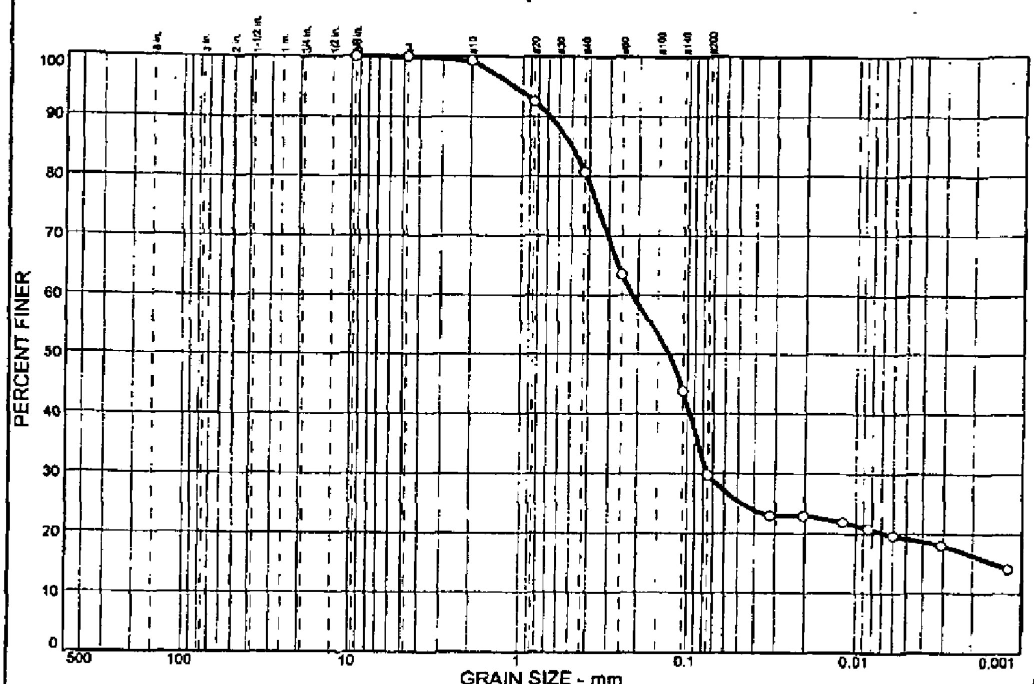
REV 1, 9/17/03

AASHTO R18

JOB NAME :	Saltstone Vault # 2, Task 8				CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	34'-36'	SAMPLE NO. :	ST4	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Tannish yellow silty sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX , % :	N.P.	MOISTURE , % :	-	FINES , % :	16
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SM	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	70.2	10.6	19.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	99.3		
#20	92.5		
#40	80.7		
#60	63.5		
#140	43.8		
#200	29.8		

* (no specification provided)

Soil Description

Reddish Brown Clayey sand

Atterberg Limits

PL= 19 LL= 44 PI= 25

Coefficients

D_{85}= 0.505 D_{60}= 0.216 D_{50}= 0.130
D_{30}= 0.0754 D_{15}= 0.0015 D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-7(2)

Remarks

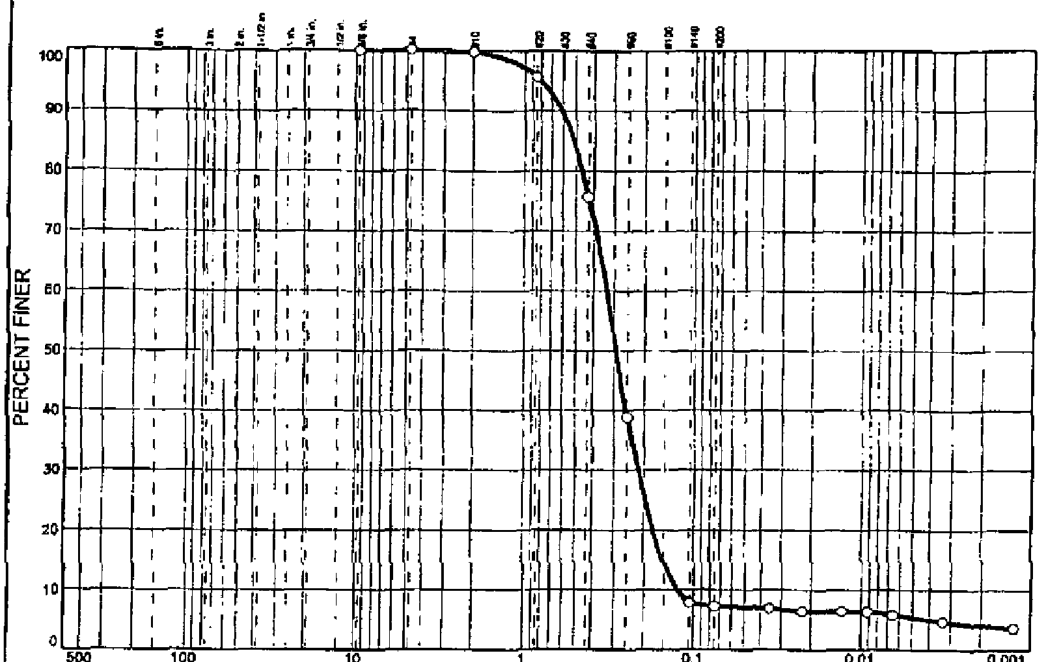
Tested by: HJ Reviewed by: JW

Sample No.: ST-1 **Source of Sample:** ZV2B3U **Date:** 6/26/05
Location: ZV2B3U **Elev./Depth:** 10.0 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltston Vault # 2Performance Assessment
Contract No. AB80188N Task Order No. 17
Project No: GTX G0847 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	92.8	1.9	5.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	99.8		
#20	95.6		
#40	75.7		
#60	38.8		
#140	7.8		
#200	7.2		

* (no specification provided)

Soil Description

Purple & Tan Brown Poorly graded sand with silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.519 D_{60}= 0.336 D_{50}= 0.293
D_{30}= 0.216 D_{15}= 0.153 D_{10}= 0.125
C_u= 2.68 C_c= 1.11

Classification

USCS= SP-SM AASHTO= A-3

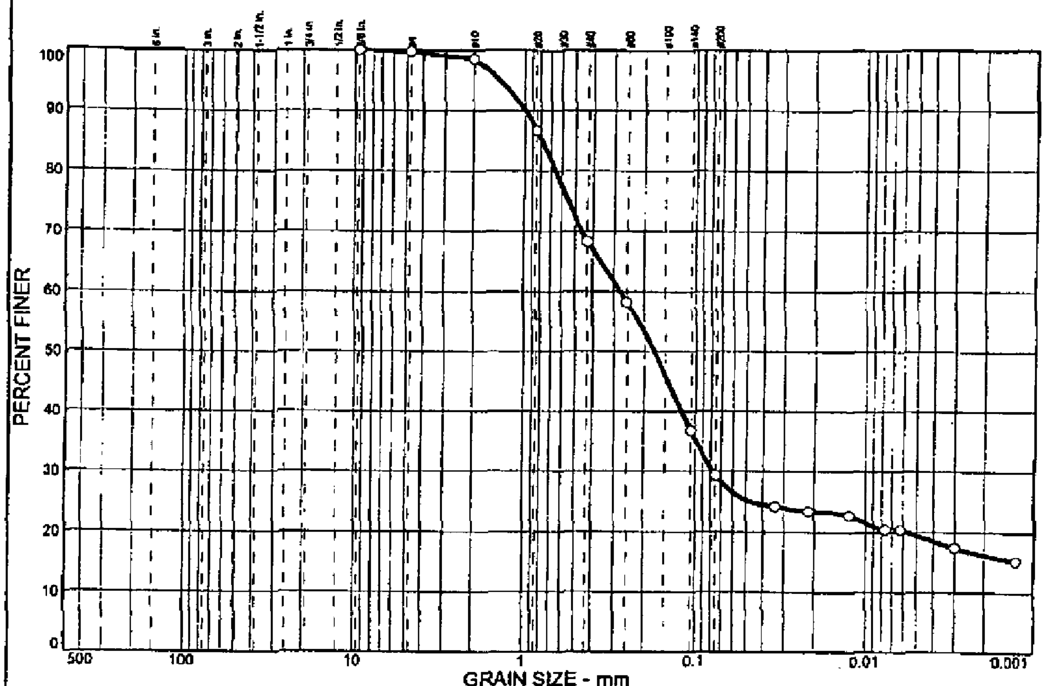
Remarks

Tested by: HJ Reviewed by: JW

Sample No.: ST-3 **Source of Sample:** ZV2B3U **Date:** 6/26/05
Location: ZV2B3U **Elev./Depth:** 29.0 Ft.

GeoTesting Express Inc.	**Client:** Westinghouse Savannah River Company **Project:** Saltston Vault # 2Performance Assessment Contract No. AB80188N Task Order No. 17 **Project No:** GTX G0847 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	70.5	9.9	19.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	99.8		
#10	98.6		
#20	86.7		
#40	68.3		
#60	58.2		
#140	36.7		
#200	29.3		

* (no specification provided)

Soil Description

Brown Clayey sand

Atterberg Limits

PL= 17 LL= 38 PI= 21

Coefficients

D_{85}= 0.792 D_{60}= 0.274 D_{50}= 0.176
D_{30}= 0.0781 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-6(2)

Remarks

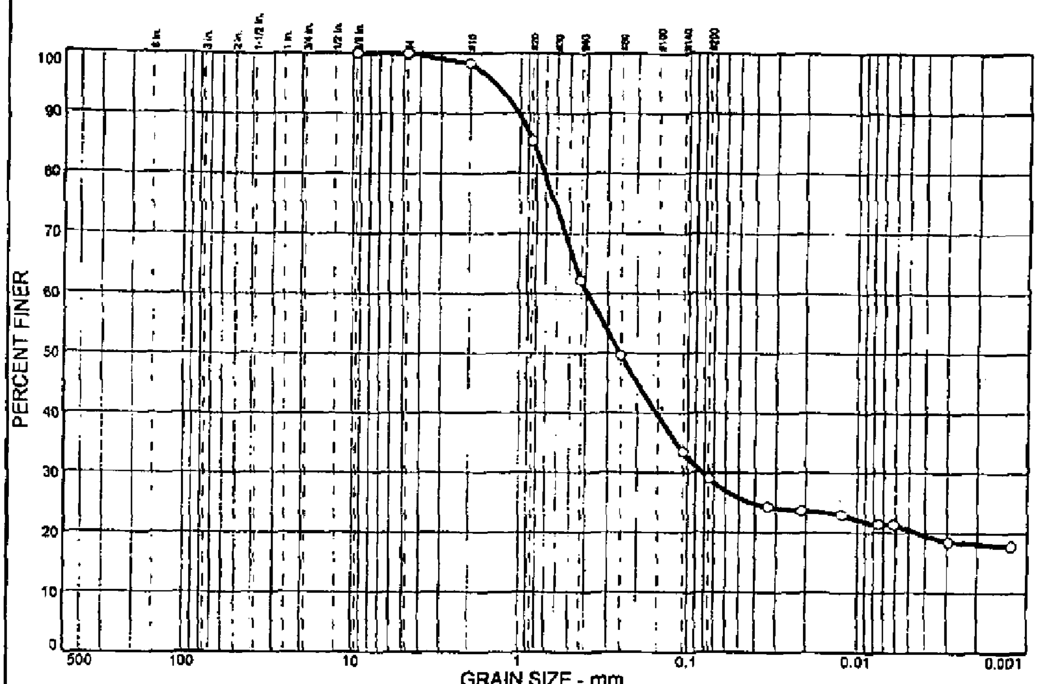
Tested by: JW Reviewed by: HJ

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/26/05
Location: TP-1 Composite **Elev./Depth:** 2-12 ft

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	71.1	8.4	20.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	98.4		
#20	85.4		
#40	62.3		
#60	49.6		
#140	33.5		
#200	28.9		

* (no specification provided)

Soil Description

Brown Clayey sand

Atterberg Limits

PL= 20 LL= 39 PI= 19

Coefficients

D_{85}= 0.838 D_{60}= 0.392 D_{50}= 0.255
D_{30}= 0.0825 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-6(1)

Remarks

Tested by: JW Reviewed by: HJ

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/26/05
Location: TP-2 Composite **Elev./Depth:** 4-12 ft

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

QORE PROPERTY SCIENCES

WATER CONTENT
(ASTM D 2216)

Rev. 122002

AAP AASHTO R18

JOB NAME : Saltstone Vault # 2,Task 8

CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8

JOB NO. : | REPORT NO. : | DATE : | REVIEWED BY :

SAMPLE TYPE	BORING/ PIT NO.	DEPTH / ELEV.	SAMPLE NO.	PAN NO.	WEIGHT OF PAN (A) (GRAMS)	WEIGHT OF WET SOIL + PAN (B) (GRAMS)	WEIGHT OF OVEN DRIED SOIL + PAN (C) (GRAMS)	WEIGHT OF WATER (D) D=(B-C) (GRAMS)	WEIGHT OF OVEN DRIED SOIL (E) E=(C - A) (GRAMS)	WATER CONTENT (F) F=(D / E)*100 %	REMARKS
SS	B2	12'-14'	2	-	60.14	111.04	102.62	8.42	42.48	19.8	
		60'-62'	15	-	60.35	105.59	95.09	10.50	34.74	30.2	
		65'-67'	16	-	60.12	114.80	103.43	11.37	43.31	26.3	
		90'-92'	21	-	59.94	82.60	76.90	5.70	16.96	33.6	
		110'-112'	25	-	60.12	77.02	70.56	6.46	10.44	61.9	
-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	
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-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	-	-	

WATER CONTENT - IS DETERMINED BY DRYING SOIL IN OVEN AT 110°C+-5°C TO CONSTANT WEIGHT

GeoTesting
express

GTX TECHNICAL PROCEDURE T-03
UNIT WEIGHT OF SAMPLE

Project No.:	GTX G0847	Boring No.:	ZV2B1U
Lab No:	13046	Depth:	6.0 Ft.
Project Name:	Saltstone Vault # 2 Performance Assessment	Sample ID:	ST-1
Tested By:	HJ	Reviewed By:	JW
Date:	06/17/05	Date:	06/30/05

Total Sample Height, *inches*		Inside Diameter of Cut Tube, *inches*		Moisture Content		
1	5.57			Tare No.	A-6	
2	5.57	Top	2.846	Tare Weight	16.87	*grams*
3	5.57	Bottom	2.842	Wet Weight + Tare	111.19	*grams*
Average	5.57	Average	2.844	Dry Weight + Tare	99.93	*grams*
				Moisture Content	13.6	%

Total Weight of Soil + Tube Section	1117.14	*grams*
Weight of Clean, Dry Tube Section	0.00	*grams*
Wet Weight of Soil	2.46	*lbs*
Volume of Sample	0.020	ft^3

RESULT SUMMARY

Moisture Content	**13.6**	**%**
Wet Density	**120.3**	*pcf*
Dry Density	**105.9**	*pcf*
Specific Gravity	**2.63**	
Porosity	**0.35**	

Remarks: Task Order No. 17
Contract No. AB80188N

GeoTesting express

GTX TECHNICAL PROCEDURE T-03
UNIT WEIGHT OF SAMPLE

Project No.:	GTX G0847	Boring No.:	ZV2B1U
Lab No:	13047	Depth:	31.0 Ft.
Project Name:	Saltstone Vault # 2 Performance Assessment	Sample ID:	ST-3
Tested By:	HJ	Reviewed By:	JW
Date:	06/17/05	Date:	06/30/05

Total Sample Height, *inches*		Inside Diameter of Cut Tube, *inches*		Moisture Content		
1	4.41			Tare No.	A-39	
2	4.41	Top	2.821	Tare Weight	17.22	*grams*
3	4.41	Bottom	2.806	Wet Weight + Tare	113.10	*grams*
Average	4.41	Average	2.814	Dry Weight + Tare	99.17	*grams*
				Moisture Content	17.0	%

Total Weight of Soil + Tube Section	889.50	*grams*
Weight of Clean, Dry Tube Section	0.00	*grams*
Wet Weight of Soil	1.96	*lbs*
Volume of Sample	0.016	ft^3

RESULT SUMMARY

Moisture Content	**17.0**	%
Wet Density	**123.6**	*pcf*
Dry Density	**105.6**	*pcf*
Specific Gravity	**2.65**	
Porosity	**0.36**	

Remarks: Task Order No. 17
Contract No. AB80188N

GeoTesting express

GTX TECHNICAL PROCEDURE T-03
UNIT WEIGHT OF SAMPLE

Project No.:	GTX G0847	Boring No.:	ZV2B3U
Lab No:	13044	Depth:	10.0 Ft.
Project Name:	Saltstone Vault # 2 Performance Assessment	Sample ID:	ST-1
Tested By:	HJ	Reviewed By:	JW
Date:	06/17/05	Date:	06/30/05

Total Sample Height, *inches*		Inside Diameter of Cut Tube, *inches*		Moisture Content		
1	5.57			Tare No.	A-20	
2	5.57	Top	2.859	Tare Weight	16.57	*grams*
3	5.57	Bottom	2.847	Wet Weight + Tare	118.62	*grams*
Average	5.57	Average	2.853	Dry Weight + Tare	105.39	*grams*
				Moisture Content	14.9	%

Total Weight of Soil + Tube Section	1191.27	*grams*
Weight of Clean, Dry Tube Section	0.00	*grams*
Wet Weight of Soil	2.63	*lbs*
Volume of Sample	0.021	ft^3

RESULT SUMMARY

Moisture Content	**14.9**	**%**
Wet Density	**127.4**	***pcf***
Dry Density	**110.9**	***pcf***
Specific Gravity	**2.65**	
Porosity	**0.33**	

Remarks: Task Order 17

Contract No. AB80188N

GeoTesting express

GTX TECHNICAL PROCEDURE T-03
UNIT WEIGHT OF SAMPLE

Project No.:	GTX G0847	Boring No.:	ZV2B3U
Lab No:	13045	Depth:	29.0 Ft.
Project Name:	Saltstone Vault # 2 Performance Assessment	Sample ID:	ST-3
Tested By:	HJ	Reviewed By:	JW
Date:	06/17/05	Date:	06/30/05

Total Sample Height, *inches*		**Inside Diameter of Cut Tube, *inches***		**Moisture Content**		
1	5.57			Tare No.	A-26	
2	5.57	Top	2.815	Tare Weight	16.95	*grams*
3	5.57	Bottom	2.805	Wet Weight + Tare	115.28	*grams*
Average	5.57	Average	2.810	Dry Weight + Tare	105.04	*grams*
				Moisture Content	11.6	%

Total Weight of Soil + Tube Section	1053.96	*grams*
Weight of Clean, Dry Tube Section	0.00	*grams*
Wet Weight of Soil	2.32	*lbs*
Volume of Sample	0.020	ft^3

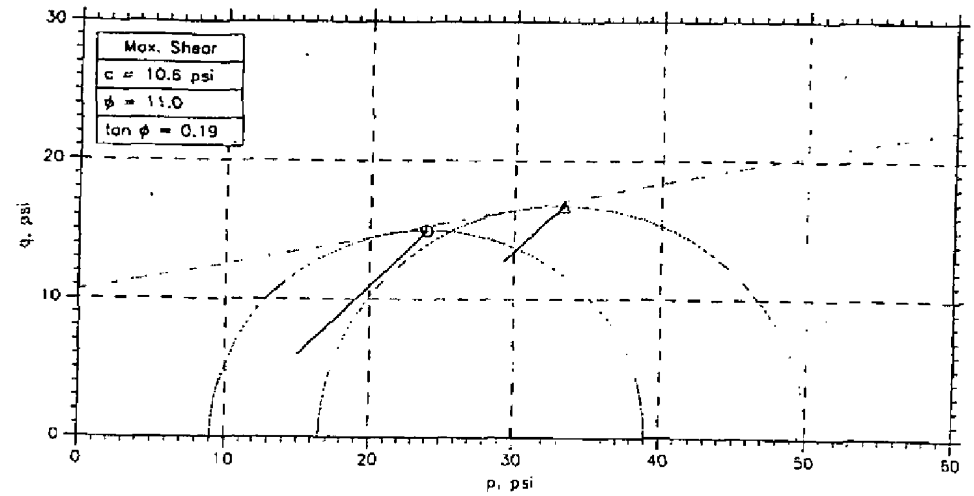
RESULT SUMMARY

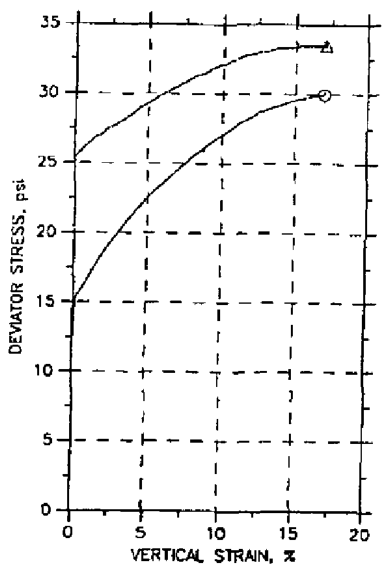
Moisture Content	**11.6**	**%**
Wet Density	**116.2**	***pcf***
Dry Density	**104.1**	***pcf***
Specific Gravity	**2.63**	
Porosity	**0.37**	

Remarks: Task Order No. 17

Contract No. AB80188N

CONSOLIDATED UNDRAINED TRIAXIAL TEST





	Symbol	⊙	△		
	Sample No.	PS-1	PS-1		
	Test No.	12981.1	12981.2		
	Depth	104.5-105	105-105.5		
Initial	Diameter, in	2.833	2.852		
	Height, in	5.543	5.545		
	Water Content, %	32.4	34.9		
	Dry Density, pcf	92.56	87.59		
	Saturation, %	108.0	103.3		
	Void Ratio	0.801	0.903		
Before Shear	Water Content, %	28.8	31.6		
	Dry Density, pcf	94.17	90.37		
	Saturation*, %	100.0	100.0		
	Void Ratio	0.77	0.845		
	Back Press., psi	36	44		
	Ver. Eff. Cons. Stress, psi	9.029	16.53		
	Shear Strength, psi	14.98	16.75		
	Strain at Failure, %	17	17.1		
	Strain Rate, %/min	0.1	0.1		
	B-Value	0.95	0.95		
	Measured Specific Gravity	2.67	2.67		
	Liquid Limit	---	---		
	Plastic Limit	---	---		

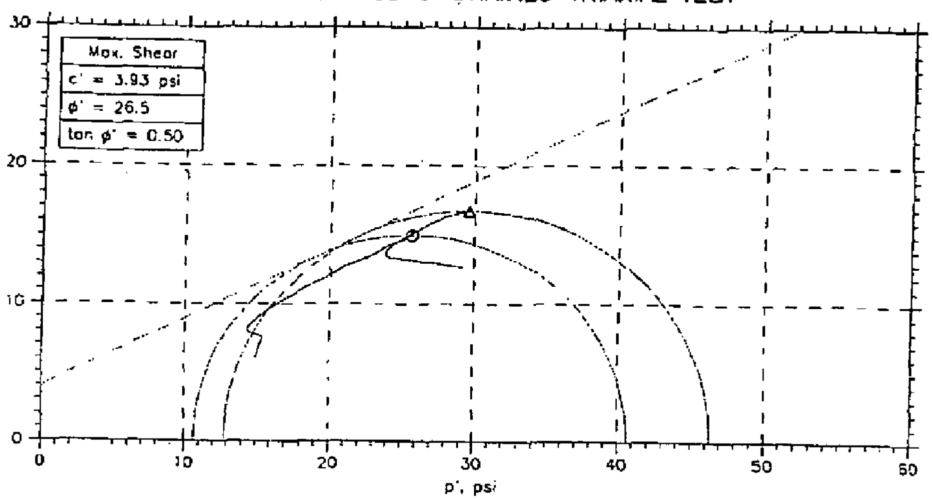
GeoTesting express the groundwork for success	Project: Saltstone Vault #2/Task13
	Location: ZB2B1U
	Project No.: GTX 50821
	Boring No.: ZB2B1U
	Sample Type: Shelby Tube
	Description: Yellowish Brown Clayey Sand
	Remarks: ASTM D4767/GTX-T07. Contract No: AB8018BN,Task 13

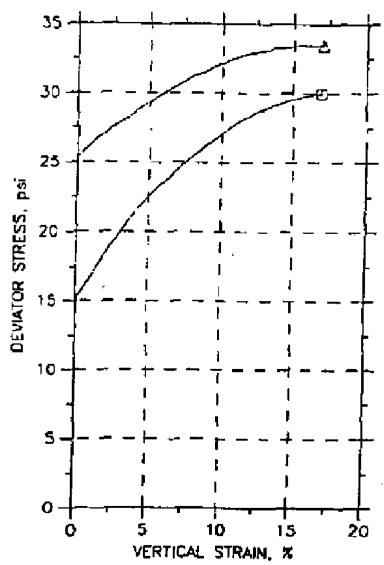
Thu, 23-JUN-2005 12:02:14

Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST





Symbol		⊙	Δ		
Sample No.		PS-1	PS-1		
Test No.		12981.1	12981.2		
Depth		104.5-105	105-105.5 ft		
Initial	Diameter, in	2.833	2.852		
	Height, in	5.543	5.545		
	Water Content, %	32.4	34.9		
	Dry Density, pcf	92.56	87.59		
	Saturation, %	108.0	103.3		
	Void Ratio	0.801	0.903		
Before Shear	Water Content, %	28.8	31.6		
	Dry Density, pcf	94.17	90.37		
	Saturation*, %	100.0	100.0		
	Void Ratio	0.77	0.845		
	Back Press., psi	36	44		
Ver. Eff. Cons. Stress, psi		9.029	18.53		
Shear Strength, psi		14.98	16.75		
Strain at Failure, %		17	17.1		
Strain Rate, %/min		0.1	0.1		
B-Value		0.95	0.95		
Measured Specific Gravity		2.67	2.67		
Liquid Limit		---	---		
Plastic Limit		---	---		

GeoTesting express the groundwork for success

Project: Saltstone Vault #2/Task13
Location: ZB2B1U
Project No.: GTX G0821
Boring No.: ZB2B1U
Sample Type: Shelby Tube
Description: Yellowish Brown Clayey Sand
Remarks: ASTM D4767/GTX-T07. Contract No: AB80188N,Task 13
Phase calculations based on start and end of test.
* Saturation is set to 100% for phase calculations.

Thu, 23-JUN-2005 12:01:49

QORE

TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

JOB NAME:	Saltstone Vault # 2, Task 8			CONTRACT AB 80187N, SPECIFICATION K-SPC-G-00013 REV8	
JOB NO.:	24150-H	REPORT NO. :	N/A	REVIEWED BY :	DATE: 6/17/05
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	6'-8'	SAMPLE NO. : ST1	TYPE: UD
SAMPLE LOCATION :	N/A				
SOIL DESCRIPTION :	Orange brown clayey sand.				
LL, % :	43	PI, % :	24	FINES, % : 40	G_s : 2.70

SPECIMEN PROPERTIES		INITIAL			AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.88	2.88	2.88	D_c	2.87	2.86	2.86
HEIGHT , INCHES	H_o	5.90	5.84	5.84	H_c	5.88	5.81	5.80
WATER CONTENT, %	W_o	20.2	19.8	16.1	W_c	22.1	20.9	17.8
DRY DENSITY, PCF	γ_{dryo}	104.4	106.8	111.6	γ_{dryc}	105.5	107.7	113.9
SATURATION , %	S_o	88.9	90.1	85.0	S_c	100	100	100
VOID RATIO	e_o	0.615	0.594	0.511	e_c	0.598	0.565	0.480

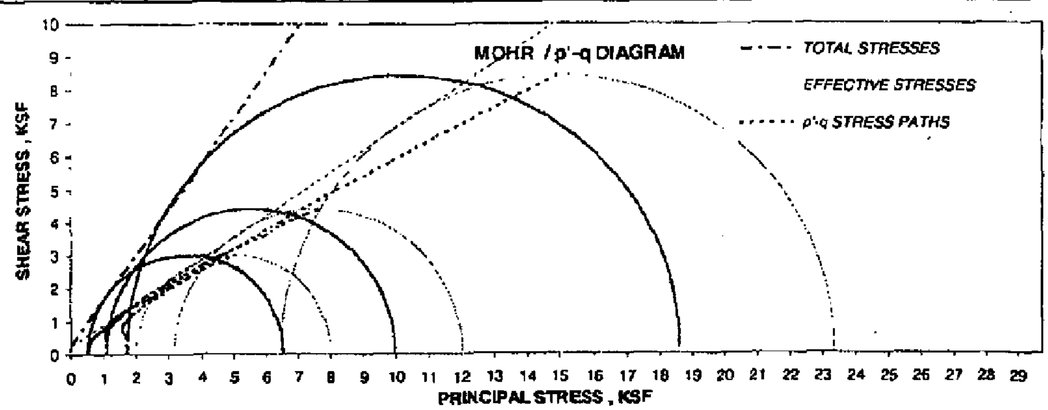
TEST PARAMETERS , TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	0.95
BACK PRESSURE, ksf	U_o	10.1	10.1	10.1
CONFINING PRESSURE , ksf	σ_3	0.5	1.1	1.7
MAX. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	6.0	8.8	16.9
ULT. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	6.0	8.8	16.9
Specimen Shape @ Failure	Bulged			

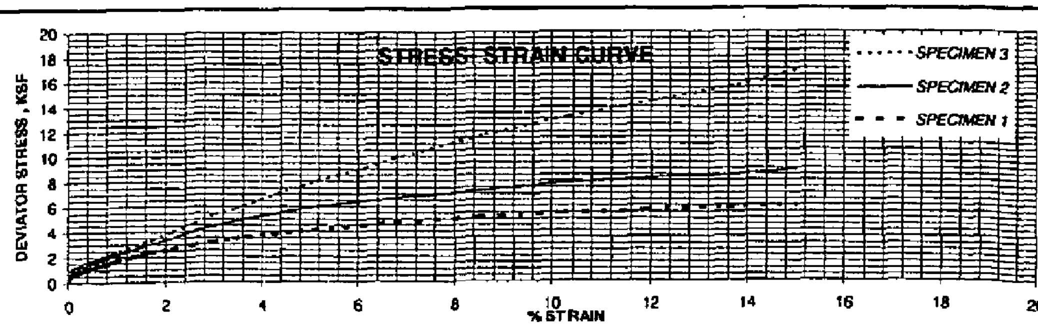
CONTROLLED : Strain @ 0.44 % per minute   T50, Minutes = 0.9

N/A

N/A

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION , c (ksf) :	0.30	APPARENT COHESION , (ksf) :	0.25
	ANGLE OF INTER. FRICTION, Φ(DEGREES) :	54.0	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	33.4





QORE

TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

JOB NAME:	Saltstone Vault # 2, Task 8			CONTRACT AB 80187N, SPECIFICATION K-SPC-G-00013 REV8			
JOB NO.:	24150-H	REPORT NO.:	N/A	REVIEWED BY:		DATE:	6/23/05
BORING / PIT NO.:	ZV2B2U	DEPTH / ELEV.:	14'-16'	SAMPLE NO.:	ST2	TYPE:	UD
SAMPLE LOCATION:	N/A						
SOIL DESCRIPTION:	Red brown and purple clayey sand.						
LL, %:	42	PI, %:	23	FINES, %:	39	G_s:	2.70

SPECIMEN PROPERTIES									
		INITIAL				AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3	
DIAMETER, INCHES	D_o	2.89	2.88	*	D_c	2.88	2.87	*	
HEIGHT, INCHES	H_o	5.90	5.83	*	H_c	5.88	5.80	*	
WATER CONTENT, %	W_o	15.7	14.1	*	W_c	18.5	15.8	*	
DRY DENSITY, PCF	γ_{dryo}	111.3	116.1	*	γ_{dryc}	112.1	118.0	*	
SATURATION, %	S_o	82.9	85.0	*	S_c	100	100	*	
VOID RATIO	e_o	0.511	0.448	*	e_c	0.500	0.426	*	

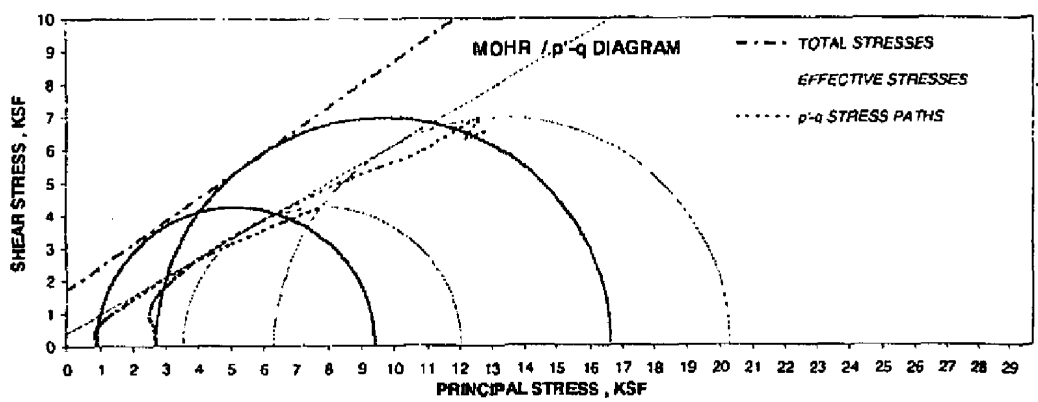
TEST PARAMETERS, TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	*
BACK PRESSURE, ksf	U_o	10.1	10.1	*
CONFINING PRESSURE, ksf	σ_3	0.9	2.7	*
MAX. DEVIATOR STRESS, ksf	$\sigma_1-\sigma_3$	8.5	14.0	*
ULT. DEVIATOR STRESS, ksf	$\sigma_1-\sigma_3$	8.5	13.2	*
Specimen Shape @ Failure	Bulged			*

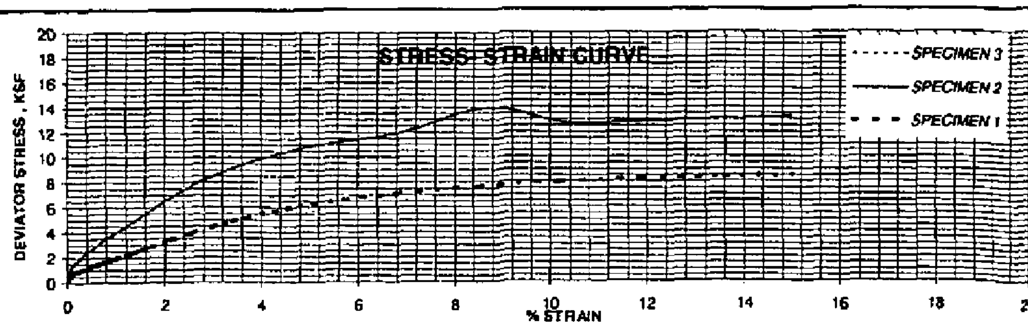
CONTROLLED : Strain @ 0.15 % per minute T50, Minutes = 2.6

Note : * Due to limited sample recovery. Only two specimens were tested as requested by SRS.

Note : Strength parameters were estimated as requested by client based on two specimens.

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION, C (ksf) :	1.70	APPARENT COHESION, (ksf) :	0.38
	ANGLE OF INTER. FRICTION, Φ (DEGREES) :	35.0	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	30.0





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TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

AAP

REV. 1/13/04

JOB NAME:	*Saltstone Vault # 2 ,Task 8*			CONTRACT AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO.:	*24150-H*	REPORT NO. :	*N/A*	REVIEWED BY :		DATE:	*8/1/05*
BORING / PIT NO. :	*ZV2B2U*	DEPTH / ELEV. :	*28'-30'*	SAMPLE NO. :	*ST3*	TYPE:	*UD*
SAMPLE LOCATION :	*N/A*						
SOIL DESCRIPTION :	*Orange brown clayey sand.*						
LL, % :	*38*	PI , % :	*20*	FINES, % :	*16*	G_s :	*2.68*

SPECIMEN PROPERTIES								
		INITIAL			AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.88	2.88	2.89	D_c	2.87	2.87	2.87
HEIGHT , INCHES	H_o	5.74	5.92	5.88	H_c	5.72	5.89	5.85
WATER CONTENT, %	W_o	16.4	14.6	14.6	W_c	19.8	19.4	18.7
DRY DENSITY, PCF	γ_{dryo}	107.8	108.3	109.7	γ_{dryc}	109.3	110.0	111.5
SATURATION ,%	S_o	79.7	72.2	74.9	S_c	100	100	100
VOID RATIO	e_o	0.551	0.544	0.524	e_c	0.530	0.520	0.501

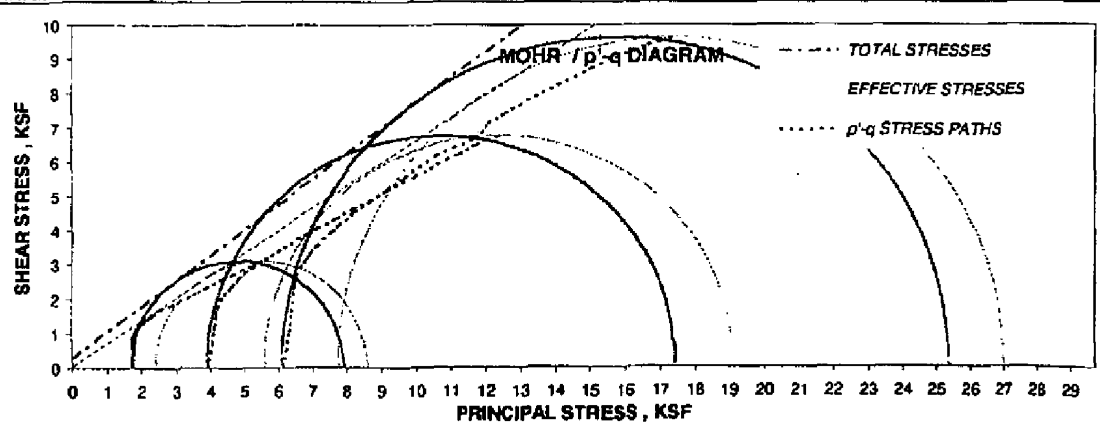
TEST PARAMETERS , TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	0.95
BACK PRESSURE, ksf	U_o	10.1	10.0	10.0
CONFINING PRESSURE , ksf	σ_3	1.7	3.9	6.1
MAX. DEVIATOR STRESS ,ksf	σ_1-σ_3	6.2	13.5	19.3
ULT. DEVIATOR STRESS , ksf	σ_1-σ_3	6.2	13.5	19.3
Specimen Shape @ Failure	Bulged			

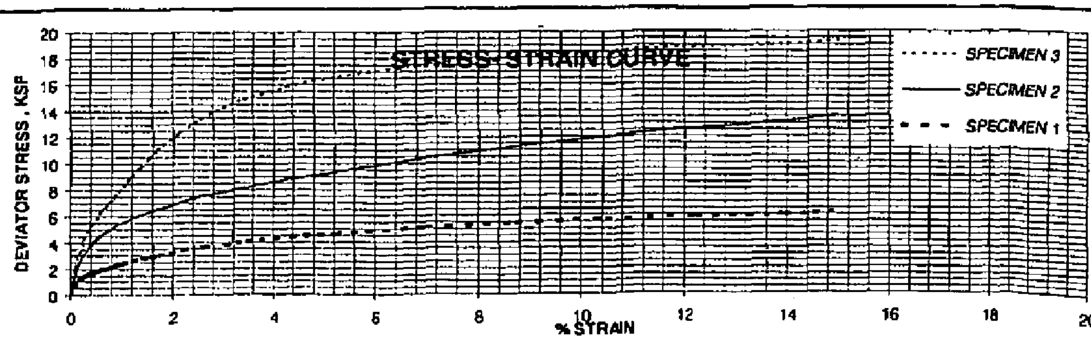
CONTROLLED : Strain @ 3.64 % per minute T50, Minutes = 0.11

N/A

N/A

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION , C (ksf) :	0.25	APPARENT COHESION , (ksf) :	0.05
	ANGLE OF INTER. FRICTION, Φ(DEGREES)	36.8	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	33.3





QORE

TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

AAP

REV. 1/13/04

JOB NAME:	Saltstone Vault # 2, Task 8			CONTRACT AB 80187N, SPECIFICATION K-SPC-G-00013 REV8			
JOB NO.:	24150-H	REPORT NO. :	N/A	REVIEWED BY :		DATE:	6/17/05
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	34'-36'	SAMPLE NO. :	ST4	TYPE:	UD
SAMPLE LOCATION :	N/A						
SOIL DESCRIPTION :	Tannish yellow silty sand.						
LL, % :	N.P.	PI, % :	N.P.	FINES, % :	16	G_s :	2.63

SPECIMEN PROPERTIES								
		INITIAL			AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER, INCHES	D_o	2.87	2.88	2.88	D_c	2.86	2.86	2.86
HEIGHT, INCHES	H_o	5.94	5.86	5.91	H_c	5.91	5.84	5.86
WATER CONTENT, %	W_o	17.8	19.3	19.8	W_c	24.2	22.5	23.0
DRY DENSITY, PCF	γ_{dryo}	99.0	101.6	99.8	γ_{dryc}	100.2	102.9	102.3
SATURATION, %	S_o	71.3	82.3	80.8	S_c	100	100	100
VOID RATIO	e_o	0.658	0.615	0.644	e_c	0.637	0.595	0.605

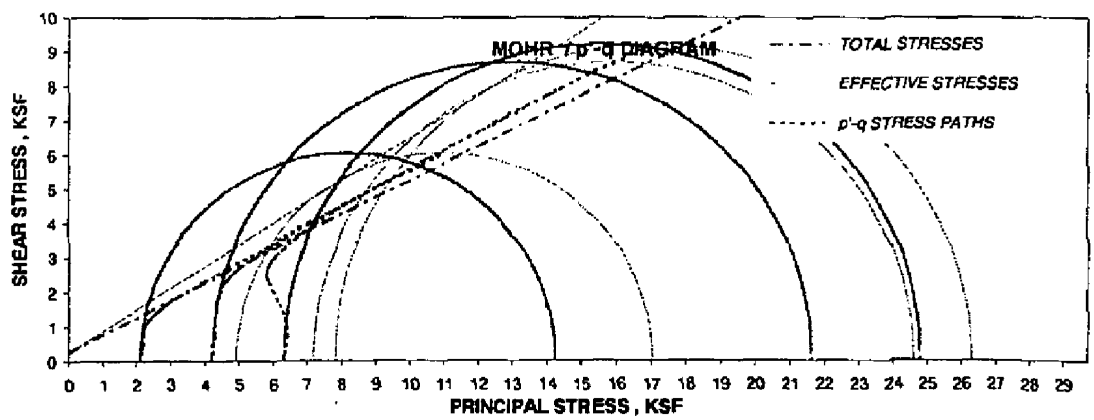
TEST PARAMETERS, TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	0.95
BACK PRESSURE, ksf	U_o	10.0	10.0	10.1
CONFINING PRESSURE, ksf	σ_3	2.1	4.2	6.3
MAX. DEVIATOR STRESS, ksf	$\sigma_1-\sigma_3$	12.1	17.4	18.5
ULT. DEVIATOR STRESS, ksf	$\sigma_1-\sigma_3$	12.1	17.4	18.5
Specimen Shape @ Failure	Bulged			

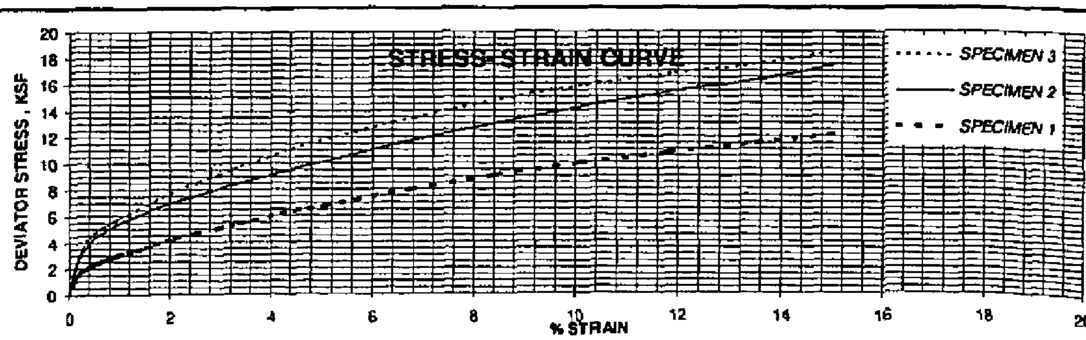
CONTROLLED : Strain @ 3.64 % per minute T50, Minutes = 0.11

N/A

N/A

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION, C (ksf) :	0.25	APPARENT COHESION, (ksf) :	0.26
	ANGLE OF INTER. FRICTION, Φ(DEGREES)	26.6	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	32.0



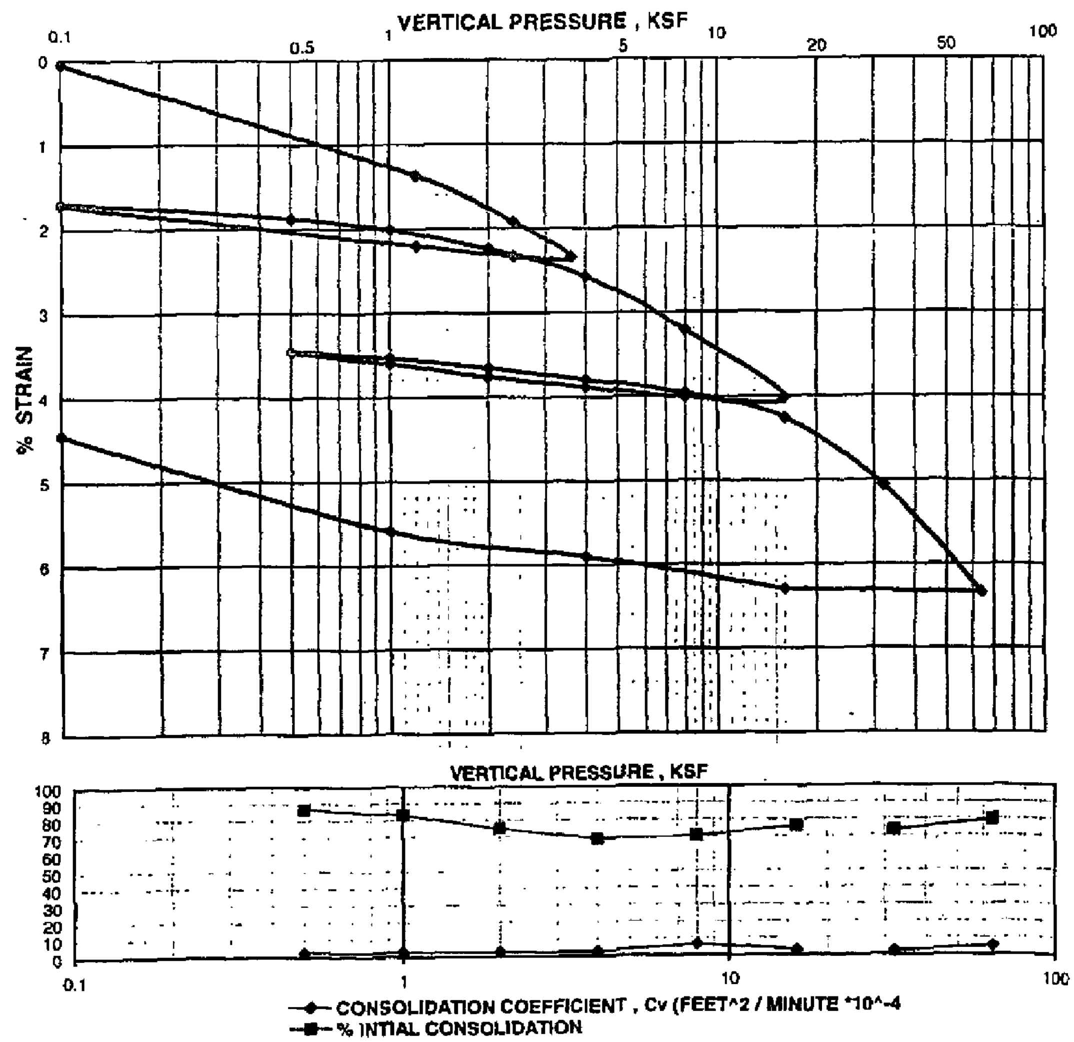


QORE

CONSOLIDATION TEST REPORT
(ASTM D 2435)

REV0:11/15/02

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 60187N , SPECIFICATION K-SPC-G-00013 REV5	
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/22/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	29'-31'	REVIEWED BY :	
SAMPLE LOCATION :	-				
WET DENSITY, γ_{wet}, PCF :	117.2	DRY DENSITY, γ_{DRY}, PCF :	100.1	SAMPLE TYPE :	UD
VOID RATIO, e_o :	0.653	MOISTURE CONTENT, %. :	17.2	SAMPLE NO. :	ST2
SATURATION, S_O, % :	69.7	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY, G_S :	2.65
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX, % :	N.P.	FINES, % :	14
SOIL DESCRIPTION :	Reddish brown sand with silt.				
REMOLDED PROPERTIES :	-				



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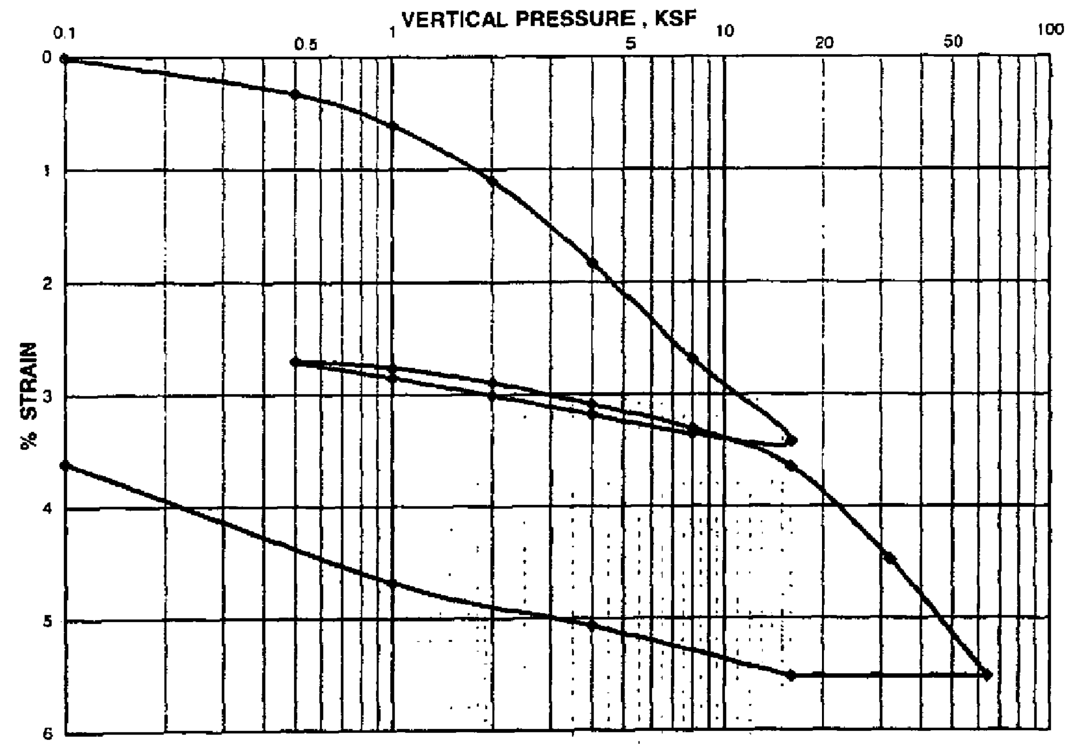
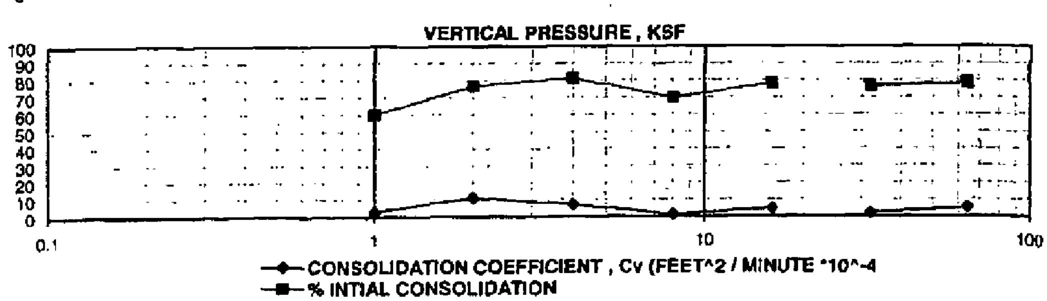
CONSOLIDATION TEST REPORT
(ASTM D 2435)

REV0,11/15/02

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AS 80187N SPECIFICATION K-SPC-G-00013 REV.8	
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	7/27/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	29'-31'	REVIEWED BY :	
SAMPLE LOCATION :	*Retested*				
WET DENSITY, γ_{wet}, PCF :	118.9	DRY DENSITY, γ_{DRY}, PCF :	101.9	SAMPLE TYPE :	UD
VOID RATIO, e_o :	0.622	MOISTURE CONTENT, %. :	16.7	SAMPLE NO. :	ST2
SATURATION, S_O, % :	71.1	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY, G_S :	2.65
LIQUID LIMIT, % :	*N.P.*	PLASTICITY INDEX, % :	*N.P.*	FINES, % :	*14*
SOIL DESCRIPTION :	Reddish brown sand with silt.				
REMOLDED PROPERTIES : -					





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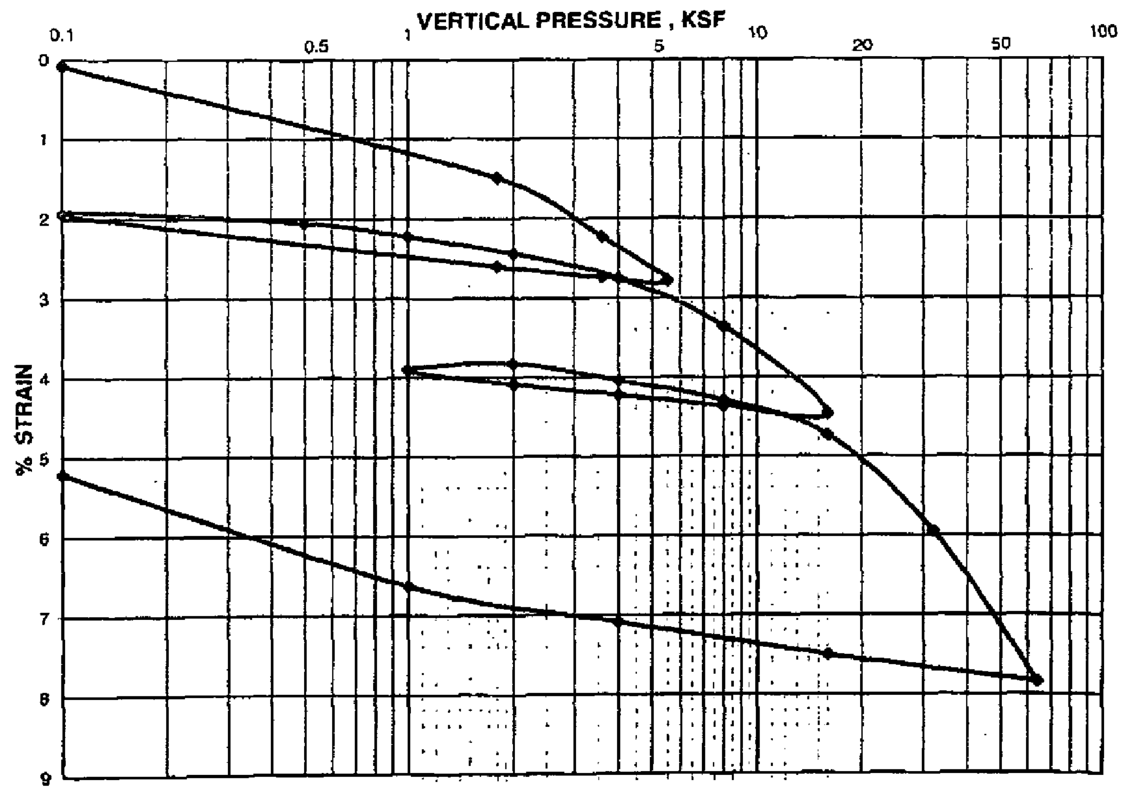
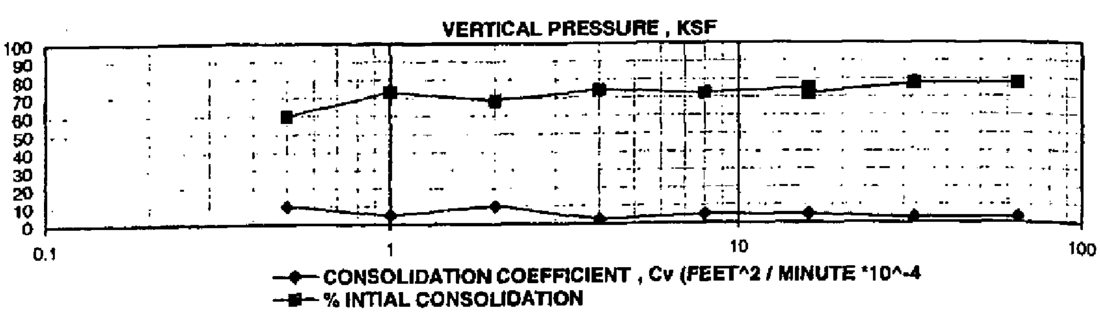
CONSOLIDATION TEST REPORT
(ASTM D 2435)

REV0 11/15/02

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80167N , SPECIFICATION K-SPC-G-00013 REV8	
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/24/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	45'-47'	REVIEWED BY :	
SAMPLE LOCATION :	-				
WET DENSITY, γ_{wet}, PCF :	118.6	DRY DENSITY, γ_{DRY}, PCF :	102.8	SAMPLE TYPE :	UD
VOID RATIO, e_o :	0.609	MOISTURE CONTENT, %. :	15.4	SAMPLE NO. :	ST4
SATURATION, S_o, % :	67.0	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY, G_s :	2.65
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX, % :	N.P.	FINES, % :	18
SOIL DESCRIPTION :	Reddish brown siltysand.				
REMOLDED PROPERTIES :	-				





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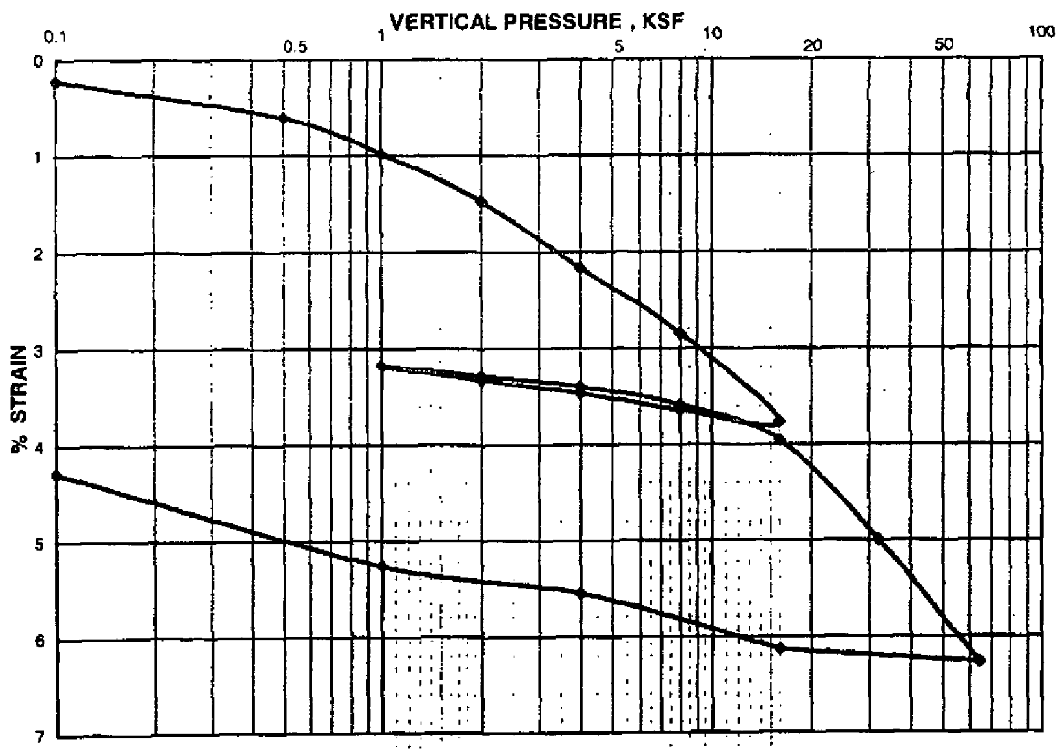
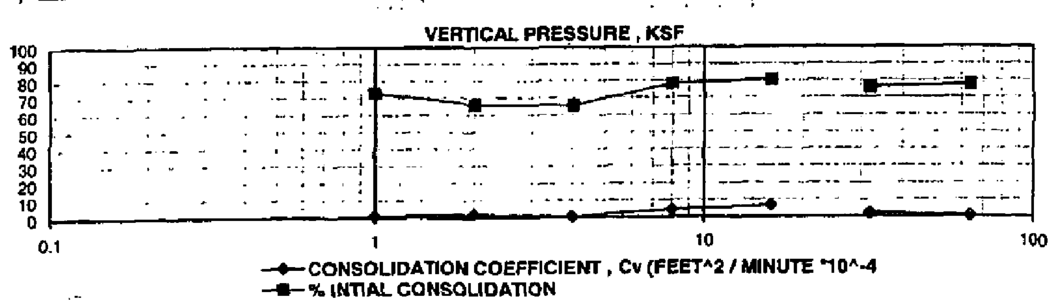
CONSOLIDATION TEST REPORT

(ASTM D 2435)

REV0, 11/15/02

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV6	
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	7/27/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	45'-47'	REVIEWED BY :	
SAMPLE LOCATION :	*Retested*				
WET DENSITY, γ_{wet}, PCF :	119.5	DRY DENSITY, γ_{DRY}, PCF :	101.6	SAMPLE TYPE :	UD
VOID RATIO, e_o :	0.628	MOISTURE CONTENT, % :	17.7	SAMPLE NO. :	ST4
SATURATION, S_o, % :	74.7	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY, G_S :	2.65
LIQUID LIMIT, % :	*N.P.*	PLASTICITY INDEX, % :	*N.P.*	FINES, % :	*18*
SOIL DESCRIPTION :	Reddish brown silty sand.				
REMOLDED PROPERTIES :	-		-		-





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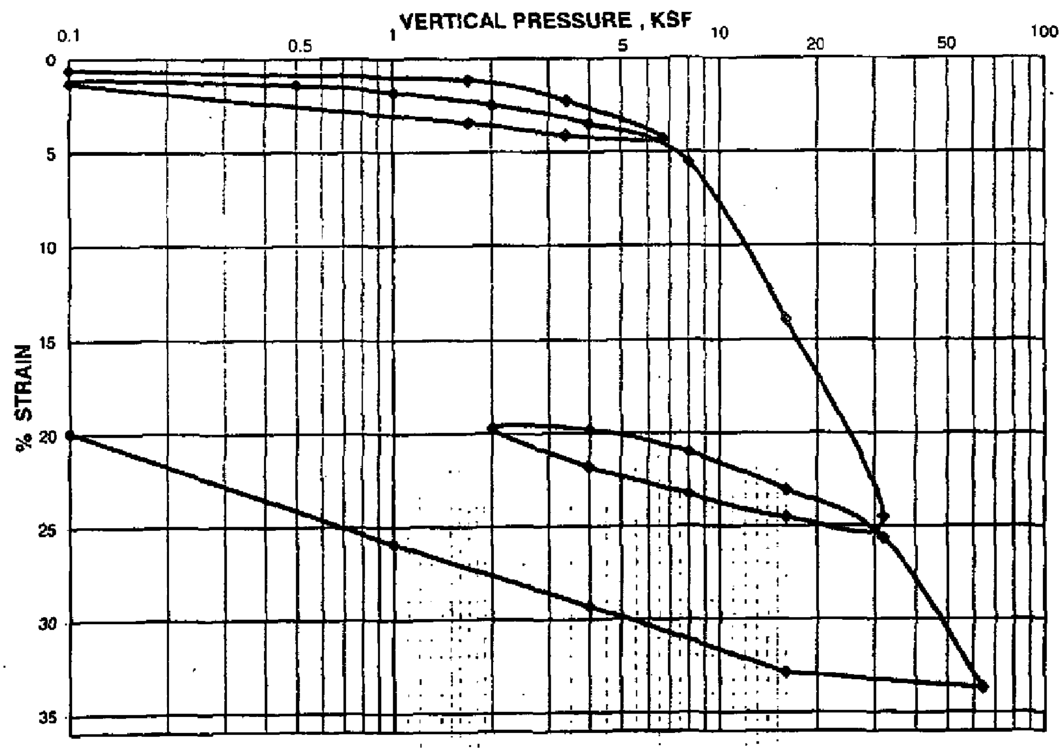
CONSOLIDATION TEST REPORT

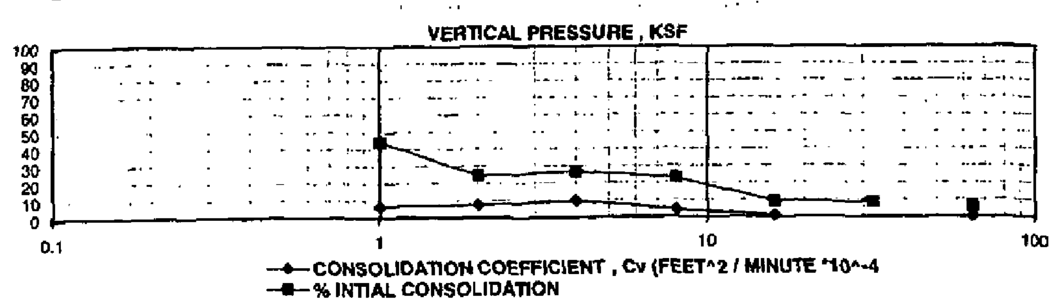
(ASTM D 2435)

AAP

REV0, 11/15/02

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8	
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/24/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	62'-64'	REVIEWED BY :	
SAMPLE LOCATION :	-				
WET DENSITY, γ_{wet}, PCF :	101.0	DRY DENSITY , γ_{DRY} , PCF :	69.5	SAMPLE TYPE :	UD
VOID RATIO , e_o :	1.378	MOISTURE CONTENT , %. :	45.3	SAMPLE NO. :	ST5
SATURATION , S_O , % :	87.1	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY , G_S :	2.65
LIQUID LIMIT , % :	53	PLASTICITY INDEX , % :	36	FINES , % :	41
SOIL DESCRIPTION :	Tan gray clayey sand.				
REMOLDED PROPERTIES :	-		-		-





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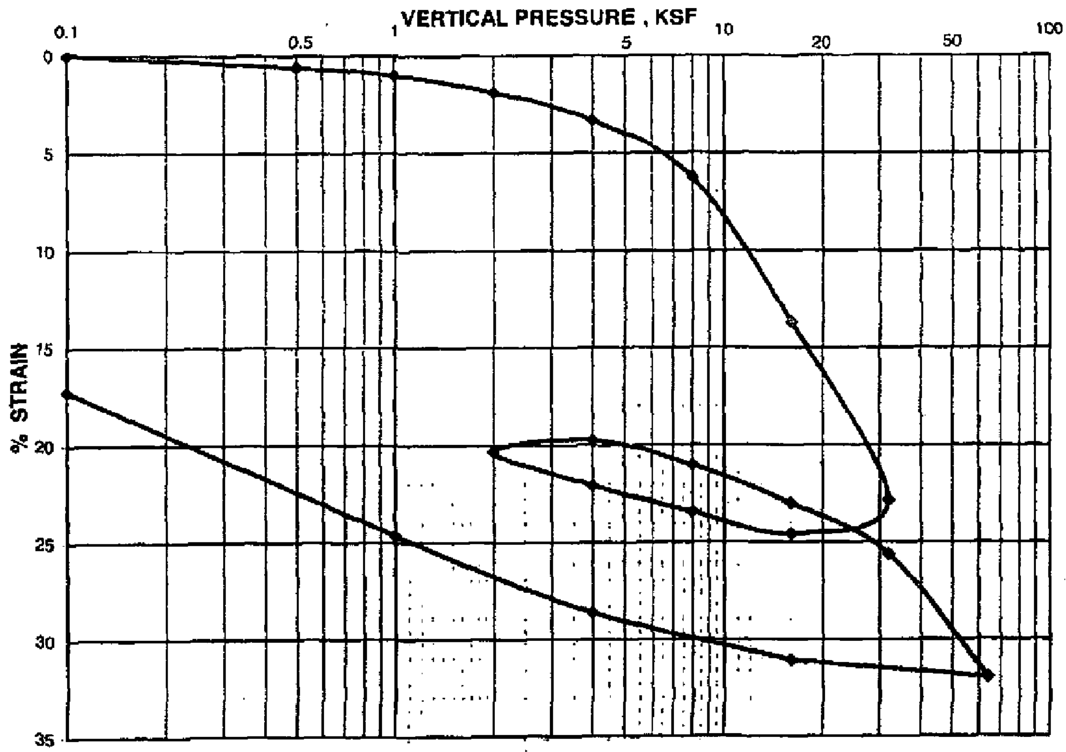
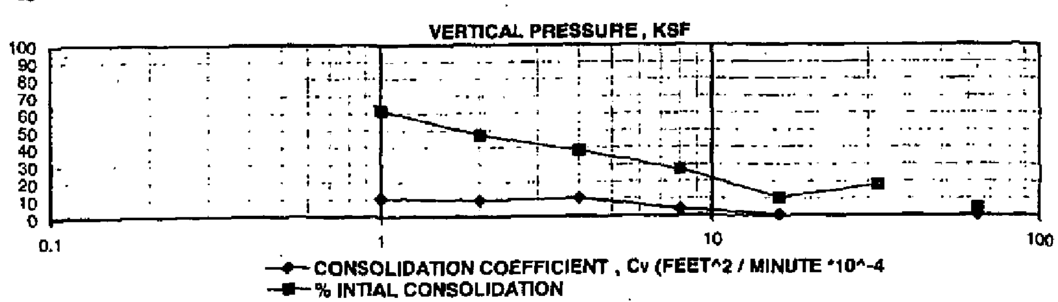
CONSOLIDATION TEST REPORT
(ASTM D 2435)

REV.0,11/15/02

CONTRACT NO. AB 30187N , SPECIFICATION K-SPC-G-00012 REV.

JOB NAME :	Saltstone Vault # 2,Task 8				
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	7/27/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	62'-64'	REVIEWED BY :	
SAMPLE LOCATION :	Retested				
WET DENSITY, γ_{wet}, PCF :	102.8	DRY DENSITY, γ_{DRY}, PCF :	69.2	SAMPLE TYPE :	UD
VOID RATIO, e_o :	1.391	MOISTURE CONTENT, %. :	48.7	SAMPLE NO. :	ST5
SATURATION, S_o, % :	92.8	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY, G_s :	2.65
LIQUID LIMIT, % :	53	PLASTICITY INDEX, % :	36	FINES, % :	41
SOIL DESCRIPTION :	Tan gray clayey sand.				
REMOLDED PROPERTIES :	-				





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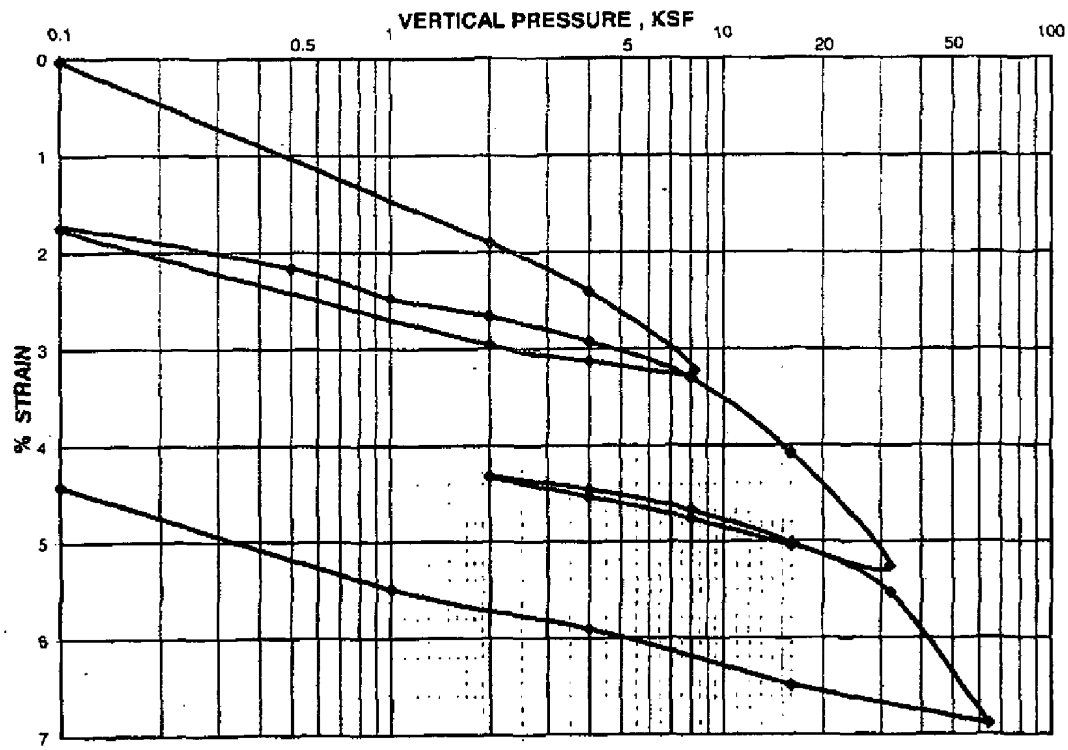
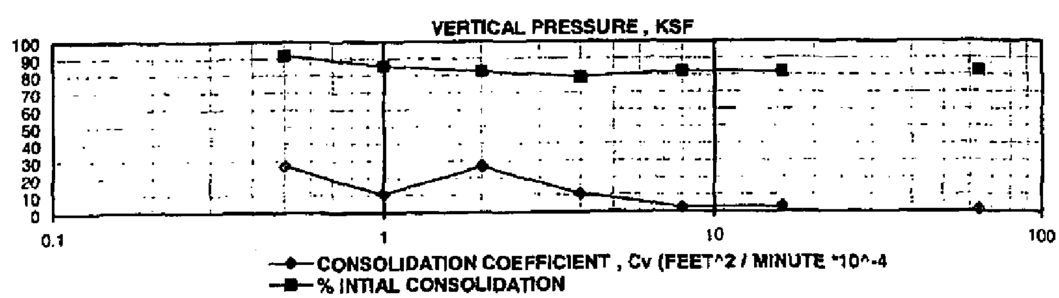
CONSOLIDATION TEST REPORT
(ASTM D 2435)

REV0,11/15/02

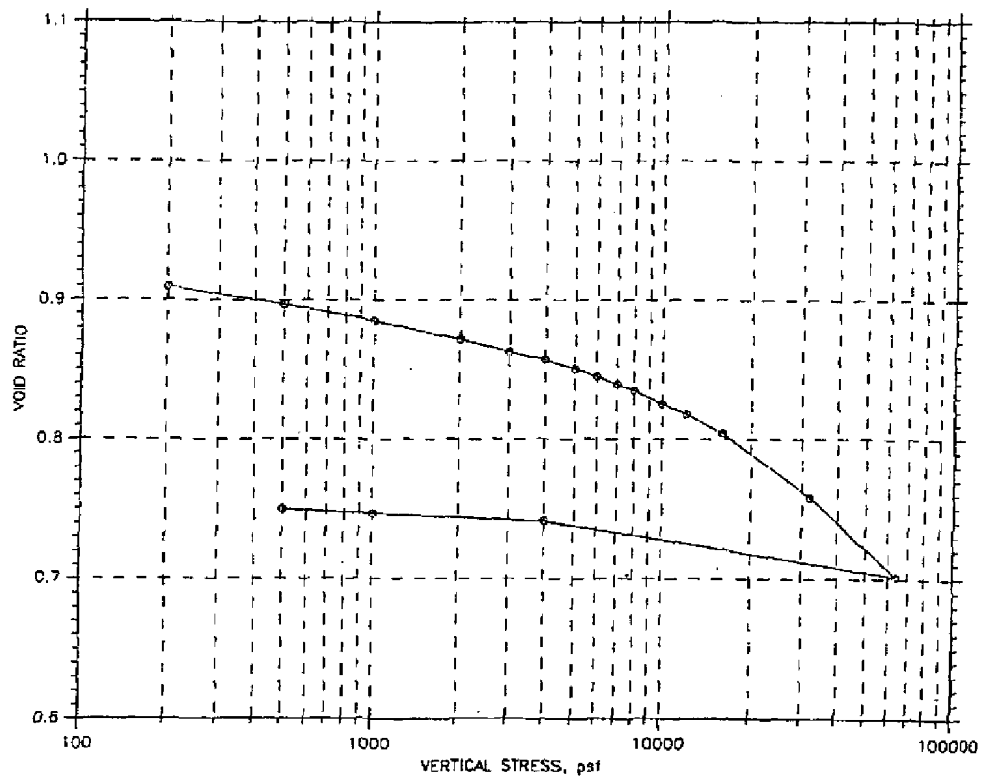
JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80167N SPECIFICATION K-SPC-G-00013 REV8	
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/24/2005
BORING / PIT NO. :	ZV2B1U	DEPTH / ELEV. :	87'-89'	REVIEWED BY :	
SAMPLE LOCATION :	-				
WET DENSITY, γ_{wet}, PCF :	123.9	DRY DENSITY, γ_{DRY}, PCF :	105.3	SAMPLE TYPE :	UD
VOID RATIO, e_0 :	0.571	MOISTURE CONTENT, % :	17.7	SAMPLE NO. :	ST7
SATURATION, S_0, % :	82.0	LOAD VS. TIME PLOT :	Log of time	SP. GRAVITY, G_S :	2.65
LIQUID LIMIT, % :	N.P.	PLASTICITY INDEX, % :	N.P.	FINES, % :	5
SOIL DESCRIPTION :	*Gray sand.*				
REMOLDED PROPERTIES : -					





CONSOLIDATION TEST DATA

SUMMARY REPORT



				Before Test	After Test
Overburden Pressure: 0 psf			Water Content, %	35.51	32.27
Preconsolidation Pressure: 0 psf			Dry Unit Weight, pcf	86.67	95.27
Compression Index: 0			Saturation, %	102.68	114.97
Diameter: 2.5 in		Height: 1 in	Void Ratio	0.92	0.75
LL: ---	PL: ---	PI: ---	GS: 2.67		

GeoTesting express the groundwork for success	Project: Saltstone Vault #2/Task 1	Location: Z8281U	Project No.: GTX G0821
	Boring No.: Z8281U	Tested By: JW	Checked By: HJ
	Sample No.: PS-1	Test Date: 5/27/05	Depth: 105.2 Ft.
	Test No.: 12981	Sample Type: Shelby Tube	Elevation: N/A
	Description: Yellowish Brown Clayey Sand		
	Remarks: Contract No.: AB80188N, Task 13		

Thu, 23-JUN-2005 11:49:59

QORE

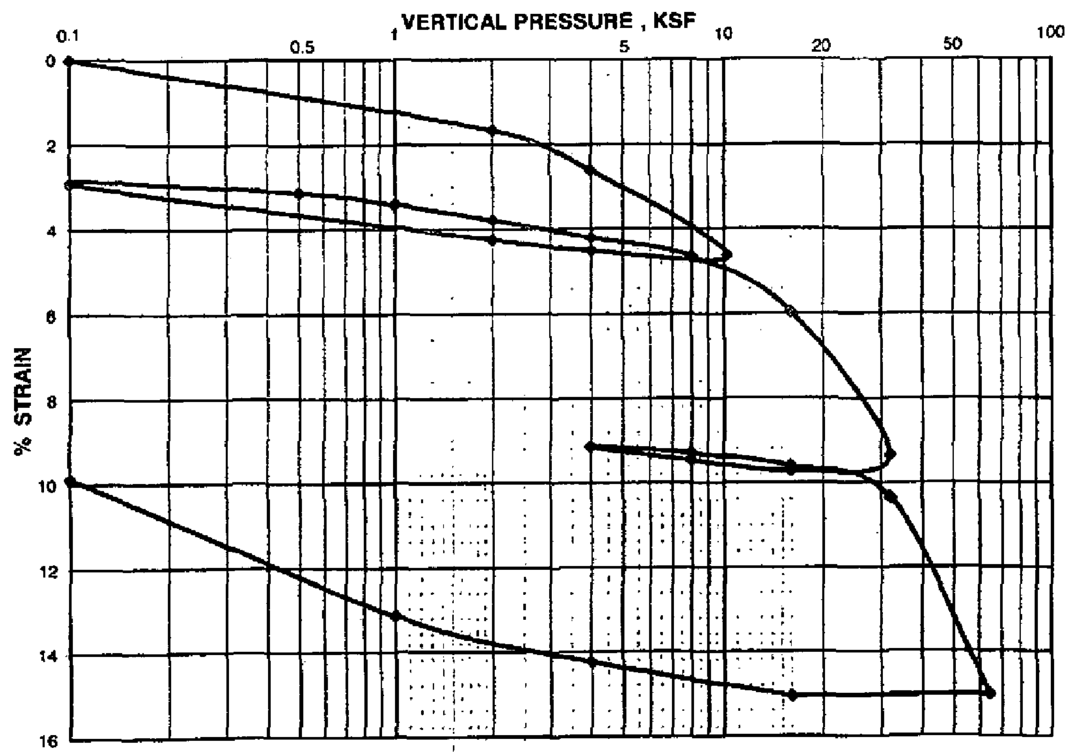
CONSOLIDATION TEST REPORT

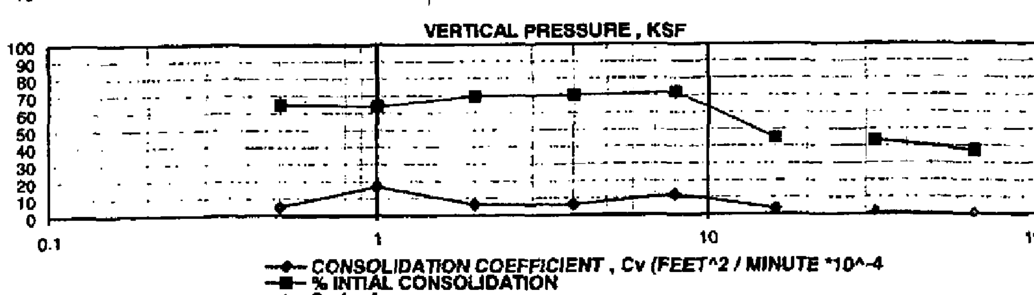
(ASTM D 2435)

REV0, 11/15/02

CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00012 REV8

JOB NAME :	Saltstone Vault # 2,Task 8				
JOB NO. :	24150-H	**REPORT NO. :**	-	**DATE :**	6/27/2005
BORING / PIT NO. :	ZV2B1U	**DEPTH / ELEV. :**	122'-124'	**REVIEWED BY :**	
SAMPLE LOCATION :	-				
WET DENSITY, γ_{wet} , PCF :	114.2	**DRY DENSITY , γ_{DRY} , PCF :**	86.1	**SAMPLE TYPE :**	UD
VOID RATIO , e_o :	0.921	**MOISTURE CONTENT , % :**	32.6	**SAMPLE NO. :**	PS3
SATURATION , S_0 , % :	93.9	**LOAD VS. TIME PLOT :**	Log of time	**SP. GRAVITY , G_S :**	2.65
LIQUID LIMIT , % :	N.P.	**PLASTICITY INDEX , % :**	N.P.	**FINES , % :**	27
SOIL DESCRIPTION :	Gray silty sand.				
REMOLDED PROPERTIES :	-				





Appendix B

K-CLC-Z-00010, Liquefaction Analysis for Saltstone Vault No. 2

Rev. 0, March 2006

(39 pages)

OSR 45-24 (Rev 11-15-2004)

Calculation Cover Sheet

Project Saltstone Vault No. 2	Calculation No. K-CLC-Z-00010	Project No. N/A
Title Liquefaction Analysis for Saltstone Vault No. 2	Functional Classification PS	Sheet 1 of 38
	Discipline Geotechnical	
Calc Level ☒ Type 1 ☐ Type 2	Type 1 Calc Status ☐ Preliminary ☒ Confirmed	
Computer Program No. ☒ N/A	Version/Release No. N/A	

Purpose and Objective

This calculation provides the evaluation of liquefaction potential and estimates settlement due to liquefaction and partial liquefaction

Summary of Conclusion

see last section.

UNCLASSIFIED
DOES NOT CONTAIN UNCLASSIFIED CONTROLLED NUCLEAR INFORMATION
ADC & Reviewing Official: Michael D. McHood
(Name and Title)
Date: 3-22-2006

Revisions

Rev No.	Revision Description
0	Initial issue

Sign Off

Rev No.	Originator (Print) Sign/Date	Verification/ Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign//Date
0	William Li William Li	Document review	Michael D. McHood 3-15-06 MICHAEL D. MCHOOD	M.K. Lewis 3/22/06 M.K. Lewis

Design Authority — (Print)	Signature	Date
Release to Outside Agency – (Print)	Signature	Date
Security Classification of the Calculation		

Table of Contents

1. Introduction

This calculation provides the evaluation of liquefaction potential for the Z-Area Saltstone Vault No. 2 site (Ref. 1) for post closure conditions. This calculation also estimates settlement of the facility due to liquefaction and partial liquefaction for post closure conditions.

2. Input

2.1 Project Site

Figure 1 shows the project site along with geotechnical exploratory locations (Ref. 2).

2.2 Seismic Piezocone Penetration Tests

Seven seismic piezocone penetration tests (SCPTu) were performed at the project site (Refs. 2 and 3). Table 1 provides the list of these SCPTu's. Figure 1 shows the locations of these SCPTu's along with other testing locations. Attachment A provides hard copy plots of the SCPTu data (Ref. 3).

2.3 Soil Properties

Laboratory sieve analyses were performed on soil samples retrieved from geotechnical boreholes at the project site (Ref. 2). Table 2 provides the boreholes used in this calculation. Figure 1 shows the locations of these boreholes along with other boreholes drilled and test pits dug during the investigation. Table 3 summarizes percent fines (i.e., percent passing #200 sieve) used in this calculation.

2.4 Stratigraphy

The stratigraphy at the project site was evaluated (Ref. 4). Engineering layers are identified as Layers S1/2, C2, S3, S4, and M1. 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 SCPTu location. Subsurface cross sections of the project site are provided in Reference 2. 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.

2.5 Groundwater Elevation

The groundwater elevation at the project site is 225 feet, MSL with estimated seasonal fluctuation of + 5 feet (Ref. 5). For this calculation a conservative groundwater elevation of 230 feet, MSL is used. Only saturated soils are analyzed for potential liquefaction.

2.6 Design Basis Earthquake

The design base earthquake is a response spectra based on IBC Section 1615 (Ref. 6). The peak ground acceleration (PGA) is 0.21 g. Attachment B provides the PGA computation details.

The USGS has performed a site specific Probabilistic Seismic Hazard Analysis (PSHA) for the SRS (Ref. 7, Attachment C). The SRS PGA hazard dissagregation for the 2,500 year return period is presented in Table 5. The hazard dissagregation is used to establish weights for averaging earthquake results based on various magnitudes for the DBE.

3. Evaluation and Computation

3.1 Evaluation of Liquefaction Potential

A modified version of the Simplified Procedure for Evaluating Soil Liquefaction Potential (Refs. 8 and 9) 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.

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 (Ref. 10) and
r_d = 1.000 – 0.00765 z for z • 9.15 m
= 1.174 – 0.0267 z for 9.15 m < z • 23 m
= 0.744 – 0.0080 z for 23 m < z • 30 m
= 0.500 for z > 30 m
where z is the depth in meters

Factor of Safety = $CRR_{7.5}\, K_{\sigma}\, K_{age}\, K_{\alpha}$ MSF/CSR
where K_{α} is the static driving shear correction factor (Ref. 10). 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. 11) where σ'_{vo} is the effective vertical overburden pressure in tsf. Figure 2 shows the equation. The NCEER recommended K_σ curves (Ref. 8) are also shown for comparison.

MSF is the earthquake magnitude scaling factor (Ref. 10). Values of the MSF are provided in Figure 3 and Table 6.

Computation of CRR using Shear Wave Velocity

CRR can be computed from shear wave velocity (Ref. 12):

$$CRR = 0.022(V_{s1}/100)^2 + 2.8[1/(V^*_{s1} - V_{s1}) - 1/V^*_{s1}]$$

where V_{s1} is overburden stress-corrected shear wave velocity
V^*_{s1} is the limiting value of V_{s1} for cyclic liquefaction occurrence

V_{s1} is computed using the equation:

$$V_{s1} = V_s (P_a / \sigma'_{vo})^{0.25}$$

where V_s is shear wave velocity
P_a is reference stress of about atmospheric pressure
σ'_{vo} is the effective vertical overburden stress

V^*_{s1} is computed using the equation:

V^*_{s1} = 215 m/s	for sands with fines content (FC) • 5%
V^*_{s1} = 215 – 0.5(FC – 5) m/s	for sands with 5% < FC < 35%
V^*_{s1} = 200 m/s	for sands with FC • 35%

Figure 4 presents the V^*_{s1} versus CRR curves.

Computation of CRR using CPTu Tip Resistance

CRR can also be computed based on normalized tip resistance (qt_1) using site specific equations developed at SRS (Refs. 11 and 15):

for 30% fines: $CRR = 0.125721 + 0.002537 (qt_1) + 0.000040 (qt_1)^2$
for 22.5% fines: $CRR = 0.093309 + 0.001757 (qt_1) + 0.000029 (qt_1)^2$
for 15% fines: $CRR = 0.072666 + 0.001141 (qt_1) + 0.000028 (qt_1)^2$
for 10% fines: $CRR = 0.046881 + 0.001190 (qt_1) + 0.000015 (qt_1)^2$
for 0% fines: $CRR = 0.021215 + 0.001408 (qt_1) + 0.000007 (qt_1)^2$

Where qt1 is the tip stress normalized for overburden stress at the time of data collection. The tip stress (qt) is normalized using the following equation (Ref. 8):

$$qt_1 = C_Q\, qt$$

where C_Q is the CPTu overburden normalization factor, computed from

$C_Q = (P_a / \sigma'_{vo})^n$ with $C_Q \leq 1.7$

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%). Figure 5 shows the relationship between CRR and qt1.

To use the SRS CRR equations it is necessary to determine percent fines. Percent fines were estimated using the following equation (Ref. 13):

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

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

Table 3 presents laboratory determined percent fines from samples collected in borings Z-V2-B1U and Z-V2-B2. These borings were selected based on their proximity to two of the SCPTu locations used in this calculation. Boring Z-V2-B1U is within 3 feet of SCPTu Z-V2-CP15 and boring Z-V2-B2 is within 8 feet of Z-V2-CP8 (see Figure 1). Figures 6 and 7 are plots of laboratory determined percent fines versus CPTu estimated fines for these SCPTu/Boring pairs. Estimated fine contents based on CPTu provided reasonable results.

3.2 Settlement Resulting from Liquefaction and Partial Liquefaction

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:

Volumetric Strain Curves

The volumetric strain curves developed for SRS (Refs. 14 and 15) using H-Area data are used for computing the volumetric strains:

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

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

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

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

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

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

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

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

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

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

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

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

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

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 8 shows the relation between the volumetric strain and the Factors of Safety for various qt_1.

Table 5 gives the PGA hazard dissagregation for the 2,500-year return period (Ref. 7). Using Table 5, the weight for a given magnitude is the sum of the various distances for a given magnitude. For example, the sum for magnitude 4.5 to 5.0 is 15.52, while the sum for magnitude 7.0 to 8.0 is 47.70. Settlement due to liquefaction or partial liquefaction is calculated using the methodology above using various magnitudes (i.e., 4.75, 5.25, 5.75, 6.25, 6.75, and 7.5). The settlement results are then weighted according to the appropriate magnitude from the hazard dissagregation.

4. Results and Conclusion

Liquefaction Potential

The "Simplified Procedure for Evaluating Soil Liquefaction Potential" was used to quantitatively evaluate liquefaction susceptibility. Both the methods of shear wave (V_{s1}) and the CPTu (qt_1) were used to determine CRR in conjunction with the DBE having a PGA of 0.21g. Percent fines required to determine CRR were determined using the CPTu estimated results, which are based on correlation with laboratory determined percent fines.

Method 1 Cyclic Resistance Ratio using Shear Wave Velocity

Liquefaction susceptibility calculations were performed for the DBE using V_{s1} data from the seven SCPTu soundings. Liquefaction factor of safety versus elevation for SCPTu location Z-V2-CP15 is presented on Figure 9 for several different magnitudes. Note that SCPTu location Z-V2-CP15 was selected for presentation as it has the lowest factor of safety values of the seven SCPTu for the site.

Settlements were not calculated using the shear wave safety factors, as the liquefaction strain curves (see Figure 8) are functions of CPTu tip stress. The factor of safety calculated from shear wave velocity is comparable to that calculated using the CPTu tip stress method (see Figures 9 and 10).

Method 2 Compute Cyclic Resistance Ratio using CPTu Tip Stress

Liquefaction potential and resulting settlements were computed using the CPTu method. Liquefaction factor of safety versus depth for SCPTu location Z-V2-CP15 is presented on Figure 10 for several different magnitudes.

Liquefaction analyses using shear wave velocity and CPTu tip stress suggest 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. Figures 9 and 10 compare reasonably well indicating that the two methods give similar results. As expected, both methods show decreasing factor of safety with increasing earthquake magnitude.

Settlement due to liquefaction and partial liquefaction

Settlement versus depth for SCPTu location Z-V2-CP15 is presented on Figure 11 for several different magnitudes. Note that SCPTu location Z-V2-CP15 was selected for presentation as it has the highest calculated settlement of the seven SCPTu for the site for a magnitude 7.5 earthquake (see Table 7).

Settlements were calculated using the SRS volumetric strain relationship for each of the seven SCPTu and for all magnitudes in the USGS seismic hazard for SRS. The results as well as the weighted average are summarized in Table 7.

Settlement due to partial liquefaction ranges from near zero to nearly 2¼ inch depending on SCPTu location and earthquake magnitude. The weighted average using the USGS PGA hazard dissagregation for weighting is less than an inch for the 2,500-year earthquake set.

5. References

(1) M-TC-Z-00004, Task Requirements and Criteria Document, Saltstone Facility Cylindrical Vault #2 Project Blud 451-002Z (U), Rev. 1, October 2005.

(2) SRS Calculation No. K-CLC-Z-00008, Evaluation of Test Data, Rev. 0, February 2006.

(3) Subcontract No. AC39054N, Task 3, Applied Research Associates, Inc., Cone Penetrometer Data Report, July 2005.

(4) SRS Calc. K-CLC-Z-00005, Stratigraphy for the Saltstone Vault No. 2, Rev. 0, July 2005.

(5) SRS Calc. K-CLC-Z-00006, Soil Properties for the Design of Vault No. 2, Rev. 0, July 2005.

(6) ICC, IBC International Building Code 2003.

(7) Frankel, A., USGS, Letter to R. C. Lee, Re: Results of USGS calculation of SRS PSHA, March 1, 1999.

(8) Youd, T. L., et. al., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, Vol. 127, No. 10, October, 2001.

(9) Seed, H. B. and I. M. Idriss, Simplified Procedure for Evaluating Soil Liquefaction Potential, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 97, SM9, New York, NY, 1971.

(10) NCEER, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, National Center for Earthquake Engineering Research, Technical Report No. NCEER-97-0022, State University of New York at Buffalo, NY, December 31, 1997.

(11) WSRC-TR-95-0057, In-Tank Precipitation Facility (ITP) and H-Tank Farm (HTF) Geotechnical Report (U), Rev. 0, September 1995.

(12) Andrus, Ronald D., and Kenneth H. Stokoe, Liquefaction Resistance of Soils from Shear-Wave Velocity, Journal of Geotechnical and Environmental Engineering, Vol. 126, No. 11, November, 2000.

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

(14) SRS calculation No. K-CLC-H-00064, Volumetric Strain, Nov. 1994.

(15) SRS calculation No. C-CLC-H-00815, Probabilistic Liquefaction Settlement Evaluation for ITP (U), Rev. 0, Nov. 1995.

Table 1 Seismic Piezocone Penetration Tests

I.D.	North Coordinate (feet)	East Coordinate (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
Z-V2-CP5	77,192	67,008	287.0	114.0
Z-V2-CP6	77,293	66,929	283.0	107.0
Z-V2-CP7	77,282	67,082	279.4	110.0
Z-V2-CP8	77,360	66,983	278.8	113.0
Z-V2-CP9	77,483	67,019	275.3	111.5
Z-V2-CP10	77,470	66,880	279.0	103.0
Z-V2-CP15	77,270	67,020	281.7	143.5

Table 2 Geotechnical Boreholes

I.D.	Type	SRS North (feet)	SRS East (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
Z-V2-B2	SPT	77,353	66,983	279.0	132.0
Z-V2-B1U	UD	77,271	67,018	281.9	123.5

Table 3 Percent Fines

Sample No.	Top Elevation (feet, MSL)	Bottom Elevation (feet, MSL)	Engineering Layer	Fines (passing #200) (%)
Z-V2-B2-SS2	267.0	265.0	S1/2	31.9
Z-V2-B2-SS6	259.0	257.0	S1/2	30.6
Z-V2-B2-SS8	252.0	250.0	S1/2	13.7
Z-V2-B2-SS10	244.0	242.0	S1/2	15.0
Z-V2-B2-SS12	234.0	232.0	S1/2	12.0
Z-V2-B2-SS14	224.0	222.0	C2	19.3
Z-V2-B2-SS15	219.0	217.0	C2	19.4
Z-V2-B2-SS16	214.0	212.0	C2	17.0
Z-V2-B2-SS17	209.0	207.0	S3	15.4
Z-V2-B2-SS18	204.0	202.0	S3	7.0
Z-V2-B2-SS19	199.0	197.0	S3	8.2
Z-V2-B2-SS20	194.0	192.0	S3	9.7
Z-V2-B2-SS21	189.0	187.0	S3	32.2
Z-V2-B2-SS22	184.0	182.0	S4	12.6
Z-V2-B2-SS23	179.0	177.0	S4	30.6
Z-V2-B2-SS24	174.0	172.0	S4	43.6
Z-V2-B2-SS25	169.0	167.0	S4	44.3
Z-V2-B1U-ST1	267.9	265.9	S1/2	25.1
Z-V2-B1U-ST2	252.9	250.9	S1/2	13.9
Z-V2-B1U-ST3	250.9	248.9	S1/2	17.6
Z-V2-B1U-ST4	236.9	234.9	S1/2	18.3
Z-V2-B1U-ST5	219.9	217.9	C2	40.6
Z-V2-B1U-ST7	194.9	192.9	S3	5.2
Z-V2-B1U-PS1T	177.9	177.4	S4	13.0
Z-V2-B1U-PS1B	177.4	175.9	S4	15.4
Z-V2-B1U-PS3	159.9	157.9	S4	27.0

Table 4 Engineering Layers

SCPTu	Surface Elevation (feet, MSL)	Elevation at contact of S1/2 and C2 (feet, MSL)	Elevation at contact of C2 and S3 (feet, MSL)	Elevation at contact of S3 and S4 (feet, MSL)
Z-V2-CP5	287.0	227	208	176
Z-V2-CP6	283.0	230	212	178
Z-V2-CP7	279.4	223	207	176
Z-V2-CP8	278.8	224	212	184
Z-V2-CP9	275.3	212	197	166
Z-V2-CP10	279.0	228	213	186
Z-V2-CP15	281.7	223	208	181
Average	280	223	208	179

Table 5 Peak Ground Acceleration Hazard Dissagregation for 2,500-year Return Period in %
(Bedrock PGA = 0.15g)

Distance (km)	M_W = 4.5 to 5.0	M_W = 5.0 to 5.5	M_W = 5.5 to 6.0	M_W = 6.0 to 6.5	M_W = 6.5 to 7.0	M_W = 7.0 to 8.0	Total
7.5	7.126	3.559	1.728	0.776	0.420	0.000	-
20	4.931	3.636	2.351	1.276	0.775	0.000	-
37.5	3.105	3.896	4.038	3.237	2.723	0.000	-
75	0.329	0.926	2.049	3.242	0.000	16.448	-
150	0.026	0.142	0.588	1.676	0.000	29.245	-
250	0.000	0.000	0.005	0.036	0.000	1.633	-
350	0.000	0.000	0.000	0.002	0.000	0.076	-
Σ	15.516%	12.158%	10.759%	10.246%	3.918%	47.404%	100.0%

Table 6 Magnitude Scaling Factors Used for this Calculation

Magnitude Mw	MSF
4.75	3.4
5.25	2.8
5.75	2.2
6.25	1.7
6.75	1.4
7.5	1.0

Note: see Figure 3

Table 7 Liquefaction and Partial Liquefaction Settlement Using CPTu CRR Stress Methodology

	Settlement (inch)					
SCPTu	M_W = 4.75 MSF = 3.4	M_W = 5.25 MSF = 2.8	M_W = 5.75 MSF = 2.2	M_W = 6.25 MSF = 1.7	M_W = 6.75 MSF = 1.4	M_W = 7.50 MSF = 1.0
Z-V2-CP5	0.655	0.913	1.084	1.158	1.319	2.068
Z-V2-CP6	0.000	0.000	0.000	0.038	0.174	1.035
Z-V2-CP7	0.000	0.000	0.000	0.050	0.266	1.417
Z-V2-CP8	0.013	0.018	0.031	0.150	0.409	1.941
Z-V2-CP9	0.000	0.000	0.006	0.080	0.297	1.582
Z-V2-CP10	0.005	0.000	0.000	0.013	0.089	0.569
Z-V2-CP15	0.000	0.000	0.009	0.124	0.449	2.237
Average Settlement (S) by Magnitude	0.096	0.133	0.161	0.230	0.429	1.550
Magnitude Weight (Σ) (from Table 5)	0.155	0.122	0.108	0.102	0.039	0.474
S x Σ	0.015	0.016	0.017	0.024	0.017	0.735
Weighted Average (inch)						0.82

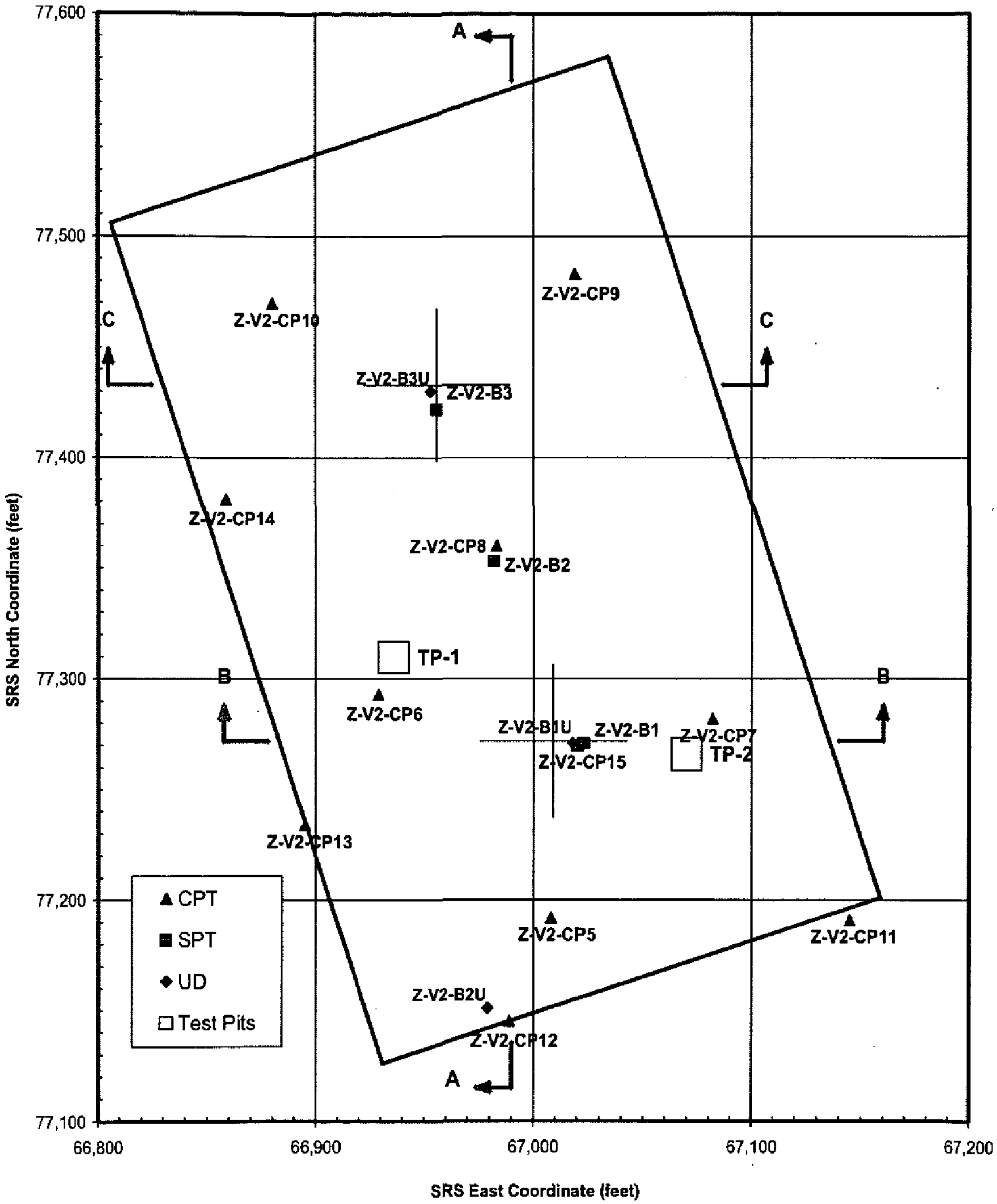


Figure 1 Geotechnical exploratory map

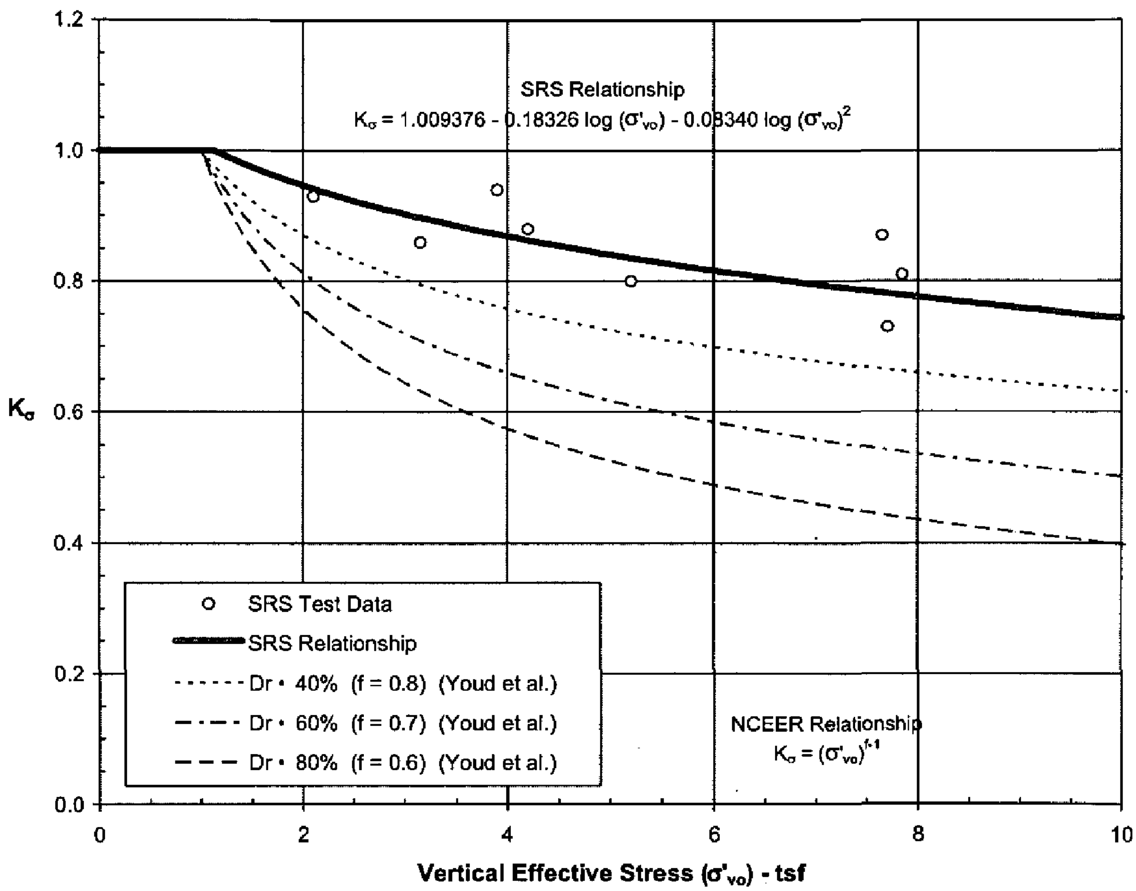


Figure 2 Comparison of effective overburden correction factors (Kσ)

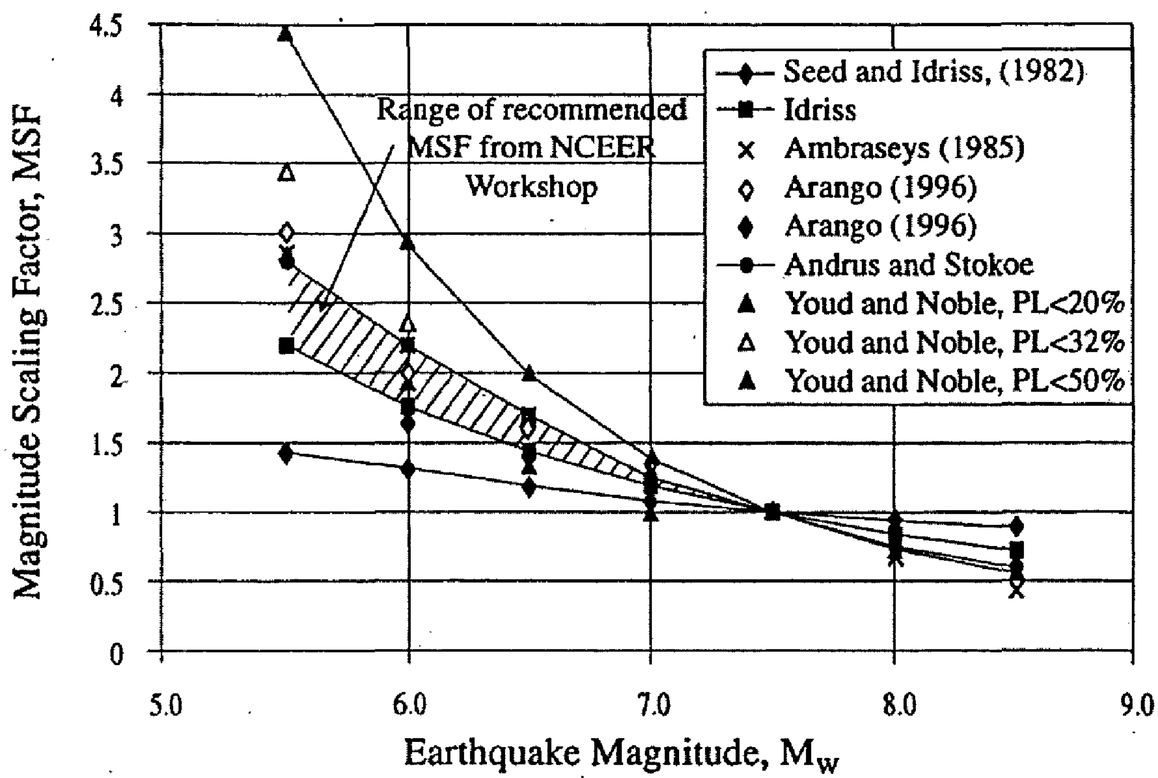


Figure 3 Magnitude scaling factors proposed by various Investigators with range recommended by NCEER workshop

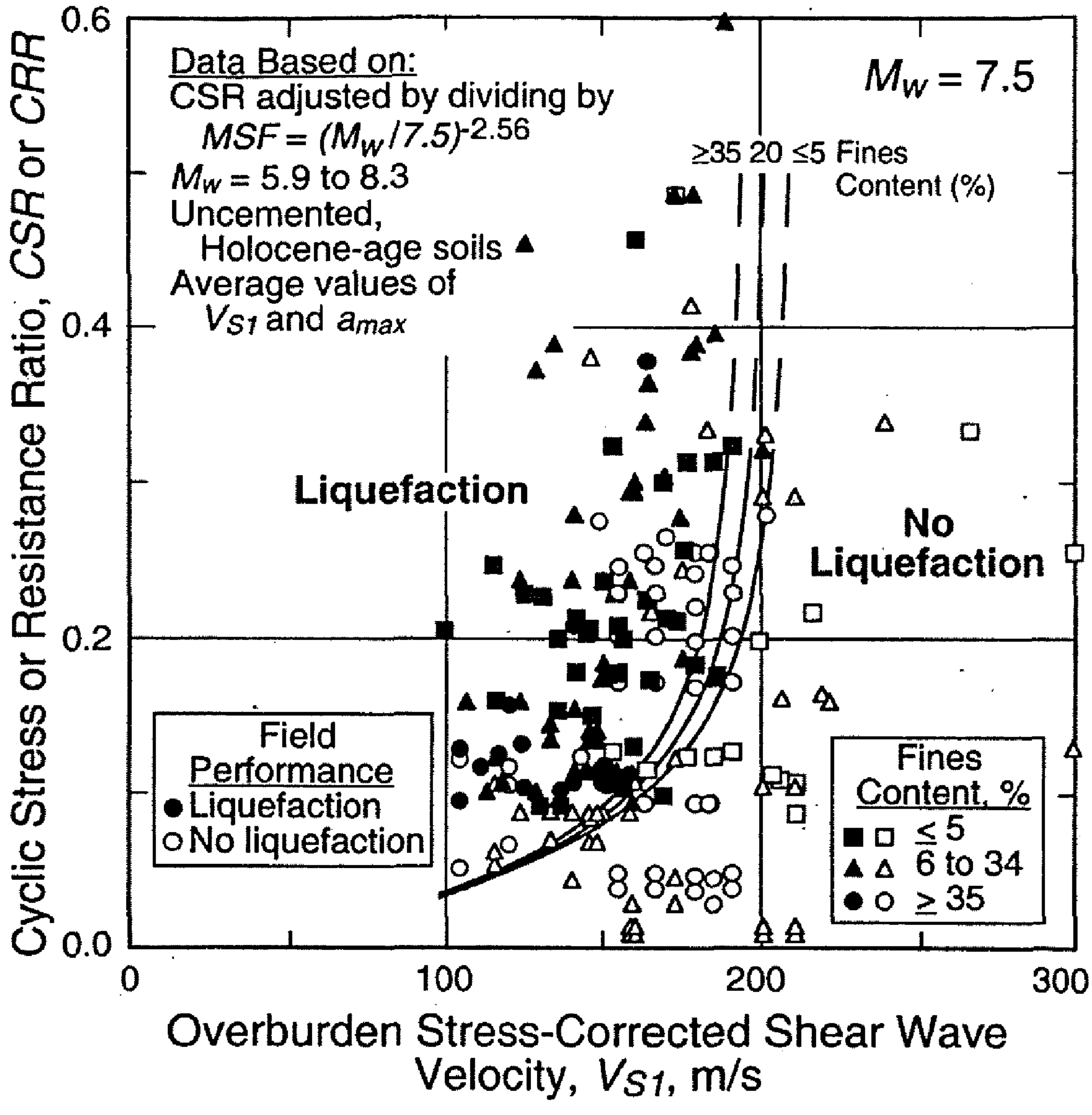


Figure 4 Relationship Between Cyclic Resistance Ratio (CRR) and Overburden Stress-Corrected Shear Wave Velocity (V_{s1})

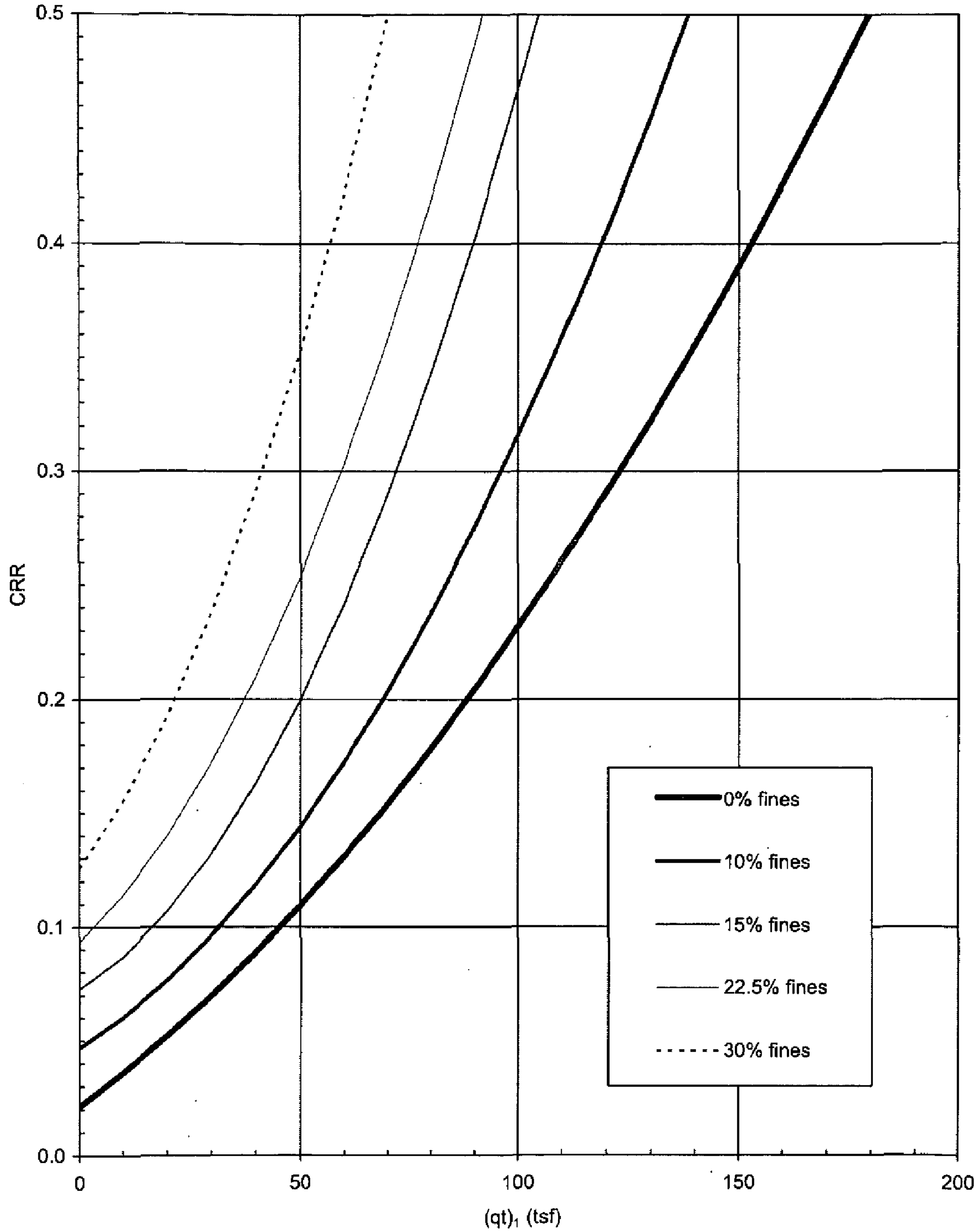


Figure 5 Relationship between cyclic resistance ratio (CRR) and normalized CPTu tip resistance (qt_1) for SRS

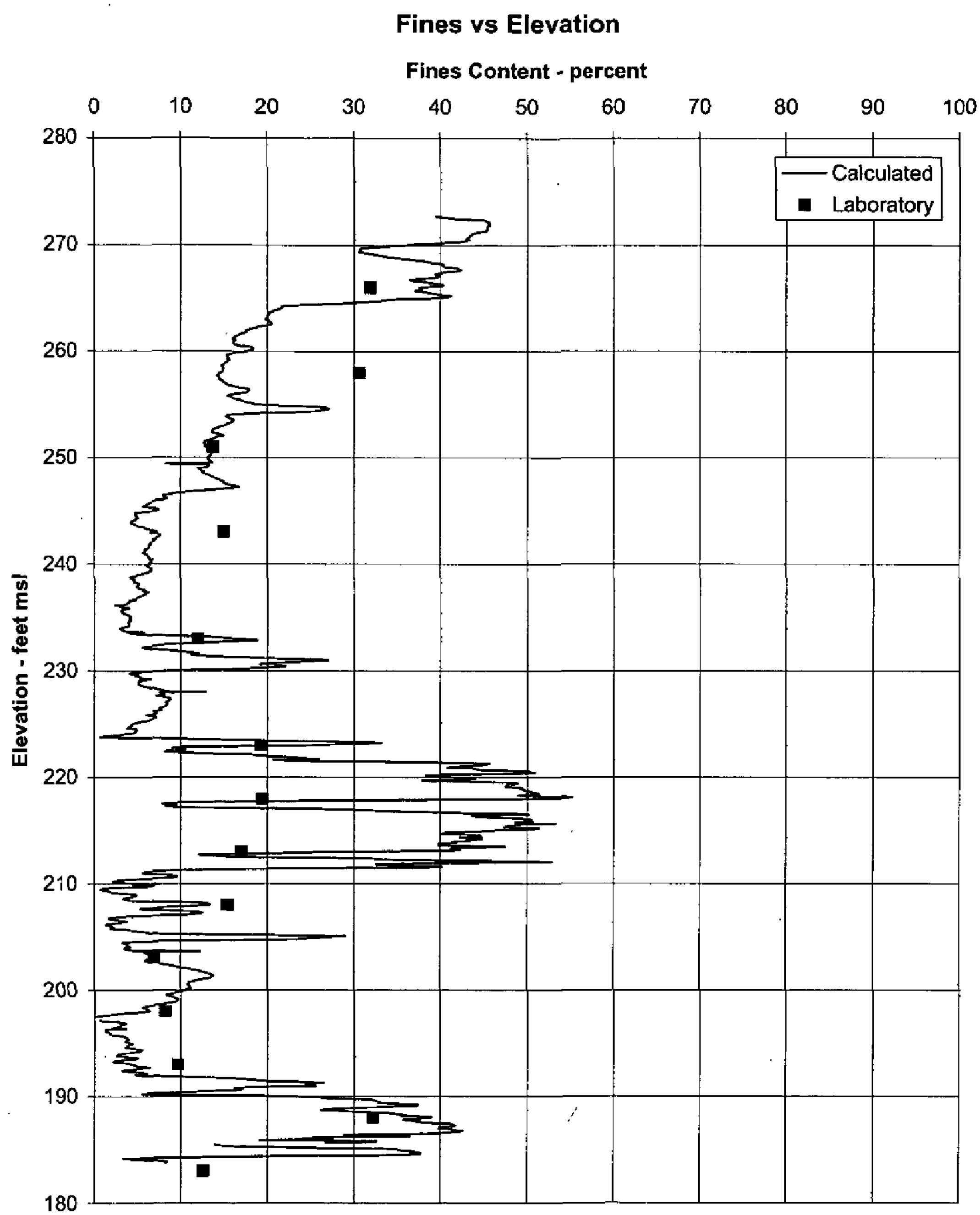


Figure 6 Comparison of fine contents estimated using CPTu data at Z-V2-CP8 with laboratory tested results from Boring Z-V2-B2

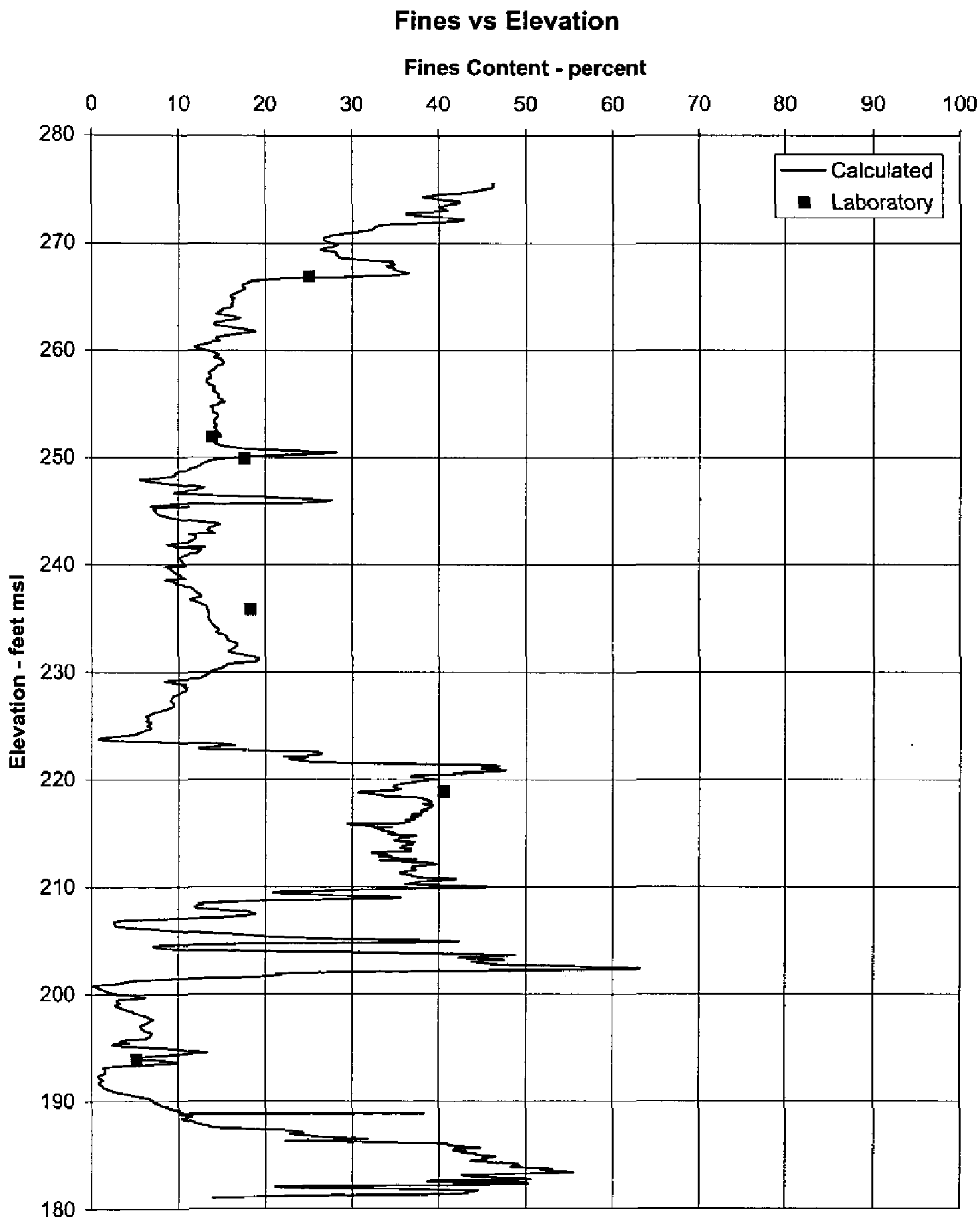


Figure 7 Comparison of fine contents estimated using CPTu data at Z-V2-CP15 with laboratory tested results from Boring Z-V2-B1U

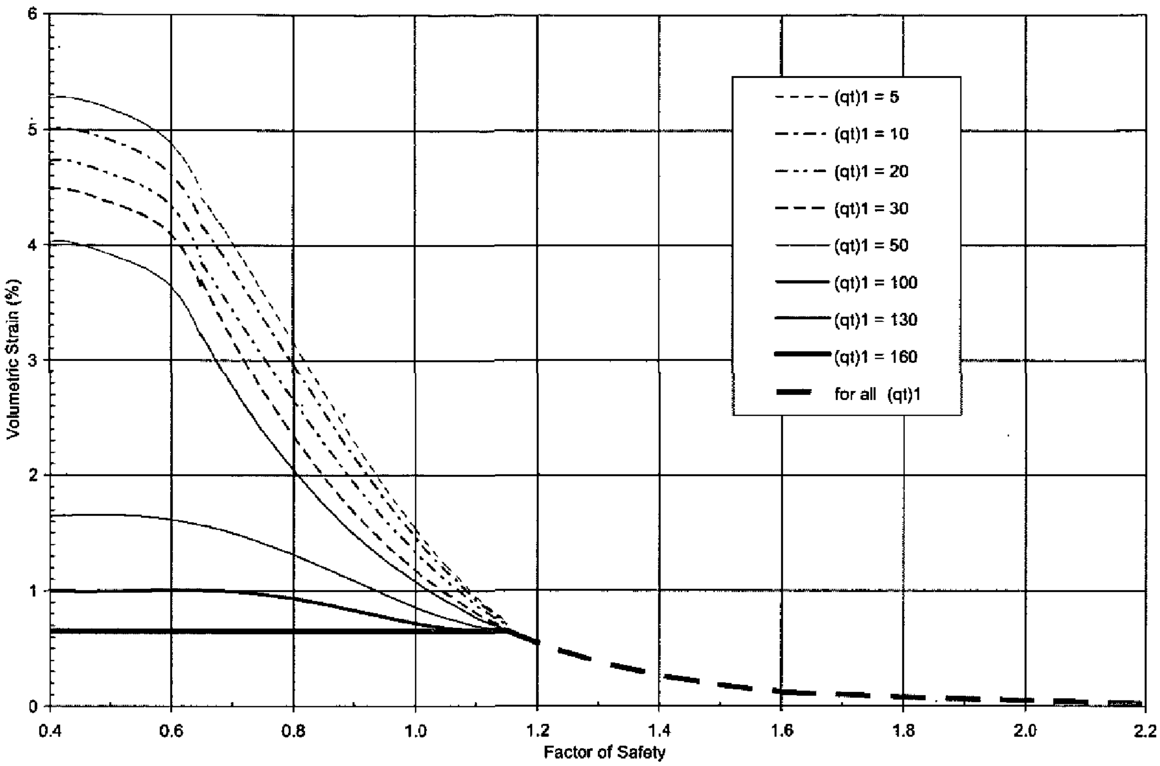


Figure 8 SRS Relationship between volumetric strain normalized CPTu tip resistance qt_1 and factor of safety against liquefaction

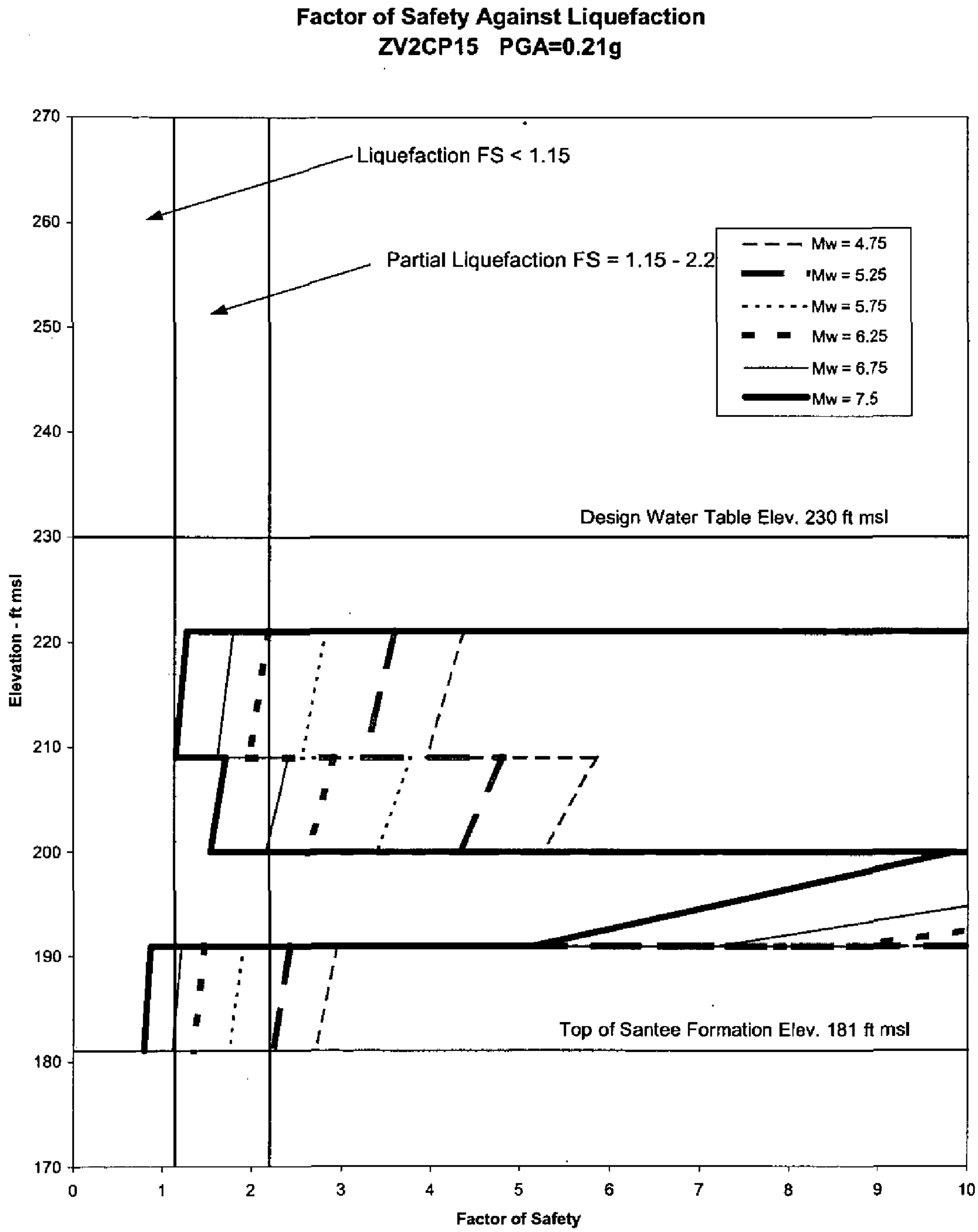


Figure 9 Factor of safety using shear wave velocity method and data from Z-V2-CP15

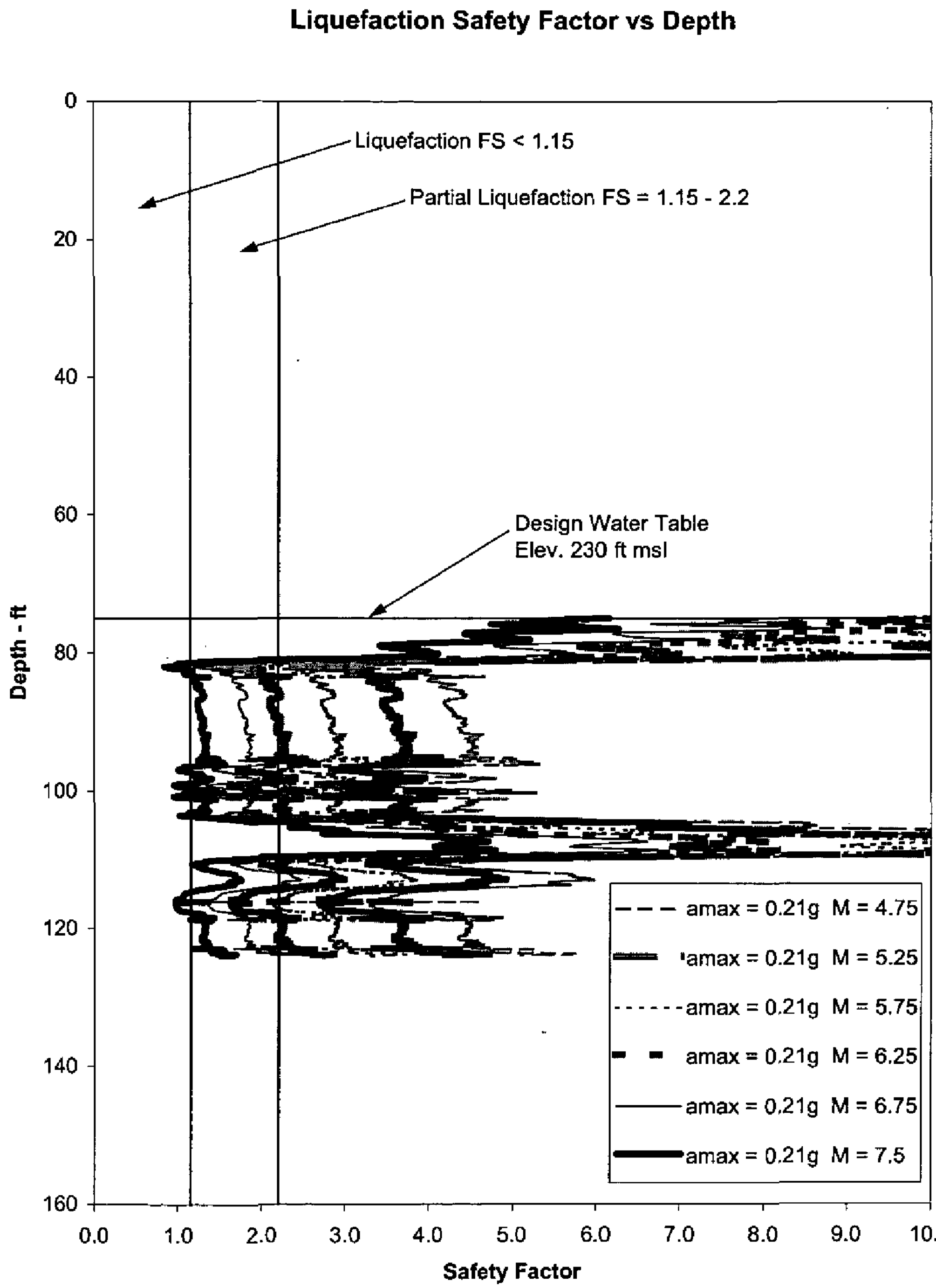


Figure 10 Factor of safety using CPTu tip resistance method and data from Z-V2-CP15

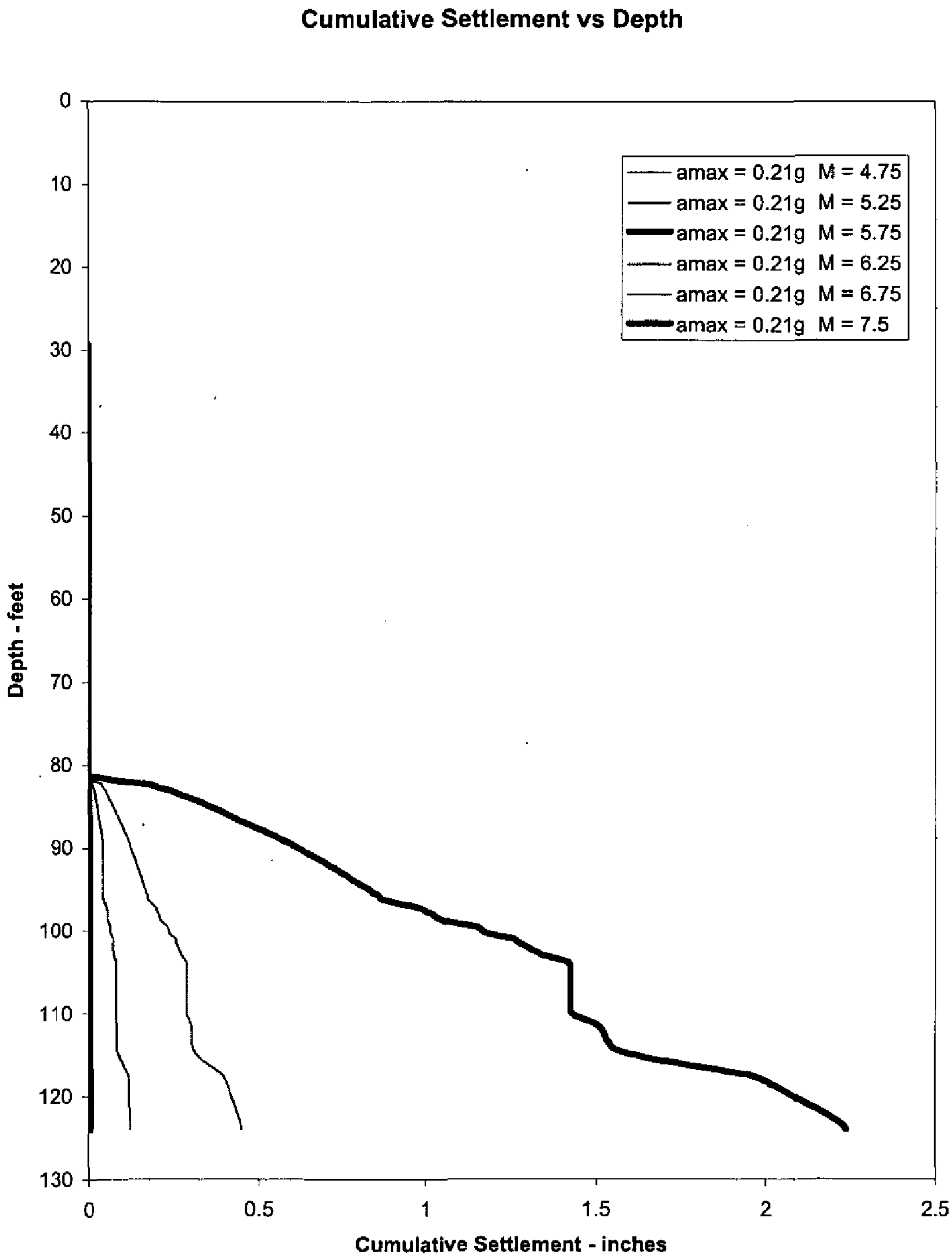
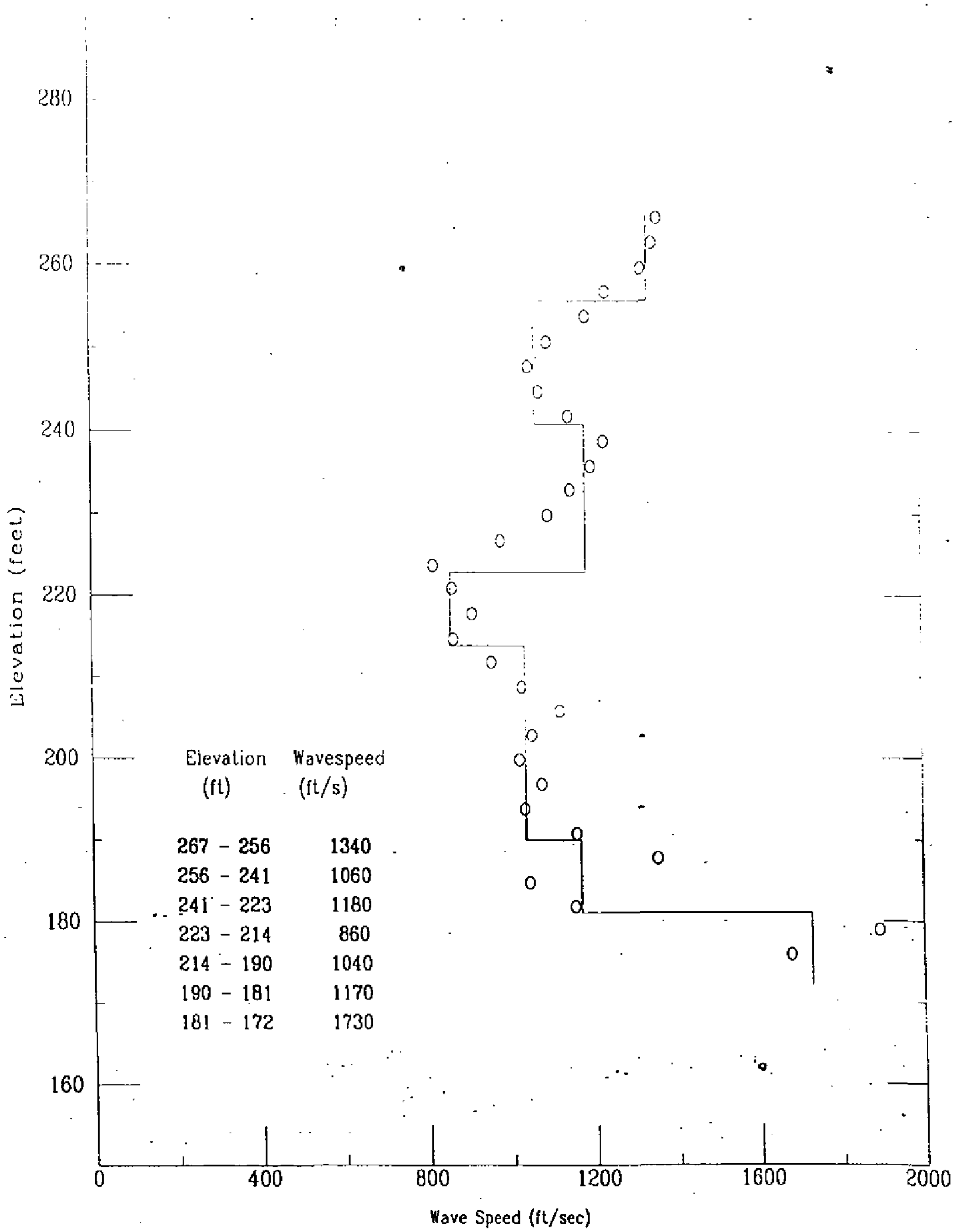


Figure 11 Settlement versus elevation for SCPTu location Z-V2-CP15 is presented

Attachment A SCPTu Data

ZV2 CP5 APPLIED RESEARCH ASSOCIATES, INC. 12/Apr/2005

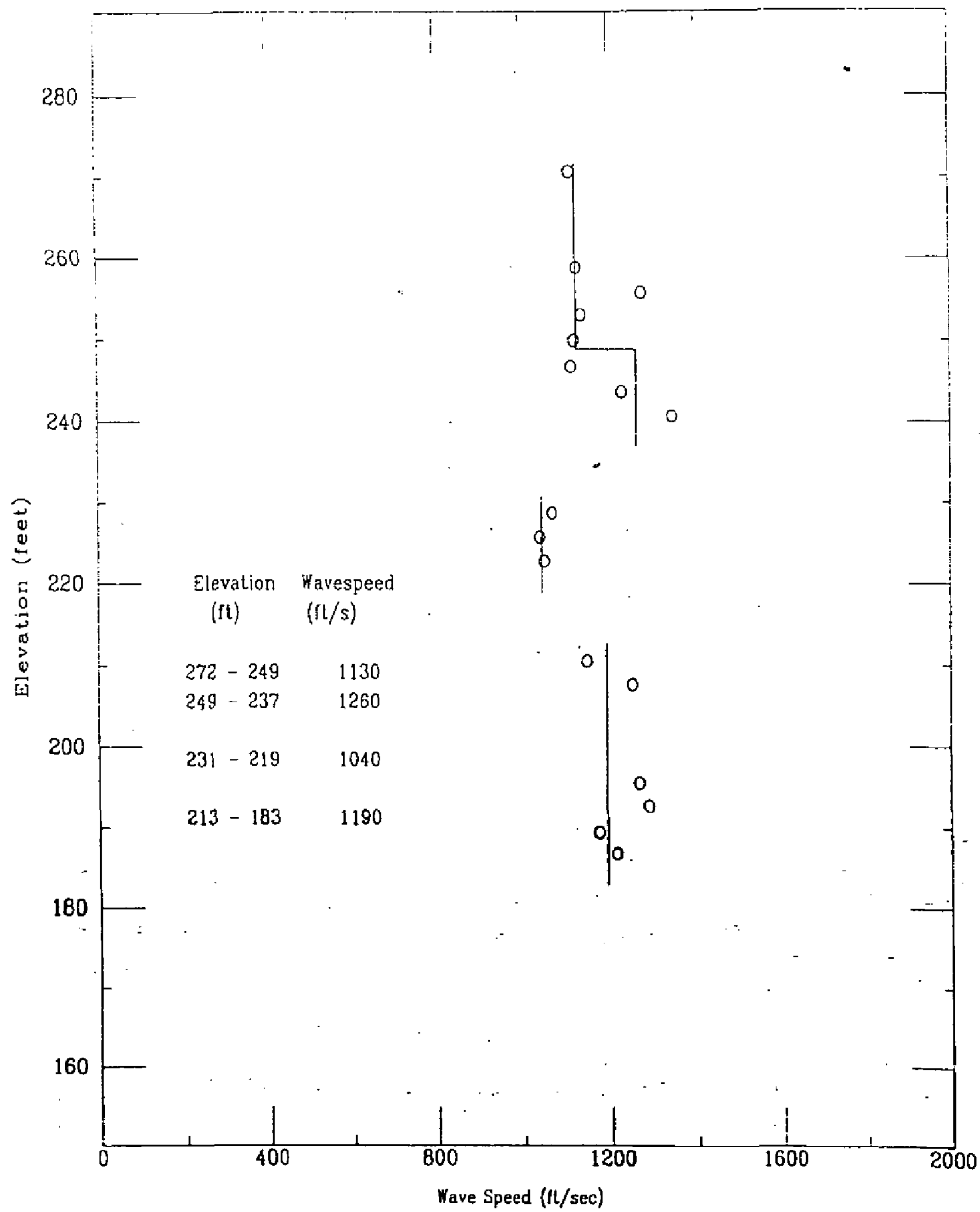
Shear Wave Speeds



File 212a0501

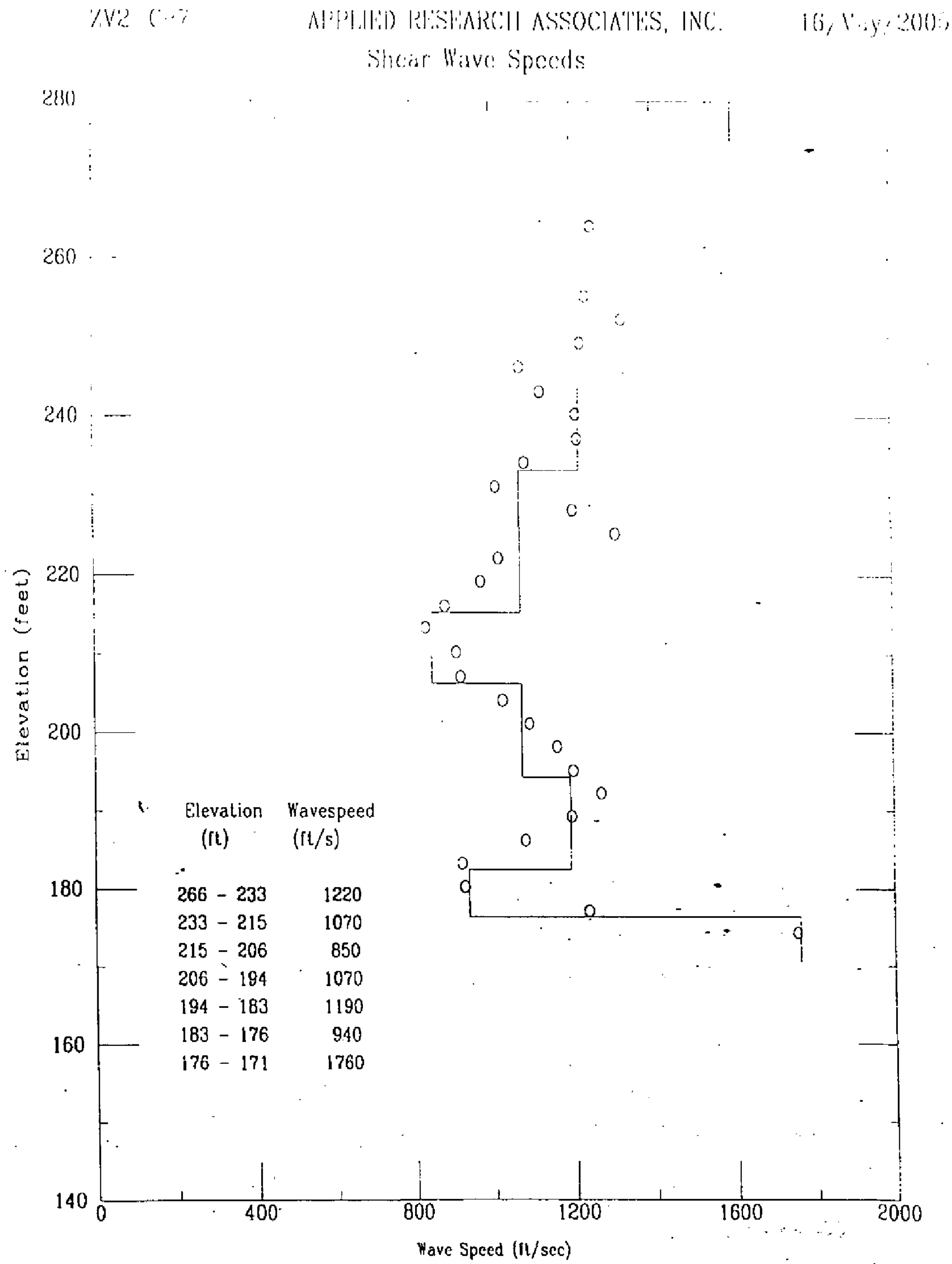
ZV2-CP6 APPLIED RESEARCH ASSOCIATES, INC. 17/May/2005

Shear Wave Speeds



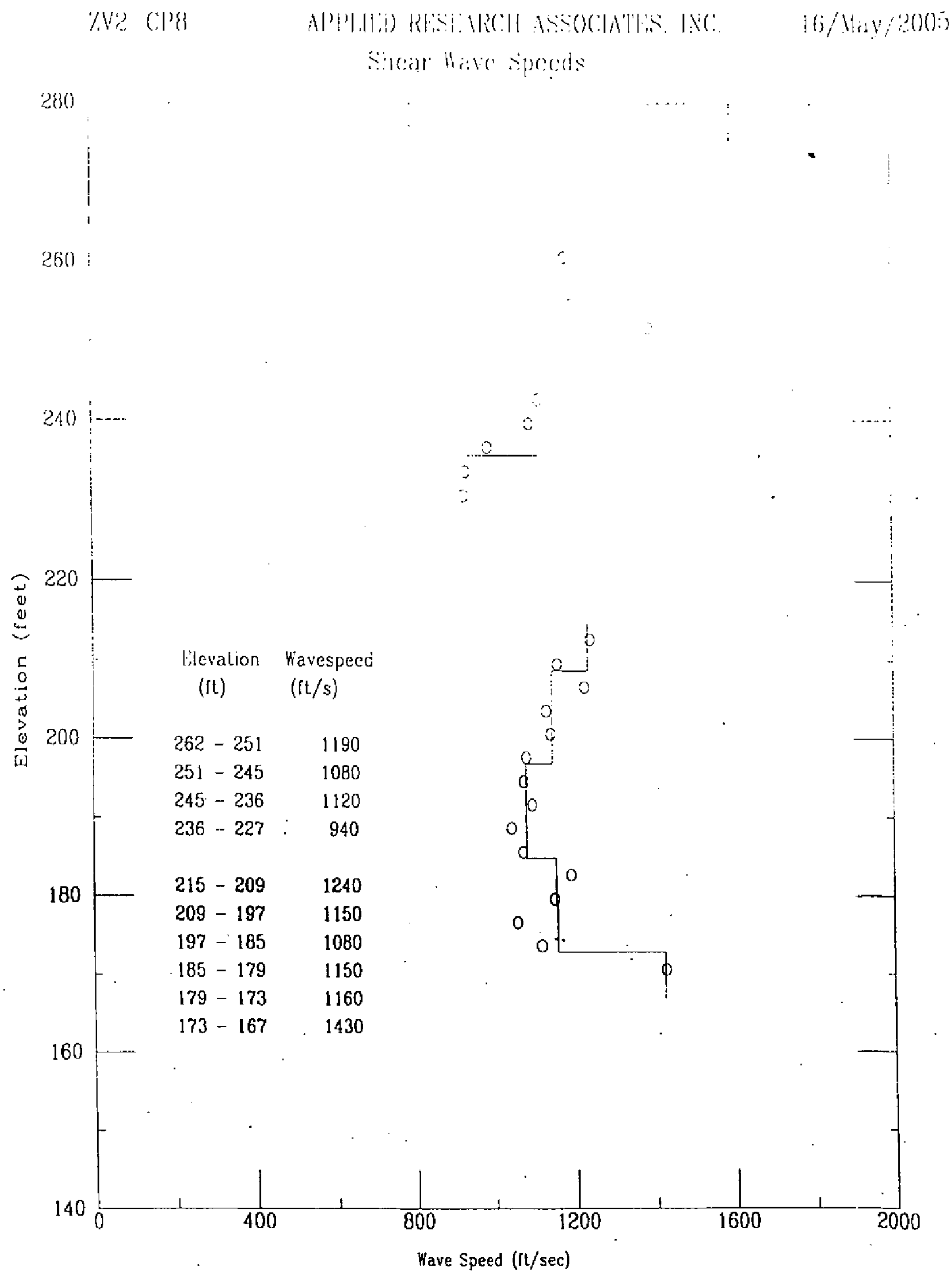
File 217y0501

Attachment A SCPTu Data



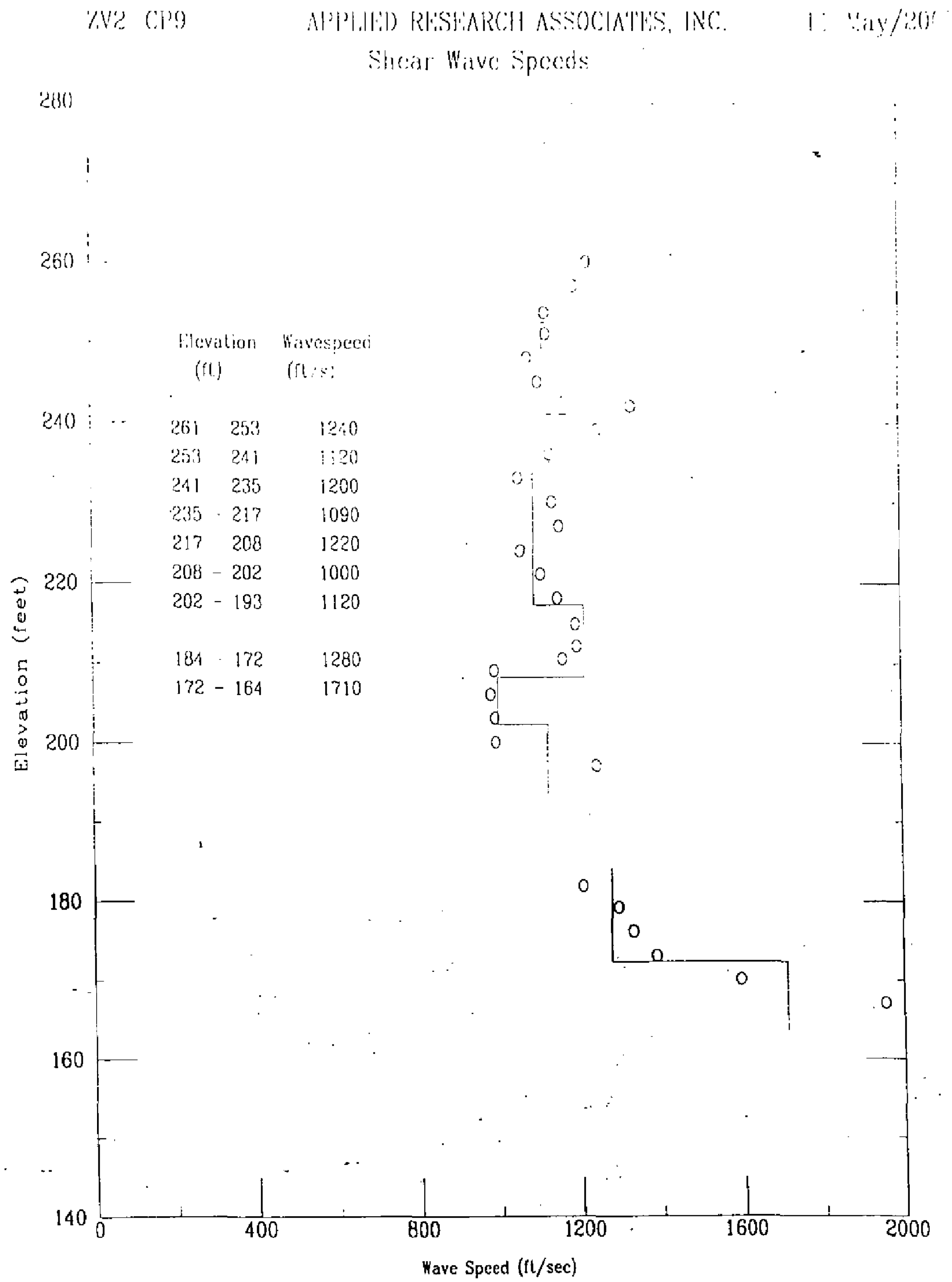
File 216y0502

Attachment A SCPTu Data



File 216y0501

Attachment A SCPTu Data

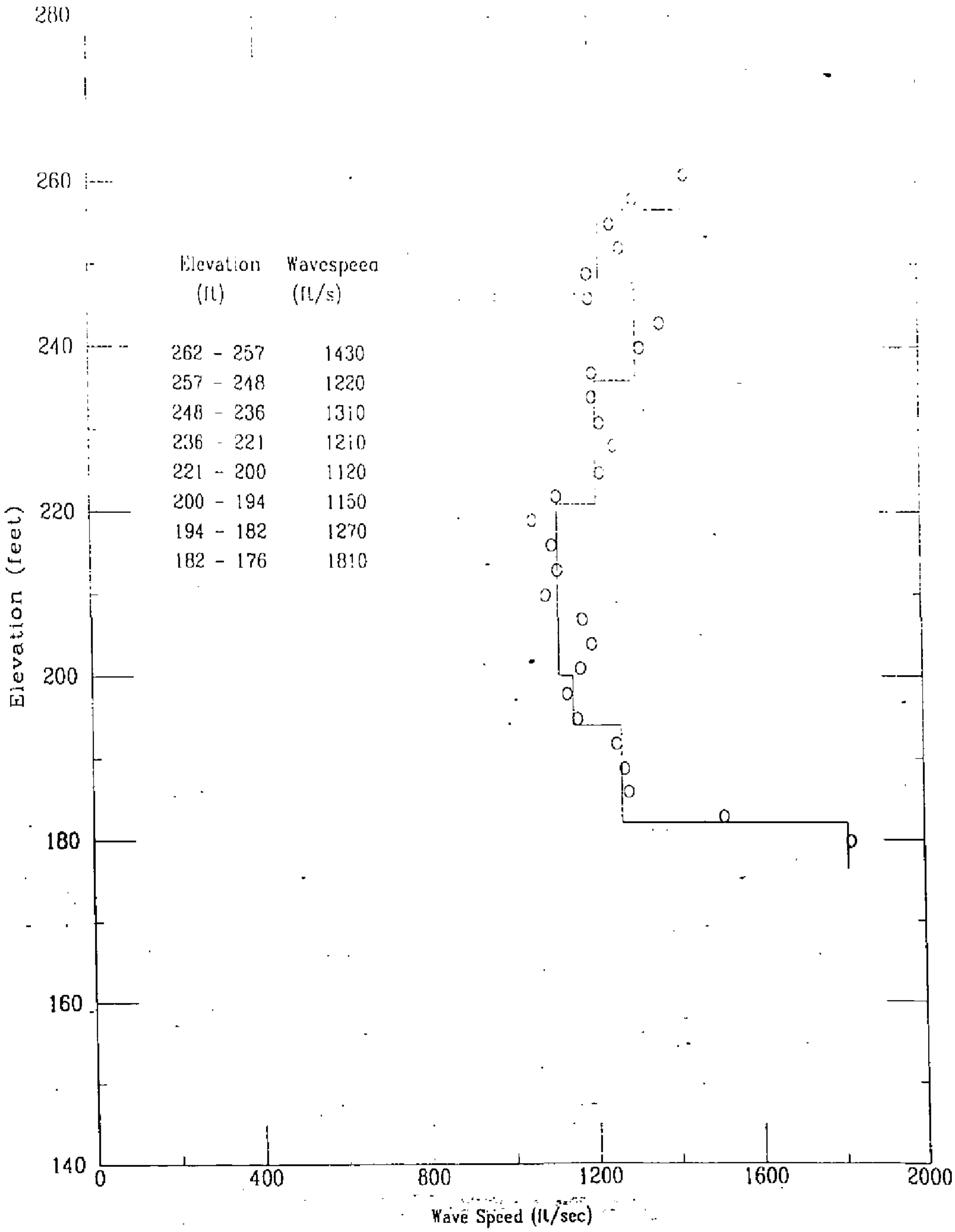


File 211y0501

Attachment A SCPTu Data

ZV2 CP10 APPLIED RESEARCH ASSOCIATES, INC. 13/May/2005

Shear Wave Speeds

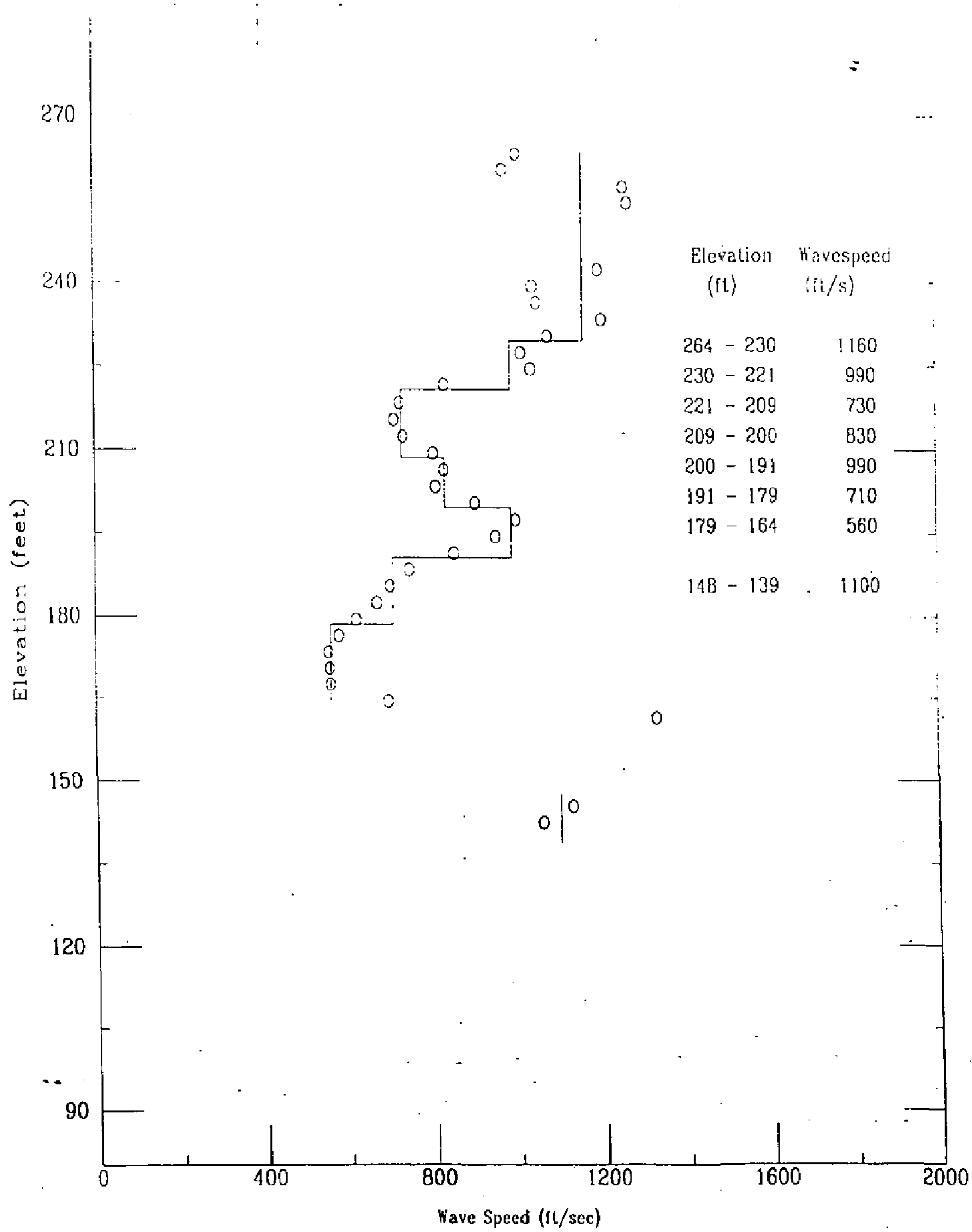


File 213y0501

Attachment A SCPTu Data

ZV2 CP15 APPLIED RESEARCH ASSOCIATES, INC. 16 May/2005

Shear Wave Speeds



File 216y0503

Attachment B Response Spectra based on International Building Code

Period T	Frequency	IBC
0.01	100.00	0.214
0.02	50.00	0.241
0.03	33.33	0.268
0.03333	30.00	0.277
0.04	25.00	0.295
0.05	20.00	0.323
0.06	16.67	0.350
0.07	14.29	0.377
0.08	12.50	0.404
0.09	11.11	0.431
0.10	10.00	0.459
0.103	9.71	0.467
0.515	1.94	0.467
0.54	1.85	0.445
0.75	1.33	0.320
1.0	1.00	0.240
1.5	0.67	0.160
2.0	0.50	0.120
2.5	0.40	0.096
3.0	0.33	0.080
4.0	0.25	0.060
5.0	0.20	0.048
10	0.10	0.024

Ss = 0.50g, from IBC Figure 1615(1) or USGS web site with Lat. 33.3 and Long. -81.63
S_1 = 0.17g, from IBC Figure 1615(2) or USGS web site with Lat. 33.3 and Long. -81.63
Fa = 1.40, from IBC Table 1615.1.2(1) Site Class D
Fv = 2.12, from IBC Table 1615.1.2(2) Site Class D

$S_{MS} = 0.70$ $S_{MS} = Fa \times Ss$ (IBC Equation 16-16)
$S_{M1} = 0.36$ $S_{M1} = Fv \times S_1$ (IBC Equation 16-17)
$S_{DS} = 0.467$ $S_{DS} = (2/3)S_{MS}$ (IBC Equation 16-18)
$S_{D1} = 0.240$ $S_{D1} = (2/3)S_{M1}$ (IBC Equation 16-19)
$T_0 = 0.103$ $T_0 = 0.2(S_{D1}/S_{DS})$ from IBC Section 1615.1.4
$T_S = 0.515$ $T_S = (S_{D1}/S_{DS})$ from IBC Section 1615.1.4

From IBC Section 1615.1.4:
$Sa = 0.6(S_{DS}/T_O)T + 0.4S_{DS}$ for periods less than or equal to T_O
$Sa = S_{DS}$ for periods greater than or equal to T_O and less than or equal to T_S
$Sa = S_{D1}/T$ for periods greater than T_S

PGA corresponds to zero period, or for this spectrum PGA is approximately 0.21g.

Earthquake Hazards Program Seismic Hazard Mapping Project

```
The ground motion values for the requested point:
LOCATION                 33.30 Lat.   -81.63 Long.
DISTANCE TO
  NEAREST GRID POINT     2.79401733965331 kms
NEAREST GRID POINT       33.30000  Lat.    -81.60000  Long.
Probabilistic ground motion values, in %g, at the Nearest Grid point are:
            10%PE in 50 yr   5%PE in 50 yr   2%PE in 50 yr
   PGA          7.566638        13.14327        23.69023
0.2 sec SA      17.18540        28.02243        50.20057
0.3 sec SA      13.55839        23.43204        40.36540
1.0 sec SA      5.531499        9.420448        17.34955
- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
```

The program has detected a zero latitude and has assumed the end of valid input data.

PROJECT INFO: Home Page
SEISMIC HAZARD: Hazard by Lat/Lon

United States Department of the Interior

U.S. GEOLOGICAL SURVEY

Arthur Frankel
U.S. Geological Survey
MS 966, Box 25046
Denver Federal Center
Denver, CO 80225
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

Attachment C Letter from Frankel to Lee, March 1, 1999

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.

/

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

Attachment D List of computer files

Computer files from ARA containing CPT data (i.e., tip stress, sleeve stress, pore pressure, etc.) are listed below:

ZV2-CP5.csv	7/18/2005	08:48a	171,163
ZV2-CP6.csv	7/18/2005	08:49a	180,227
ZV2-CP7.csv	7/18/2005	08:49a	204,572
ZV2-CP8.csv	7/18/2005	08:50a	213,040
ZV2-CP9.csv	7/18/2005	08:51a	218,434
ZV2-CP10.csv	7/18/2005	08:51a	185,683
ZV2-CP15.csv	7/18/2005	08:54a	253,763

Excel spreadsheets created to perform the calculations are listed below:

ZV2CP5-liq.xls	2/15/2006	01:53p	4,265,984
ZV2CP6-liq.xls	2/15/2006	01:56p	4,534,784
ZV2CP7-liq.xls	2/15/2006	02:07p	5,036,544
ZV2CP8-liq.xls	2/21/2006	05:04p	4,835,840
ZV2CP9-liq.xls	2/15/2006	02:13p	5,500,928
ZV2CP10-liq.xls	2/15/2006	02:15p	4,422,656
ZV2CP15-liq.xls	3/13/2006	04:05p	5,137,920

zv2cp5-Vs.xls	3/10/2006	02:42p	543,232
zv2cp6-Vs.xls	3/10/2006	04:51p	570,368
zv2cp7-Vs.xls	3/10/2006	01:18p	611,840
zv2cp8-Vs.xls	3/10/2006	04:24p	585,728
zv2cp9-Vs.xls	3/10/2006	03:39p	673,792
zv2cp10-Vs.xls	3/10/2006	01:13p	557,568
zv2cp15-Vs.xls	3/10/2006	01:12p	675,328

Appendix C

K-CLC-Z-00006, Soil Properties for the Design of Vault No. 2

Rev. 0, August 2005

(58 pages)

OSR 45-24# (Rev. 1-10-2000)

Calculation Cover Sheet

Project N/A	Calculation Number K-CLC-Z-00006	Project Number
Title Soil Properties for the Design of Vault No. 2 (U)	Functional Classification PS	Sheet 1 of 57
	Discipline Geotechnical	

☐ Preliminary ☒ Confirmed

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

Purpose and Objective

Provide the soil properties for the design of the vaults

Summary of Conclusion

See last section.

ENGINEERING DOC. CONTROL-SRS

00798857

UNCLASSIFIED
DOES NOT CONTAIN UNCLASSIFIED CONTROLLED NUCLEAR INFORMATION
ADC & Reviewing Official [signature]
(Name and Title)
Date: 8-15-05

Revisions

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	[signature] W. Li	Document review	[signature] 7-28-05 M. McHood	[signature] M. Lewis

Release to Outside Agency – Design Authority (Print)	Signature	Date

Security Classification of the Calculation

Unclassified

TABLE OF CONTENTS

1. INTRODUCTION

This calculation provides selected soil properties for the design of Saltstone Vault No. 2 (Ref. 1). Figure 1 shows the layout of the project site along with the geotechnical exploratory locations.

The foundations of the vaults will be at elevation 268 feet, MSL, approximately. Soils to be excavated will be used as the fill around the vaults.

2. INPUT DATA

The input data include:

2.1 Field Exploration

Type of Exploration	I.D.	North Coordinate (feet)	East Coordinate (feet)	Ground Elevation (feet MSL)	Total Depth (feet)
SCPTU	Z-V2-CP5	77,192	67,008	287.0	114.0
SCPTU	Z-V2-CP6	77,293	66,929	283.0	107.0
SCPTU	Z-V2-CP7	77,282	67,082	279.4	110.0
SCPTU	Z-V2-CP8	77,360	66,983	278.8	113.0
SCPTU	Z-V2-CP9	77,483	67,019	275.3	111.5
SCPTU	Z-V2-CP10	77,470	66,880	279.0	103.0
CPTU	Z-V2-CP11	77,191	67,145	283.0	40.0
CPTU	Z-V2-CP12	77,145	66,989	289.7	40.0
CPTU	Z-V2-CP13	77,234	66,895	286.2	40.0
CPTU	Z-V2-CP14	77,381	66,859	280.8	40.0
SCPTU	Z-V2-CP15	77,270	67,020	281.7	143.5
SPT Boring	Z-V2-B1	77,272	67,024	281.7	132.0
SPT Boring	Z-V2-B2	77,353	66,983	279.0	132.0
SPT Boring	Z-V2-B3	77,422	66,957	278.1	132.0
UD Boring	Z-V2-B2U	77,151	66,979	289.7	36.0
Test Pit	TP-1	77,311	66,936	281.5	12.0
Test Pit	TP-2	77,266	67,070	280.9	12.0

Appendix A provides the CPT logs (Ref. 2). Appendix B provides the borehole logs.

2.2 Dissipation Tests

CPT I.D.	Ground Elevation (feet, MSL)	Test Depth (feet)	Test Elevation (feet, MSL)	Pressure (psi)	Groundwater Elevation (feet, MSL)
Z-V2-CP5	287.0	76.1	210.9	3.80	219.7
Z-V2-CP9	275.3	62.0	213.3	7.21	229.9

Dissipation test reports are included in Appendix A, CPT logs.

2.3 Laboratory Tests

1) Index tests (Ref. 3):

Borehole or Test Pit No.	Sample No.	Ground Elevation (ft, msl)	Depth from (feet)	Depth to (feet)	Elevation from (ft, msl)	Elevation to (ft, msl)
Z-V2-B2U	ST1	289.7	6.0	8.0	283.7	281.7
Z-V2-B2U	ST2	289.7	14.0	16.0	275.7	273.7
TP-1	Bag 4	281.5	2.0	4.0	279.5	277.5
TP-1	Bag 6	281.5	4.0	6.0	277.5	275.5
TP-1	Bag 8	281.5	6.0	8.0	275.5	273.5
TP-1	Bag 10	281.5	8.0	10.0	273.5	271.5
TP-1	Bag 12	281.5	10.0	12.0	271.5	269.5
TP-1	Composite	281.5	2.0	12.0	279.5	269.5
TP-2	Bag 4	280.9	2.0	4.0	278.9	276.9
TP-2	Bag 6	280.9	4.0	6.0	276.9	274.9
TP-2	Bag 8	280.9	6.0	8.0	274.9	272.9
TP-2	Bag 10	280.9	8.0	10.0	272.9	270.9
TP-2	Bag 12	280.9	10.0	12.0	270.9	268.9
TP-2	Composite	280.9	2.0	12.0	278.9	268.9

2) Modified proctor tests (Ref. 3):

Test Pit I.D.	Sample	Ground Elev. (ft, MSL)	Depth From (feet)	Depth To (feet)	Elev. From (ft, MSL)	Elev. to (ft, MSL)	Max. Density (pcf)	Optimal Water Content (%)
TP-1	Composite	281.5	2.0	12.0	279.5	269.5	124.2	10.2
TP-2	Composite	280.9	4.0	12.0	276.9	268.9	123.4	10.7

3) Unit weights from undisturbed soil samples above elevation 268 (Ref. 4):

Bore hole I.D.	Sample I.D.	Depth from (feet)	Depth to (feet)	Elev. from (ft, MSL)	Elev. to (ft, MSL)	Test No.	Dry unit weight (pcf)	Moisture contents (%)	Wet unit weight (pcf)
Z-V2-B2U	ST1	6.0	8.0	283.7	281.7	1	104.4	20.2	125.5
						2	105.8	19.8	126.7
						3	111.6	16.1	129.6
Z-V2-B2U	ST2	14.0	16.0	275.7	273.7	1	111.3	15.7	128.8
						2	116.1	14.1	132.5
Average							109.8	17.2	128.6

4) Triaxial strength test results on undisturbed soil samples above elevation 268 (Ref. 4).

Bore hole I.D.	Sample I.D.	Depth From (feet)	Depth To (feet)	Elev. From (ft, MSL)	Elev. to (ft, MSL)	Total Friction Angle (degree)	Total Cohe-sion (psf)	Eff. Friction Angle (degree)	Eff. Cohe-sion (psf)
Z-V2-B2U	ST1	6.0	8.0	283.7	281.7	54.0	300	33.4	250
Z-V2-B2U	ST2	14.0	16.0	275.7	273.7	35.0	1,700	30.0	380
Z-V2-B2U	ST3	28.0	30.0	261.7	259.7	36.8	250	33.3	50
Z-V2-B2U	ST4	34.0	36.0	255.7	253.7	26.6	250	32.0	260

Appendix C provides the laboratory test reports.

3. COMPUTATIONS

3.1 Groundwater Elevation

Ground water elevation was obtained from Reference 5 (Appendix D) and verified by dissipation test results. The groundwater elevation at the project site is estimated to be 225 ft, MSL. A seasonal groundwater fluctuation is estimated to be ± 5 feet.

3.2 Unit Weight of Fills

Soils excavated above 268 feet, MSL will be backfilled and compacted. Based on the proctor test results on remolded samples and the evaluation of the density test results on undisturbed samples, wet density of the compacted fill is estimated to be 125 pcf.

3.3 Lateral Earth Pressure Coefficients of the Fills

A friction angle (ϕ) of 30 degrees is estimated for the compacted fill. Based on the review of the index tests as well as the strength test results, a friction angle of 30 degrees for the fill appears conservative. Using the friction angle of 30 degrees, the lateral earth pressure coefficients for the fill are:

Active earth pressure coefficient, $k_a = \tan^2(45 - \phi/2) = 0.33$

Passive earth pressure coefficient, $k_p = \tan^2(45 + \phi/2) = 3.0$

At-rest earth pressure coefficient, $k_o = 1 - \sin(\phi) = 0.5$

3.4 Subgrade Modulus

The subgrade modulus is a problem-specific observed result and is not a soil property. The subgrade modulus depends on the size of the loaded area and needs to be properly evaluated in each particular application. This calculation provides subgrade moduli to be used for the following two types of applications:

(1) Loading over large areas:

Reference 7 provides the modulus of subgrade reaction using the settlement data from the Vitrification Building, a range of subgrade modulus k_s between approximately 13 and 21 pci were calculated. Since the proposed vault is 28 percent wider than the Vitrification Building, subgrade modulus for computing the elastic settlement of the proposed vault would be less.

Based on settlement data from large structures a best estimate range of subgrade modulus k_s is between 10 to 20 pci. However, these values should not be used for point loads or smaller loading areas when the soil elastic deformation is limited within a shallow depth. For example, using the above subgrade modulus, an over-conservative immediate settlement of 1 to 2 inches would be calculated for a parked H-20 truck.

(2) Loading over smaller areas:

During the design of the slab, when the stress of a slab is computed, the size of the loaded area under the slab is computed simultaneously. The loaded area is relatively small, no more than several feet (Ref. 8, pp. 247). The elastic deformation of the soil reaches to a shallow depth only. Elastic properties of deep soil do not affect the subgrade modulus used in this type of computation, neither the stress of the slab.

Subgrade modulus for soils at elevation 268 feet, MSL will be used to design the foundation of the vaults. Subgrade modulus at the project site was estimated based on soil classification, CPTU tip stress and friction ratio, as well as SPT blow count and CPTU tip stress.

Based on the soil classification at the project site, majority of the soils are SC. The range of the subgrade modulus is between 200 and 250 pci (Ref. 6 and Appendix E).

Average CPT tip stresses and friction ratios below the proposed foundation elevation were used to estimate the subgrade modulus (Ref. 6 and Appendix E). Average values of soils considering various thicknesses below the proposed foundation were used. The estimated subgrade moduli are:

Thickness below Foundation (feet)	Range of Elevations (Feet, MSL)	Average Tip Stress q_c (tsf)	Average Tip Stress q_c (kPa)	Average Friction ratio R_f (%)	$q_c/100R_f$ (kPa/%)	Subgrade Modulus K_1 (pci)
10	268 to 258	110.6	10,594	1.72	61.6	246
20	268 to 248	122.9	11,772	1.36	86.6	252
30	268 to 238	120.5	11,542	1.10	104.9	256
40	268 to 228	155.5	14,895	1.14	130.7	263
50	268 to 218	147.9	14,167	1.24	114.2	258

Average SPT blow counts and CPT tip stresses were also used to estimate the subgrade modulus (Ref. 6 and Appendix E). Measured blow counts were converted to energy-corrected blow counts N_{55}, corresponding to 55% of input energy using:

$$N_{55} = (ER/55)\, N_{MEASURED}$$

where ER, the energy ratio per ASTM D 4633, is 76% (Appendix B).

Subgrade moduli estimated based on average blow counts considering various thicknesses under the proposed foundation are:

Thickness below Foundation (feet)	Range of Elevations (Feet, MSL)	Average Tip Stress q_c (tsf)	Average Tip Stress q_c (kPa)	Average Blow Count N_{55}	$q_c/100N_{55}$ (kPa)	Subgrade Modulus K_1 (pci)
10	268 to 258	110.6	10,594	32	3.3	205
20	268 to 248	122.9	11,772	32	3.7	215
30	268 to 238	120.5	11,542	30	3.8	218
40	268 to 228	155.5	14,895	32	4.7	244
50	268 to 218	147.9	14,167	31	4.6	241

Average SPT blow counts were also used based on the following relation (Ref. 8):

$k_1 = 6\ N$

where k_1 is the subgrade modulus in tsf. Based on average N_{55} using N_{60} of 29, k_1 is computed to be 201 pci. Based on soil classification, CPTU tip stress and friction ratio, SPT blow count and CPTU tip stress, as well as SPT blow counts the range of subgrade modulus for a 1' x 1' plate, K_1 is between 200 pci to 260 pci.

Based on the above computation, the range of k_1 is between 200 and 260 pci. To find subgrade modulus for a 30 inch plate as required by the ACI code (Ref. 9) the following equation is used:

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

where B is the width of the load (30 inches or 2.5 feet in this case). Therefore,

$k_{2.5} = 0.49\ k_1$

Using the ACI Code for design, the range of subgrade modulus, $K_{2.5}$ (i.e., subgrade modulus for a 30 inch plate) is between about 100 pci to 130 pci. This applies to point loads or loads on smaller areas.

3.5 Bearing Capacity

The allowable bearing capacity for the proposed and existing structures were calculated based on modifications to the general Terzaghi bearing capacity equation (Ref. 6):

$q_u = cN_cS_cD_c + 1/2B\gamma'_H\ N_\gamma\ S_\gamma\ D_\gamma + \gamma'_D D\ N_q\ S_q\ D_q$

Where N_c, N_q, and N_γ are the bearing capacity factors; S_c, S_q, and S_γ are the shape factors; and D_c, D_q, and D_γ are the depth factors. It should be noted that the allowable bearing capacities given within this section does not consider the settlement.

Using the general equations for these variables as given in the reference; conservative soil properties of $\phi = 32^\circ$, c = 50 psf, and γ = 120 pcf; and dimension D = 0, B = L = 133 feet:

$N_c = \cot\phi\ (N_q - 1) = 35.49$

$N_q = e^{\pi \tan\phi} \tan^2 (\pi/4 + \phi/2) = 23.18$

$N_\gamma = 2(N_q + 1) \tan\phi \tan (\pi/4 + \phi/5) = 37.85$

Using the equation for bearing capacity factor N_γ suggested by Hansen in 1970:

$N_\gamma = 1.5\ (N_q - 1) \tan\phi = 20.79$

In calculating the bearing capacity, the smaller N_γ of 20.79 was use to compute the bearing capacity.

Shape factors S_c, S_q, and S_γ are:

$S_c = 1 + (B/L)\ (N_q / N_c) = 1.653$

$S_q = 1 + (B/L) \tan\phi = 1.625$

$S_\gamma = 1 - 0.4\ B/L = 0.6$

Depth factors D_c, D_q, and D_γ are:

$D_c = 1 + 0.4K = 1.000$

$D_q = 1 + 2 \tan\phi\ (1 - \sin\phi)^2\ K = 1.000$

$D_\gamma = 1.000$

where $K = D/B$ if $D \leq B$

$K = \tan^{-1} (D/B)$ if $D > B$

For this site, the groundwater table is at an elevation below the foundation but within the influence zone of some of the foundation system. Therefore, based on the reference, a factored value of the unit weight, termed the effective unit weight, should be used in the portions of the general equation affected by the groundwater table. In cases where the groundwater table influences the failure zone, the effective unit weight is given by:

$\gamma_H = \gamma_{HSUB} + \gamma_W * (Z_W/2*B)$

where: γ_{HSUB} = submerged soil unit weight

γ_W = unit weight of water

Z_W = depth from foundation to groundwater table

A factor of safety of 3 is used to compute the allowable bearing capacity q_a:

$q_a = q_u/3 = 20$ ksf

Therefore, the soil at elevation 268 feet, MSL will be able to support the foundation pressure of 3,500 psf with sufficient factor of safety.

4. RESULTS AND CONCLUSION

The results are summarized as follows:

- Average groundwater table elevation is 225 feet mean sea level with seasonal fluctuation of ± 5 feet. Areas of perched water can be expected to occur above the groundwater table elevations.
- Wet density of the fill is 125 pcf.
- Lateral earth pressure coefficients for the fill are:
  - Active earth pressure coefficient, $k_a = 0.33$
  - Passive earth pressure coefficient, $k_p = 3.0$
  - At-rest earth pressure coefficient, $k_o = 0.5$
- Modulus of subgrade reaction k_s depends on the size of the loaded area; 10 to 20 pci for the full 150 ft diameter foundation and 100 to 130 pci for a 30-inch diameter plate.
- Soils at elevation 268 feet, MSL will support the foundation pressure of 3,500 psf with sufficient factor of safety.

5. REFERENCES

1. M-TC-Z-00004, Saltstone Facility Cylindrical Vault #2 Project, Bldg 451-002Z (U).
2. Subcontract No. AC39054N, Task 3, Applied Research Associates, Inc., Cone Penetrometer Data Report, July 2005.
3. Subcontract No. AB80188N, Task 13, GeoTesting Express Laboratory Test Report, June 2005.
4. Subcontract No. AB80187N, Task 8, QORE Laboratory Test Report, June 2005.
5. WSRC-TR-98-0045, *The Regional Water Table of the Savannah River Site and Related Coverages*, September 1998.
6. Fang, Foundation Engineering Handbook, 2nd Edition, 1991.
7. K-CLC-S-00008, GWSB#2 Facility Settlement and Bearing Capacity, Rev. 0, June 2002
8. Scott, R. F., Foundation Analysis, Prentice-Hall, 1981.
9. American Concrete Institute, *Design of Slabs on Grade*, ACI 360R-92, Reapproved 1997.

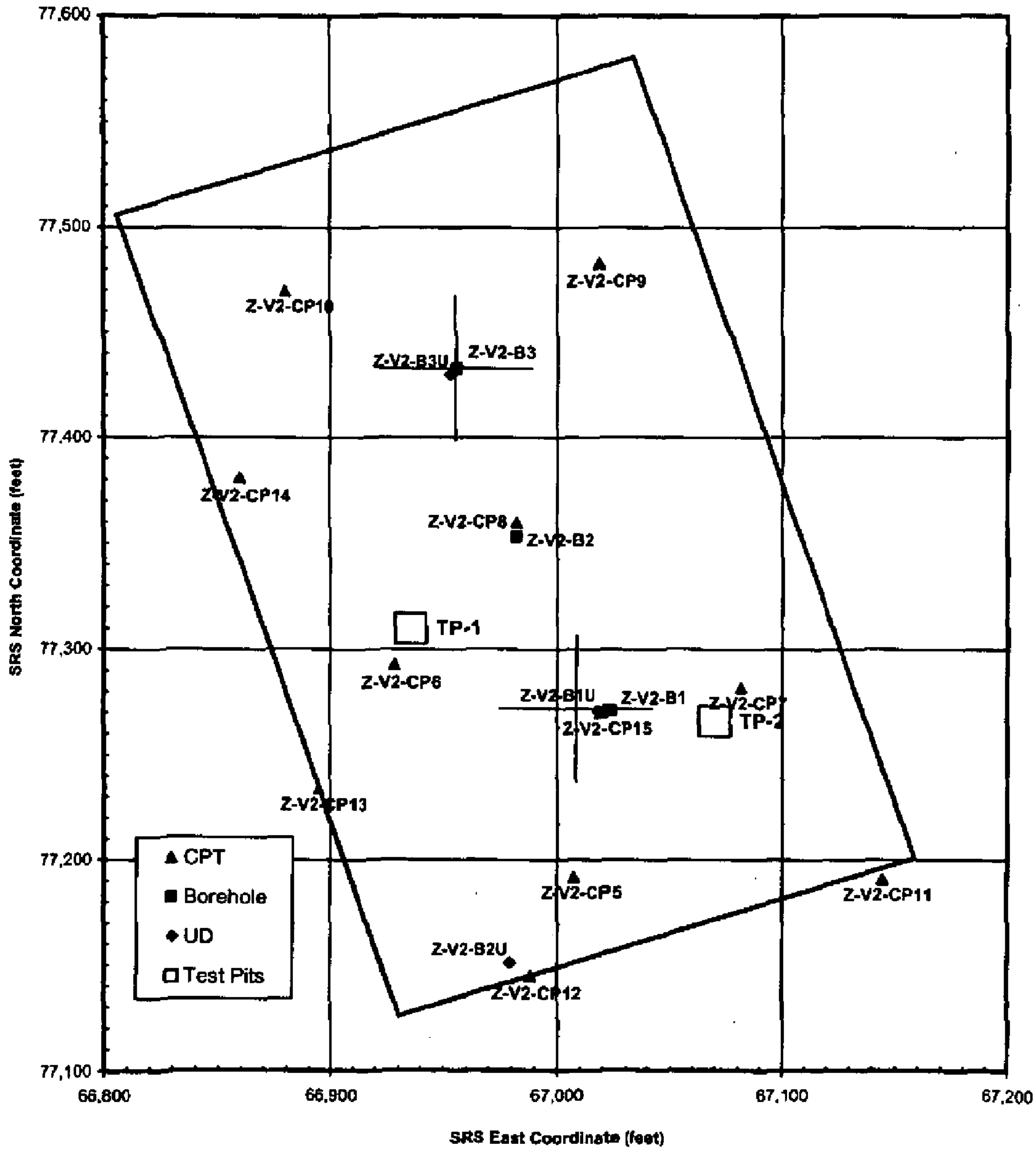
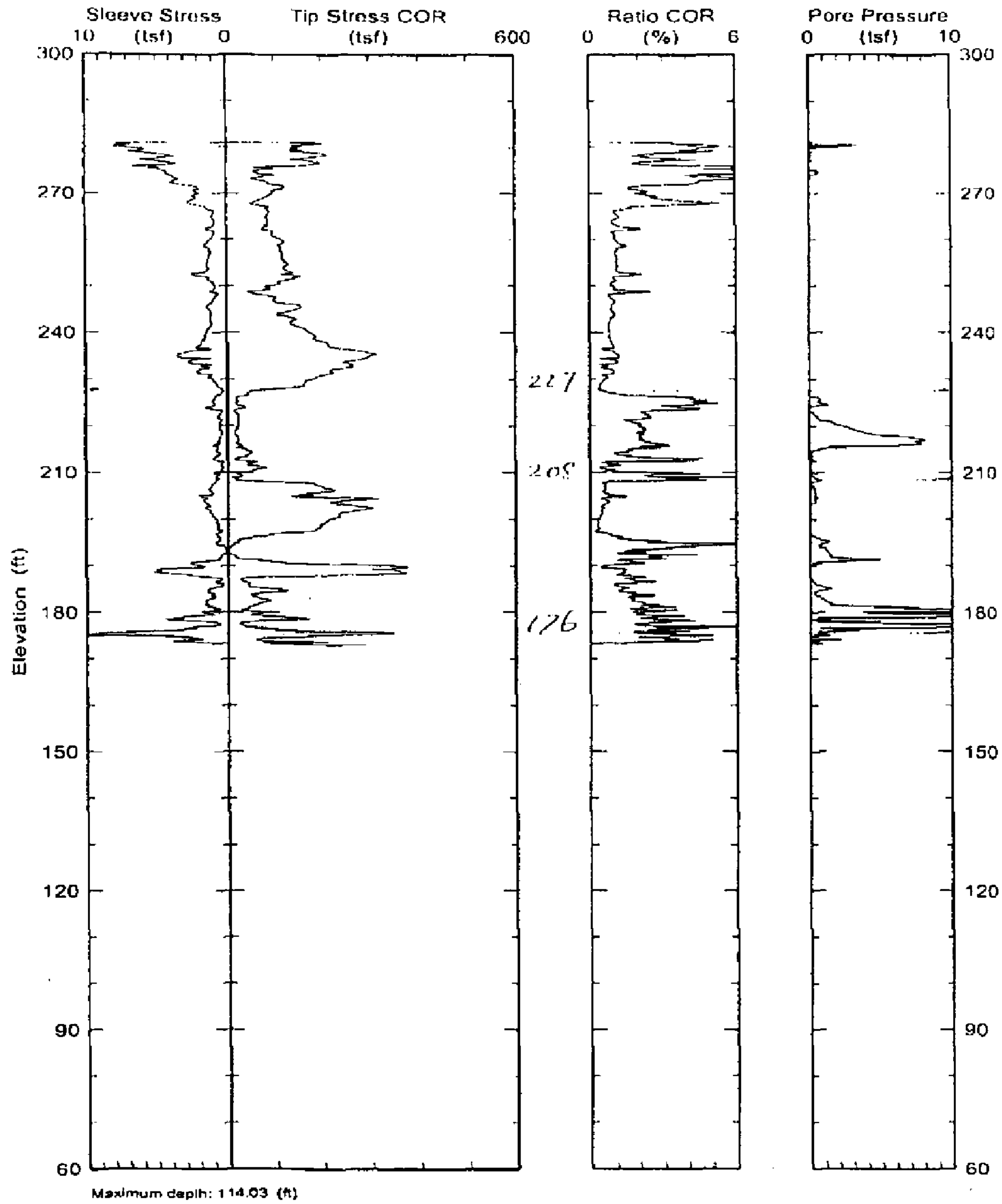
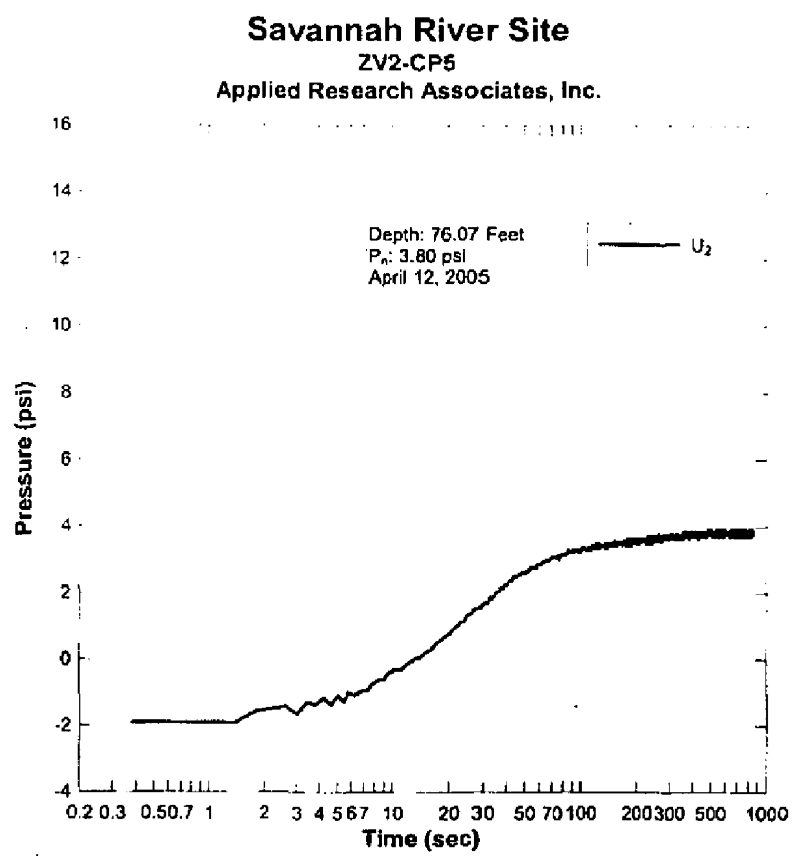


Figure 1 Facility layout and geotechnical exploration locations

Appendix A CPTU Report

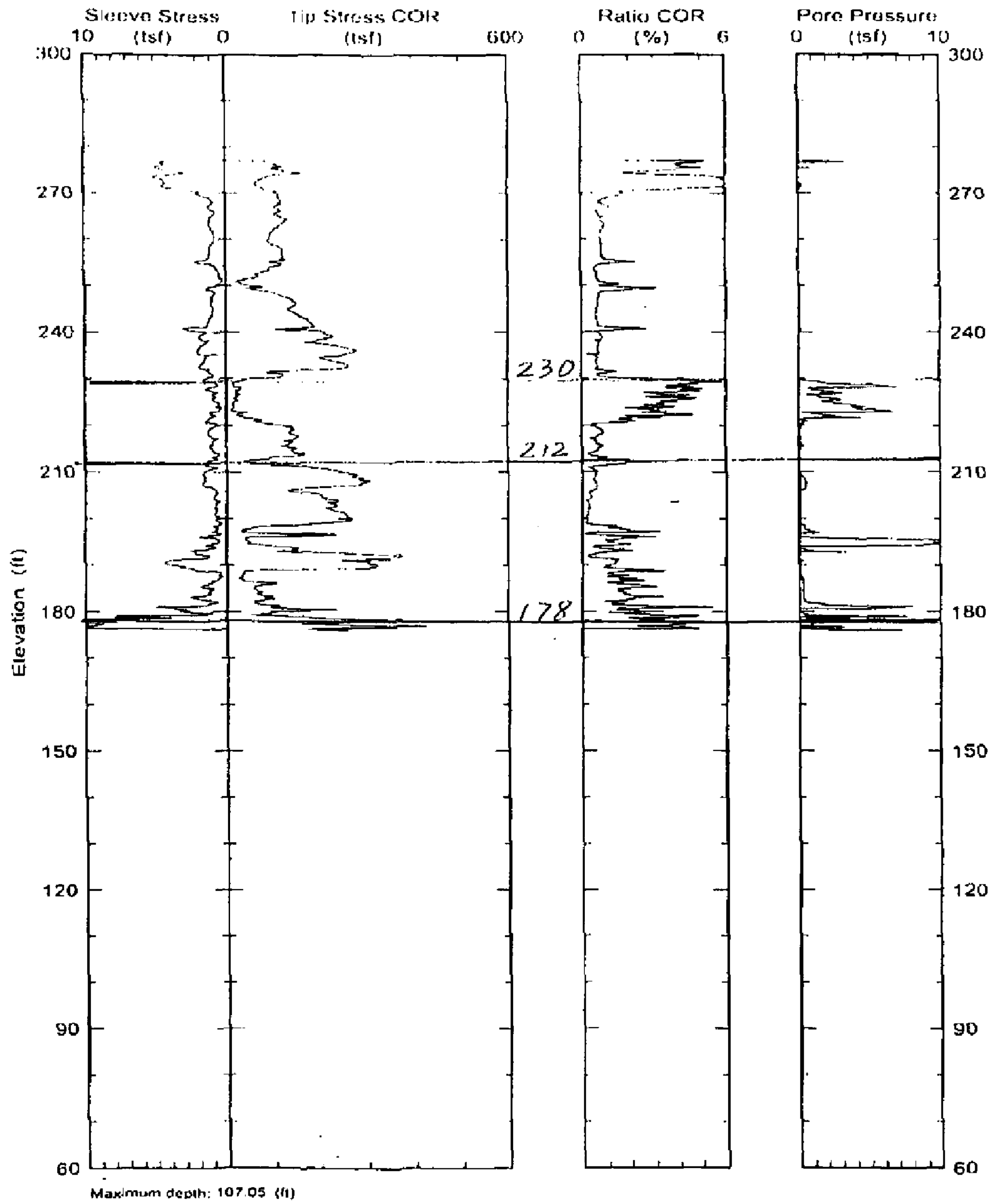
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 12/Apr/2005 Test ID: ZV2-CP5 Northing: 77192.0 Easting: 67008.0 Elevation: 287.0




Savannah River Site
ZV2-CP5
Applied Research Associates, Inc.
Depth: 76.07 Feet
P₀: 3.80 psi
April 12, 2005
U2
Pressure (psi)
16
14
12
10
8
6
4
2
0
-2
-4
0.2 0.3 0.5 0.7 1 2 3 4 5 6 7 10 20 30 50 70 100 200 300 500 1000
Time (sec)

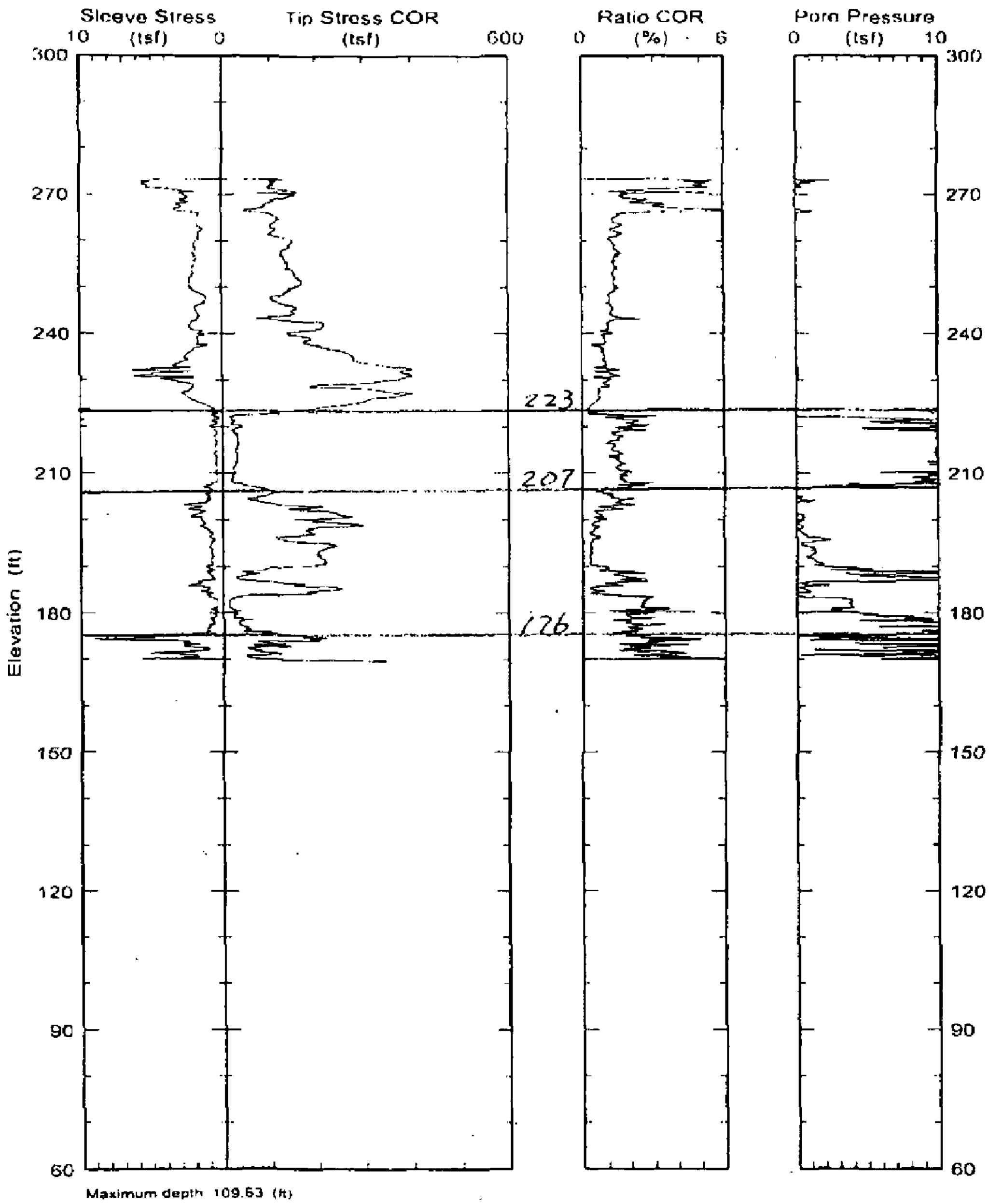
Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 17/May/2005 Test ID: ZV2-CP6 Northing: 77293.0 Easting: 66929.0 Elevation: 283.0



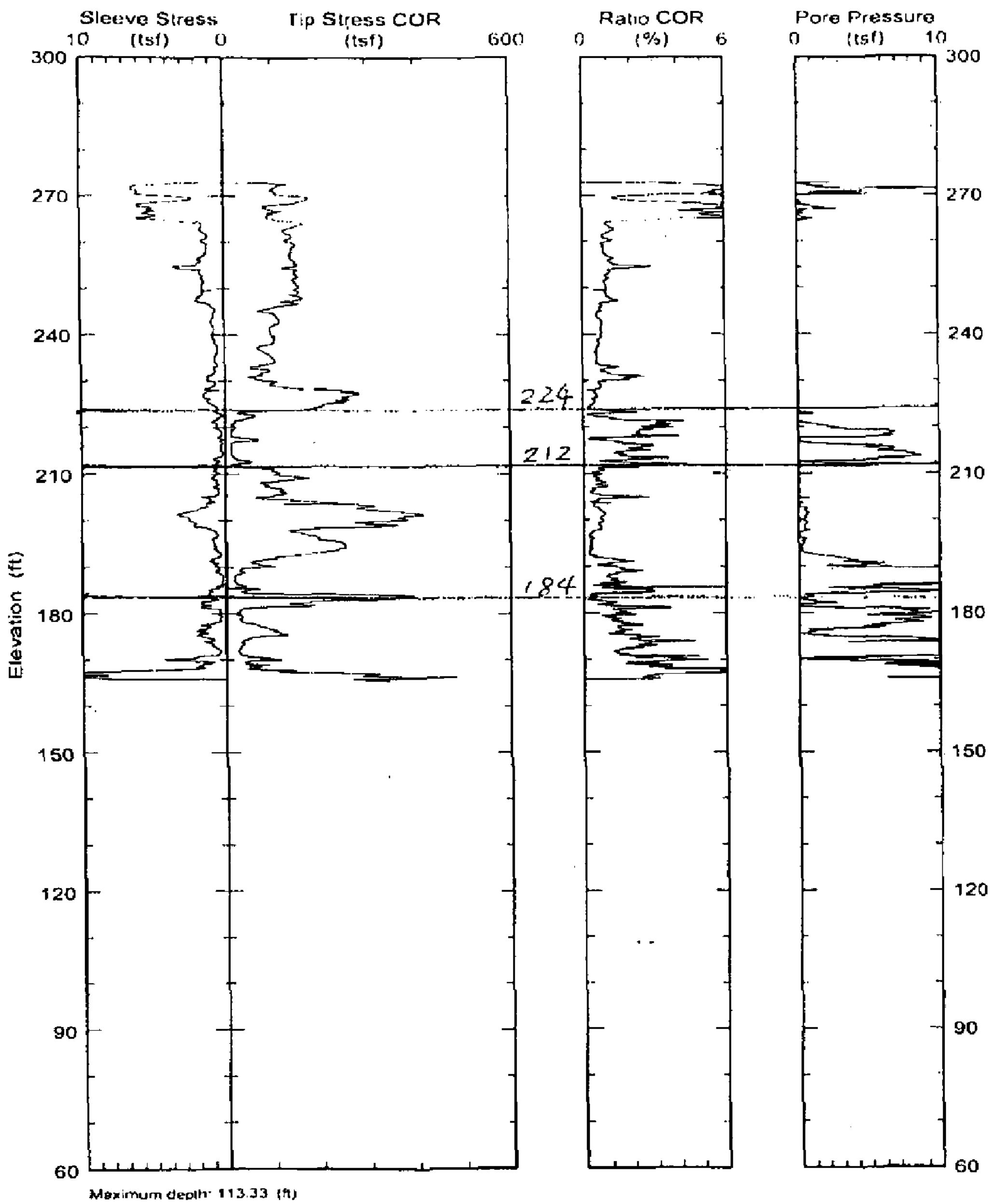
Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 16/May/2005 Test ID: ZV2-CP7 Northing: 77282.0 Easting: 67082.0 Elevation: 279.4



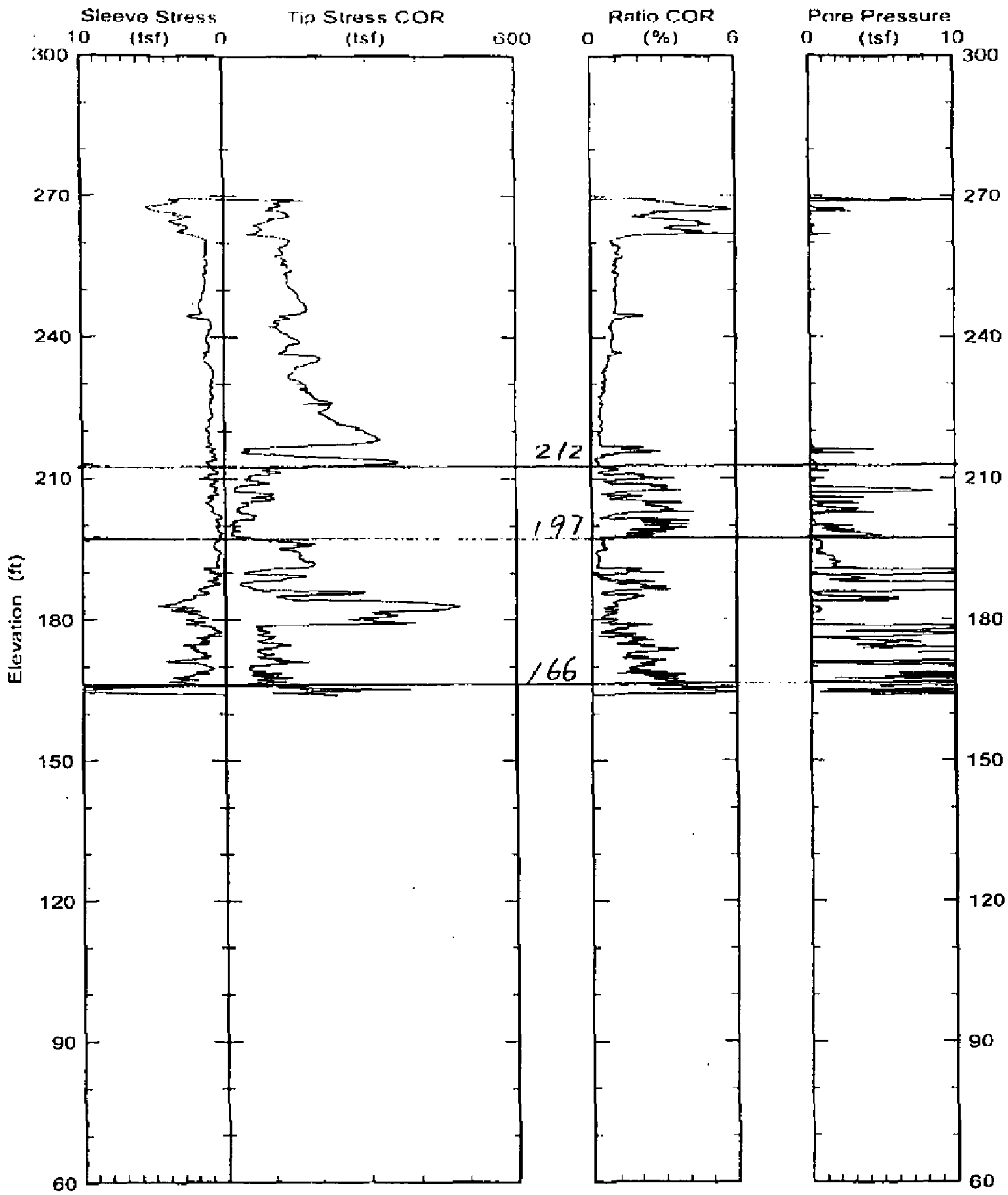
Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 16/May/2005 Test ID: ZV2-CP8 Northing: 77359.9 Easting: 66983.0 Elevation: 278.8

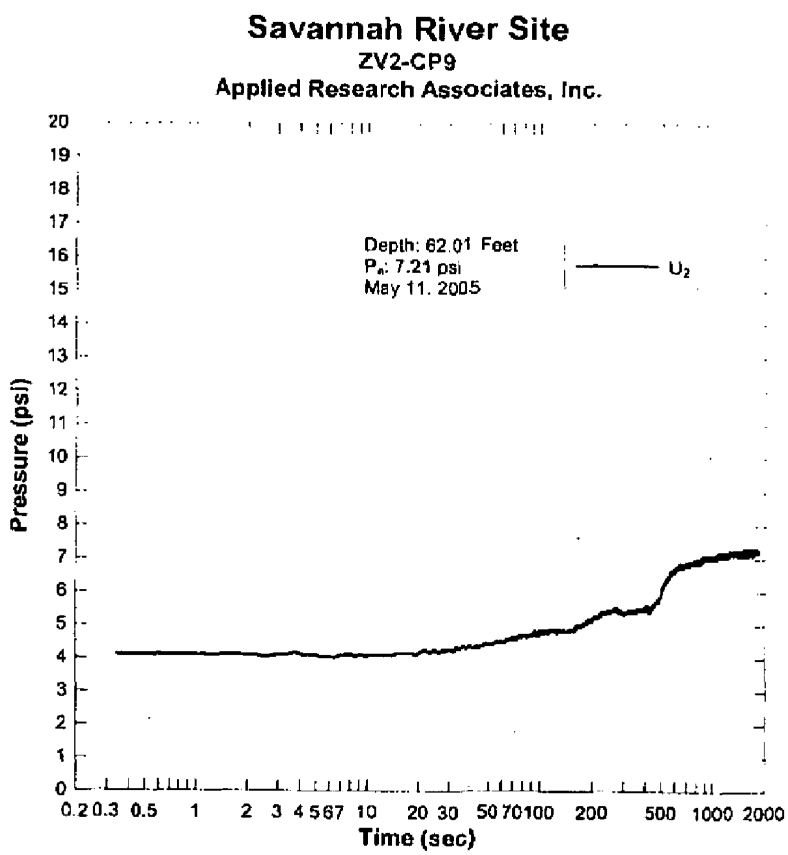


Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 11/May/2005 Test ID: ZV2-CP9 Northing: 77483.0 Easting: 67019.0 Elevation: 275.3

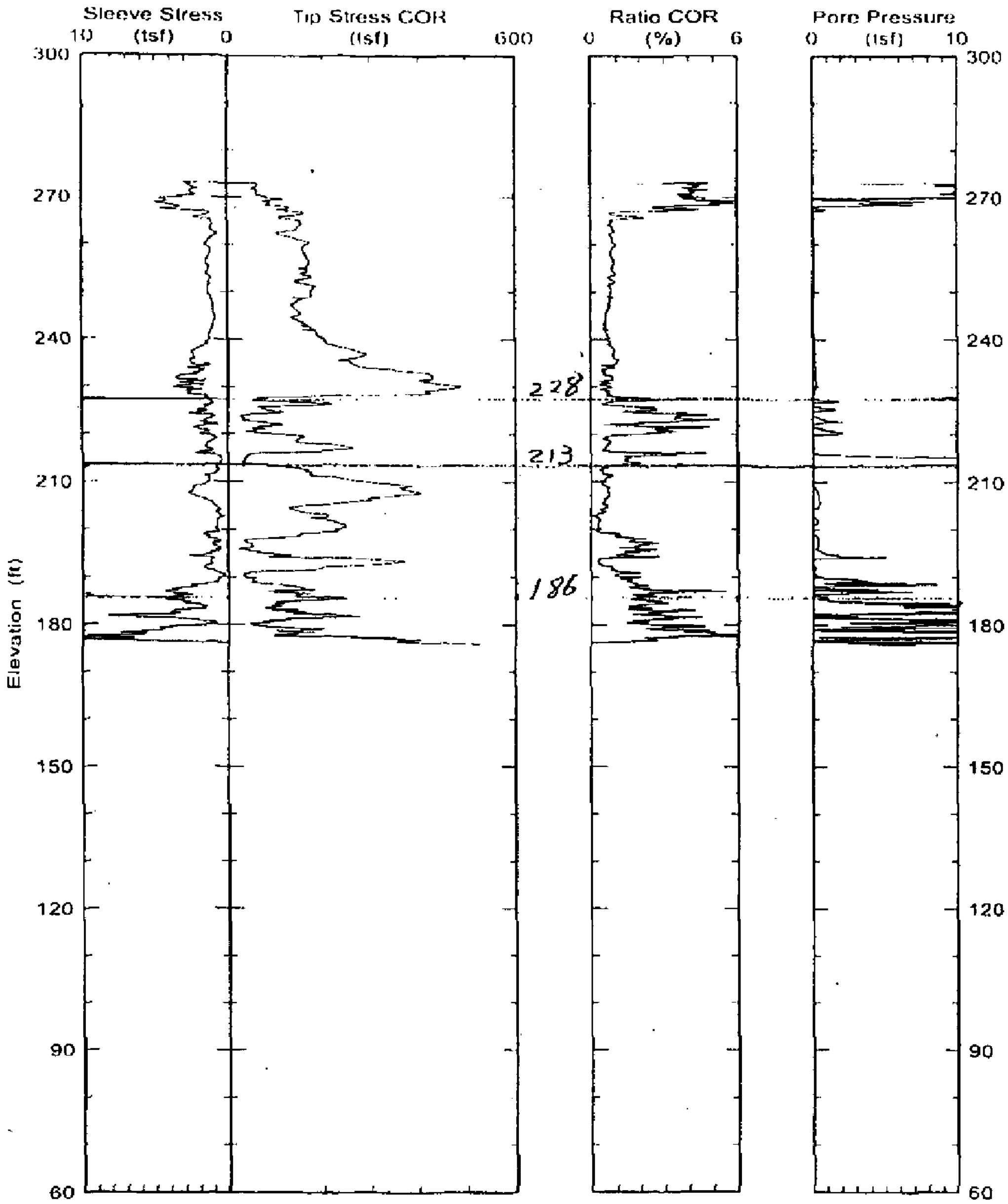


Maximum depth: 111.36 (ft)


Savannah River Site
ZV2-CP9
Applied Research Associates, Inc.
Depth: 62.01 Feet
Pₐ: 7.21 psi
May 11. 2005
U2
Pressure (psi)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
0.2 0.3 0.5 1 2 3 4 5 6 7 10 20 30 50 70 100 200 500 1000 2000
Time (sec)

Appendix A CPTU Report

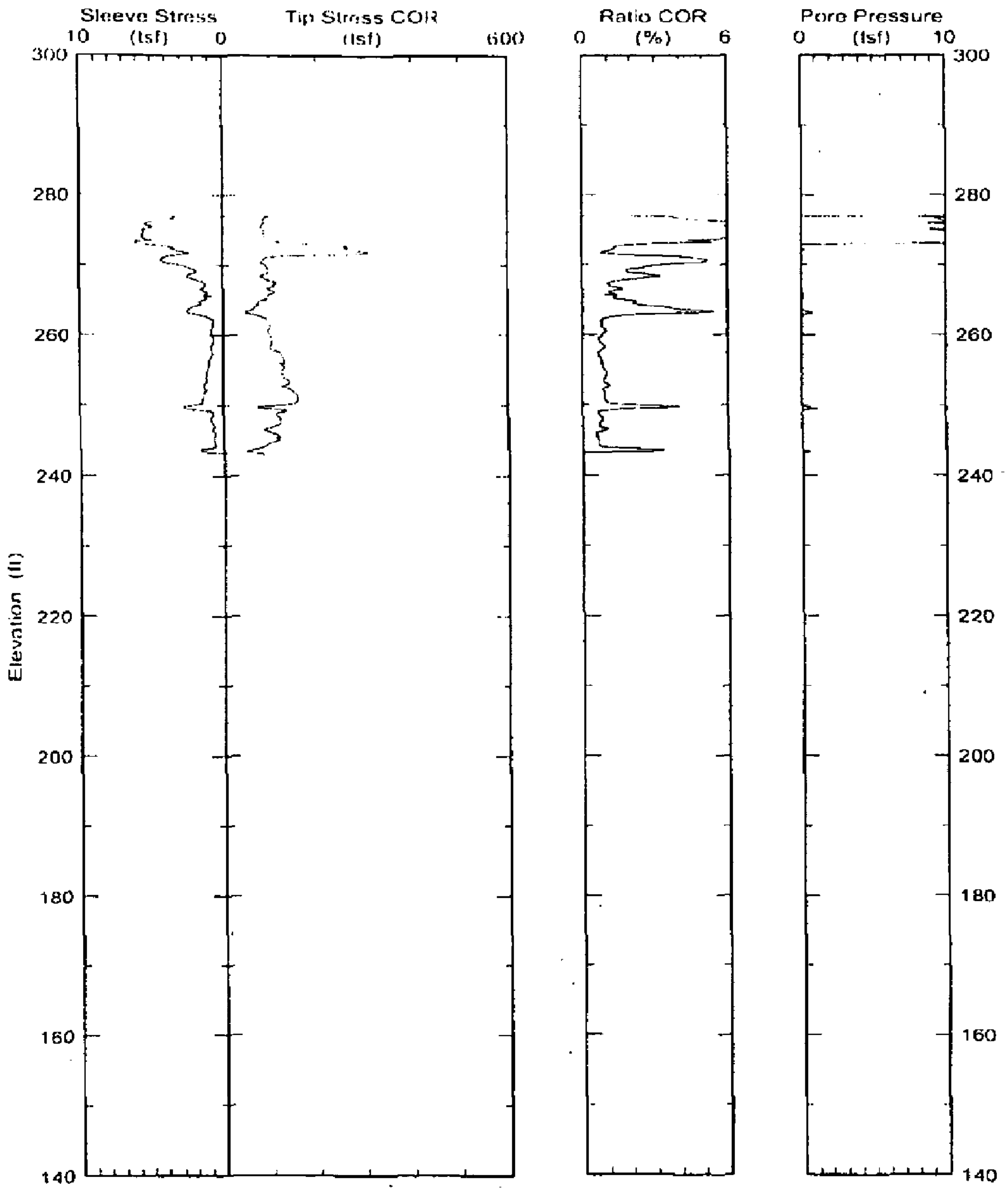
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 13/May/2005 Test ID: ZV2-CP10 Northing: 77470.0 Easting: 66880.0 Elevation: 279.0



Maximum depth: 103.07 (ft)

Appendix A CPTU Report

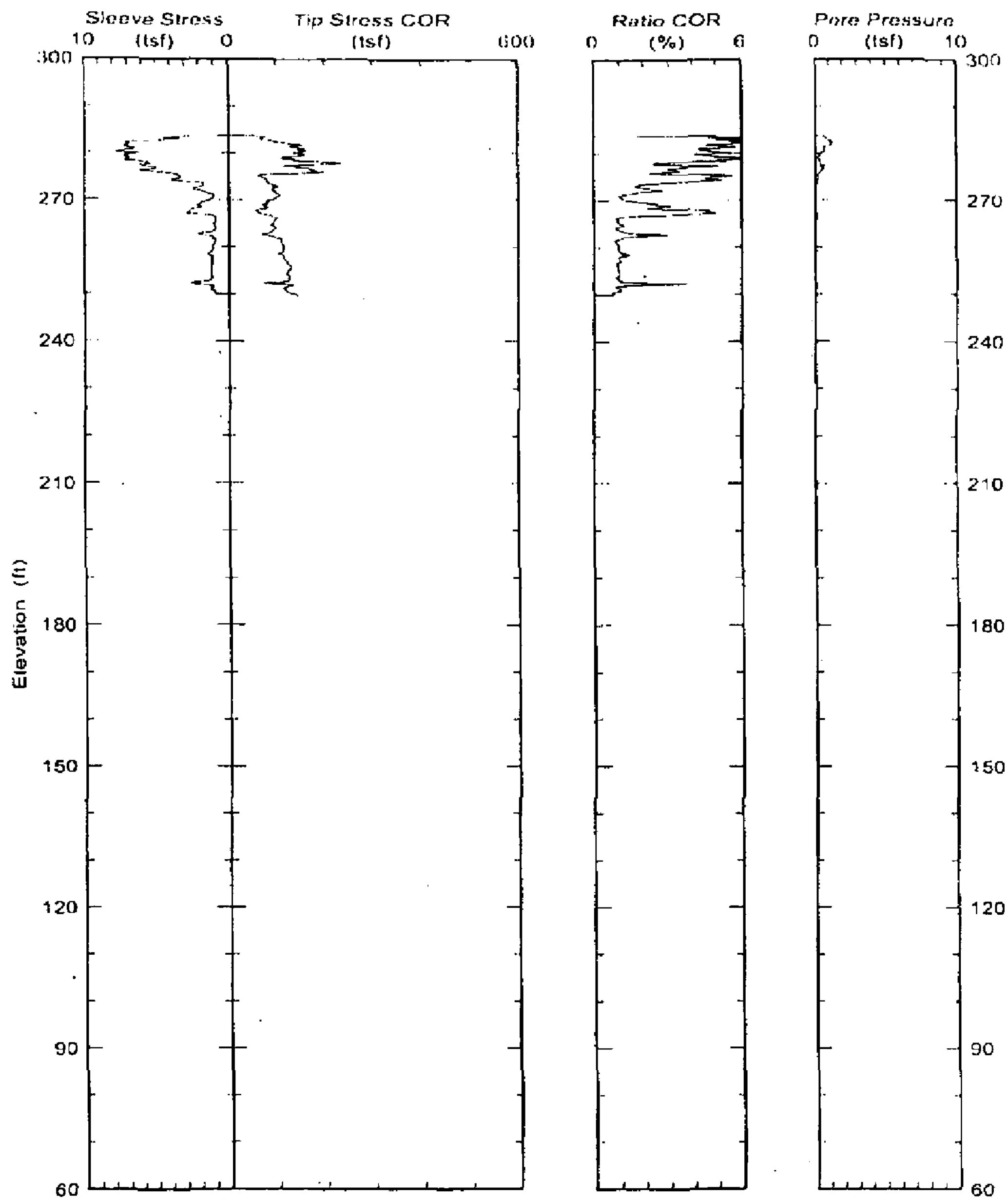
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 18/May/2005 Test ID: ZV2-CP11 Northing: 77191.0 Easting: 67145.0 Elevation: 283.0



Maximum depth: 40.10 (ft)

Appendix A CPTU Report

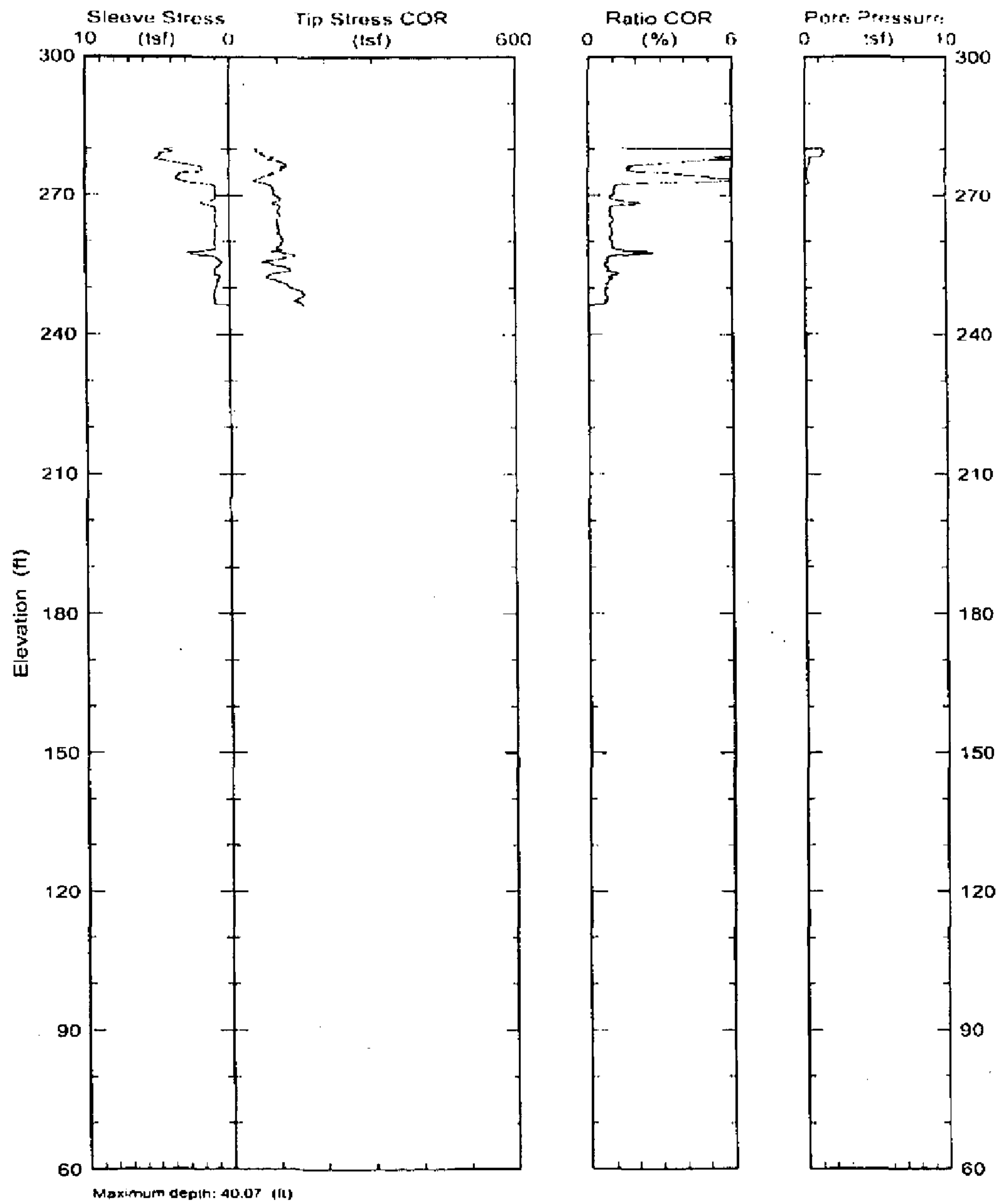
Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 12/Apr/2005 Test ID: ZV2-CP12 Northing: 77145.0 Easting: 66989.0 Elevation: 289.7



Maximum depth: 40.19 (ft)

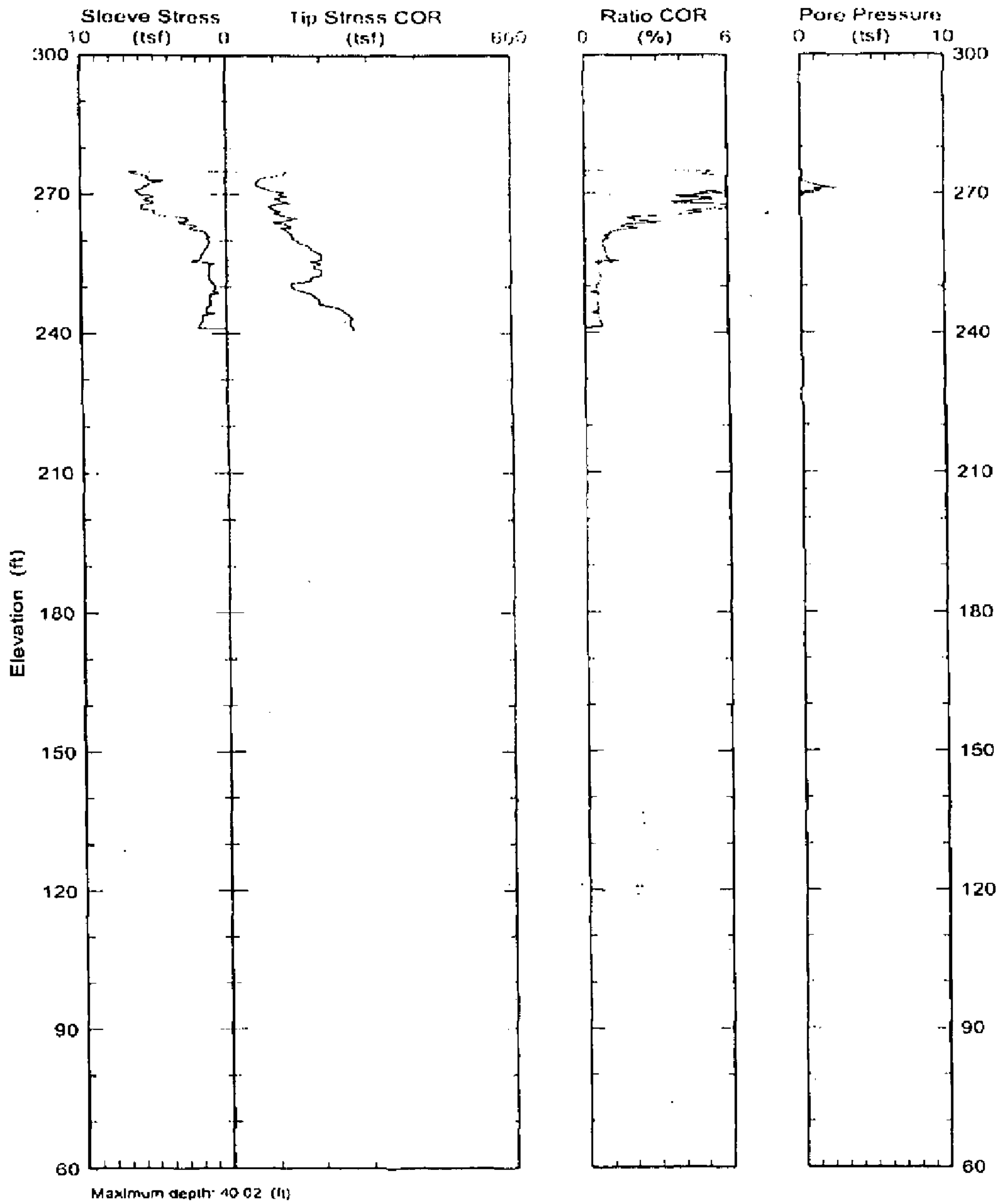
Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 12/Apr/2005 Test ID: ZV2-CP13
	Northing: 77324.0 Easting: 66895.0 Elevation: 286.2



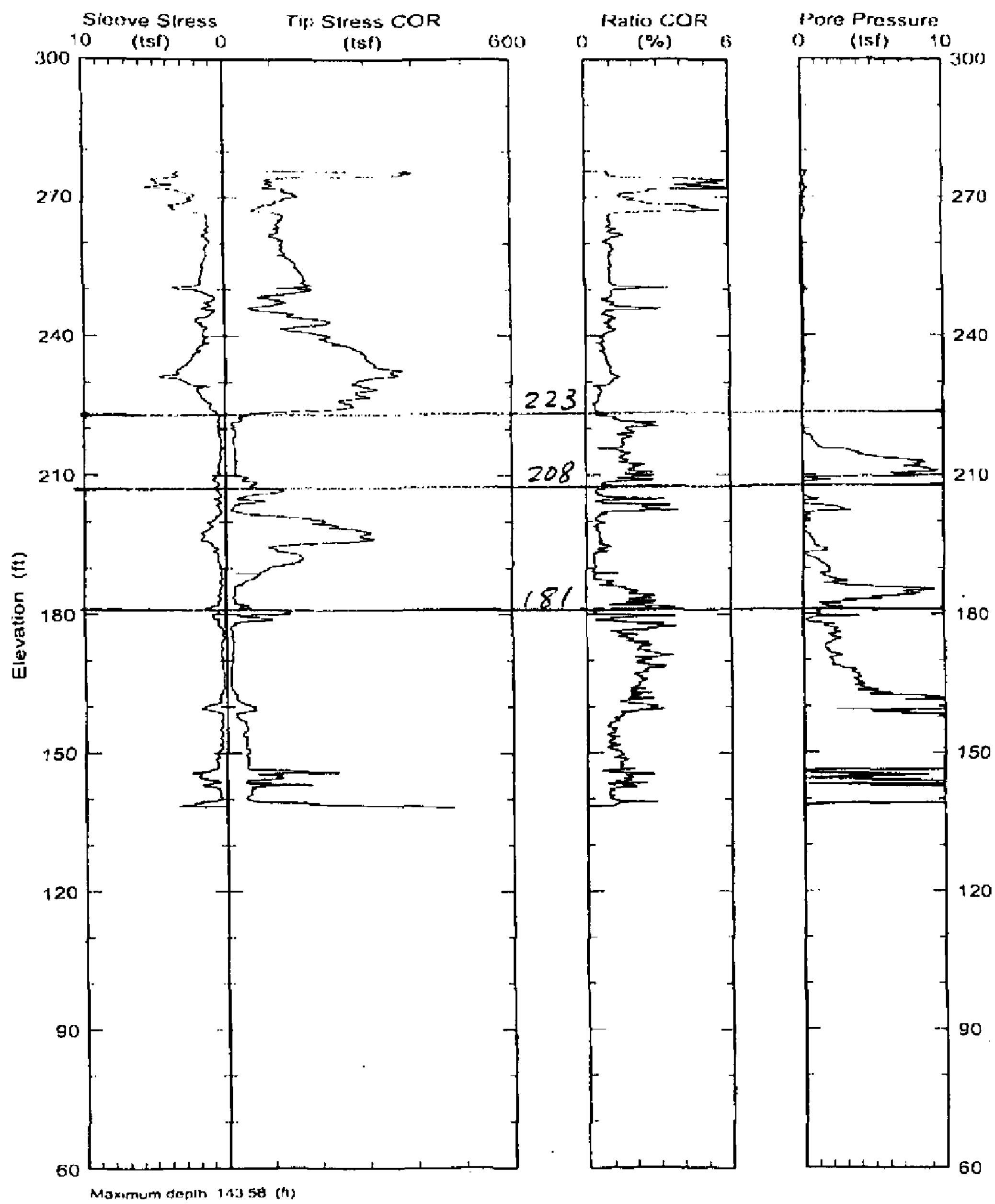
Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 18/May/2005 Test ID: ZV2-CP14 Northing: 77381.0 Easting: 66859.0 Elevation: 280.8



Appendix A CPTU Report

Applied Research Associates, Inc. South Royalton, VT 05068 802-763-8348 cpt@ned.ara.com www.ara.com	Date: 16/May/2005 Test ID: ZV2-CP15 Northing: 77270.0 Easting: 67020.0 Elevation: 281.7



FIELD BORING LOG

PROJECT	SALTSTONE VAULT 2
JOB NO.	
SHEET NO.	1 OF 7
HOLE NO.	ZV2B1
SITE	Z AREA VAULT 2
COORDINATES	N 77271.[illegible] E [illegible]
WEATHER CONDITIONS	CLOUDY 70°F
BEGUN	5-5-05
COMPLETED	
DRILLING CO./DRILLER	GREGG DRILLING
DRILL MAKE AND MODEL	CME 7[illegible]
HOLE SIZE	4"
SAMPLE HAMMER WEIGHT/FALL	CME AUTOMATIC HAMMER
TOTAL DEPTH	132
GROUND EL.	281.70
GROUND WATER DEPTH/DATE	
TECHNICAL OVERSIGHT BY:	F H SYMS
REVIEWED BY:	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						5-5-05 THR
						CLOUDY 70°F
						0-15' DRILL OUT
	5					
	10					
	15					
SS-1		6	0"/24"		RD BRN CLAYEY SAND (SC) POORLY SORTED F-C	
		12			MED DENSE, FINES PLASTIC, MOIST, WHITE WHT	
		15			KAOLIN LENSES	
		17				
		7				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

FIELD BORING LOG	PROJECT SALTSTONE VAULT 2	JOB NO.	SHEET NO. 2 of 7	HOLE NO. ZV2B1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-2		10	18"/24"		SAA	
		12				
		13				
SS-3		7	24/24		SAA BECOMING SLIGHTLY MORE BRN	
	20	10				
		12				
		13				
SS-4		9	24/24"		SAA CLAY CONTENT INCREASING + 40%	
		11				
		14				
		15				
SS-5		7	18"/24"		LT BRN SILTY SAND (SM) WELL SORTED F.S. IN	10:00 AM
		12			SILTY MATRIX ~35% FINES 10% CLAY, MOIST	
		13			LOW PLASTICITY, MED DENSE	
		15				
	25					
SS-6		10	20/24"		RD BRN SILTY SAND w/ CLAY (SM-SC) WELL SORTED	
		14			M.S.	
		15				
		19				
SS-7		11	18"/24"		SAA	
		14				
		16				
		16				
SS-8		10	16"/24"		BECOMING POORLY SORTED	11:00 AM
	30	16				
		16				
		14				
SS-9		12	15"/24"		RD BRN CLAYEY SAND (SC) WELL SORTED M.S.	
		14			WET ~20% FINES	
		8				
		6				
SS-10		3	18"/24"		BRN CLAYEY SAND (SC) POORLY SORTED F.C	11:20 RAIN
		3			SD, WET	
		5				
		5				
	35					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

SKS

Appendix B Geotechnical Borehole Logs

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALSTONE VAULT 2		3 of 7	ZV2B1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	40					
		6			RD BRN CLAY SAND (SC) [illegible] F-C	1:15 PM LIGHT
SS-11		11	22"		[illegible]	RAIN
		14	/24"			
		14				
	45					
		11			BRN SAND w/SILT (SW-SM) WIDELY GRADED	2:00 PM
SS-12		17	21"		F-C SAND ~10% FINES WET	
		19	/24"			RE-CONDITIONING
		21				MUD
	50					
		13	20"		RD BRN SAND w/SILT (SW-SM) WIDELY GRADED	2:30 PM
SS-13		18			F-C, WET, DENSE	
		20	/24"			
		23				
	55					
		17			RD BRN SAND (SW) WIDELY GRADED F-C, WET	3:15 PM
SS-14		22	10"		DENSE	RAIN OUT
		28				
		27	/24"			

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B1

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		4 of 7	ZVZB1

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	60					
SS-15		WR, 4, 4, 6	23/24"		TOP OF SAND (SM) ... WET, LT. BRN SILTY SAND (SM) V.F.S. TR MAG OXIDE WET SOFT	
	65					
SS-16		2, 3, 3, 4	24/24"		LT. BRN CLAY W/SAND (CL) MOD. PLASTIC, WET MAG OXIDE V.F.S.	
	70					
SS-17		WR, 4, 4, 4	20/24"		LT. BRN SILTY SAND (SM), SOFT, MOIST LAMINATED TR. MAG OXIDE	MONDAY SUNNY 85°F 9:30 TRUCK OUT OF WATER
	75					
SS-18		WR, 2, 3, 4	23"/24"		LT. BRN SAND W/SILT (SP-SM), SOFT, WET, V.F.S.	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB1

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2			JOB NO.	SHEET NO. 1 of 7	HOLE NO. ZV2B2		
SITE: Z-AREA VAULT 2	COORDINATES: N 77353.7 E 66982.6			WEATHER CONDITIONS: SUNNY 75°F			
BEGUN: 4-28-05	COMPLETED: 5-04-05	DRILLING CO./DRILLER: GREGG IN-SITU	DRILL MAKE AND MODEL: CME 75	HOLE SIZE: 4"	SAMPLE HAMMER WEIGHT/FALL: CME AUTOMATIC HAMMER	TOTAL DEPTH	
GROUND EL.: 279.00	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY: FRANK H. SYMS	REVIEWED BY				

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						4-28-05 TIME SUNNY 75°F. 0-10' DRILL OUT
	5					
SS-1	10	6, 9, 9, 11	18"/24"		RDW BRN CLAYEY SAND (SC) WIDELY GRADED LAMINATED, DRY, MED DENSE	
SS-2		6, 8, 9, 12	20"/24"		SAA	11:05
SS-3	15	7, 11, 13, 17	18"/24"		RD BRN, LT. BRN, PUR CLAYEY SAND (SC) WIDELY GRADED, MED DENSE. C.S. MOTTLED	
SS-4		9, 9, 12	18"/24"		RD BRN, LT BRN, WHT SAND (SW-SC) W/ CLAY WIDELY GRADED M-C, MED DENSE	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

SRS

Appendix B Geotechnical Borehole Logs

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 2 of 7 | HOLE NO.: ZV2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
		12				
SS-5		8	17"/24"		LT. BRN ~~SILTY~~ CLAYEY SAND (SC) WIDELY GRADED, M-C	11:45 AM
		12			MOD DENSE, MOIST, HIGHLY PLASTIC FINES	
		12				
	20	14				
SS-6		7	20"/24"		SAA SLIGHTLY MORE SANDY	
		11				
		11				
		12				
						OVERDRILLED 1½' TO 25' RESUMED CONT. SAMPLING @ 25'
SS-7	25	6	18"/24"		LT. BRN SAND W/ SILT (SP-SM) NARROWLY GRADED	
		8			M.S. NON PLASTIC, WET, MOD DENSE TO	
		8			LOOSE	
		10				7:00 PM
SS-8		8	20"/24"		SAA BECOMING SLIGHTLY FINER GRAINED SAND	
		10			AND RED BRN, WET	
		10				
		12				
SS-9		8	20"/24"		SAA, WET	2:15 PM
	30	9				
		9				
		9				
SS-10	35	4	20"/24"		SAA FINES CONTENT INCREASING SLIGHTLY, WET	2:55 PM
		7				
		8				
		8				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG

PROJECT: SALTSTONE VAULT 2 | JOB NO.: | SHEET NO.: 3 of 7 | HOLE NO.: ZV2B2

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-11	40	5 8 7 7	20"/24"		RD BRN, LT. BRN CLAYEY SAND, SAND w/CLAY (SC, SW-SC) WIDELY GRADED, WET	
SS-12	45	5 8 10 15	10"/24"		BRN, RD BRN, SAND w/ SILT (SP-SM) POORLY GRADED, LAMINATED, MOD DENSE WET	FRIDAY
SS-13	50	9 12 15 18	12"/24"		BRN SAND (SW) WIDELY GRADED F-C, DENSE, WET	10:00 AM
SS-14	55	4 3 3 4	14"/24"		BRN, RD. BRN CLAYEY SAND (SC) WIDELY GRADED F-C, SOFT, WET, LAMINATED BEDS OF CLAY AND SAND	

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B2

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		1 of	ZV2B3

SITE	COORDINATES	WEATHER CONDITIONS
VAULT 2	N ~~77433~~ 77411.88 E 66956.75	SUNNY 75°F

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH
4-21-05		GREGG DRILLING	MOBILE		140 lb/30" ATO	132

GROUND EL.	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:
278.1		FH SYMS	

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
						4-21-05 SUNNY 75°F
					AUTOMATIC HAMMER	0-5 [illegible]
	5	10	18"/24"		RED/BRN/PURPLE CLAYEY SAND (SC) WIDELY GRADED M-C. DENSE. MOIST	
SS-1		15				
		16				
		17				
	7	8	18"/24"		RED/BRN/WHITE CLAYEY SAND (SC) SAA WIDELY GRADED C-M DENSE MOIST MOD. PLASTIC	
SS-2		14				
		16				
		21			BOT 6" PUR/WHITE SILTY SAND (SM)	
	9	6	15"/24"		RED/BRN/WHT CLAYEY SAND (SC) WIDELY GRADED C-M DENSE MOIST. LAMINATED W/ WHT KAOLIN. MOD. PLASTIC	
SS-3	10	11				
		12				
		16				
	11	5	16"/24"		SAA BECOMING V. MOIST ALMOST WET	
SS-4		6				
		10				
		10				
	13	7	13"/24"		SAA V MOIST	
SS-5		13				
		14				
		14				
	15	8	14"/24"		LT. BRN/WHT CLAYEY SAND W/SILT (SM) WIDELY GRADED. MED DENSE MOIST	
SS-6		11				
		12				
		12				
	17	6				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3

FIELD BORING LOG

PROJECT: SALTSTONE | JOB NO.: | SHEET NO.: 2 OF | HOLE NO.: ZVZ83

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
SS-7	18	14, 13, 13	16/24"		SAA	
	19					
SS-8	20	9, 10, 12, 13	13/24"		SAA	
	21					
SS-9	25	8, 10, 11, 12	18"/24"		LT. PUR SAND w/SILT (SP-SM) POORLY GRADED M-F SAND MOIST MED. DENSE NON PLASTIC	2:00 PM
	27					
SS-10	30	6, 6, 8, 12	17"/24"		LT BRN SAND w/SILT (SP-SM/SP) POORLY GRADED F SAND, MOIST MED DENSE, NON PLASTIC	
	32					
SS-11	35	8, 9, 10, 10	18"/24"		RED BRN SAND w/CLAY (SP-SC) POORLY GRADED M-F SAND 15-20% FINES, WET, MED DENSE, NON PLASTIC	3:15 PM

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES HOLE NO. ZVZ83

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
SALTSTONE VAULT 2		3 of	ZV2B3

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT/ PRESSURE (psi)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	38					
	40					
SS-12		10	20/24		RED BRN CLAYEY SAND (SC) POSSIBLE (SP-SC)	3:50 PM
		11			M-C SAND, WET, MED DENSE NON PLAST	
		14				
		15				
	45					
SS-13		12	18"/24"		RED BRN CLAYEY SAND (SC) M-F SAND, WET,	4:30 END OF DAY
		16			DENSE MOD PLASTIC	
		17				
	47	23				
	50					
SS-14		15	12"/24"		LT. BRN SAND (SP) POORLY GRADED M. SAND	4-22-05 7:00 AM
		25			WET, DENSE	
		25				
		30				HYDRAULIC MAINT
	52					
	55					
SS-15		4	22"/24"		LT. BRN, RED BRN, CLAYEY SAND (SC) WELL GRADED	10:00 AM.
		3			POORLY SORTED M-C SAND, WET, SOFT	
		3				
	57	7				

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZV2B3

FIELD BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
VAULT 2		1 of	ZVZB2U

SITE	COORDINATES	WEATHER CONDITIONS
Z AREA	N E	SUNNY 95 F

BEGUN	COMPLETED	DRILLING CO./DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	SAMPLE HAMMER WEIGHT/FALL	TOTAL DEPTH
[illegible]	[illegible]	[illegible]	CME 75	[illegible]	N/A	

GROUND EL	GROUND WATER DEPTH/DATE	TECHNICAL OVERSIGHT BY:	REVIEWED BY:
28[illegible]		H SIMS	

SAMP. TYPE AND NO.	DEPTH (FT)	BLOW COUNT/ PRESSURE (PSI)	REC./PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	5					
	7					SEE UD LOG
ST-1			[illegible]			
	8					
	10					
	14					SEE UD LOG
ST2	15		17"/24"			
	16					

SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB2U

SRS

FIELD BORING LOG

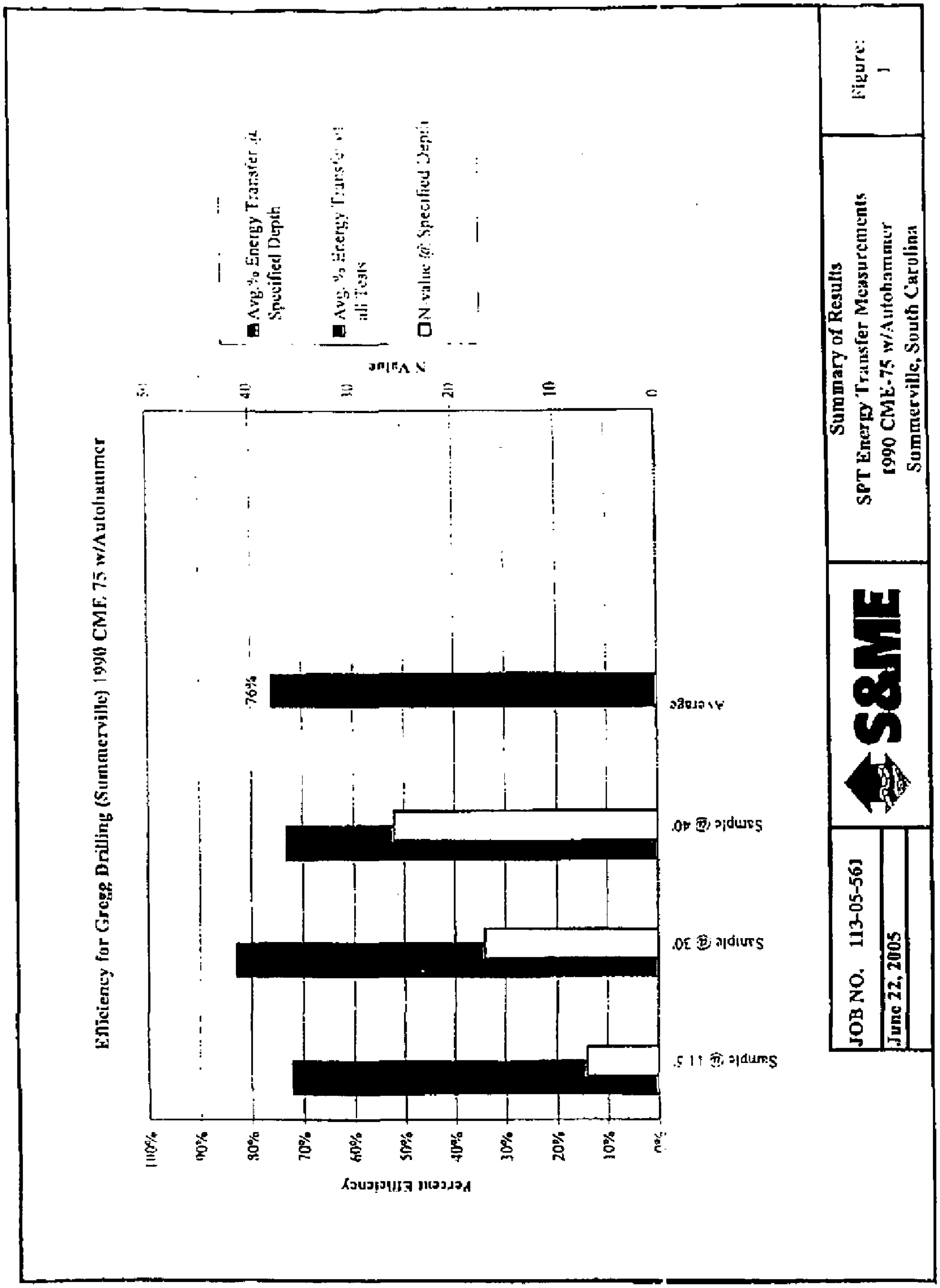
PROJECT | JOB NO | SHEET NO. | HOLE NO. ZVZB2U

SAMP. TYPE AND NO.	DEPTH (ft)	BLOW COUNT PRESSURE (psi)	REC. PEN.	GRAPHICS	DESCRIPTION AND CLASSIFICATION	NOTES
	25					
	28					SEE UD LOG
ST-3						
	30					
	34					SEE UD LOG
ST-4	35		21"/24"			
	36					

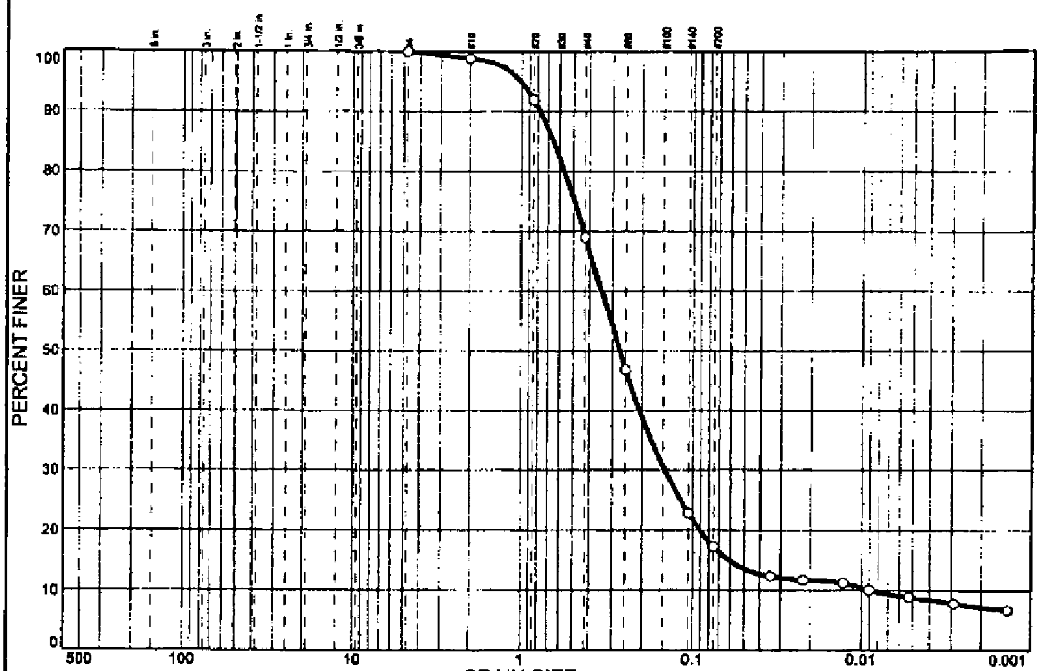
SS = SPLIT SPOON; ST = SHELBY TUBE; PS = PISTON; PB = PITCHER; CR = CORE

SITE GEOTECHNICAL SERVICES

HOLE NO. ZVZB2U


Efficiency for Gregg Drilling (Summerville) 1998 CME 75 w/Autohammer
Avg. % Energy Transfer @ Specified Depth
Avg. % Energy Transfer of all Tests
N-value @ Specified Depth
N Value
Percent Efficiency
76%
Sample @ 11.5'
Sample @ 30'
Sample @ 40'
Average
Summary of Results
SPT Energy Transfer Measurements
1990 CME-75 w/Autohammer
Summerville, South Carolina
S&ME
JOB NO. 113-05-561
June 22, 2005
Figure: 1

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	82.8	8.4	8.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.9		
#20	91.9		
#40	69.0		
#60	46.8		
#140	22.8		
#200	17.2		

* (no specification provided)

Soil Description

Tan Silty Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.654 D_{60}= 0.344 D_{50}= 0.271

D_{30}= 0.147 D_{15}= 0.0608 D_{10}= 0.0088

C_u= 39.32 C_c= 7.13

Classification

USCS= AASHTO= A-2-4(0)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05

Location: TP-1 **Elev./Depth:** 2-4 Ft.

GeoTesting Express Inc.

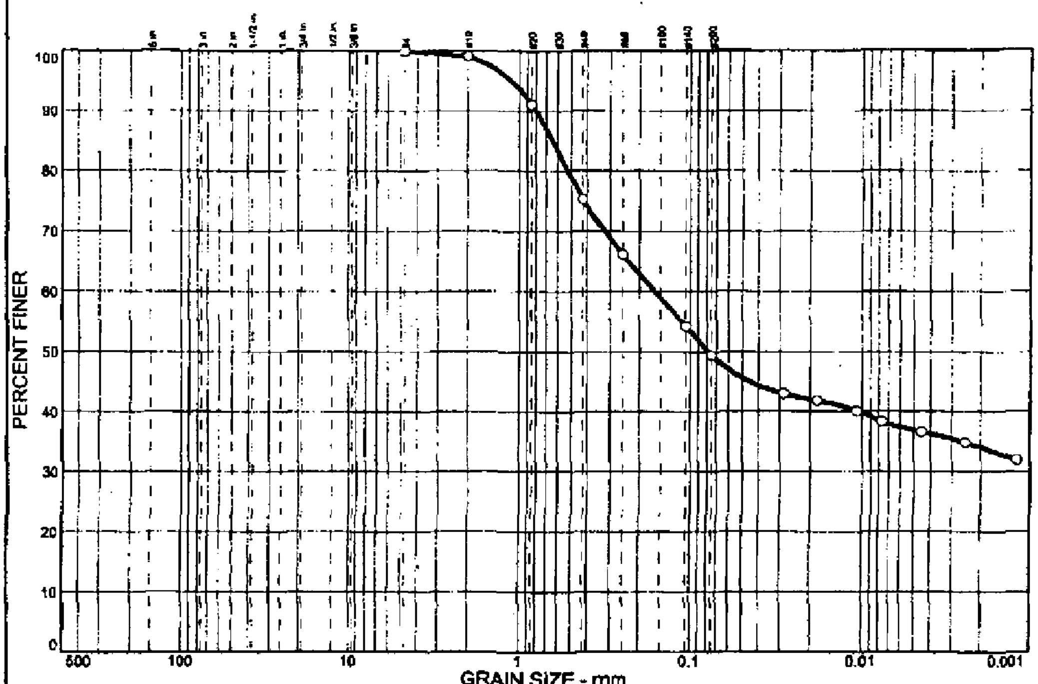
Client: Westinghouse Savannah River Company

Project: Saltstone Vault #2 Bulk Samples

Task Order Number 13 Contract No. AB80188N

Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	50.6	12.5	36.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.1		
#20	91.1		
#40	75.3		
#60	66.1		
#140	54.2		
#200	49.4		

* (no specification provided)

Soil Description

Light Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.640 D_{60}= 0.161 D_{50}= 0.0787
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-4(0)

Remarks

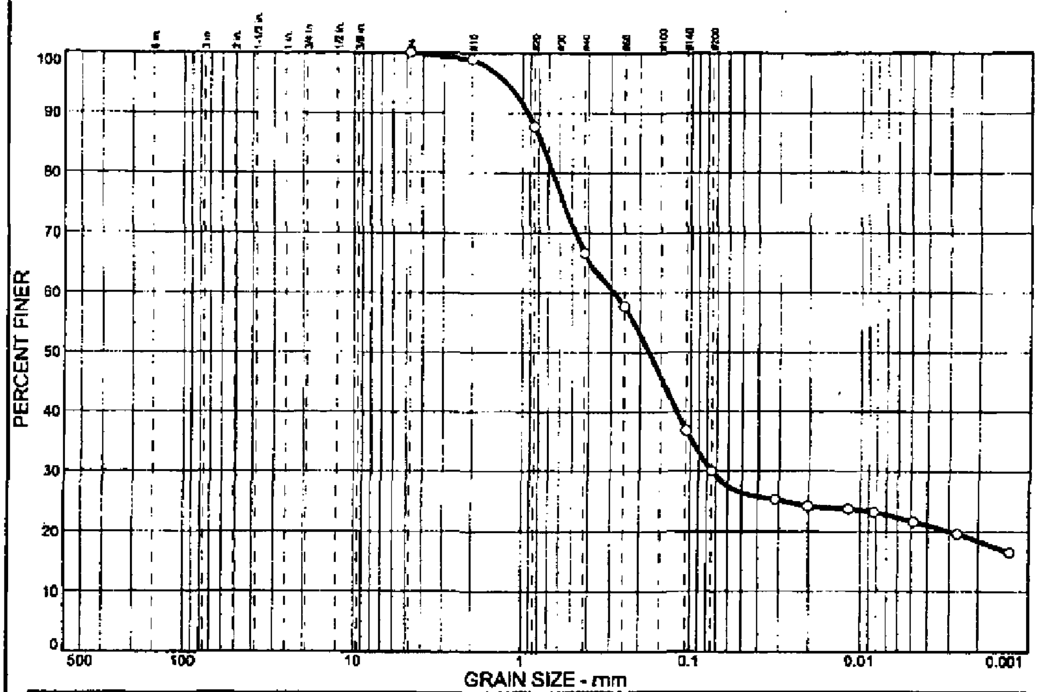
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 4-6 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	70.0	8.3	21.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.0		
#20	87.8		
#40	66.7		
#60	57.7		
#140	36.8		
#200	30.0		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.769 D_{60}= 0.288 D_{50}= 0.177
D_{30}= 0.0750 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

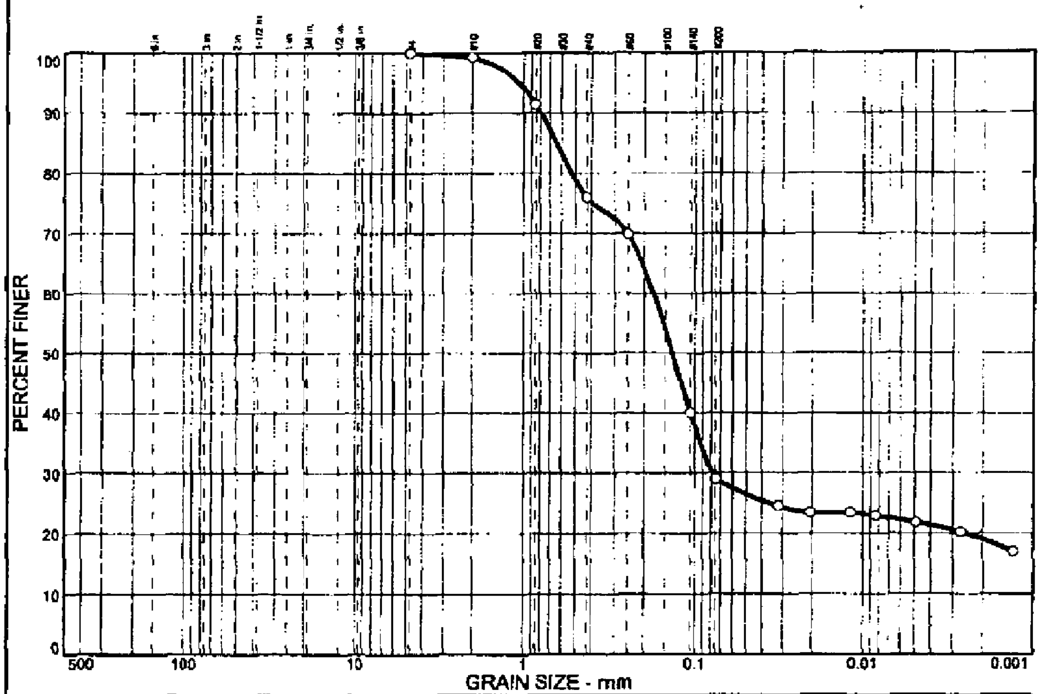
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 8 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	71.0	7.1	21.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.3		
#20	91.4		
#40	76.0		
#60	69.8		
#140	40.0		
#200	29.0		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.644 D_{60}= 0.177 D_{50}= 0.136

D_{30}= 0.0782 D_{15}= D_{10}=

C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05

Location: TP-1 **Elev./Depth:** 10 Ft.

GeoTesting Express Inc.

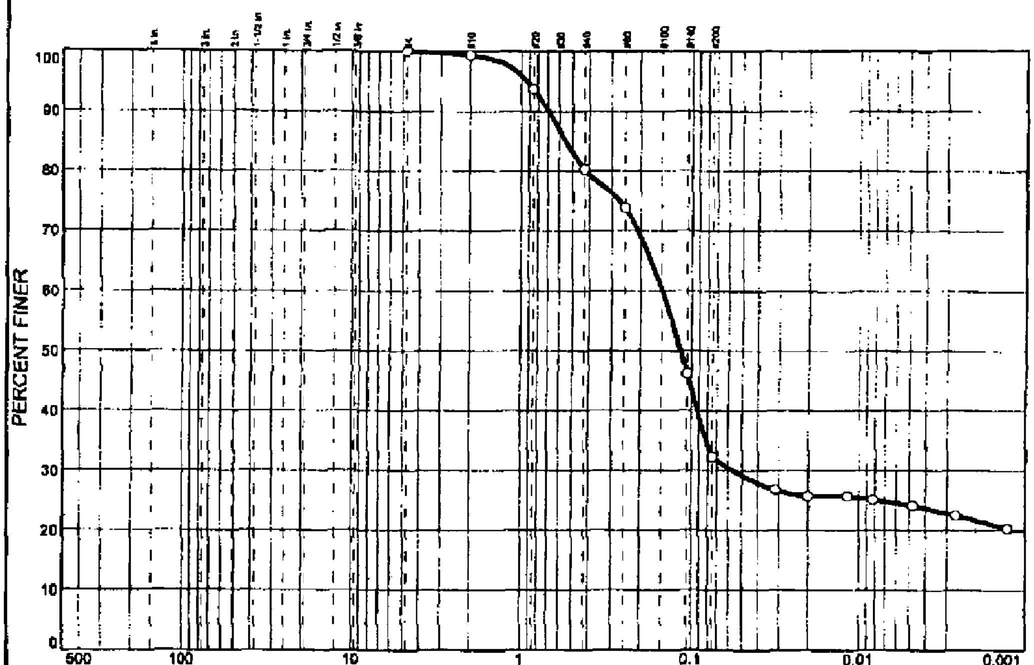
Client: Westinghouse Savannah River Company

Project: Saltstone Vault #2 Bulk Samples

Task Order Number 13 Contract No. AB80188N

Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	67.7	8.1	24.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.4		
#20	93.7		
#40	80.3		
#60	73.8		
#140	46.2		
#200	32.3		

* (no specification provided)

Soil Description

Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.550 D_{60}= 0.150 D_{50}= 0.116
D_{30}= 0.0564 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

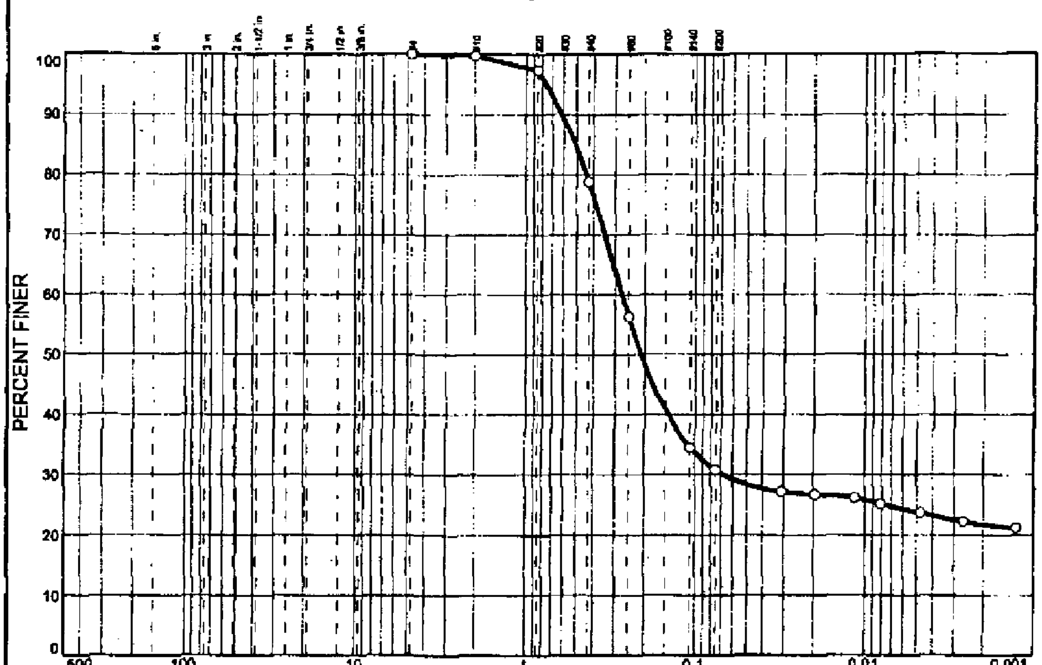
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/12/05
Location: TP-1 **Elev./Depth:** 12 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	69.2	7.0	23.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#20	97.4		
#40	78.8		
#60	56.3		
#140	34.5		
#200	30.8		

* (no specification provided)

Soil Description

Tan Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.511 D_{60}= 0.274 D_{50}= 0.210
D_{30}= 0.0673 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

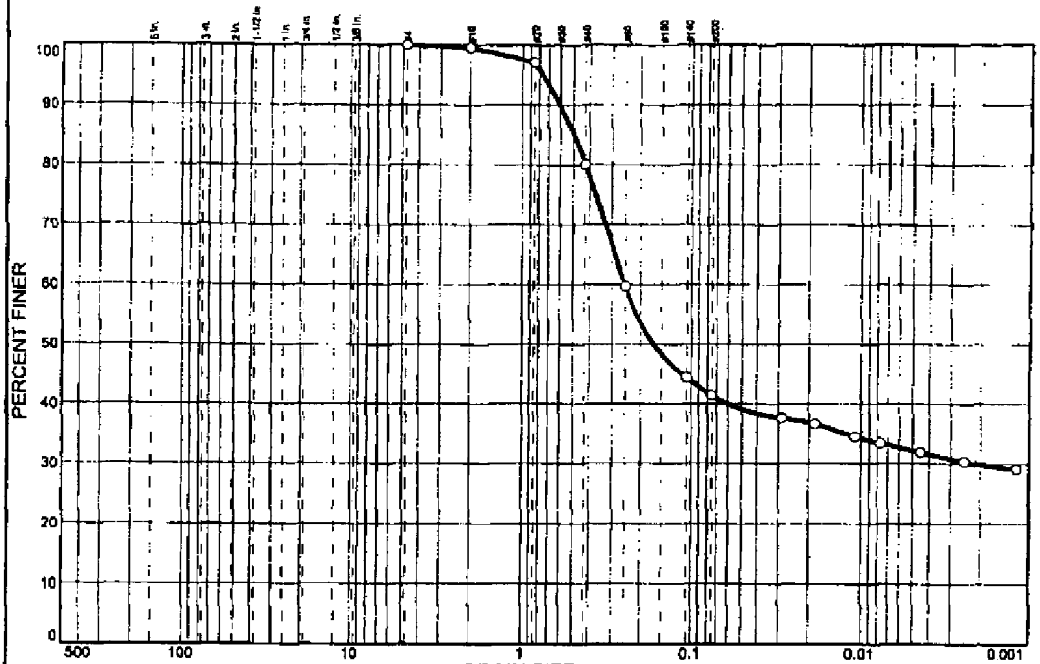
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 4 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	58.6	9.2	32.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.6		
#20	97.1		
#40	80.1		
#60	59.7		
#140	44.4		
#200	41.4		

* (no specification provided)

Soil Description

Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.496 D_{60}= 0.252 D_{50}= 0.168
D_{30}= 0.0022 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-4(0)

Remarks

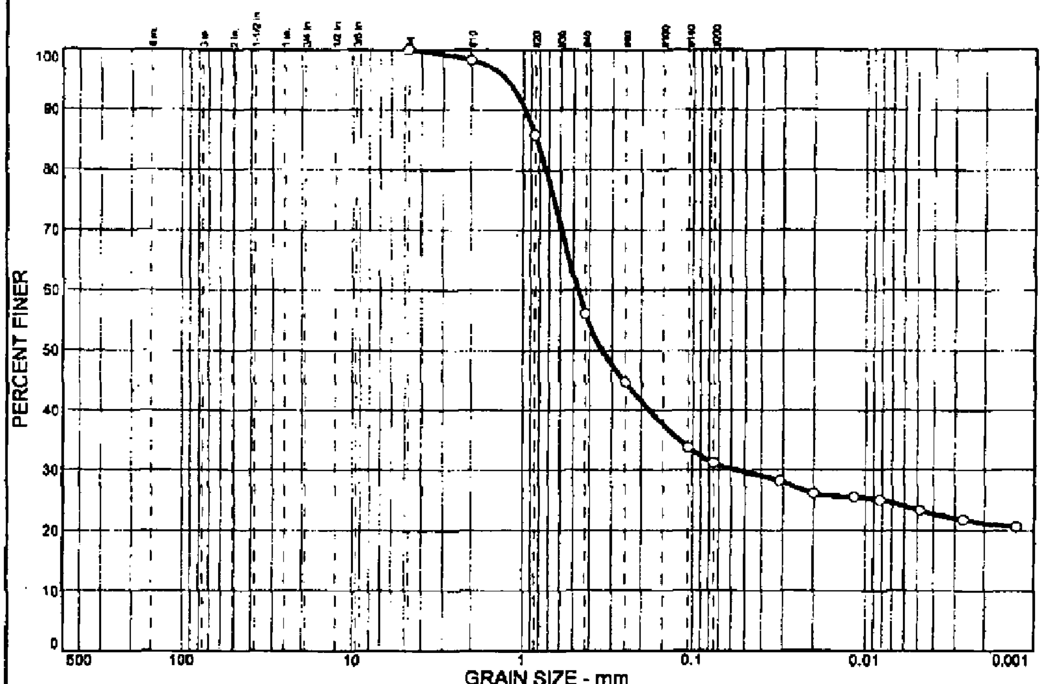
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 6 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	68.8	7.8	23.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.3		
#20	85.8		
#40	56.1		
#60	44.7		
#140	33.8		
#200	31.2		

* (no specification provided)

Soil Description

Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.831 D_{60}= 0.471 D_{50}= 0.340
D_{30}= 0.0562 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

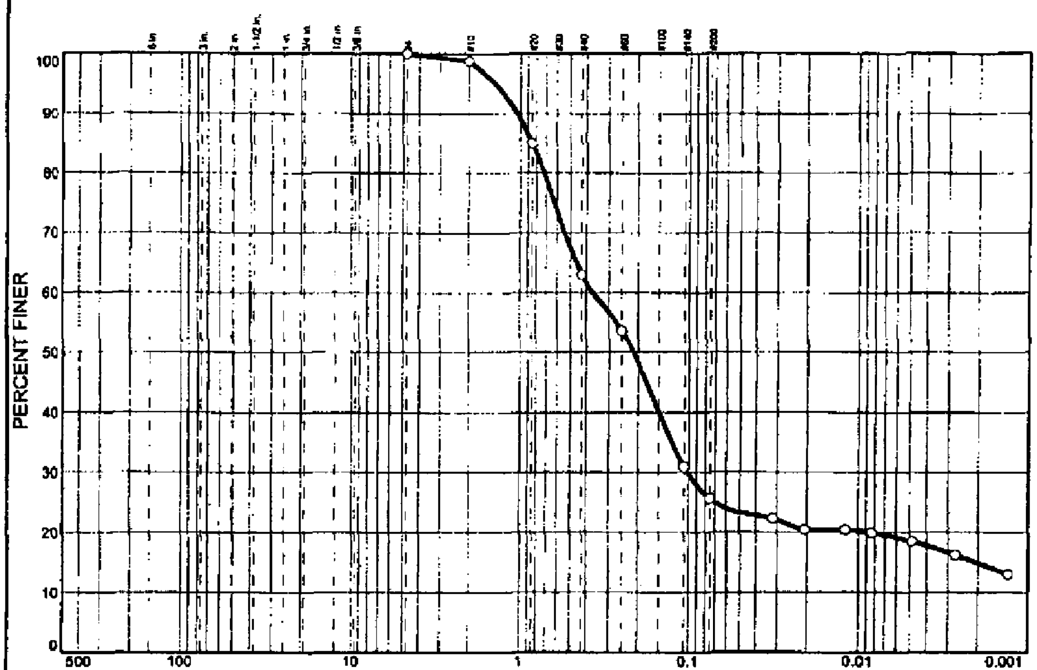
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 8 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	74.4	7.0	18.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.8		
#20	85.0		
#40	63.0		
#60	53.6		
#140	30.9		
#200	25.6		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.850 D_{60}= 0.367 D_{50}= 0.214
D_{30}= 0.101 D_{15}= 0.0021 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

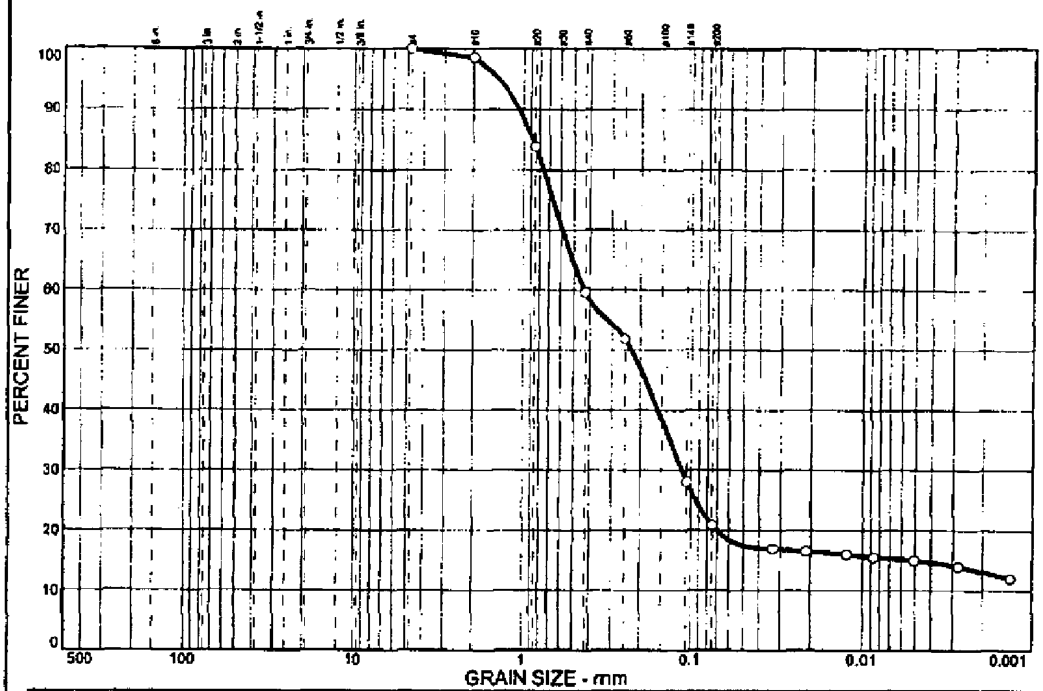
Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 10 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	79.2	5.8	15.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.6		
#20	83.9		
#40	59.6		
#60	51.8		
#140	28.1		
#200	20.8		

* (no specification provided)

Soil Description

Purple Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.881 D_{60}= 0.432 D_{50}= 0.228
D_{30}= 0.114 D_{15}= 0.0050 D_{10}=
C_u= C_c=

Classification

USCS= AASHTO= A-2-4(0)

Remarks

Tested by: HJ Reviewed by: JW

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/12/05
Location: TP-2 **Elev./Depth:** 12 Ft.

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

QORE PROPERTY SCIENCES

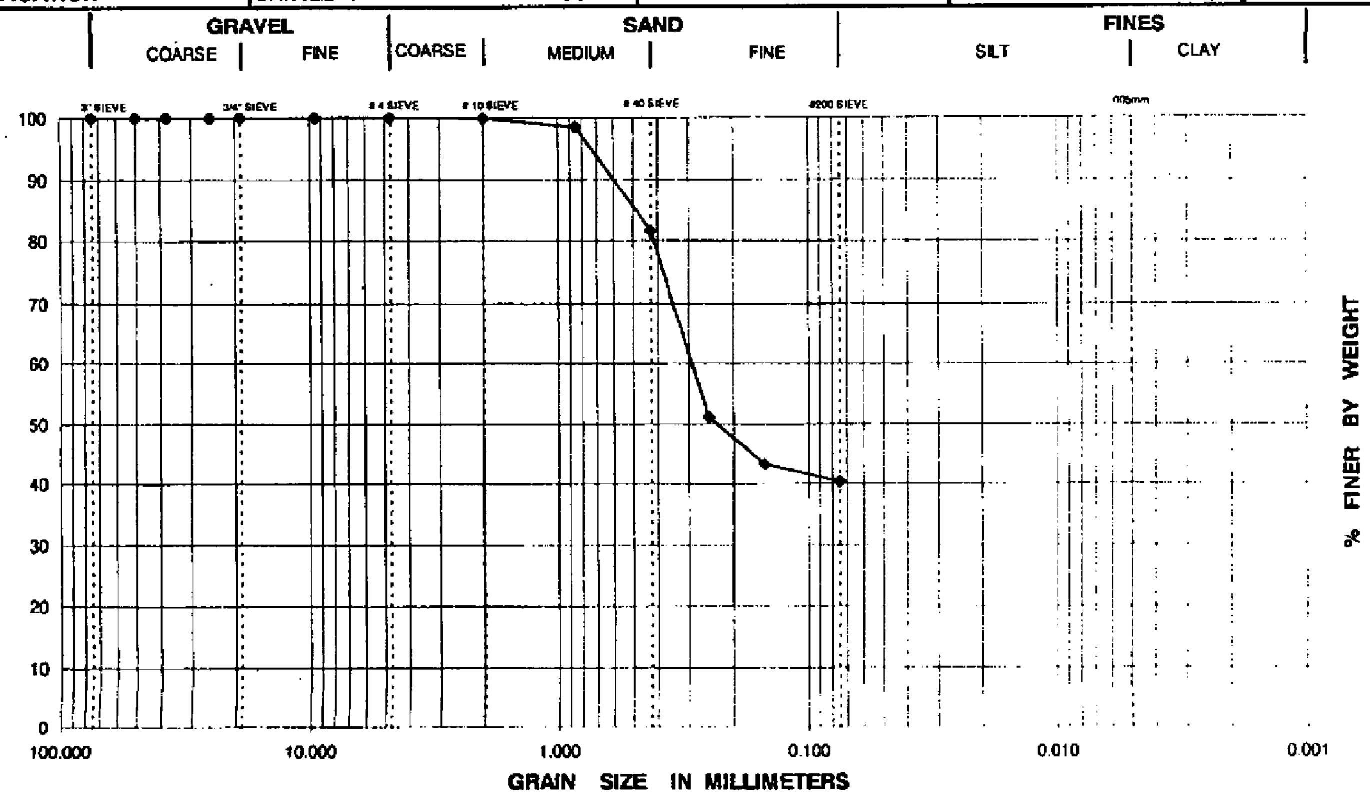
PARTICLE- SIZE DISTRIBUTION TEST REPORT

SIEVE AND HYDROMETER ASTM D422 -63

AASHTO R18

REV1,9/17/03

JOB NAME :	Saltstone Vault # 2,Task 8			CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/23/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	6'-8'	SAMPLE NO. :	ST1	SAMPLE TYPE :	UD
SAMPLE LOCATION : -							
SOIL DESCRIPTION :	Orange brown clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	43	PLASTICITY INDEX , % :	24	MOISTURE , % :	-	FINES , % :	40
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



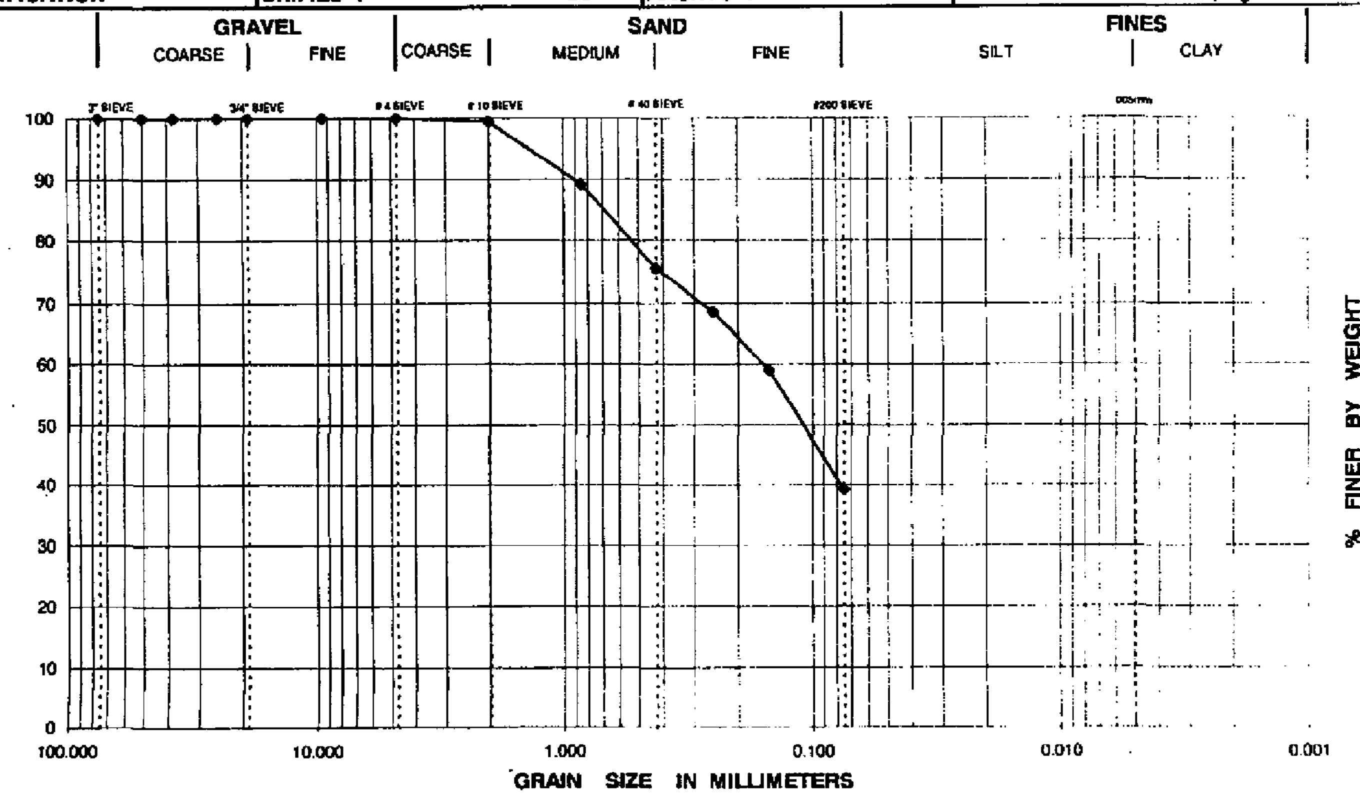
QORE

PARTICLE- SIZE DISTRIBUTION TEST REPORT

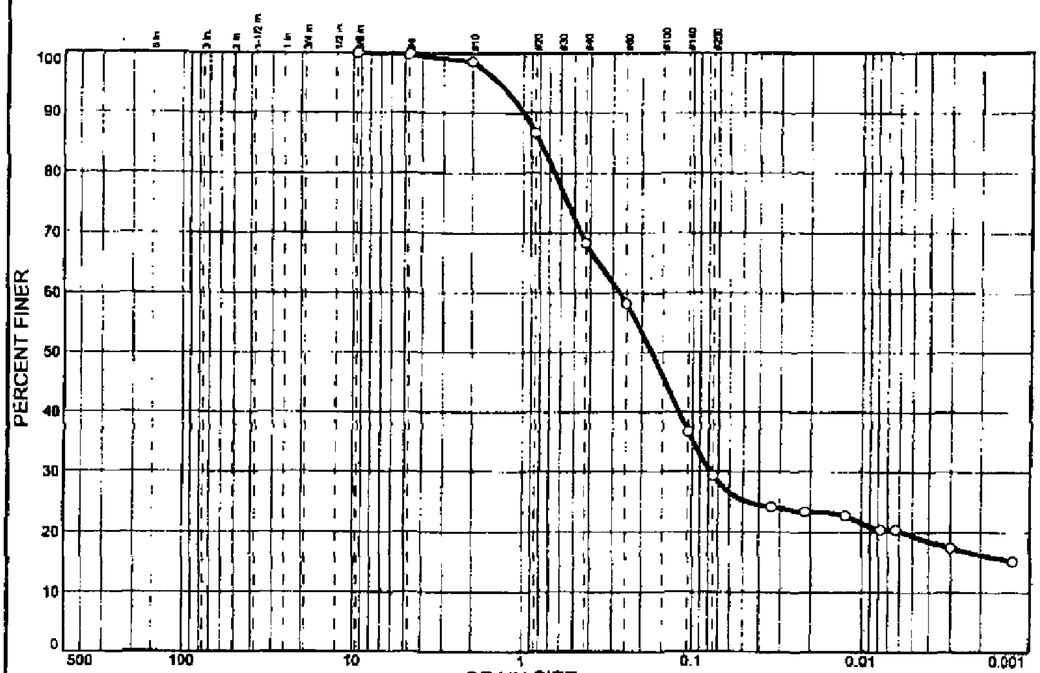
SIEVE AND HYDROMETER ASTM D422 -63

REV1,8/17/03

JOB NAME :	Saltstone Vault # 2,Task 8				CONTRACT NO. AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO. :	24150-H	REPORT NO. :	-	DATE :	6/28/05	REVIEWED BY :	
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	14'-16'	SAMPLE NO. :	ST2	SAMPLE TYPE :	UD
SAMPLE LOCATION :	-						
SOIL DESCRIPTION :	Red brown and purple clayey sand.					SP. GRAVITY, Gs :	-
LIQUID LIMIT, % :	42	PLASTICITY INDEX , % :	23	MOISTURE , % :	-	FINES , % :	39
D10 , MM :	N/A	D30 , MM :	N/A	D60 , MM :	N/A	COEFF. OF CURVATURE , C_c :	N/A
CLASSIFICATION		UNIFIED :	SC	AASHTO :	N/A	COEFF. OF UNIFORMITY , C_u :	N/A



Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	70.5	9.9	19.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	99.8		
#10	98.6		
#20	86.7		
#40	68.3		
#60	58.2		
#140	36.7		
#200	29.3		

* (no specification provided)

Soil Description

Brown Clayey sand

Atterberg Limits

PL= 17 LL= 38 PI= 21

Coefficients

D_{85}= 0.792 D_{60}= 0.274 D_{50}= 0.176
D_{30}= 0.0781 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-6(2)

Remarks

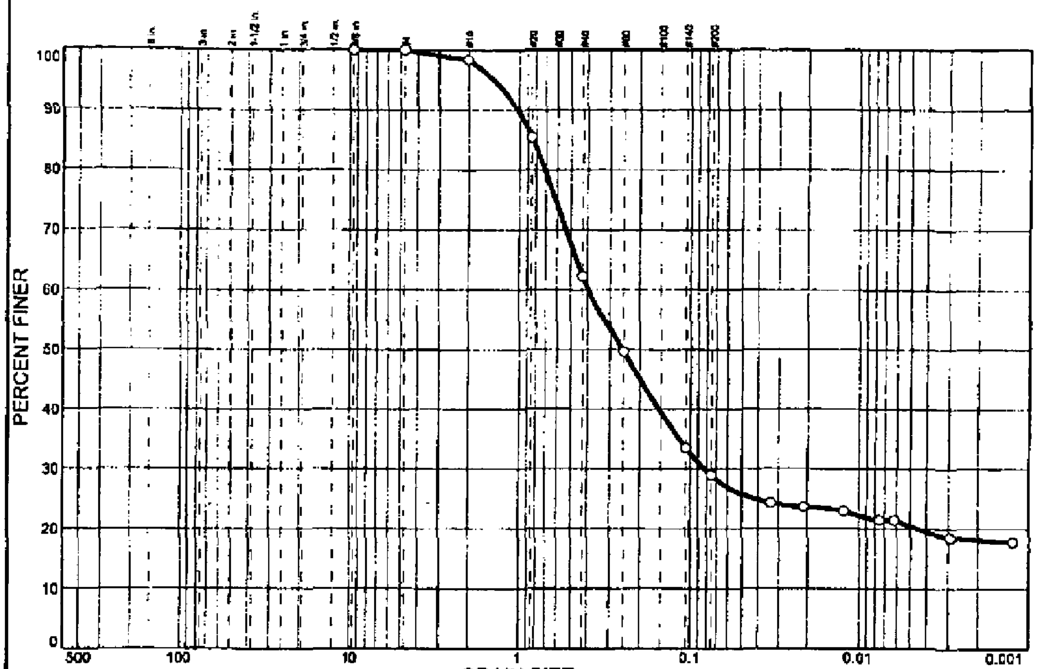
Tested by: JW Reviewed by: HJ

Sample No.: Bag **Source of Sample:** TP-1 **Date:** 5/26/05
Location: TP-1 Composite **Elev./Depth:** 2-12 ft

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

Particle Size Distribution Report ASTM D422/ASTM D1140



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	71.1	8.4	20.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	98.4		
#20	85.4		
#40	62.3		
#60	49.6		
#140	33.5		
#200	28.9		

* (no specification provided)

Soil Description

Brown Clayey sand

Atterberg Limits

PL= 20 LL= 39 PI= 19

Coefficients

D_{85}= 0.838 D_{60}= 0.392 D_{50}= 0.255
D_{30}= 0.0825 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO= A-2-6(1)

Remarks

Tested by: JW Reviewed by: HJ

Sample No.: Bag **Source of Sample:** TP-2 **Date:** 5/26/05
Location: TP-2 Composite **Elev./Depth:** 4-12 ft

GeoTesting Express Inc.

Client: Westinghouse Savannah River Company
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Project No: GTX G0821 **Figure**

COMPACTION TEST REPORT

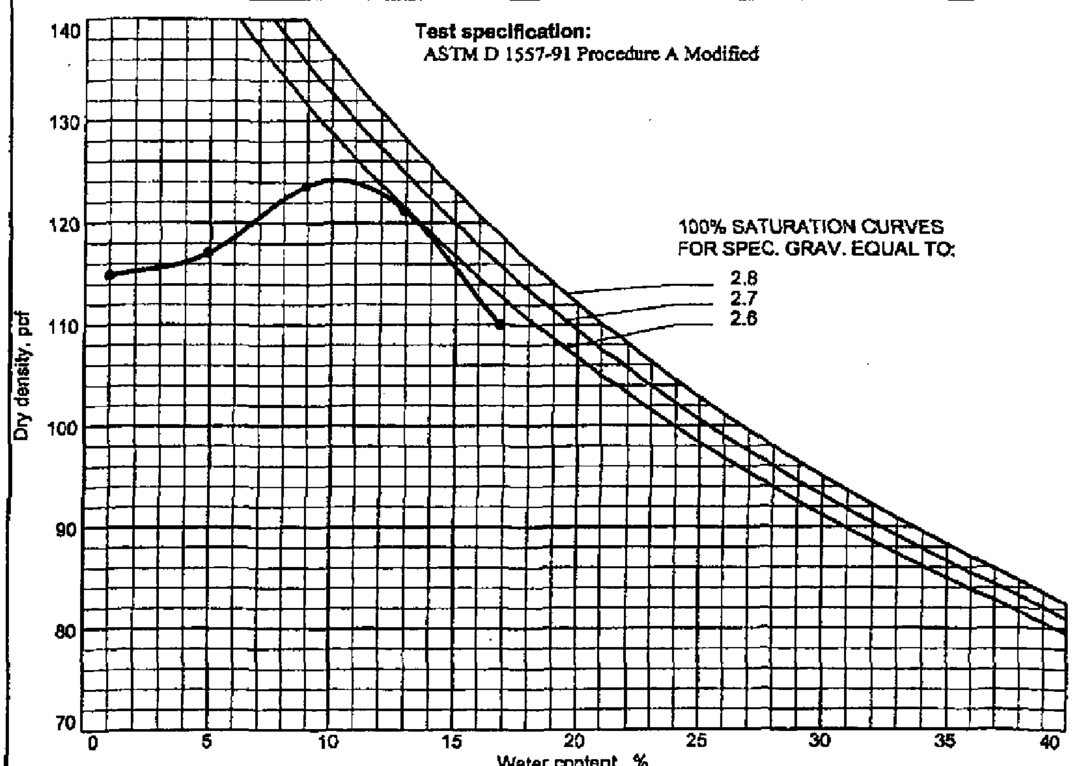
Project No.: GTX G0821 **Date:** 5/24/05
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Location: TP-1 Composite
Elev./Depth: 2-12 ft **Sample No.** Bag
Remarks: Tested by: SS Reviewed by: HJ

MATERIAL DESCRIPTION

Description: Brown Clayey sand

Classifications - **USCS:** SC **AASHTO:** A-2-6(2)
Nat. Moist. = **Sp.G. =** 2.7
Liquid Limit = 38 **Plasticity Index =** 21
% > No.4 = 0.2 % **% < No.200 =** 29.3 %

TEST RESULTS
Maximum dry density = 124.2 pcf
Optimum moisture = 10.2 %



Figure

GeoTesting Express Inc.

COMPACTION TEST REPORT

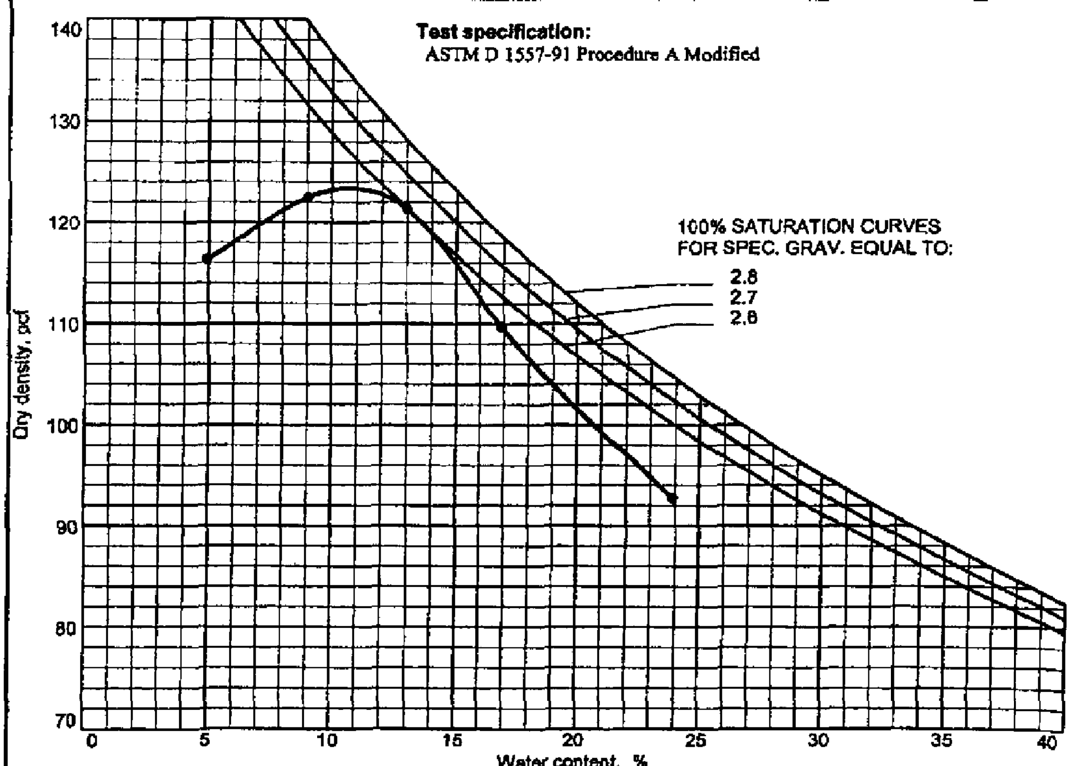
Project No.: GTX G0821 **Date:** 5/24/05
Project: Saltstone Vault #2 Bulk Samples
Task Order Number 13 Contract No. AB80188N
Location: TP-2 Composite
Elev./Depth: 4-12 ft **Sample No.** Bag
Remarks: Tested by: SS Reviewed by: HJ

MATERIAL DESCRIPTION

Description: Brown Clayey sand

Classifications - **USCS:** SC **AASHTO:** A-2-6(1)
Nat. Moist. = **Sp.G. =** 2.70
Liquid Limit = 39 **Plasticity Index =** 19
% > No.4 = 0.0 % **% < No.200 =** 28.9 %

TEST RESULTS
Maximum dry density = 123.4 pcf
Optimum moisture = 10.7 %



Figure

QORE

TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

REV 1/13/01

JOB NAME:	Saltstone Vault # 2, Task 8	CONTRACT AB 80187N . SPECIFICATION K-SPC-G-00013 REV8	
JOB NO.: 24150-H	REPORT NO. : N/A	REVIEWED BY :	DATE: 6/17/05
BORING / PIT NO. : ZV2B2U	DEPTH / ELEV. : 6'-8'	SAMPLE NO . : ST1	TYPE: UD
SAMPLE LOCATION : N/A			
SOIL DESCRIPTION : Orange brown clayey sand.			
LL, % : 43	PI, % : 24	FINES, % : 40	G_s : 2.70

SPECIMEN PROPERTIES

	INITIAL				AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.88	2.88	2.88	D_c	2.87	2.86	2.86
HEIGHT , INCHES	H_o	5.90	5.84	5.84	H_c	5.88	5.81	5.80
WATER CONTENT, %	W_o	20.2	19.8	16.1	W_c	22.1	20.9	17.8
DRY DENSITY, PCF	γ_{dryo}	104.4	105.8	111.6	γ_{dryc}	105.5	107.7	113.9
SATURATION , %	S_o	88.9	90.1	85.0	S_c	100	100	100
VOID RATIO	e_o	0.615	0.594	0.511	e_c	0.598	0.565	0.480

TEST PARAMETERS , TEST TYPE : CU/PP

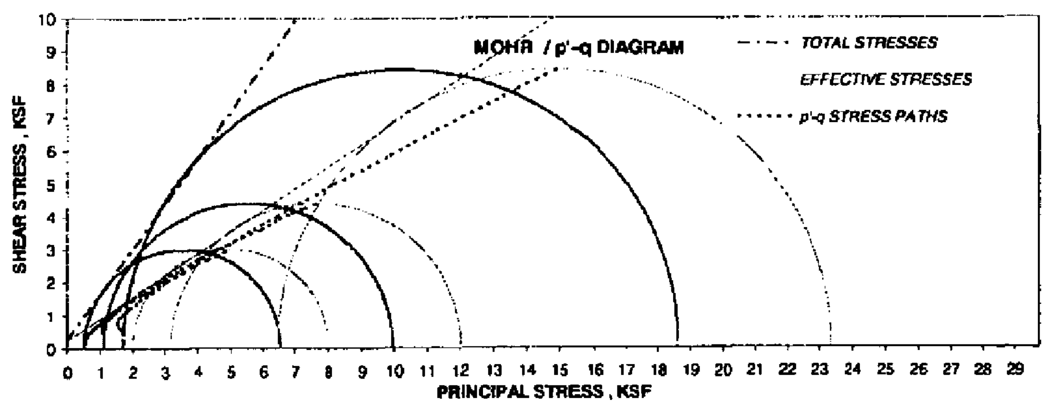
SPECIMEN NO.		1	2	3
B Value		0.95	0.96	0.95
BACK PRESSURE, ksf	U_o	10.1	10.1	10.1
CONFINING PRESSURE , ksf	σ_3'	0.5	1.1	1.7
MAX. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	6.0	8.8	16.9
ULT. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	6.0	8.8	16.9
Specimen Shape @ Failure	Bulged			

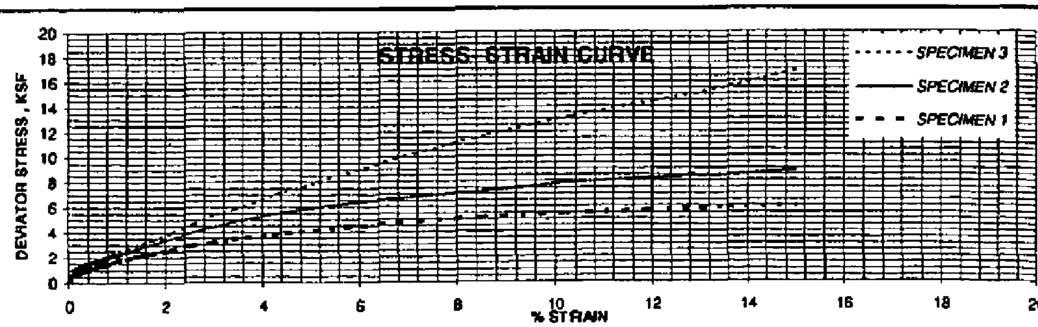
CONTROLLED : Strain @ 0.44 % per minute T50, Minutes = 0.9

N/A

N/A

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION , C (ksf) :	0.30	APPARENT COHESION , (ksf) :	0.25
	ANGLE OF INTER. FRICTION, Φ (DEGREES) :	54.0	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	33.4





QORE

TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

JOB NAME:	Saltstone Vault # 2 ,Task 8		CONTRACT AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO.:	24150-H	REPORT NO. : N/A	REVIEWED BY :	DATE:	6/23/05
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. : 14'-16'	SAMPLE NO. : ST2	TYPE:	UD
SAMPLE LOCATION :	N/A				
SOIL DESCRIPTION :	Red brown and purple clayey sand.				
LL, % :	42	PI , % : 23	FINES, % : 39	G_s :	2.70

SPECIMEN PROPERTIES		INITIAL			AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.89	2.88	*	D_c	2.88	2.87	*
HEIGHT , INCHES	H_o	5.90	5.83	*	H_c	5.88	5.80	*
WATER CONTENT, %	W_o	15.7	14.1	*	W_c	18.5	15.8	*
DRY DENSITY, PCF	γ_{dryo}	111.3	116.1	*	γ_{dryc}	112.1	118.0	*
SATURATION ,%	S_o	82.9	85.0	*	S_c	100	100	*
VOID RATIO	e_o	0.511	0.448	*	e_c	0.500	0.426	*

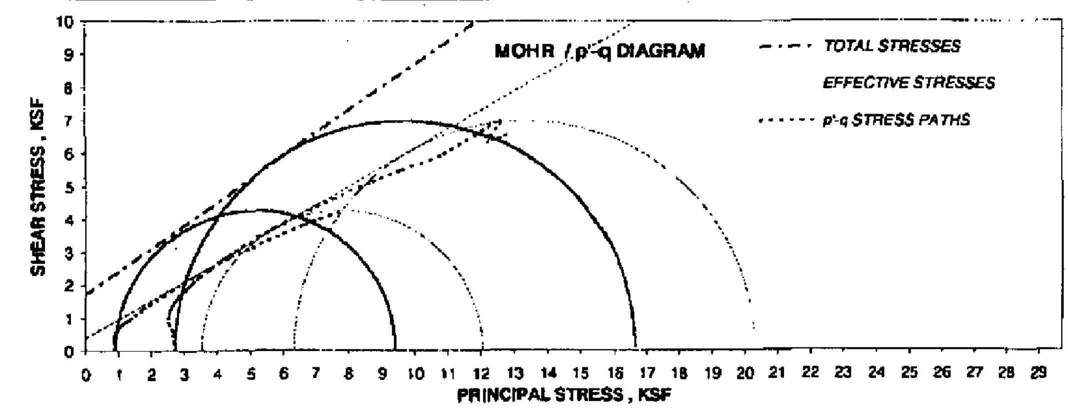
TEST PARAMETERS , TEST TYPE : CU/PP		1	2	3
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	*
BACK PRESSURE, ksf	U_o	10.1	10.1	*
CONFINING PRESSURE , ksf	σ_3	0.9	2.7	*
MAX. DEVIATOR STRESS ,ksf	σ_1-σ_3	8.5	14.0	*
ULT. DEVIATOR STRESS , ksf	σ_1-σ_3	8.5	13.2	*
Specimen Shape @ Failure	Bulged			*

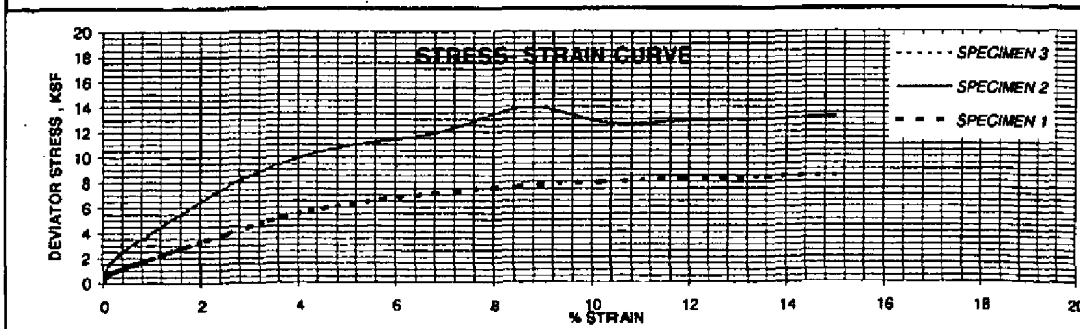
CONTROLLED : Strain @ 0.15 % per minute T50, Minutes = 2.6

Note : * Due to limited sample recovery.Only two specimens were tested as requested by SRS.

Note : Strength parameters were estimated as requested by client based on two specimens.

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION , C (ksf) :	1.70	APPARENT COHESION , (ksf) :	0.38
	ANGLE OF INTER. FRICTION, Φ(DEGREES) :	35.0	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	30.0





TRIAXIAL SHEAR TEST REPORT
(ASTM D 4767) -88

JOB NAME:	Saltstone Vault # 2 ,Task 8		CONTRACT AB 80187N , SPECIFICATION K-SPC-G-00013 REV8		
JOB NO.:	24150-H	REPORT NO. : N/A	REVIEWED BY :	DATE:	8/1/05
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. : 28'-30'	SAMPLE NO. : ST3	TYPE:	UD
SAMPLE LOCATION :	N/A				
SOIL DESCRIPTION :	Orange brown clayey sand.				
LL, % :	38	PI , % : 20	FINES, % : 16	G_s :	2.68

SPECIMEN PROPERTIES								
	INITIAL				AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.88	2.88	2.89	D_c	2.87	2.87	2.87
HEIGHT , INCHES	H_o	5.74	5.92	5.88	H_c	5.72	5.89	5.85
WATER CONTENT, %	W_o	16.4	14.6	14.6	W_c	19.8	19.4	18.7
DRY DENSITY, PCF	γ_{dryo}	107.8	108.3	109.7	γ_{dryc}	109.3	110.0	111.5
SATURATION ,%	S_o	79.7	72.2	74.9	S_c	100	100	100
VOID RATIO	e_o	0.551	0.544	0.524	e_c	0.530	0.520	0.501

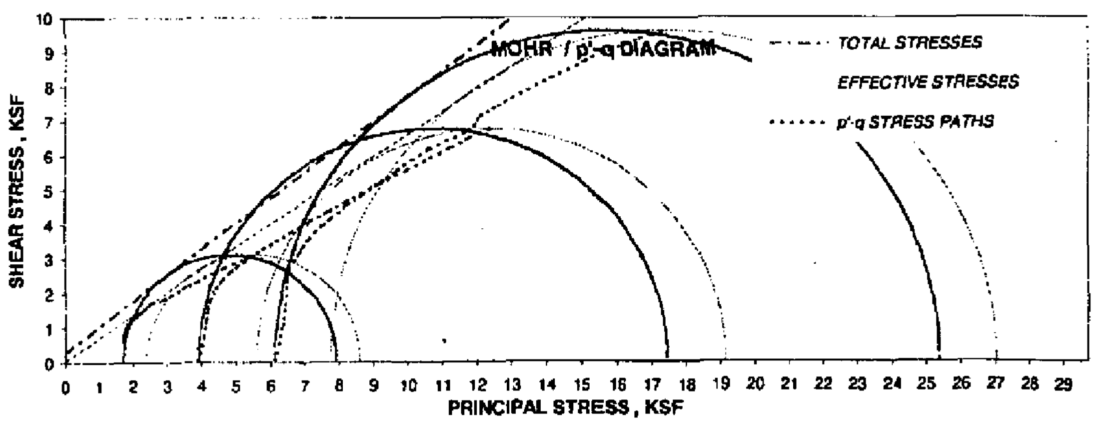
TEST PARAMETERS , TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	0.95
BACK PRESSURE, ksf	U_o	10.1	10.0	10.0
CONFINING PRESSURE , ksf	σ_3	1.7	3.9	6.1
MAX. DEVIATOR STRESS ,ksf	$\sigma_1-\sigma_3$	6.2	13.5	19.3
ULT. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	6.2	13.5	19.3
Specimen Shape @ Failure	Bulged			

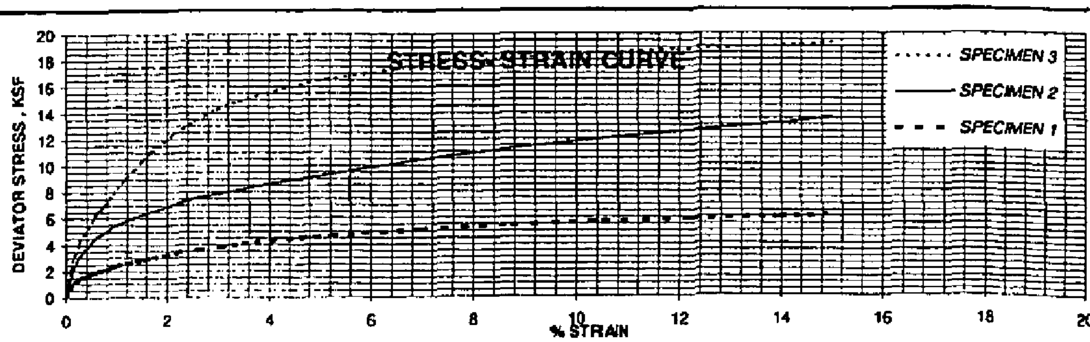
CONTROLLED : Strain @ 3.64 % per minute T50, Minutes = 0.11

N/A

N/A

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION , C (ksf) :	0.25	APPARENT COHESION , (ksf) :	0.05
	ANGLE OF INTER. FRICTION, Φ(DEGREES)	36.8	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	33.3





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TRIAXIAL SHEAR TEST REPORT

(ASTM D 4767) -88

AAP

REV. 1/13/04

JOB NAME:	Saltstone Vault # 2 , Task 8			CONTRACT AB 80187N , SPECIFICATION K-SPC-G-00013 REV8			
JOB NO.:	24150-H	REPORT NO. :	N/A	REVIEWED BY :		DATE:	6/17/05
BORING / PIT NO. :	ZV2B2U	DEPTH / ELEV. :	34'-36'	SAMPLE NO . :	ST4	TYPE:	UD
SAMPLE LOCATION :	N/A						
SOIL DESCRIPTION :	Tannish yellow silty sand.						
LL, % :	N.P.	PI , % :	N.P.	FINES, % :	16	G_s :	2.63

SPECIMEN PROPERTIES								
	INITIAL				AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.87	2.88	2.88	D_c	2.86	2.86	2.86
HEIGHT , INCHES	H_o	5.94	5.86	5.91	H_c	5.91	5.84	5.86
WATER CONTENT, %	W_o	17.8	19.3	19.8	W_c	24.2	22.6	23.0
DRY DENSITY, PCF	γ_{dryo}	99.0	101.6	99.8	γ_{dryc}	100.2	102.9	102.3
SATURATION ,%	S_o	71.3	82.3	80.8	S_c	100	100	100
VOID RATIO	e_o	0.658	0.615	0.644	e_c	0.637	0.595	0.605

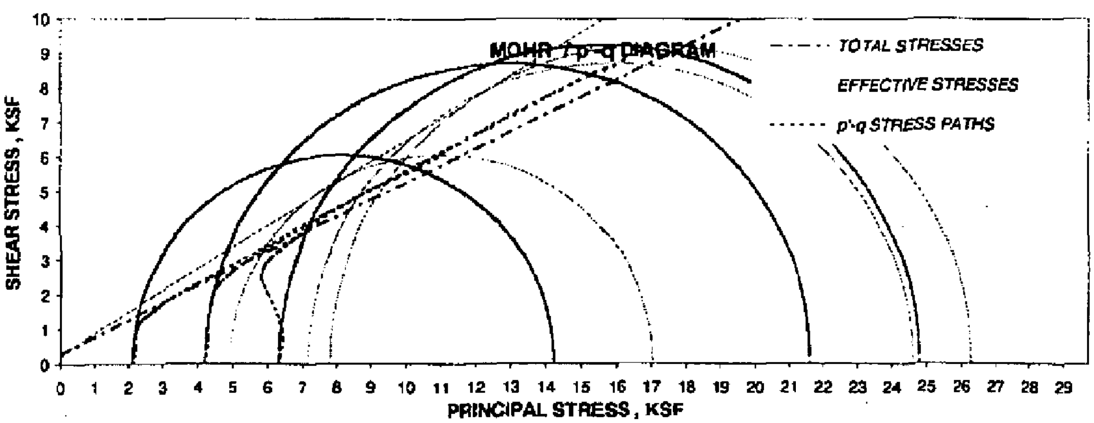
TEST PARAMETERS , TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.95	0.95	0.95
BACK PRESSURE, ksf	U_o	10.0	10.0	10.1
CONFINING PRESSURE , ksf	σ_3	2.1	4.2	6.3
MAX. DEVIATOR STRESS ,ksf	$\sigma_1-\sigma_3$	12.1	17.4	18.5
ULT. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	12.1	17.4	18.5
Specimen Shape @ Failure	Bulged			

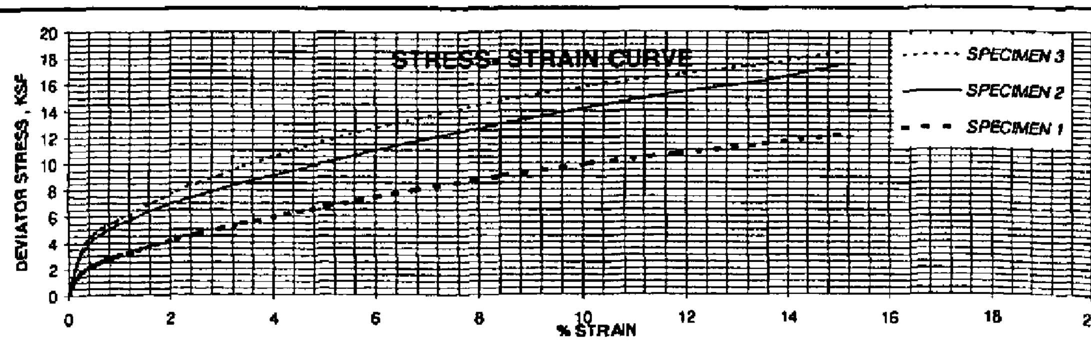
CONTROLLED : Strain @ 3.64 % per minute T50, Minutes = 0.11

N/A

N/A

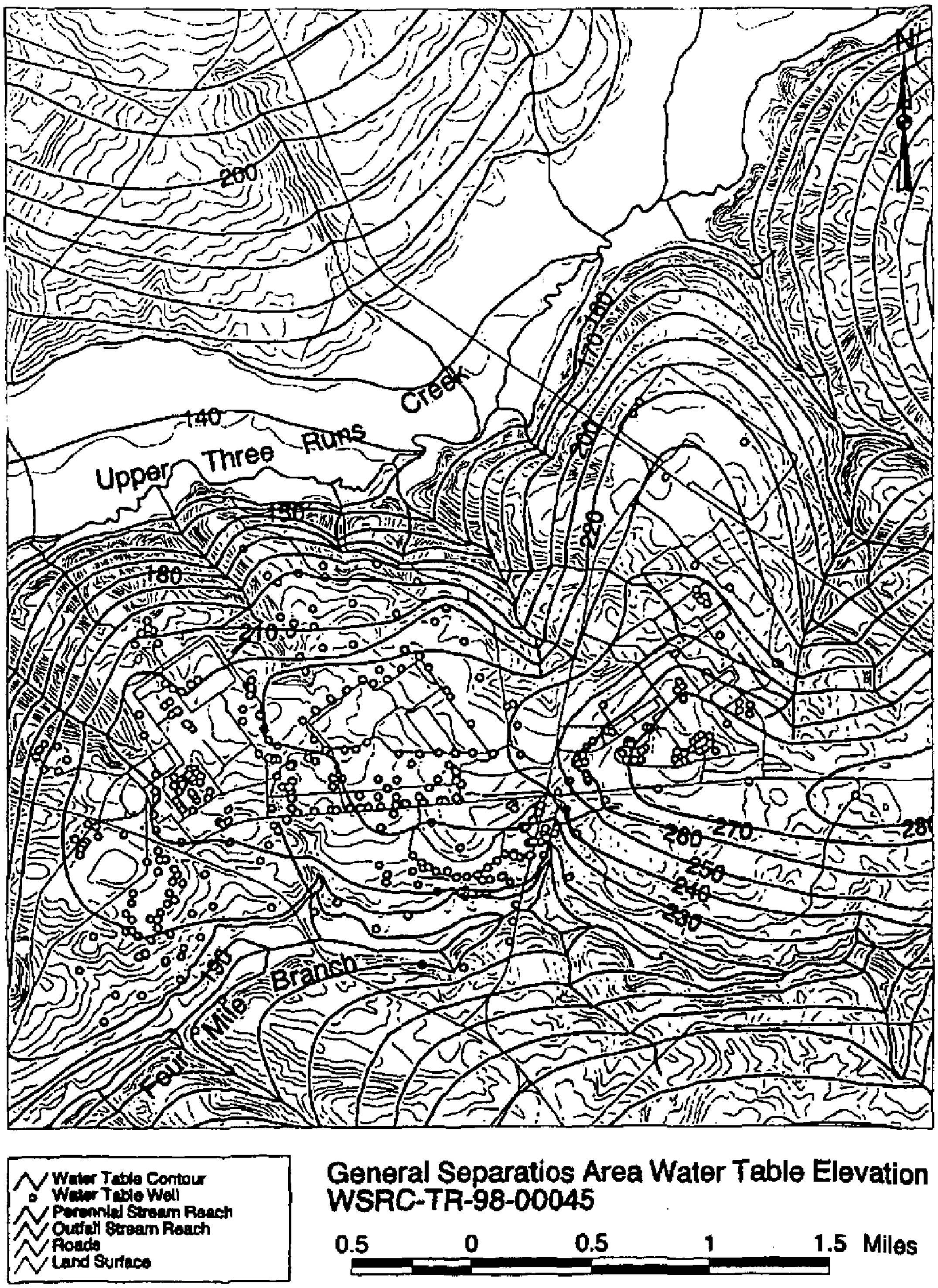
SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION , C (ksf) :	0.25	APPARENT COHESION , (ksf) :	0.26
	ANGLE OF INTER. FRICTION, Φ(DEGREES)	26.6	ANGLE OF INTER. FRICTION, Φ' (DEGREES) :	32.0



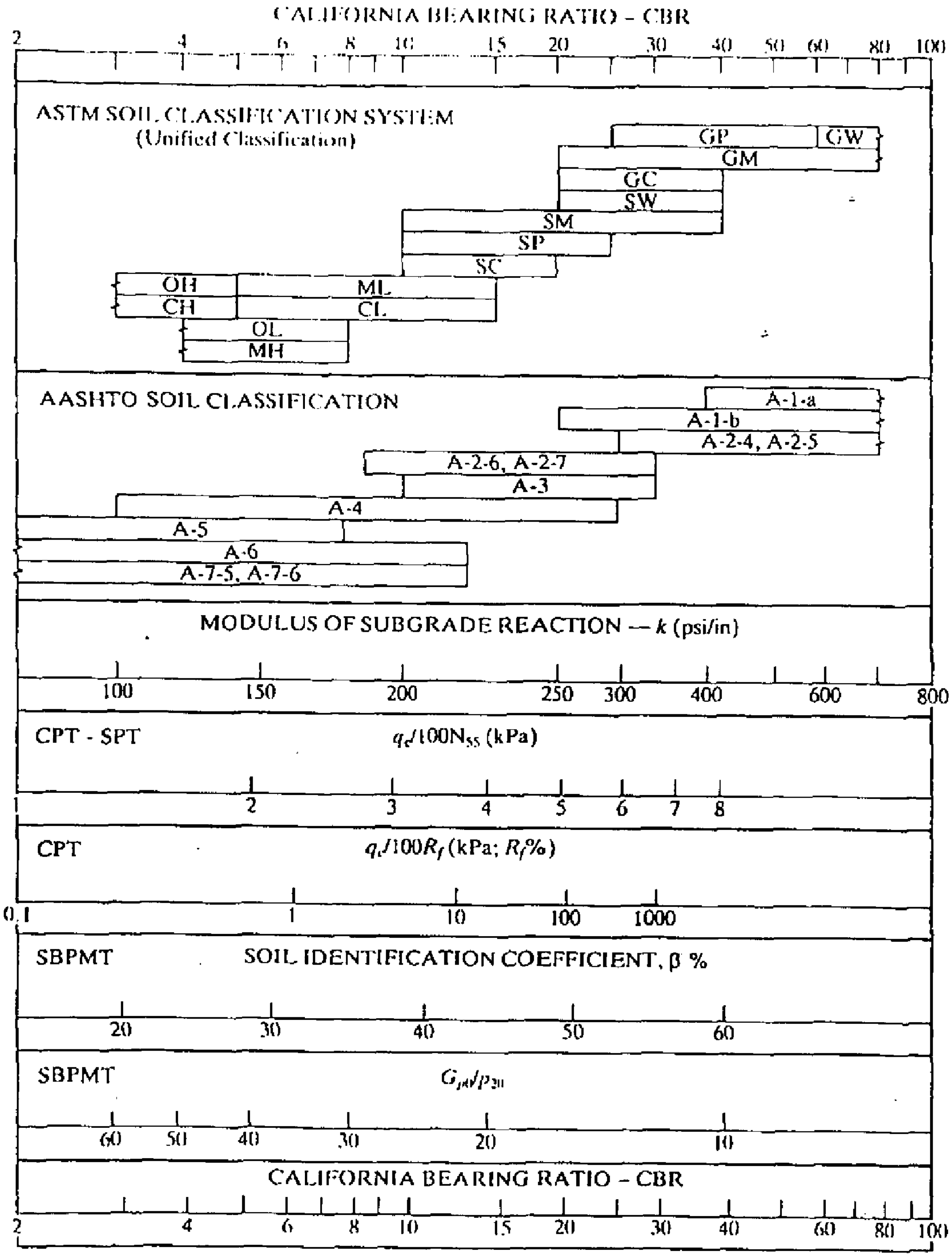


Appendix D Excerpts from Reference 5

Figure 3 General Separations Area Water Table Elevation



Appendix E Excerpts from Reference 6



Appendix D

K-CLC-Z-00005

Stratigraphy for the Saltstone Vault No. 2

Rev. 0, July 2005

(6 pages)

OSR 45-24 (Rev 11-15-2004)

Calculation Cover Sheet

Project	Calculation No.	Project No.
Saltstone Vault No. 2	K-CLC-Z-00005	N/A
Title: Stratigraphy for the Saltstone Vault No. 2	Functional Classification: PS	Sheet 1 of 5
	Discipline: Geotechnical	
Calc Level: ☒ Type 1 ☐ Type 2	Type 1 Calc Status: ☐ Preliminary ☒ Confirmed	
Computer Program No.: ☒ N/A	Version/Release No.	

Purpose and Objective

Develop engineering and geologic stratigraphy for the Saltstone Vault No. 2 area for engineering analysis.

Summary of Conclusion

See results and conclusions section.

UNCLASSIFIED
DOES NOT CONTAIN
UNCLASSIFIED CONTROLLED
NUCLEAR INFORMATION
ADC & Reviewing Official: [signature]
(Name and Title)
Date: 7-26-2005

Revisions

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	F. SYMS [signature] 6-30-05	Document review	L. BAGWELL [signature] 6/30/05	M. LEWIS [signature] 7/26/05

Design Authority – (Print) NA	Signature NA	Date NA
Release to Outside Agency – (Print) NA	Signature NA	Date NA

Security Classification of the Calculation

UNCLASSIFIED

ENGINEERING DOC. CONTROL – SRS

00699440

1. INTRODUCTION

This calculation identifies engineering stratigraphic layers for CPTU soundings performed for the Saltstone Vault No. 2. Engineering stratigraphic layers and nomenclature were based on previous subsurface investigations performed within Z Area. Geologic formations were correlated from regional type wells and sediment samples acquired from the SPT borings and CPT soundings as part of previous investigations as well as exploration completed specifically for the Vault No. 2 location.

2. INPUT DATA

Subsurface data acquired for the Saltstone Vault No. 2 investigation included six (6) seismic piezocone penetration test (SCPTU) soundings pushed to the point of refusal, four (4) shallow piezocone penetration test soundings (CPTU), three (3) standard penetration test (SPT) borings and, three (3) undisturbed (UD) borings (Figure 1). One of the six SCPTU soundings (ZV2CP15) was pushed near SPT boring ZV2B1 for the purpose of acquiring data through a soft interval noted in the SPT boring. The four shallow CPTU soundings were utilized for slope stability analysis and the stratigraphy for these borings was not interpreted as part of this effort.

3. METHOD FOR DETERMINATION OF ENGINEERING LAYERS

Engineering layers developed for the Saltstone Vault No. 2 area followed a scheme previously used by other investigations in Z Area. Utilization of this same layering provided a means to compare subsurface conditions between investigation sites. A primary S and C designation was used to divide layers into predominantly sand and clay units respectively. Layers were secondarily divided based on clay and silt content. This layering system was extended to the Saltstone Vault No. 2 area by correlating CPT curve signatures to SPT N-values and soil descriptions.

Of most significance beneath the Saltstone Vault No. 2 site is the presence of a thick, soft clay unit correlative to layer C2, which was extensively mapped during previous investigations throughout Z Area. Thus, the top and bottom of the C2 layer were delineated from the seven deep SCPTU soundings.

4. METHOD FOR DETERMINATION OF STRATIGRAPHIC LAYERS

Geologic stratigraphy (specifically the top of the Santee/Tinker Formation) was determined directly from the three SPT borings drilled as part of this investigation. 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 formations overlying the Santee/Tinker Formation were not differentiated as part of this study.

The top of the Santee/Tinker Formation was determined visually from soil samples as a marked change in the gradation, color, mineralogy, and reaction to dilute 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.

5. RESULTS AND CONCLUSIONS

Engineering stratigraphy and top of the Santee/Tinker Formation for Saltstone Vault No. 2 CPT soundings and SPT borings are summarized in Table 1.

6. REFERENCES

K-CLC-Z-00003, Subsurface Stratigraphy for Saltstone Vault No. 4, Rev 0.

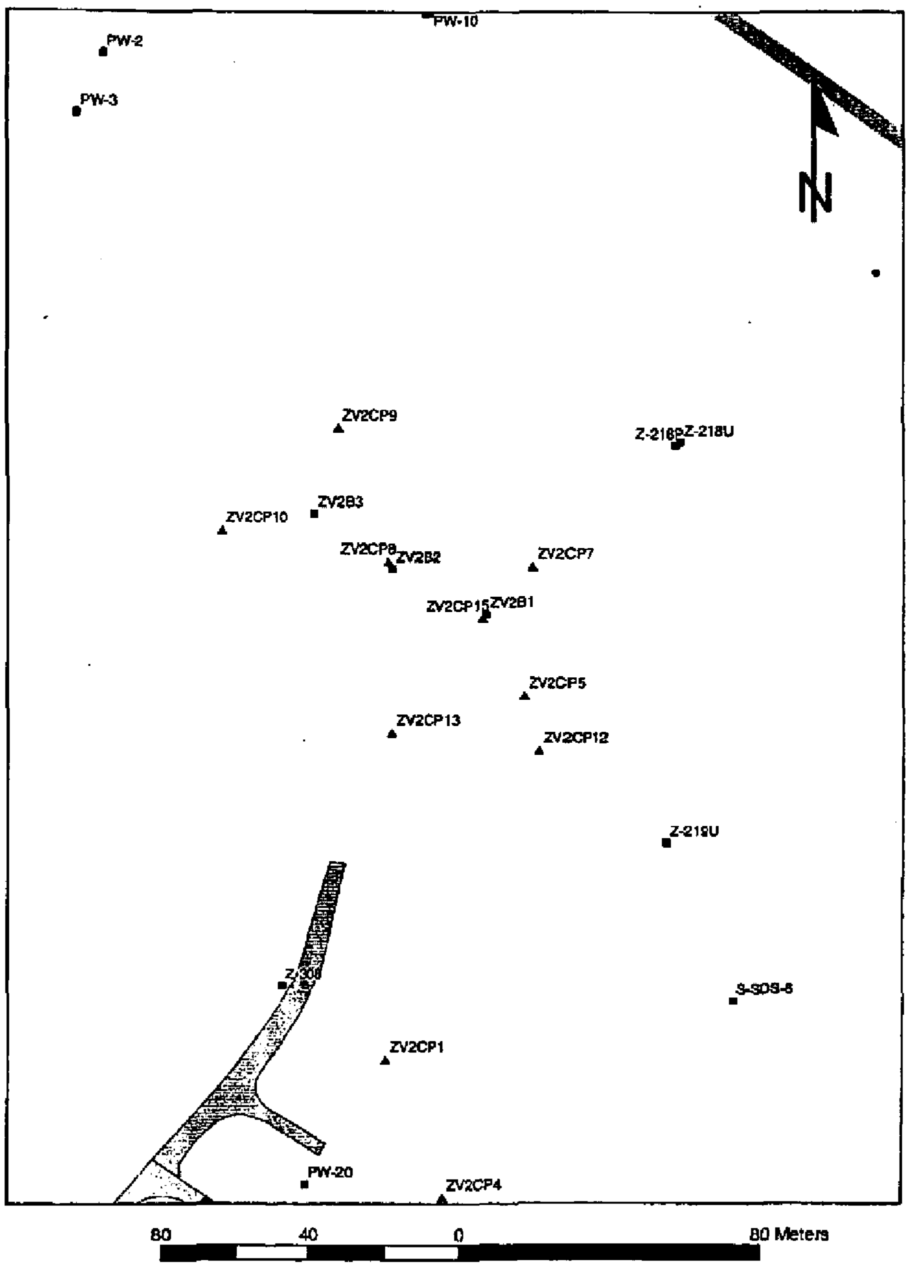

PW-2
PW-3
PW-10
N
ZV2CP9
Z-218U
ZV2B3
ZV2CP10
ZV2CP8
ZV2B2
ZV2CP7
ZV2CP15
ZV2B1
ZV2CP5
ZV2CP13
ZV2CP12
Z-219U
S-SDS-6
ZV2CP1
PW-20
ZV2CP4
80
40
0
80 Meters

Figure 1

Calculation No. K-CLC-Z-00005, Rev. 0

ID	Surf Elv (MSL)	UTM_E	UTM_N	Surface	PICK_ELEV	PICK_DEPTH
ZV2CP10	279	440310.52	3685228.28	top C2	228	51
ZV2CP10	279	440310.52	3685228.28	bot C2	213	66
ZV2CP10	279	440310.52	3685228.28	top S4	186	93
ZV2CP15	281	440380.88	3685204.06	top C2	223	58
ZV2CP15	281	440380.88	3685204.06	bot C2	208	73
ZV2CP15	281	440380.88	3685204.06	top S4	181	100
ZV2CP5	287	440391.92	3685182.70	top C2	227	60
ZV2CP5	287	440391.92	3685182.70	bot C2	208	79
ZV2CP5	287	440391.92	3685182.70	top S4	176	111
ZV2CP6	283	440354.33	3685193.42	top C2	230	53
ZV2CP6	283	440354.33	3685193.42	bot C2	212	71
ZV2CP6	283	440354.33	3685193.42	top S4	178	105
ZV2CP7	279	440393.89	3685218.14	top C2	223	57
ZV2CP7	279	440393.89	3685218.14	bot C2	207	73
ZV2CP7	279	440393.89	3685218.14	top S4	176	104
ZV2CP8	279	440355.63	3685219.59	top C2	224	55
ZV2CP8	279	440355.63	3685219.59	bot C2	212	67
ZV2CP8	279	440355.63	3685219.59	top S4	184	95
ZV2CP9	275	440342.44	3685256.35	top C2	212	63
ZV2CP9	275	440342.44	3685256.35	bot C2	197	78
ZV2CP9	275	440342.44	3685256.35	top S4	166	109

Appendix E
K-CLC-Z-00011
Bearing Capacity and Static Settlement
Rev. 0, April 2006

(29 pages)

OSR 45-24 (Rev 11-15-2004)

Calculation Cover Sheet

Project	Calculation No.	Project No.
Saltstone Vault No. 2	K-CLC-Z-00011	N/A
Title Bearing Capacity and Static Settlement	Functional Classification PS	Sheet 1 of 28
	Discipline Geotechnical	
Calc Level X Type 1 Type 2	Type 1 Calc Status Preliminary X Confirmed	
Computer Program No. X N/A	Version/Release No. N/A	

Purpose and Objective

This calculation provides the bearing capacity and the static settlement.

Summary of Conclusion

see last section.

UNCLASSIFIED
DOES NOT CONTAIN
UNCLASSIFIED CONTROLLED
NUCLEAR INFORMATION
ADC & Reviewing Official (Name and Title)
Date: 4-12-2006

Revisions

Rev No.	Revision Description
0	Initial issue

Sign Off

Rev No.	Originator (Print) Sign/Date	Verification/ Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign//Date
0	William Li 4/12/06	Document review	DAVID J HUIZENGA 4/12/06	M.R. Lewis 4/12/06

Design Authority — (Print)	Signature	Date
Release to Outside Agency – (Print)	Signature	Date

Security Classification of the Calculation

TABLE OF CONTENTS

1. INTRODUCTION

This calculation provides the bearing capacity and static settlement for the Z-Area Vault No. 2 (Ref. 1).

2. INPUT DATA

Input data include:

(1) Facility configurations

Elevation at Bottom of mud mat: 268 feet, MSL (Ref. 2, Attachment 5.4)

Area to be excavated: approximately 250 feet by 400 feet (Ref. 2, Attachment 5.4)

Vault inside diameter: 150 feet (Ref. 2, Section 3.1.2.1)

Vault inside height: 22 feet (Ref. 2, Section 3.1.2.1)

Weight of the fluid Saltstone : 110 psf (Ref. 2, 3.1.2.10.A)

Closure cap thickness: 13 feet to 23.5 feet (Ref. 1, Section 3.2.1)

(2) Stratigraphy

Stratigraphy at SCPTu locations (Ref. 3 and Ref. 4):

Exploratory I.D.	Ground Elevation (feet MSL)	Contact Between S1/2 and C2 (ft, MSL)	Contact Between C2 and S3 (ft, MSL)	Contact Between S3 and S4 (ft, MSL)
Z-V2-CP5	287.0	227	208	178
Z-V2-CP6	283.0	230	212	178
Z-V2-CP7	279.4	223	207	176
Z-V2-CP8	278.8	224	212	184
Z-V2-CP9	275.3	212	197	166
Z-V2-CP10	279.0	228	213	186
Z-V2-CP15	281.7	223	208	181

Elevations of area will be excavated: El. 274 ft, MSL northeast to 289 ft, MSL southwest.

Groundwater elevation: 225 $\pm$ 5 feet) (Ref. 4, Section 3.1)

(3) Soil properties:

Weight of soil above El. 268 ft, MSL (soil to be excavated and backfill) :125 pcf (Ref. 4, Section 3.2)

Weight of soil below El. 268 ft, MSL: 120 pcf (Ref. 5, Section 4.0)

Effective friction angle: 32 degrees (Ref. 5, Table 13)

Effective cohesion: 50 psf (Ref. 5, Table 13)

Energy Corrected average blow count for S1/2 layer: 31 (Ref. 5, Table 8)

Consolidation properties for layer C2 (Ref. 5, Pages 123-124, Table 14):

Sample No.	e_O	OCR	C_c	C_r
Z-V2-B1U-ST5	1.378	1.1	0.781	0.102
Z-V2-B1U-ST5a	1.391	1.0	0.725	0.083
Average	1.4	1	0.75	0.09

(4) Settlement Data

Actual measured settlement data was obtained from the Vitrification Building. Settlement and heave data of the settlement points at the four corners of the building are (Ref. 6, Appendix F):

Location	North-west	North-east	South-west	south-east
Settlement Point No.	3	18	14	9
Heave after excavation (in)	-0.276	0.000	-0.276	-0.468
Settlement 10 years after construction completion (in)	3.234	3.180	1.903	1.680
Total (in)	3.510	3.180	2.179	2.148

3. COMPUTATIONS

3.1 Bearing Capacities

This section provides the bearing capacity parameters including ultimate bearing capacity, allowable static bearing capacity, allowable dynamic bearing capacity, design bearing capacity, and strength reduction factor.

3.1.1 Ultimate Bearing Capacity

Ultimate bearing capacity q_o is computed using the equations originated by Terzaghi and later modified by others (Ref. 7):

$$q_o = q_c + q_q + q_\gamma$$

where $q_c = c\, N_c S_c D_c G_c$

$$q_q = q\, N_q S_q D_q G_q$$

$q_\gamma = \gamma\ (B/2)\ N_\gamma\ S_\gamma\ D_\gamma\ G_\gamma$

q is the overburden or surcharge pressure at the foundation base, γ is the effective soil unit weight of 120 pcf, ϕ is the friction angle of 32 degrees, c is the cohesion of 50 psf, and B is the foundation width of 150 feet. N_c, N_q, and N_γ are the bearing capacity factors; S_c, S_q, and S_γ are the shape factors; D_c, D_q, and D_γ are the depth factors; and G_c, G_q, and G_γ are the inclination factors.

The bearing capacity factors N_c and N_q are:

$$N_q = e^{\pi \tan\phi} \tan^2 (\pi/4 + \phi/2) = 23.18$$

$$N_c = \cot \phi\ (N_q - 1) = 35.49$$

The bearing capacity factor N_γ suggested by Chen in 1975 is:

$$N_\gamma = 2(N_q + 1) \tan \phi \tan (\pi/4 + \phi/5) = 37.85$$

while N_γ suggested by Hansen in 1970 is:

$$N_\gamma = 1.5\ (N_q - 1) \tan \phi = 20.79$$

The smaller N_γ is to be used to compute the bearing capacity

The shape factors S_c, S_q, and S_γ suggested by Hansen are:

$$S_c = 1 + (B/L)\ (N_q / N_c) = 1.653$$

$$S_q = 1 + (B/L) \tan \phi = 1.625$$

$$S_\gamma = 1 - 0.4\ B/L = 0.600$$

The depth factors D_c, D_q, and D_γ suggested by Hansen are:

$$D_q = 1 + 2 \tan\phi\ (1 - \sin\phi)^2\ k = 1.000$$

$$D_c = D_q - (1 - D_q)/(N_c \tan \phi) = 1.000 \text{ for } \phi > 0^\circ$$

$$D_\gamma = 1.000$$

where $k = D/B \quad (D < B)$

The inclination Factors G_c, G_q, and G_γ are 1.0 for this calculation.

Since the distance z_w from the foundation to the groundwater table is less than the foundation width B, the effective unit weight of the soil below the foundation base is taken as:

$$\gamma = \gamma' + (\gamma_m - \gamma')z_w/B$$

For this evaluation, γ_m is considered to be 120 pcf.

where γ' is the submerged unit weight and γ_m is the moist unit weight of soil. Furthermore, since the width of the foundation is larger than 6 feet, a reduction factor of (Ref. 8).

$$r_\gamma = 1 - 0.25 \log (B/6) = 0.651$$

is applied to the component of the ultimate bearing capacity, q_γ.

The components of the ultimate bearing capacity are:

$$q_c = c\ N_c\ S_c\ D_c\ G_c = 2{,}933 \text{ psf}$$

$$q_q = q\ N_q\ S_q\ D_q\ G_q = 0 \text{ psf}$$

$q_\gamma = r_\gamma \gamma (B/2) N_\gamma S_\gamma D_\gamma G_\gamma = 45{,}931$ psf

The ultimate pressure applied to foundation to cause a bearing failure is:

$q_o = q_c + q_q + q_\gamma = 48{,}864$ psf or 48,000 psf

3.1.2 Allowable Bearing Capacity

Consider a factor of safety of 3, the allowable bearing capacity is:

$q_a = q_o/3 = 48{,}864/3 = 16{,}288$ psf or 16,000 psf

3.1.3 Design Bearing Capacity

The design bearing capacity Φq_o was computed using the same equations for computing the ultimate bearing capacity. However, the strength parameters including cohesion and friction angle were reduced using appropriate strength reduction factors. The cohesion is reduced by applying a factor f_c of 0.5. For cohesion c = 50 psf, the reduced cohesion is:

$c_{reduced} = f_c c = 0.5 \times 50 = 25$ psf

The friction angle is reduced by applying a factor f_ϕ of 0.8 to the tangent of the friction angle. For friction angle $\phi = 32$ degrees, $\tan \phi = 0.625$, the reduced tangent of the friction angle is:

$f_\phi \tan \phi = 0.8 \times 0.625 = 0.500$

or $\tan(\phi_{reduced}) = 0.500$

The reduced friction angle is:

$\phi_{reduced} = 26.6$ degrees

Using a reduced cohesion $c_{reduced}$ and a reduced friction angle $\phi_{reduced}$, the bearing capacity factors, shape factors; and depth factors are:

$N_q = 23.18$

$N_c = 12.59$

$N_\gamma = 8.69$

$S_c = 1.543$

$S_q = 1.500$

$S_\gamma = 0.600$

$D_c = 1.000$

$D_q = 1.000$

$D_\gamma = 1.000$

The components of the design bearing capacity are:

$q_c = 2{,}738$ psf

$q_q = 0$ psf

$q_\gamma = 26{,}962$ psf

The design bearing capacity is:

$$\Phi q_o = q_c + q_q + q_\gamma = 21{,}938 \text{ psf or } 22{,}000 \text{ psf}$$

3.1.4 Strength Reduction Factor

Based on the ultimate bearing capacity and design bearing capacity, the strength reduction factor Φ is:

$$\Phi = (\Phi q_o)/q_o = 21{,}938/48{,}864 = 0.45$$

3.2 Settlement

Settlement is computed based on the static loadings, soil properties, and thickness of each engineering layer. Appropriate methods were chosen for cohesionless or cohesive soils. The following sections provide the static loadings, layer thicknesses, and computations of settlements.

3.2.1 Static Loadings

Static loading under the foundation, at elevation 268 feet, MSL, is used to compute the static settlement. This loading changes at various phases of the project.

Current ground surface elevation is between 275.3 to 287.0 feet, MSL. At elevation 268 feet MSL, the existing overburden pressure is from 7.3 to 19 feet of soil, or 900 to 2,400 psf. At the end of excavation, the overburden pressure reduces to zero.

At the end of construction, outside the tank, approximately 24 feet of structural fill is placed with a overburden pressure of approximately 125 pcf x 24 feet = 3,000 psf. Under the tank, assuming the weight of the foundation and mud mat is 300 psf, the roof is 25 psf, and the wall including the miscellaneous items is 175 psf, the overburden pressure is approximately a total of 500 psf. When the tank is filled with Saltstone , additional weight of 22 feet of Saltstone , or 110 psf x 22 feet = 2,420 psf plus 100 psf of grout above the Saltstone , will make the total foundation pressure approximately 3,000 psf. Therefore, for the pre-closure case, a uniform overburden pressure of 3,000 psf is considered at elevation 268 feet, MSL.

After the 13-foot to 23-foot thick closure cap is completed, additional 125 pcf x 13 feet to 125 pcf x 23.5 feet or 1,600 to 3,000 psf is added to the pressure. The total pressure is 4,600 to 6,000 psf.

Phase / Range	Existing overburden (psf)	End of Excavation (psf)	Pre-Closure (psf)	Post-Closure (psf)
Minimum	900	0	3,000	4,600
Maximum	2,400	0	3,000	6,000

3.2.2 Layer Thicknesses

Based on the stratigraphy at various SCPTu locations, the thicknesses of the layers at these SCPTu locations are:

Exploratory I.D.	Thickness to be excavated above El. 268 ft, MSL (feet)	Layer S1/2 thickness below El. 268 ft, MSL (feet)	Layer C2 thickness (feet)	Layer S3 thickness (feet)
Z-V2-CP5	19.0	41	19	30
Z-V2-CP6	15.0	38	18	34
Z-V2-CP7	11.4	45	16	31
Z-V2-CP8	10.8	44	12	28
Z-V2-CP9	7.3	56	15	31
Z-V2-CP10	11.0	40	15	27
Z-V2-CP15	13.7	45	15	27
Average	12.6	44.1	15.7	29.7

Soils in Layer S1/2 and S3 are cohesionless while soils in Layer C2 are cohesive. Following sections describe methods chosen for the computations of different types of soils.

3.2.3 Historical Settlement Data

Historical settlement data from similar site under similar conditions may be available for estimating the settlement. Settlement data is available for the Vitrification Building located southwest of the project site, less than 4,000 feet away.

Stratigraphy at the Vitrification Building site contains the same types of engineering layers as the Saltstone site. Groundwater at the Vitrification Building site is approximately 30 to 35 feet below the foundation compare to an average of 43 feet below the foundation at the Saltstone site. Consolidation properties obtained from laboratory tests are very similar (Ref. 9). The foundation of the Vitrification Building is 117 feet by 362 feet, the bottom elevation of the foundation is 270 feet, MSL, and the static building load is between 5,000 to 5,500 psf. These dimensions and load are comparable to the Saltstone Facility.

Based on the similarities of the stratigraphy, soil properties, and facility configurations, as well as the availability of the settlement data. Settlement data from the Vitrification Building was used to estimate the settlement of the project site.

The total settlement of the Vitrification Building over a 10-year period since excavation is between 2.148 to 3.510 inches. Since most of the layers are cohesionless, it is assumed that the settlement is linearly elastic and the subgrade modulus is computed as:

$$K = p/D$$

where p is the load and D is the settlement. Using an average pressure of 5,250 psf, the subgrade modulus can be computed as a minimum value of 10.39 pci and a maximum value of 16.97 pci. For phases after closure cap completed, minimum settlements were computed considering a minimum cap thickness of 13 feet and a maximum subgrade modulus while maximum settlements were computed considering a maximum cap thickness of 23.5 feet and a minimum subgrade modulus. The average heave is the average of four heaves computed using minimum and maximum subgrade moduli as well as minimum and maximum depths of excavation. The average settlement is the average of four settlements computed using minimum and maximum subgrade moduli as well as minimum and maximum cap thicknesses. The results are:

Phase	Cumulative Minimum Settlement (inches)	Cumulative Maximum Settlement (inches)	Cumulative Average Settlement (inches)
Heave after Excavation	-0.4	-1.6	-0.9
10 years after operation completed	1.2	2.0	1.6
10 years after closure cap completed	1.9	4.0	2.9

Note that settlements include the recompressions of heave.

3.2.4 Settlement of Cohesionless Layers

Settlement of cohesionless layers was computed using Burland and Burbidge method and Schmertmann's method.

3.2.4.1 Burland and Burbidge Method:

Burland and Burbidge method estimates the immediate settlement ρ_i using the relation (Ref. 10, p.3-10):

$$\rho_i = f_s\, f_1\, [(\Delta P'_{ave} - 2/3\, \sigma'_p)\, B^{0.7} I_c] \text{ for } \Delta P'_{ave} > \sigma'_p$$

$$\rho_i = f_s\, f_1\, \Delta P'_{ave} I_c/3 \text{ for } \Delta P'_{ave} < \sigma'_p$$

where f_s is the shape factor, $[(1.25L/B)/(L/B + 0.25)]^2$

f_1 is the layer correction factor, $H/Z_1(2 - H/Z_1)$

$\Delta P'_{ave}$ is the average effective pressure, $q_{0ave} + \sigma'_{0ave}$

q_{0ave} is the average pressure in stratum from foundation load,

σ'_{0ave} is the effective overburden pressure in stratum H

σ'_p is the maximum effective past pressure

H is the thickness of the layer

$Z_1 = 1.35B^{0.75}$ is the depth of influence of loaded area

N_{ave} is the average SPT blow count over depth influence by loaded area.

I_c is the compressibility influence factor, $0.23/N_{ave}^{1.4}$ with coefficient of correlation 0.848

The lower bound, upper bound, and average settlement are computed as follows:

Lower bound settlement is computed using $I_{cmin} = 0.08/N_{ave}^{1.3}$

Upper bound settlement is computed using $I_{cmax} = 1.34/N_{ave}^{1.67}$

Average settlement is computed using compressibility influence factor, I_c, defined earlier.

Settlement after time t at least 3 years following construction may be estimated by:

$\rho_c = f_t \rho_i$

where $f_t = 1 + R_i + R_c \log (t/3)$

R_i and R_c are the time–dependent settlement ratios, for t = 30 years

$R_i = 0.3$ and $R_c = 0.2$

$f_t = 1 + 0.3 + 0.2 \log (30/3) = 1.5$

Therefore, primary and secondary settlement is:

$\rho_c = 1.5\ \rho_i$

Burland and Burbidge method considers the settlement of the layer up to the depth of $Z_1 = 1.35B^{0.75}$ or 59 feet. Since the average thickness of Layer S1/2 is 44.1 feet and the average combined thickness of layers S1/2 and C2 exceeds 59 feet, only layer S1/2 with a thickness of 44.1 feet was considered as the thickness H. Dimension B is considered as 250 feet and L is considered to be 400 feet.

The lower bound, upper bound, and average settlements were computed using I_{cmin}, I_{cmax}, and I_c as described earlier. For phases after closure cap completed, lower bound settlements were computed considering a minimum cap thickness of 13 feet while upper bound settlements were computed considering a maximum cap thickness of 23.5 feet. The average settlement is the average of four settlements computed using lower and upper bounds as well as minimum and maximum cap thicknesses. The results are summarized in the following table:

Phase	Lower Bound Settlement (inches)	Upper Bound Settlement (inches)	Average Settlement (inches)
Immediate after operation completed	0.9	5.3	2.1
Immediate after closure cap completed	1.2	8.3	3.1
30 Years after closure cap completed	1.9	12.4	4.6

3.2.4.2 Schmertmann Method

Schmertmann method estimates the immediate settlement ρ_i using the equation (Ref. 10, p.3-6):

$\rho_i = C_1 C_t \Delta p \Sigma (\Delta z_i I_{zi} / E_{si})$

where C_1 is the correction to account from strain relief from embedment

C_t is the correction for time-dependent increase in settlement and $C_t = 1 + 0.2 \log (t/0.1)$

t is time in years, for t = 30 years, $C_t = 1 + 0.2 \log (30/0.1) = 1.5$.

Δp is the net applied footing pressure

Δz_i is the depth increment i

I_{zi} is the influence factor of soil layer i

$E_{si} = 2.5\, q_t$ is the elastic modulus of soil layer I for axisymmetric footings L/B = 1

where q_t is the average tip stress of soil layer i in tsf. For this evaluation, thickness of each soil layer is 2 feet.

Previous analyses performed for the Vitrification Building found that the results obtained from Schmertmann method were approximately 2.3 times the actual measurements (Ref. 11). Realistic results can be obtained by increasing the elastic modulus E_{si}, i.e., multiply E_{si} by an adjustment factor of 2.3. Since the resulting settlement is a linear function of elastic modulus, in lieu of multiply the elastic modulus by an adjustment factor, the resulting settlement was divided by the same adjustment factor. For this calculation the results were divided by a conservative factor of 2. The results are summarized in the following table:

Settlements were computed using data from each of the seven deep SCPTu's. For phases after closure cap completed, both the minimum cap thickness of 13 feet and the maximum cap thickness of 23.5 feet were considered. The average settlement is the average of 14 settlements computed using seven SCPTu's as well as minimum and maximum cap thicknesses. The results are summarized in the following table:

Phase	Minimum Settlement (inches)	Maximum Settlement (inches)	Average Settlement (inches)
Immediate after operation completed	0.7	4.1	1.6
Immediate after closure cap completed	1.1	8.5	3.0
30 Years after closure cap completed	1.6	12.8	4.4

3.2.5 Settlement of Cohesive Layer

Settlement of cohesive Layer C2 was computed using the one-dimensional consolidation equation (Ref. 10, p.3-29):

$\rho_p = H_0\, \Delta e/(1 + e_0)$

where H_o = initial height of the layer

e_0 = 1.4, initial void ratio

Δe = change in void ratio

For a normally consolidated soil

$\Delta e = C_c \log (P_f / P_o)$

For an overconsolidated soil

$\Delta e = C_r \log (P_f / P_p) + C_c \log (P_p/P_o)$

where C_c = 0.75 is the virgin compression index

C_r = 0.090 is the recompression index

P_o = initial effective overburden pressure

P_p = preconsolidation stress

P_f = final applied effective pressure

Layer C2 is approximately 15 feet thick and 56 feet below the foundation at the north side and 19 feet thick 41 feet below the foundation at the south side. Previous investigations in the S-area suggested that reasonable and conservative settlement of Layer C2 can be obtained using one-dimensional consolidation theory assuming half of the layer experience virgin compression and half experience recompression index (Ref. 9, p.22). Recompression index of 0.09 was used for overburden less than P_o and average compression index of (0.75 + 0.09)/2 or 0.42 was used for overburden greater than P_o.

The average heave is the average of the heaves considering minimum and maximum depths of excavation. The average settlement is the average of the settlements considering minimum and maximum cap thicknesses. The estimated settlements of cohesive layer C2 are:

Phase	Minimum Settlement/ Heave (inches)	Maximum Settlement/ Heave (inches)	Average Settlement/ Heave (inches)
Heave after Excavation	-0.2	-1.0	-0.6
Immediate after operation completed	1.5	2.5	2.0
Immediate after closure cap completed	3.7	5.0	4.3

Note that settlements include the recompressions of heave.

Site experience in H- and S-Areas also indicates that the rate of secondary consolidation for the site is small, on the order of less than 1/4 inch over 30 years, for structural loading in the range of 3 to 6 ksf. Based on site experience, secondary compression for Vault No. 2 is estimated to less than 1/4 inch over the life of the facility (30 years). It is expected that this settlement will be uniform and not contribute to differential settlement.

3.2.6 Summary of Settlements and heave

Estimation of settlement was performed by combining the settlement computed for cohesionless layers and settlement computed for the cohesive layer. The results were compared with the settlement using the historical data and conservative judgments were made to estimate these values, The estimated settlement are summarized as follows:

Phase	Minimum Settlement/ Heave (inches)	Maximum Settlement/ Heave (inches)	Average Settlement/ Heave (inches)
Heave after Excavation	- 1/2	- 1-1/2	- 1
Immediate after operation completed	2	7	4
Immediate after closure cap completed	5	13	7
30 Years after closure cap completed	6	18	9

Note that settlements include the recompression of heave.

The expected heave and settlement are the average computed heave and settlement shown in the above table. The differential heave or settlement at various phases is approximately the same as the total heave or settlement.

When one of the vaults is being filled with Saltstone and the other vault is left empty, differential settlement will occur due to the differential loading. The portion of the empty vault adjacent to the vault being filled will experience more settlement than the portion opposite the vault being filled. This type of differential settlement was not considered in the analysis, but can be eliminated if both vaults are filled simultaneously. However, this type of differential settlement will be diminished when both vaults are filled.

Due to the large overburden pressure, settlement was predicted with large amount of deviations. In order to verify the estimated settlement, it is recommended that settlement monitoring points be installed and settlement be monitored during construction and operation phases. Settlement data will provide important information for the actual settlement and will be used to verify and calibrate the predicted settlement as well as for the design of the closure cap.

4. RESULTS

(1) The bearing capacity parameters are:

Ultimate bearing capacity: q_o = 48,000 psf

Design bearing capacity: Φq_o = 22,000 psf

Strength reduction factor: Φ = 0.45

Allowable static bearing capacity: q_a = 16,000 psf

Allowable dynamic bearing capacity: q_d = 21,000 psf

(2) The estimated settlement and heave are:

Average and differential heaves at completion of excavation is approximately 1 inch

Average and differential settlements immediately after the completion of operation is approximately 4 inches.

Average and differential settlements immediately after the installation of the closure cap is approximately 8 inches.

Average and differential settlements 30 years after installation of the closure cap is approximately 10 inches.

5. REFERENCES

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(8) Bowles, Foundation Analysis and Design, Forth Edition, McGraw-Hill

(9) Mueser, Rutledge, Johnson & DeSimone, Geotechnical Report, Design and Construction, Defense Waste Process Facility, 200-S Area Savannah River Plant, 1984 .

(10) US Army Corp of Engineer, EM 1110-1-1904, Settlement Analysis, October 1992.

(11) K-ESR-S-00006, Glass Waste Storage Building #2 Geotechnical Baseline and Evaluation Report, Rev. 1, November 2003.

Appendix A

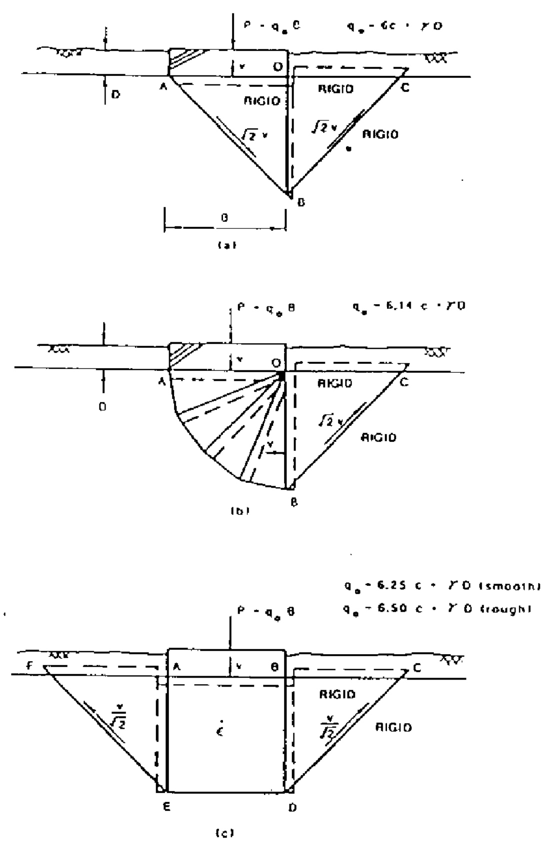


Fig. 4.5 Bearing capacity on Tresca material.

The failure mechanism shown in Figure 4.6 for a footing on a Coulomb material is that determined by Hill (1950). Since the mechanism is symmetrical, we may consider only the left side. Again we determine the solution by equating the internal and external rates of energy dissipation. The external work is due to the footing load. The internal work is due to the deforming region ADC and the differential slip along the surfaces OC, CD, and DE. No work is dissipated along the surfaces AC and AD owing to the nature of the assumed deformation in ADC. The work components are

$$W_{EXT} = \frac{Pv_p}{2}$$

$$W_{ACD} = \frac{cv_0 B \cot\phi}{8\cos(\frac{1}{4}\pi + \frac{1}{2}\phi)}(e^{\pi\tan\phi} - 1) = W_{CD}$$

$$W_{OC} = cv_0 \cos\phi \left[\frac{B}{4\cos(\frac{1}{4}\pi + \frac{1}{2}\phi)}\right]$$

$$W_{ED} = c[v_0 \cos\phi e^{(\frac{1}{2}\pi)\tan\phi}]\left[\frac{Be^{(\frac{1}{2}\pi)\tan\phi}}{4\cos(\frac{1}{4}\pi + \frac{1}{2}\phi)}\right] \quad (4.10)$$

These expressions are developed by considering the work expressions in Figure 4.4b and the kinematic relations of Figure 4.6. Collecting terms,

$$P = cB\cot\phi\,[e^{\pi\tan\phi}\tan^2(\tfrac{1}{4}\pi + \tfrac{1}{2}\phi) - 1] \quad (4.11)$$

The limit of Equation 4.11 as ϕ approaches zero is $P = (2 + \pi)Bc$, which is the well-known exact solution for a footing on a Tresca material.

Using limit analysis, Chen (1975) has evaluated the bearing capacity factors N of Equation 4.1. These values, summarized in Table 4.1, are given by the following relationships.

$$N_q = e^{\pi\tan\phi}\tan^2(\tfrac{1}{4}\pi + \tfrac{1}{2}\phi) \quad (4.12)$$

$$N_c = \cot\phi\,(N_q - 1) \quad (4.13)$$

$$N_\gamma = 2(1 + N_q)\tan\phi\tan(\tfrac{1}{4}\pi + \tfrac{1}{5}\phi) \quad (4.14)$$

It is worth pointing out that while the correct N_γ expression has for some time remained unsettled, the above values of N_c and N_q are the same as those given by Meyerhof and Brinch Hansen and are generally accepted as the correct or exact values. The differences in the reported N_γ are substantial, ranging from about one-third to double the values reported here. Andersen (1972) and Georgiadis and Michalopoulos (1985) have determined that the expression for N_γ provided by Brinch Hansen (1970) provides a lower bound to values determined experimentally. The data presented by Georgiadis and Michalopoulos indicate that Equation 4.14 provides a very good representation of the data for $\phi < 40°$ from a design standpoint. Brinch Hansen's expression has the form

$$N_\gamma = 1.5(N_q - 1)\tan\phi \quad (4.15)$$

On the basis of a statistical analysis of test results, Ingra and Baecher (1983) have suggested that for strip footings

$$N_\gamma = e^{(0.173\phi - 1.646)} \quad (4.16)$$

which gives N_γ values greater than Chen's expression (Eq. 4.14) for $\phi < 50°$.

Numerous authors have noted that the correct value of ϕ to be used in the bearing capacity equations may be different from that value measured by conventional triaxial tests. The value of ϕ measured in plane strain tests may be 10% greater

TABLE 4.1 BEARING CAPACITY FACTORS*.

ϕ	N_q	N_c	N_γ	N_q/N_c	tan ϕ
0	1.000	5.142	0.000	0.194	0.000
2	1.197	5.632	0.156	0.212	0.035
4	1.432	6.185	0.350	0.232	0.070
6	1.716	6.813	0.595	0.252	0.105
8	2.058	7.527	0.909	0.273	0.141
10	2.471	8.345	1.313	0.296	0.176
12	2.973	9.285	1.837	0.320	0.213
14	3.586	10.370	2.522	0.346	0.249
16	4.335	11.631	3.422	0.373	0.287
18	5.258	13.104	4.612	0.401	0.325
20	6.399	14.835	6.196	0.431	0.364
22	7.821	16.883	8.316	0.463	0.404
24	9.603	19.323	11.173	0.497	0.445
26	11.854	22.254	15.049	0.533	0.488
28	14.720	25.803	20.351	0.570	0.532
30	18.401	30.139	27.665	0.611	0.577
32	23.177	35.490	37.849	0.653	0.625
34	29.440	42.163	52.182	0.698	0.675
36	37.752	50.585	72.594	0.746	0.727
38	48.933	61.351	102.050	0.798	0.781
40	64.195	75.312	145.191	0.852	0.839
42	65.373	93.706	209.435	0.911	0.900
44	115.307	118.368	306.920	0.974	0.966
46	158.500	152.096	458.018	1.042	1.036
48	222.297	199.257	697.926	1.116	1.111
50	319.053	266.878	1089.456	1.195	1:192

*Chen (1975)

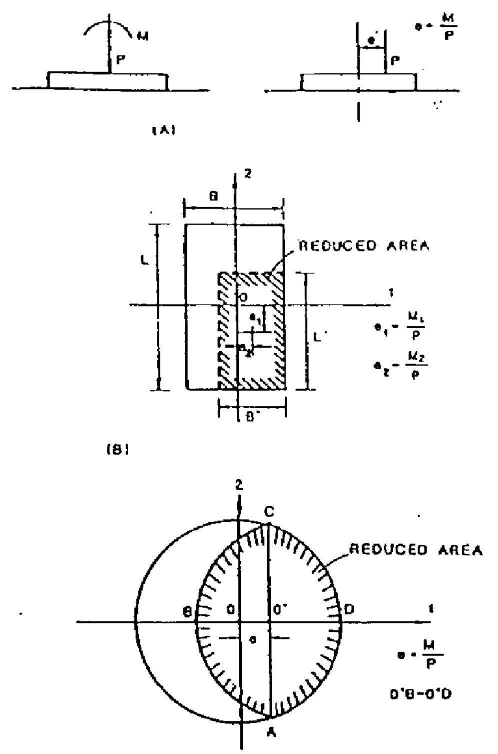


Fig. 4.10 Reduced footing area for eccentric loads. (*API, 1987.*) (A) Equivalent loadings. (B) Reduced area—rectangular footing. (C) Reduced area—circular footing.

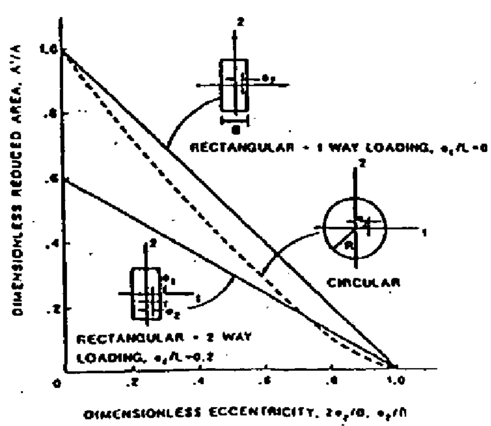


Fig. 4.11 Area reduction factors for eccentrically loaded footings. (*API, 1987.*)

footing depth factors, respectively, given in Tables 4.2 and 4.3. An iterative solution is required for inclined loads because the footing load components H and V appear in the factors i. Figure 4.12 shows the geometry for an inclined footing load. The equivalent dimensions B' and L' should be used, in the case of eccentric loads, in the expression of Tables 4.2 and 4.3. The ultimate footing load should be based on the reduced footing dimensions.

TABLE 4.2 MEYERHOF FOOTING DEPTH AND LOAD INCLINATION BEARING CAPACITY MODIFIERS*.

$$q_0 = N_c s_c i_c d_c c + N_q s_q i_q d_q q + \frac{\gamma B}{2} N_\gamma s_\gamma i_\gamma d_\gamma$$

For $D < B$:

$$d_c = 1 + 0.2\sqrt{N_\phi}\frac{D}{B}$$

$$d_q = d_\gamma = 1 \qquad (\phi = 0°)$$

$$d_q = d_\gamma = 1 + 0.1\sqrt{N_\phi}\frac{D}{B} \qquad (\phi > 10°)$$

$$i_c = i_q = (1 - \epsilon/90°)^2$$

$$i_\gamma = (1 - \epsilon/\phi)^2$$

$$N_\phi = \tan^2(\tfrac{1}{4}\pi + \tfrac{1}{2}\phi)$$

* Meyerhof (1963).

TABLE 4.3 BRINCH HANSEN FOOTING DEPTH AND LOAD INCLINATION BEARING CAPACITY MODIFIERS*.

$$q_0 = N_c s_c i_c d_c c + N_q s_q i_q d_q q + \frac{\gamma B}{2} N_\gamma s_\gamma i_\gamma d_\gamma$$

For $D < B$:

$$d_c = 1 + 0.4\frac{D}{B} \qquad (\phi = 0°)$$

$$d_c = d_q - \frac{1 - d_q}{N_c \tan\phi} \qquad (\phi > 0°)$$

$$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\frac{D}{B}$$

$$d_\gamma = 1$$

$$i_c = 1 - \frac{mH}{Ac(\pi + 2)} \qquad (\phi = 0°)$$

$$i_c = i_q - \frac{1 - i_q}{N_q \tan\phi} \qquad (\phi > 0°)$$

$$i_q = \left[1 - \frac{H}{V + Ac\cot\phi}\right]^m$$

$$i_\gamma = \left[1 - \frac{H}{V + Ac\cot\phi}\right]^{m+1}$$

$$m = m_L\cos^2\theta_n + m_B\sin^2\theta_n$$

$$m_B = \frac{2 + B/L}{1 + B/L} \qquad m_L = \frac{2 + L/B}{1 + L/B}$$

θ_n is the projected direction of load in the plane of the footing, measured from the side of length L

* As modified by Vesić (1975).

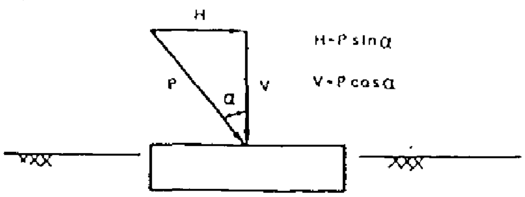


Fig. 4.12 Inclined footing load.

Bowles (1982) has suggested that the Brinch Hansen inclination factors *i* should not be used in conjunction with the shape factors *s*. The use of Meyerhof's expressions with Equation 4.24 has been verified by comparison with laboratory experiments (Hanna and Meyerhof, 1981).

4.7 FOOTING SHAPE, DEPTH, AND INCLINATION EFFECTS

The footing size, orientation and position relative to the ground surface have an influence on the bearing capacity. Brinch Hansen (1970) and Vesić (1975) have addressed these problems. The bearing capacity may be determined as

$$q_0 = N_c s_c d_c g_c b_c c + N_q s_q d_q g_q b_q q + \frac{\gamma B}{2} N_\gamma s_\gamma d_\gamma g_\gamma b_\gamma \qquad (4.25)$$

where q_0 is the vertical component of the footing load and the factors *s*, *d*, *g*, and *b* are the footing shape, footing depth, ground inclination, and base inclination factors, respectively. The expressions for the factors *s*, *b*, and *g* are given in Tables 4.4 and 4.5. The geometric parameters for inclined footings and ground surfaces are given in Figure 4.13. The equivalent dimensions B' and L' should be used, in the case of eccentric loads, in the expression of Tables 4.4 and 4.5. The ultimate footing load should be based on the reduced footing dimensions in the case of an eccentric load.

Bowles (1982) has suggested the factors *s*, *d*, *g*, and *b* should not be used in conjunction with the load inclination factors *i*. Vesić (1975) has suggested that for the sloping ground condition and $\phi = 0$ (Tresca material) soils,

$$N_\gamma = -2 \sin \omega \qquad (4.26)$$

TABLE 4.4 MEYERHOF AND BRINCH HANSEN FOOTING SHAPE BEARING CAPACITY MODIFIERS.

$$q_0 = N_c s_c i_c d_c c + N_q s_q i_q d_q q + \frac{\gamma B}{2} N_\gamma s_\gamma i_\gamma d_\gamma$$

Meyerhof (Meyerhof, 1963)

$$s_c = 1 + 0.2 N_\phi \frac{B}{L}$$

$$s_q = s_\gamma = 1.0 \qquad (\phi = 0°)$$

$$s_q = s_\gamma = 1 + 0.1 N_\phi \frac{B}{L} \qquad (\phi > 10°)$$

$$N_\phi = \tan^2 (\tfrac{1}{4}\pi + \tfrac{1}{2}\phi)$$

Brinch Hansen (After Vesić, 1975)

$$s_c = 1 + \frac{B}{L}\frac{N_q}{N_c}$$

$$s_q = 1 + \frac{B}{L} \tan \phi$$

$$s_\gamma = 1 - 0.4 \frac{B}{L}$$

For circular footing use $B/L = 1$.

TABLE 4.5 BRINCH HANSEN FOOTING AND GROUND INCLINATION BEARING CAPACITY MODIFIERS*.

$$q_0 = N_c s_c d_c g_c b_c c + N_q s_q d_q g_q b_q q \pm \frac{\gamma B}{2} N_\gamma s_\gamma d_\gamma g_\gamma b_\gamma$$

Footing Inclination Factors

$$b_q = b_\gamma = (1 - \alpha \tan \phi)^2$$

$$b_c = 1 - \frac{2\alpha}{\pi + 2} \qquad (\phi = 0°, \alpha \text{ in radians})$$

$$b_c = b_q - \frac{1 - b_q}{N_c \tan \phi} \qquad (\phi > 0°)$$

Ground Inclination Factors

$$g_q = g_\gamma = (1 - \tan \omega)^2 \qquad (\phi > 0°)$$

$$g_c = 1 - \frac{2\omega}{\pi + 2} \qquad (\phi = 0°, \omega \text{ in radians})$$

$$g_c = g_q - \frac{1 - g_q}{N_c \tan \phi} \qquad (\phi > 0°)$$

Restrictions: $\alpha < 45°$, $\omega < 45°$, $\omega < \phi$.
For ground inclination use $N_\gamma = -2 \sin \omega$.

* As modified by Vesić (1975)

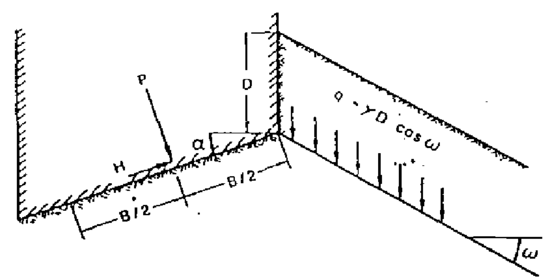


Fig. 4.13 Inclined footing and ground surface.

4.8 NONHOMOGENEOUS FOUNDATIONS AND ANISOTROPIC STRENGTH

The conditions of nonhomogeneous foundations and anisotropic soil strength have been treated by Reddy and Srinivasan (1967) using limit equilibrium procedures and by Chen (1975) using the upper-bound method. These authors examined the common condition of a strip footing supported by a two-layer undrained clay (Tresca material, $\phi = 0$) foundation. Chen also considered the case of a linearly increasing strength with depth. The results of Chen agree well with those of Redding and Srinivasan and are shown in Figure 4.14.

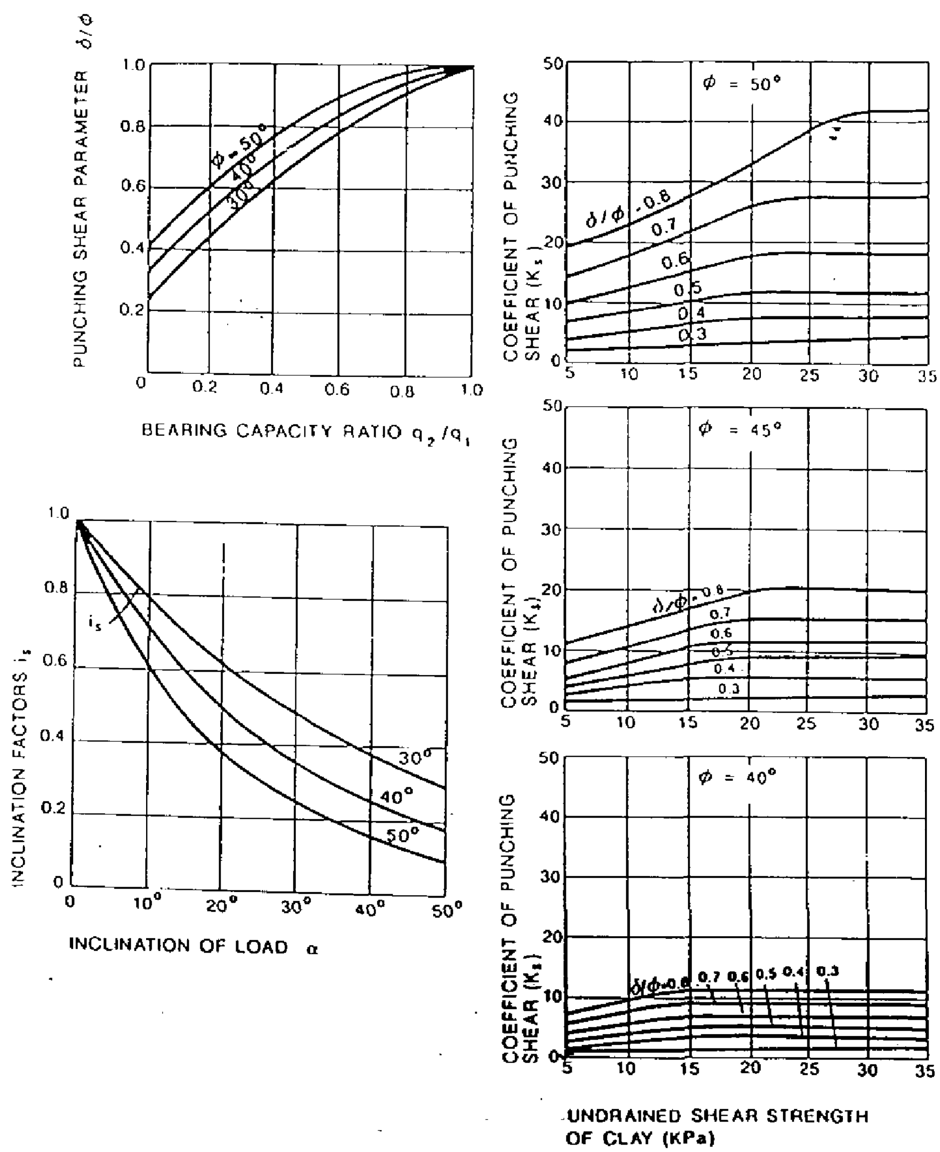


Fig. 4.17 Graphical aids for Meyerhof and Hanna solution to layered foundation.

4.9 INFLUENCE OF GROUNDWATER TABLE

The position of the groundwater table may have a significant effect on the bearing capacity of shallow foundations. Generally the submergence of soils will cause loss of the apparent cohesion due to capillary stresses or weak cementation bonds. At the same time the effective unit weight of submerged soils will be reduced to about one-half the weight of the same soils above water. Thus, through submergence, all three terms involved in the bearing capacity equation are reduced. For this reason, it is essential that the bearing capacity analysis be made assuming the highest possible groundwater level at the particular level for the expected lifetime of the structure. The assessment of this level must be made taking into consideration the probability of temporary high levels that could be expected in some locations during heavy rainstorms or floods, although they may not appear in the official records.

If the highest groundwater level is within the depth $z_w < B$ below the foundation level (Fig. 4.18) the effective weight γ of the soil below the foundation base should be taken as

$$\gamma = \gamma' + \frac{z_w}{B}(\gamma_m - \gamma') \qquad (4.28)$$

where γ' is the submerged unit weight and γ_m is the moist unit weight of soil, corresponding to the minimum moisture content of the soil above the water table (Meyerhof, 1953). If the water

TABLE 4.8 VALUES OF MINIMUM PARTIAL FACTORS*

Category	*Item*	*Load Factor*	*Resistance Factor*
Loads	Dead loads	(f_d) 1.25 (0.85)	
	Live loads, wind or earthquake	(f_l) 1.5	
	Water pressures	(f_u) 1.25 (0.85)	
Shear strength	Cohesion (c) (stability; earth pressures)		(f_c) 0.65
	Cohesion (c) (foundations)		(f_c) 0.5
	Friction ($\tan\phi$)		(f_ϕ) 0.8

Note: Load factors given in parentheses apply to dead loads and water pressures when their effects are beneficial, as for dead loads resisting instability by sliding, overturning or uplift.

* Meyerhof (1984)

factors) given in Table 4.8. The higher values in Table 4.7 are applied to the normal loads and service conditions, while the lower values are applied to the maximum loads and worst environmental conditions.

The basic philosophy using total factors of safety is that the foundation should be capable of resisting a load F_s times greater than the design load. The load and resistance factor design (LRFD) method applies separate or partial factors to the loads and soil resistance. The load factors are provided mainly for variability and pattern of loading, which differ for dead loads, live loads, environmental loads, and water pressures. The resistance factors consider the variability and uncertainty of assessment of soil resistance, which differ for the cohesive and friction components. Thus, the factored shear strength of soil at the ultimate limit state may be expressed as

$$\tau = f_c c + \sigma_n f_\phi \tan\phi \tag{4.33}$$

for the Coulomb criterion. The factors f_c and f_ϕ are the resistance factors for the cohesive and friction components, respectively. It is evident from Equation 4.33 that the total factor of safety obtained will depend on the relative contributions of the cohesive and friction components.

Whitman (1984) has recently reviewed the application of the related topic of risk analysis to geotechnical engineering.

4.13 EXAMPLE PROBLEMS

EXAMPLE 4.1

A rectangular footing (Fig. 4.21) 28 ft wide and 84 ft long is to be placed at a depth of 10 ft in a deep stratum of soft, saturated clay (bulk unit weight 105 lb/ft³). The water table is at 8 ft below ground surface. Find the ultimate bearing capacity under the following two conditions:

a. assuming that the rate of application of dead and live loads is fast in comparison with the rate of dissipation of excess pore-water pressures caused by loads, so that undrained conditions prevail at failure;

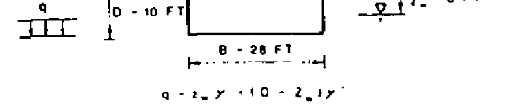


Fig. 4.21 Footing geometry

b. assuming, as the other extreme, that the rate of loading is slow enough that no excess pore-water pressures are introduced in the foundation soil.

The strength parameters of the soil, obtained from unconsolidated, undrained tests are $c_u = 0.22$ ton/ft², $\phi_u = 0$. Consolidated, drained tests give $c_d = 0.04$ ton/ft², $\phi_d = 23°$.

CONDITION (a)

Submerged unit weight of soil: $\gamma' = 105 - 62 = 43$ lb/ft³.

Overburden stress: $q = [(8)(105) + (2)(43)]/(2000)$
$= 0.463$ ton/ft².

Bearing capacity factors (Table 4.1): $N_c = 5.14$; $N_q = 1$; $N_\gamma = 0$.

Shape factors (Table 4.4, Brinch Hansen):

$$s_c = 1 + \frac{B}{L}\frac{N_q}{N_c} = 1 + (1/3)(0.19) = 1.065$$

$$s_q = 1.00$$

Ultimate bearing pressure (Eq. 4.24):

$$q_0 = cN_c s_c + qN_q s_q$$

$$q_0 = (0.22)(5.14)(1.065) + (0.463)(1)(1.00)$$

$$= 1.21 + 0.46 = 1.67 \text{ ton/ft}^2$$

CONDITION (b)

Bearing capacity factors: $N_c = 18.05$; $N_q = 8.66$; $N_\gamma = 9.70$.
Shape factors:

$$s_c = 1 + \frac{B}{L}\frac{N_q}{N_c} = 1 + (1/3)(0.48) = 1.16$$

$$s_q = 1 + \frac{B}{L}\tan\phi = 1 + (1/3)(0.42) = 1.14$$

$$s_\gamma = 1 - 0.4\frac{B}{L} = 1 - (0.4)(1/3) = 0.87$$

Ultimate bearing pressure:

$$q_0 = cN_c s_c + qN_q s_q + \frac{\gamma B}{2}N_\gamma s_\gamma$$

$$q_0 = (0.04)(18.05)(1.16) + (0.463)(8.66)(1.14)$$
$$+ (1/2)(43)(28)(9.7)(0.87)/(2000)$$
$$= 0.72 + 4.57 + 2.54 = 7.83 \text{ ton/ft}^2$$

ing reduction factor (revised from the previous edition) as follows:

$$r_\gamma = 1 - 0.25 \log\left(\frac{B}{\kappa}\right) \qquad B \geqq 6 \text{ ft or } 2 \text{ m}$$

where $\kappa = 6.0$ for fps and 2.0 for SI

This gives:

$B = 2$	2.5	3	3.5	4	5	10	20	100 m
$r_\gamma = 1.0$	0.97	0.95	0.93	0.92	0.90	0.82	0.75	0.57

One can use this reduction factor with any of the bearing capacity methods to give

$$0.5\gamma BN_\gamma s_\gamma d_\gamma r_\gamma$$

This equation is particularly applicable for large bases at small D/B ratios where the BN_γ term is predominating.

General observations about the bearing-capacity equations may be made as follows:

1. The cohesion term predominates in cohesive soil.
2. The depth term ($\bar{q}N_q$) predominates in cohesionless soils. Only a small D increases q_{ult} substantially.
3. The base width term $0.5\gamma BN_\gamma$ provides some increase in bearing capacity for both cohesive and cohensionless soils. In cases where $B < 3$ to 4 m this term could be neglected with little error.
4. **No one would place a footing on the ground surface of a cohesionless soil mass.**
5. **It is highly unlikely that one would place a footing on a cohesionless soil with a D_r (Table 3-4) less than 0.5. If the soil is loose, it would be compacted in some manner to a higher density prior to placing footings in it.**
6. **Where the soil beneath the footing is not homogeneous or is stratified, some judgment must be applied to determining the bearing capacity. In the case of stratification, later sections will consider several cases.**
7. **When a base must be designed for a particular load, except for the Terzaghi method, one must use an iterative procedure since the shape, depth, and inclination factors depend on B. A computer program such as B-31 is most useful for this type problem. It should be set to increment the base by 0.075 m or 0.25 ft (3 in) steps as this is a common multiple of base dimensions.**
8. **Inspection of Table 4-1 indicates that the Terzaghi equation is much easier to use than the other methods (see also Example 4-1) so that it has great appeal for many practitioners—particularly for bases with only vertical loads and $D/B \leqq$ Its form is also widely used for deep foundations but with adjusted N factor**
9. **Vesić (1973) recommends that depth factors d_i not be used for shallo foundations ($D/B \leqq 1$) because of uncertainties in quality of the overburde**

$$\rho_i = C_1 \cdot C_t \cdot \Delta p \cdot \sum_{i=1}^{n} \frac{\Delta z_i}{E_{si}} \cdot Izi \quad (3\text{-}8)$$

where

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$
σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D , tsf
Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf
C_t = correction for time dependent increase in settlement, $1 + 0.2 \cdot \log_{10}(t/0.1)$
t = time, years
E_{si} = elastic modulus of soil layer i , tsf
Δz_i = depth increment i , 0.2B, ft
Izi = influence factor of soil layer i, Figure 3-4

Settlement may be calculated with the assistance of the calculation sheet, Figure 3-5. The time-dependent increase in settlement is related with creep and secondary compression as observed in clays.

(1) Influence factor. The influence factor Iz is based on approximations of strain distributions for square or axisymmetric footings and for infinitely long or plane strain footings observed in cohesionless soil, which are similar to an elastic medium such as the Boussinesq distribution, Figure 1-2. The peak value of the influence factor Izp in Figure 3-4 is (item 59)

$$Izp = 0.5 + 0.1 \left[\frac{\Delta p}{\sigma'_{Izp}}\right]^{1/2} \quad (3\text{-}9a)$$

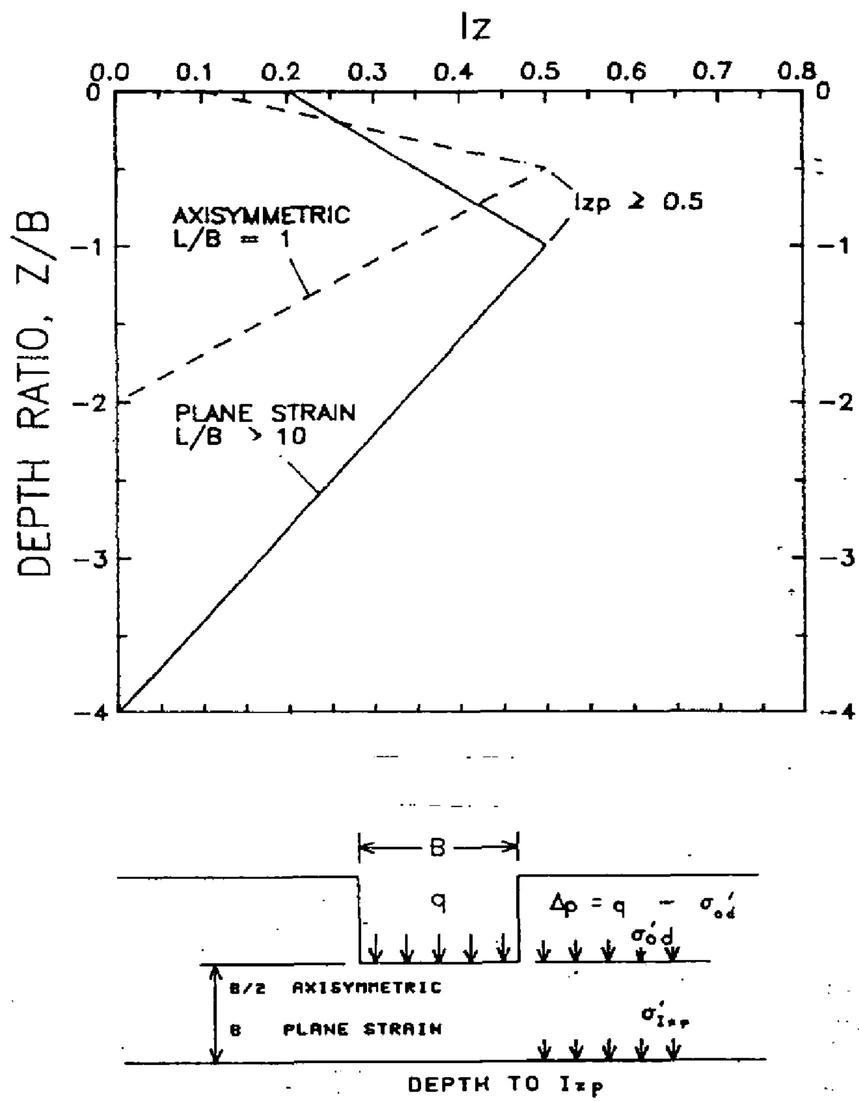
Axisymmetric: $L/B = 1$
$$\sigma'_{Izp} = 0.5 \cdot B \cdot \gamma' + D \cdot \gamma' \quad (3\text{-}9b)$$

Plane Strain: $L/B \geq 10$
$$\sigma'_{Izp} = B \cdot \gamma' + D \cdot \gamma' \quad (3\text{-}9c)$$

where

σ'_{Izp} = effective overburden pressure at the depth of Izp , tsf
γ' = effective unit weight (wet soil unit weight γ less unit weight of water) in units of ton/cubic foot
D = excavated or embeded depth, ft

The parameter σ_{Izp} may be assumed to vary linearly between Equations 3-9b and 3-9c for L/B between 1 and 10 . Iz may be assumed to vary linearly between 0.1 and 0.2 on the Iz axis at the ground surface for L/B between 1 and 10 and Z/B may be assumed to vary linearly between 2 and 4 on the Z/B axis for L/B between 1 and 10 .



Z - DEPTH BELOW FOOTING BOTTOM, FT
B - FOOTING WIDTH, FT
Iz - DEPTH INFLUENCE FACTOR
Izp - PEAK DEPTH INFLUENCE FACTOR

Figure 3-4. Recommended strain influence factors for Schmertmann's Approximation. Reprinted with permission of the American Society of Civil Engineers from the <u>Journal of the Geotechnical Engineering Division</u>, Vol 104, 1978, "Improved Strain Influence Factor diagram", by J. M. Schmertmann, J. P. Hartman, and P. R. Brown, p 1134

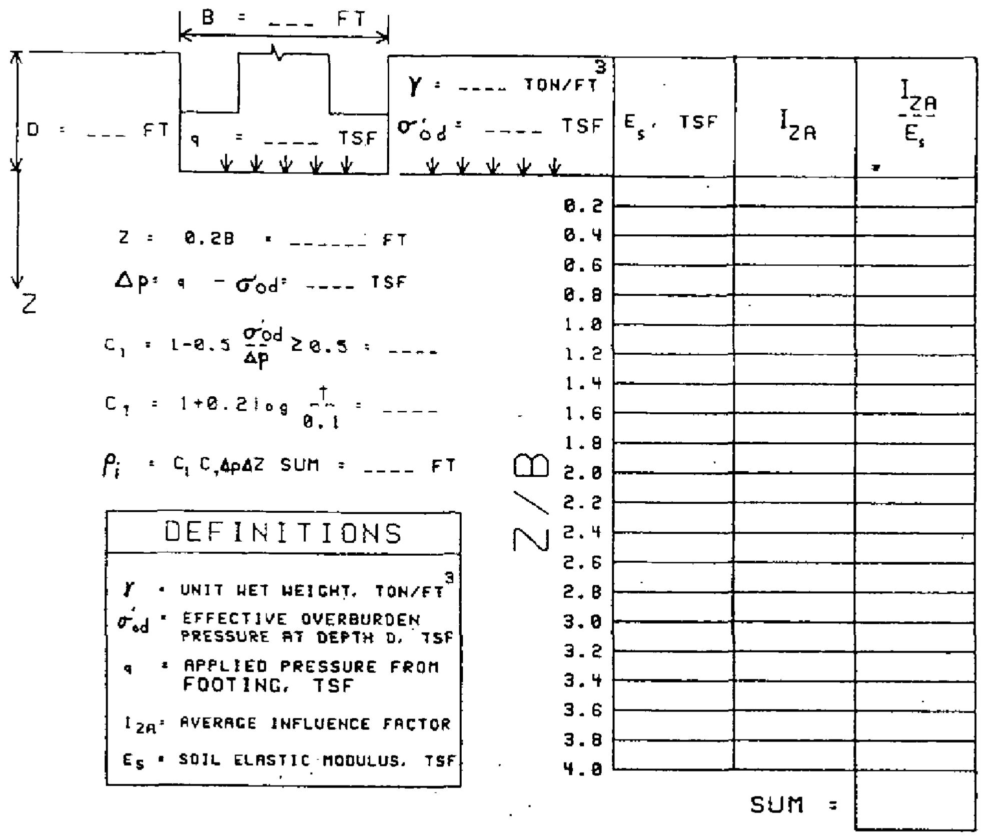


Figure 3-5. Settlement calculation sheet for cohesionless soil using Schmertmann's method

(2) Elastic modulus. Elastic modulus E_{si} may be estimated from results of the mechanical (Dutch Static) Cone Penetration Test (CPT) (item 59)

$$\text{Axisymmetric Footings: } L/B = 1 \qquad E_{si} = 2.5 \cdot q_c \tag{3-10a}$$

$$\text{Plane Strain Footings: } L/B \geq 10 \qquad E_{si} = 3.5 \cdot q_c \tag{3-10b}$$

where q_c is the cone tip bearing resistance in units of tsf. E_{si} may be assumed to vary linearly between Equations 3-10a and 3-10b for L/B between 1 and 10 . SPT data may also be converted to Dutch cone bearing capacity by the correlations in Table 3-1. The estimated average elastic modulus of each depth increment may be plotted in the E_s column of Figure 3-5.

Table 3-1

Correlations Between Dutch Cone Tip Resistance q_c and Blow Count N from the SPT (Data from Item 55)

Soil	q_c/N^*
Silts, sandy silts, slightly cohesive silt-sand	2
Clean, fine to medium sands and slightly silty sands	3.5
Coarse sands and sands with little gravel	5
Sandy gravel and gravel	6

*Units of q_c are in tsf and N in blows/ft

(3) Calculation of settlement. Iz/E_s is computed for each depth increment z/B and added to obtain SUM, Figure 3-5. Immediate settlement of the soil profile may then be calculated as shown on Figure 3-5. If a rigid base lies within z = 2B, then settlement may be calculated as shown down to the rigid base.

e. Burland and Burbidge Approximation. This procedure based on 200 SPT case studies predicts settlements less than most of these methods (item 4).

(1) Immediate settlement of sand and gravel deposits may be estimated by

$$\Delta P'_{ave} > \sigma'_p: \qquad \rho_i = f_s \cdot f_i \cdot \left[(\Delta P'_{ave} - \frac{2}{3}\sigma'_p) \cdot B^{0.7} \cdot I_c \right] \qquad (3\text{-}11a)$$

$$\Delta P'_{ave} < \sigma'_p: \qquad \rho_i = f_s \cdot f_i \cdot \Delta P'_{ave} \cdot \frac{I_c}{3} \qquad (3\text{-}11b)$$

where

f_s = shape correction factor, $[(1.25 \cdot L/B)/(L/B + 0.25)]^2$
f_1 = layer thickness correction factor, $H/z_1 \cdot (2 - H/z_1)$
$\Delta P'_{ave}$ = average effective bearing pressure, $q_{oave} + \sigma'_{oave}$, tsf
q_{oave} = average pressure in stratum from foundation load, tsf
σ'_{oave} = average effective overburden pressure in stratum H, tsf
σ'_p = maximum effective past pressure, tsf
H = thickness of layer, ft
z_1 = depth of influence of loaded area, ft
I_c = compressibility influence factor, $\approx 0.23/(N_{ave}^{1.4})$ with coefficient of correlation 0.848
N_{ave} = average SPT blowcount over depth influenced by loaded area

(a) The depth of influence z_1 is taken as the depth at which the settlement is 25 percent of the surface settlement. This depth in feet may be approximated by $1.35B^{0.75}$ where N_{ave} increases or is constant with depth. z_i is taken as 2B where N_{ave} shows a consistent decrease with depth.

(b) N_{ave} is the arithmetic mean of the measured N values within the depth of influence z_i. N_{ave} is not corrected for effective overburden pressure, but instead considers compressibility using I_c. The arithmetic mean of the measured N_{ave} should be corrected to $15 + 0.5(N_{ave} - 15)$ when $N_{ave} > 15$ for very fine and silty sand below the water table and multiplied by 1.25 for gravel or sandy gravel.

(c) The probable limits of accuracy of Equations 3-11 are within upper and lower bound values of I_c given by

$$0.08/(N_{ave})^{1.3} \le I_c \le 1.34/N_{ave}^{1.67} \qquad (3\text{-}12)$$

(2) Settlement after time t at least 3 years following construction from creep and secondary compression effects may be estimated by

$$\rho_t = f_t \cdot \rho_i \qquad (3\text{-}13)$$

where

- $f_t = 1 + R_3 + R_t \cdot \log t/3$
- R_3 = time-dependent settlement ratio as a proportion of ρ_i during first 3 years following construction, ≈ 0.3
- R_t = time-dependent settlement ratio as a proportion of ρ_i for each log cycle of time after 3 years, ≈ 0.2

Values of R_3 and R_t are conservative based on 9 case records (item 4).

f. Dilatometer Approximation. The dilatometer consists of a stainless steel blade 96 mm wide and 15 mm thick with a sharp edge containing a stainless steel membrane centered and flush with one side of the blade. The blade is preferably pushed (or driven if necessary) into the soil. A pressure-vacuum system is used to inflate/deflate the membrane a maximum movement of 1.1 mm against the adjacent soil (item 58).

(1) Calculation. This procedure predicts settlement from evaluation of one-dimensional vertical compression or constrained modulus E_d by the DMT

$$\rho_i = \frac{q_{oave} \cdot H}{E_d} \qquad (3\text{-}14)$$

where

- q_{oave} = average increase in stress caused by the applied load, tsf
- H = thickness of stratum at depth z where q_{oave} is applicable, ft
- E_d = constrained modulus, $R_D E_s$, tsf
- R_D = $(1 - \nu_s)/[(1 + \nu_s)(1 - 2\nu_s)]$, factor that varies from 1 to 3 relates E_d to Young's soil modulus E_s
- ν_s = Poisson's ratio

Table 3-5

Procedure for Calculation of Ultimate Primary Consolidation Settlement of a Compressible Stratum

Step	Description
1	Evaluate the preconsolidation stress σ'_p from results of a one-dimensional (1-D) consolidation test on undisturbed soil specimens using the Casagrande construction procedure, Table 3-6a, or by methods in paragraph 1-5a. Refer to Appendix E for a description of 1-D consolidation tests.
2	Estimate the average initial effective overburden pressure σ'_o in each compressible stratum using soil unit weights, depth of overburden on the compressible stratum, and the known groundwater level or given initial pore water pressure in the stratum. Refer to Equation 1-1, $\sigma'_{oz} = \gamma z - u_w$. $\sigma'_o = (\sigma'_{oz1} + \sigma'_{oz2})/2$ where σ'_{oz1} = effective pressure at top of compressible stratum and σ'_{oz2} = effective pressure at bottom of compressible stratum.
3	Determine the soil initial void ratio e_o as part of the 1-D consolidation test or by methods in Appendix II, EM 1110-2-1906, Laboratory Soils Testing.
4	Evaluate the compression index C_c from results of a 1-D consolidation test using the slope of the field virgin consolidation line determined by the procedure in Table 3-6a as illustrated in Figures 3-12 and 3-13, or preliminary estimates may be made from Table 3-7. Determine the recompression index C_r for an overconsolidated soil as illustrated in Figures 3-12 and 3-13; preliminary estimates may be made from Figure 3-14.
5	Estimate the final applied effective pressure σ'_f where $\sigma'_f = \sigma'_o + \sigma_{st}$. σ_{st}, soil pressure caused by the structure, may be found from Equation C-2 or Boussinesq solution in Table C-1.
6	Determine the change in void ratio Δe_j of stratum j for the pressure increment $\sigma'_f - \sigma'_o$ graphically from a data plot similar to Figure 3-12, from Equation 3-21 for a normally consolidated soil, or from Equation 3-23 for an overconsolidated soil.
7	Determine the ultimate one-dimensional consolidation settlement of stratum j with thickness H_j, from Equation 3-20 $$\rho_{cj} = \frac{\Delta e_j}{1 + e_{oj}} H_j$$
8	Determine the total consolidation ρ_c of the entire profile of compressible soil from the sum of the settlement of each stratum, Equation 3-22

Table 3-5. Concluded

Step	Description
	$$\rho_c = \sum_{j=1}^{n} \rho_{cj}$$
9	Correct ρ_c for effect of overconsolidation and small departures from 1-D compression on the initial excess pore pressure using the Skempton and Bjerrum procedure, Equation 3-24 $$\rho_{\lambda c} = \lambda \rho_c$$ where λ is found from Figure 3-15. $\lambda = 1$ if $B/H > 4$ or if depth to the compressible stratum is > 2B. The equivalent dimension of the structure when corrected to the top of the compressible stratum B_{cor} is found by the approximate distribution $B_{cor} = (B'L')^{0.5}$ where $B' = B + z$ and $L' = L + z$, B = foundation width, L = foundation length and z = depth to top of the compressible soil profile. Substitute B_{cor} for B in Figure 3-15. $\rho_{\lambda c}$ is the corrected consolidation settlement. This correction should not be applied to bonded clays.

where

ρ_{cj} = consolidation settlement of stratum j, ft
Δe_j = change in void ratio of stratum j, $e_{oj} - e_{fj}$
e_{oj} = initial void ratio of stratum j at initial pressure σ'_{oj}
e_{fj} = final void ratio of stratum j at final pressure σ'_{fj}
H_j = height of stratum j, ft

The final void ratio may be found graphically using the final pressure σ'_f illustrated in Figure 3-12a. The change in void ratio may be calculated by

$$\Delta e_j = C_c \cdot \log_{10} \frac{\sigma'_{fj}}{\sigma'_{oj}} \qquad (3\text{-}21)$$

where C_c is the slope of the field virgin consolidation curve or compression index. Figure 3-13 illustrates evaluation of C from results of a 1-D consolidation test. Table 3-7 illustrates some empirical correlations of C_c with natural water content, void ratio, and liquid limit. Refer to Chapter 3, TM 5-818-1, for further estimates of C_c.

(c) Total consolidation settlement ρ_c of the entire profile of compressible soil may be determined from the sum of the settlement of each stratum

$$\rho_c = \sum_{j=1}^{n} \rho_{cj} \qquad (3\text{-}22)$$

where n is the total number of compressible strata. This settlement is considered to include much of the immediate elastic compression settlement ρ_i, Equation 3-1.

Table 3-7

Estimates of the Virgin Compression Index, C_c

Soil	C_c
Organic soils with sensitivity less than 4	$0.009(LL - 10)$
Organic soils, peat	$0.0115W_n$
Clays	$1.15(e_o - 0.35)$ $0.012W_n$ $0.01(LL - 13)$
Varved clays	$(1 + e_o)\cdot[0.1 + 0.006(W_n - 25)]$
Uniform silts	0.20
Uniform sand	
Loose	0.05 to 0.06
Dense	0.02 to 0.03

Note: LL = liquid limit, percent
W_n = natural water content, percent
e_o = initial void ratio

final effective vertical applied pressure σ_f' exceeding the preconsolidation stress σ_p'. Reloading a specimen in the consolidometer will give the laboratory curve shown in Figure 3-12b.

(a) Reconstruction of the field virgin consolidation curve with slope C_c may be estimated by the procedure in Table 3-6b. Refer to Table 3-7 for methods of estimating C_c.

(b) The rebound loop in the laboratory curve is needed to develop the recompression line BF. Evaluation of the recompression index C_r is illustrated in Figure 3-13. The recompression index is equal to or slightly smaller than the swelling index, C_s. Approximate correlations of the swelling index are shown in Figure 3-14.

(c) Settlement ρ_{cj} of stratum j in inches may be estimated as the sum of recompression and virgin consolidation settlements. The final void ratio is found graphically from Figure 3-12b. The change in void ratio may be calculated by

$$\Delta e_j = C_r \cdot \log_{10}\frac{\sigma_{pj}'}{\sigma_{oj}'} + C_c \cdot \log_{10}\frac{\sigma_{fj}'}{\sigma_{pj}'} \qquad (3\text{-}23)$$

where C_r is the average slope of the recompression line BF. If $\sigma_{fj}' < \sigma_{pj}'$, ignore the right-hand term of Equation 3-23 containing C_c and substitute σ_{fj}' for σ_{pj}' in the term containing C_r. Settlement of stratum j is

Appendix F

K-CLC-Z-00009

Settlement due to Compression of Soft Zone

Rev. 0, March 2006

(10 pages)

OSR 45-24 (Rev 11-15-2004)

Calculation Cover Sheet

Project	Calculation No.	Project No.
Saltstone Vault No. 2	K-CLC-Z-00009	N/A
Title: Settlement due to Compression of Soft Zone	Functional Classification: PS	Sheet 1 of 9
	Discipline: Geotechnical	
Calc Level: ☒ Type 1 ☐ Type 2	Type 1 Calc Status: ☐ Preliminary ☒ Confirmed	
Computer Program No.: ☒ N/A	Version/Release No.: N/A	

Purpose and Objective

This calculation provides the ground surface settlement due to the compression of soft zone.

Summary of Conclusion

see last section.

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ADC & Reviewing Official: [signature]
(Name and Title)
Date: 3-20-2006

Revisions

Rev No.	Revision Description
0	Initial issue

Sign Off

Rev No.	Originator (Print) Sign/Date	Verification/ Checking Method	Verifier/Checker (Print) Sign/Date	Manager (Print) Sign//Date
0	[signature] 3/23/06 William T. Li	Document review	Stephen M. Mead 3/23/06 STEPHEN M. MEAD	M.R. Lewis 3/30/06 M.R. Lewis

Design Authority – (Print)	Signature	Date
Release to Outside Agency – (Print)	Signature	Date

Security Classification of the Calculation

TABLE OF CONTENTS

1. Introduction

The calculation computes the ground surface profile including the upper bounds of the settlement, slope, and the curvature of the slope due to the compression of soft zone. The settlement at ground surface will distribute in the form of an error or normal probability function (Ref. 1).

2. Input

2.1 Geometrical Configuration

The foundation of the Saltstone Vault No. 2 is approximately at 268 feet, MSL. The ground surface elevation at the project site after closure is approximately 305 feet, MSL (Ref. 2). One significant soft zone was found at the project site. The top and bottom of this soft zone are 177.6 and 163.2 feet, MSL, respectively, the width of the soft zone is within 150 feet, and the soft interval thickness is 14 feet (Ref. 3).

2.2 Soft Zone Properties

Two samples were taken from the soft zone, sample ID Z-V2-B1U-PS1V, between elevations 177.2 and 176.7 feet, MSL; and sample ID Z-V2-B1U-PS1H, between elevations 177.7 and 177.2 feet, MSL

Test results from sample Z-V2-B1U-PS1V include:

Initial void ratio e_0 is 0.92 (Ref. 3, p. 126)

Compression index C_c is 0.196 (Ref. 3, p. 14)

Over-consolidation ratio OCR is 0.9 (Ref. 3, p. 14)

Moisture contents: 35.5%, 32.4%, and 34.9% (Ref. 3, p.13)

Compression ratio CR = $C_c/(1 + e_o)$ = 0.196

Test results from sample Z-V2-B1U-PS1H include:

Initial void ratio e_0 is 0.72 (Ref. 4).

Moisture contents: 25.7%, 28.8%, and 31.6% (Ref. 4)

Based on these test results, the compression ratio, CR for initial void ratio of 0.72 is $C_c/(1 + e_o)$ or 0.114 and the average moisture content is 31.5%.

These initial void ratios, compression index, OCR, moisture contents, and compression ratio are within reasonable ranges as results found from soft zones elsewhere within the Savannah River Site (Ref. 5, p. 30 and 31).

Using the correlation developed for the soft zone at the SRS (Ref. 5, p. 45 Fig. 13 and p. 46 Fig. 14), based on an average moisture content of 31.5%, both the compression index C_c of 0.196 and the compression ratio CR of 0.114 for the project site are very consist with the results from the S-Area, the area closet to the project site and covered in Reference 5. Both the compression index and the compression ratio appear to be reasonable.

3. Computations

3.1 Soft Zone Compression

The compression of the soft zone s_S at depth is 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 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 provided in the previous section, choosing a conservative e_o of 0.72, the compression of the soft zone is:

$$s_S = H \{C_c/(1 + e_o)\} \log (1/OCR) = 14 \times 0.196/(1 + 0.72) \times \log (1/0.9) = 0.0730 \text{ feet}$$

or $s_S = 0.876$ inches.

3.2 Methodology for Computing Surface Settlement

A vertical slice of subsurface with unit thickness perpendicular to the longitudinal direction of the soft zone was considered. Ground settlement propagated from subsurface deformation was computed considering the surface settlement profile resembles the shape of an error or normal probability curve (Ref. 1). The surface settlement s(x) at any point x is:

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

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

$$i = W/(2\pi)^{1/2} \quad (2)$$

and W is the half width of the normal probability curve and may be estimated as (Ref. 1):

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

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:

$$V_L = s_S W_{SZ} \quad (4)$$

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 be increased 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. 1).

The volume of the surface settlement is:

$$V_S = R_{S/L} V_L \quad (5)$$

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):

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

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

$$s(0) = V_S/W \quad (7)$$

Substituting Equation (6) into Equation (7)

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

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

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

Figure 1 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:

$$W_{SZ} = 5 \text{ feet}$$

For the project site, at elevation of the foundation, 268 feet, MSL, the distance to the average depth of the soft zone is:

$$z = 268 - (177.6 + 163.2)/2 = 97.6 \text{ feet}$$

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. 1) at the project site, consider β = 33 degrees:

$$W = z \tan\beta + W_{SZ}/2 = 97.6 \tan(33^\circ) + 5/2 = 65.882 \text{ feet}$$

$$i = W/(2\pi)^{1/2} = 65.882/(2\pi)^{1/2} = 26.283 \text{ feet}$$

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

$$s(0) = R_{S/L}\, s_S\, W_{SZ} / W = (2/3) \times 0.876 \times 5/65.882 = 0.0443 \text{ inches}$$

Equation 1 become:

$$s(x) = 0.0443 \text{ Exp } \{-x^2/(2 \times 26.283^2)\} \text{ inches}$$

or $$s(x) = 0.0443 \text{ Exp}(-x^2/1381.59) \text{ inches}$$

3.3 Surface Settlement Considering Various Widths of Soft Zone

A wide soft zone can be considered as a series of narrow soft zones located one next to the other. Surface settlement due to a wide soft zone was computed by superimposing trough computed for narrow soft zone many times to simulate the desired width. In this calculation, a series of troughs considering 5-foot width soft zones were superimposed to simulate soft zones representing various widths of the soft zone.

Figure 2 presents the multiple troughs and their combined settlement for various widths of soft zone. Figures 3 through 5 present the settlement, slope, and curvature at the surface considering various widths of soft zone. The maximum settlement, maximum differential settlement, maximum slope, maximum curvature, and the ratio of the maximum settlement at the surface to the soft zone compression are also summarized in the following table:

Soft Zone Width (feet)	Maximum Settlement (inches)	Maximum Differential Settlement within Foundation width (inches)	Maximum Slope (feet/feet)	Maximum Curvature (per foot)	Ratio of Maximum Settlement to Soft Zone Compression (%)
25	0.214	0.202	0.00040	0.000024	24%
50	0.384	0.368	0.00064	0.000034	44%
75	0.495	0.450	0.00073	0.000029	56%
100	0.550	0.450	0.00074	0.000019	63%

125	0.574	0.389	0.00074	0.000017	66%
150	0.581	0.289	0.00074	0.000017	66%
Maximum	0.581	0.450	0.00074	0.000034	66%

3.5 Reaction Under a Rigid Foundation

Foundations can be designed as flexible foundation or rigid foundation. For the case of a flexible foundation, the foundation is considered to be deformed in the same shape and magnitude as the settlement profile provided in the above section.

For the case of a rigid foundation, soil under the slab will be settled equally but the reaction on the foundation will not be uniform.

Based on the differential settlement of 0.45 inches and an average subgrade modulus of 17 pci (Ref. 5), the reaction at the center of the trough could be less than the reaction away from the center in the amount of:

$p = k\,y = 17$ pci x 0.450 inches = 7.65 psi = 1,100 psf

4. Results

Considering the upper limits for the settlement, slope, and curvature of the slope for various width of the soft zone. The following is recommended for design:

The maximum settlement is 1/2 inch

The maximum differential settlement is 1/2 inch

For the design of a flexible foundation, consider maximum slope of 0.00075 feet/feet and maximum curvature of the slope of 0.000035 per foot.

For the design of a rigid foundation, consider the reaction of the soil under the center of the foundation is 1,100 psf less than the pressure at the perimeter of the foundation, distributed in the form of a parabola with the vertex at the center of the tank.

5. References

(1) Cording, E. J., Hansmire, W. H. MacPherson, H. H. Lenzini, P. A. and A. P. Vonderohe (1976), Displacements Around Tunnels in Soils, U. S. Department of Transportation Report No. DOT-TST 76T-22, Washington D. C.

(2) M-TC-Z-00004, Task Requirements and Criteria Document Salt stone Vault #2 Project, Bldg. 451-002Z, Rev. 1.

(3) K-CLC-Z-00008, Evaluation of Test Data for Vault 2. Rev. 0.

(4) Subcontract No. AB80188N, Sheet 014, Task 13, GeoTesting Express Laboratory Test Report, June 2005.

(5) K-ESR-F-00014, Pit Disassembly and Conversion Facility (PDCF) Soft Zone Characterization and Evaluation Report, Rev. 0, July 2005.

(6) K-CLC-Z-00006, Soil Properties for the Design of Vault No. 2, Rev. 0, August 2005.

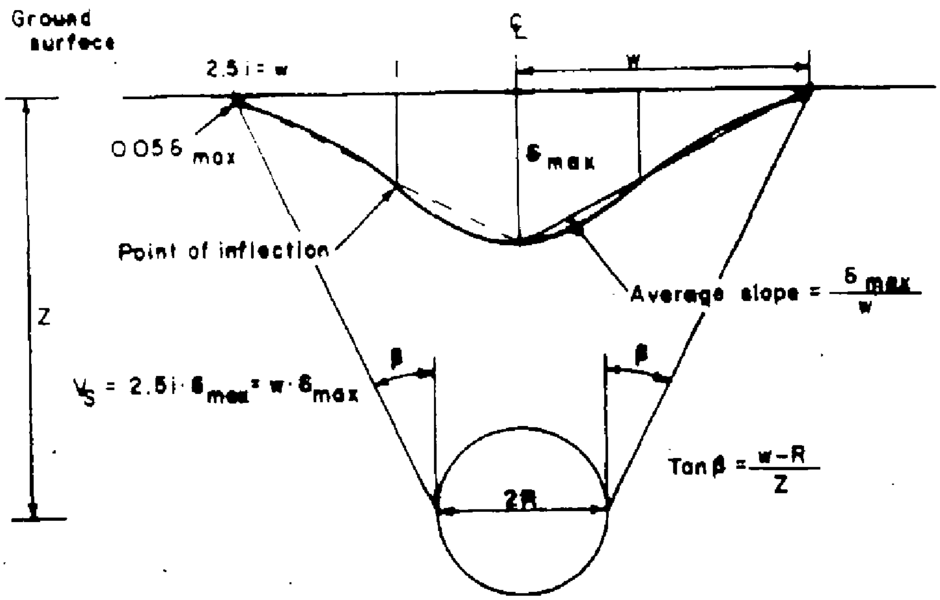

Figure 1 Geometry of surface settlement trough

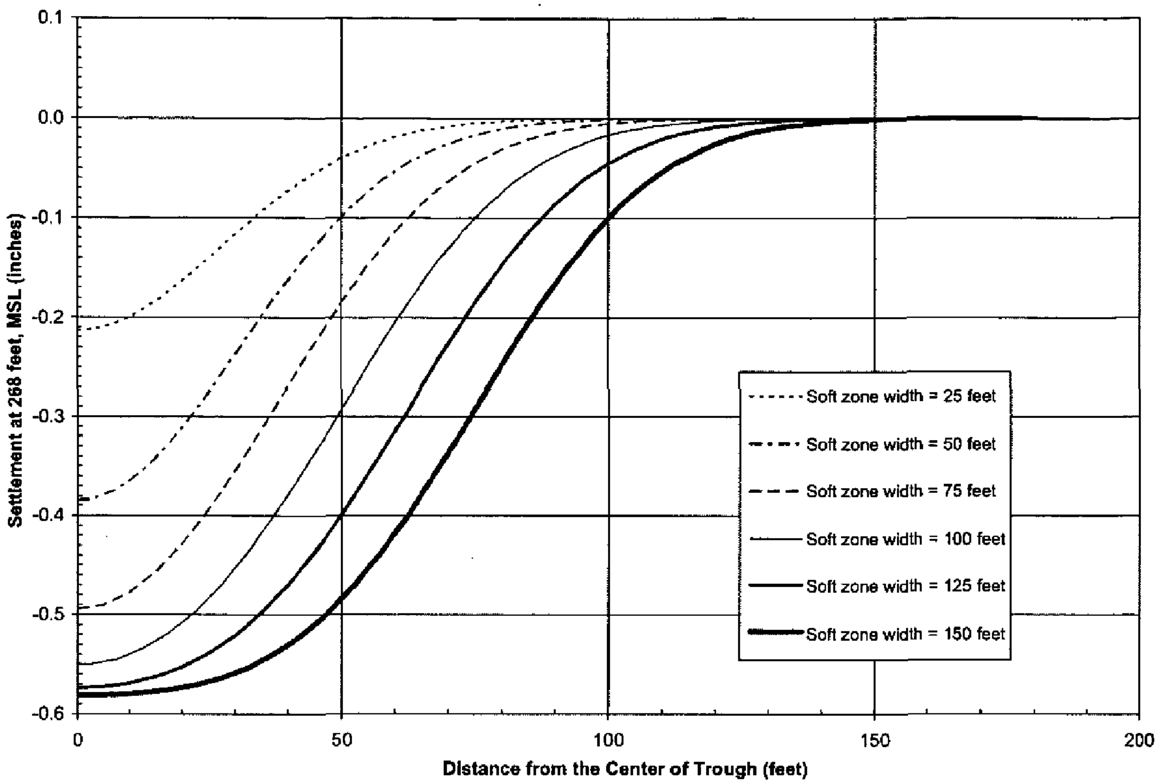

Figure 2 Surface settlement profile for various widths of soft zone

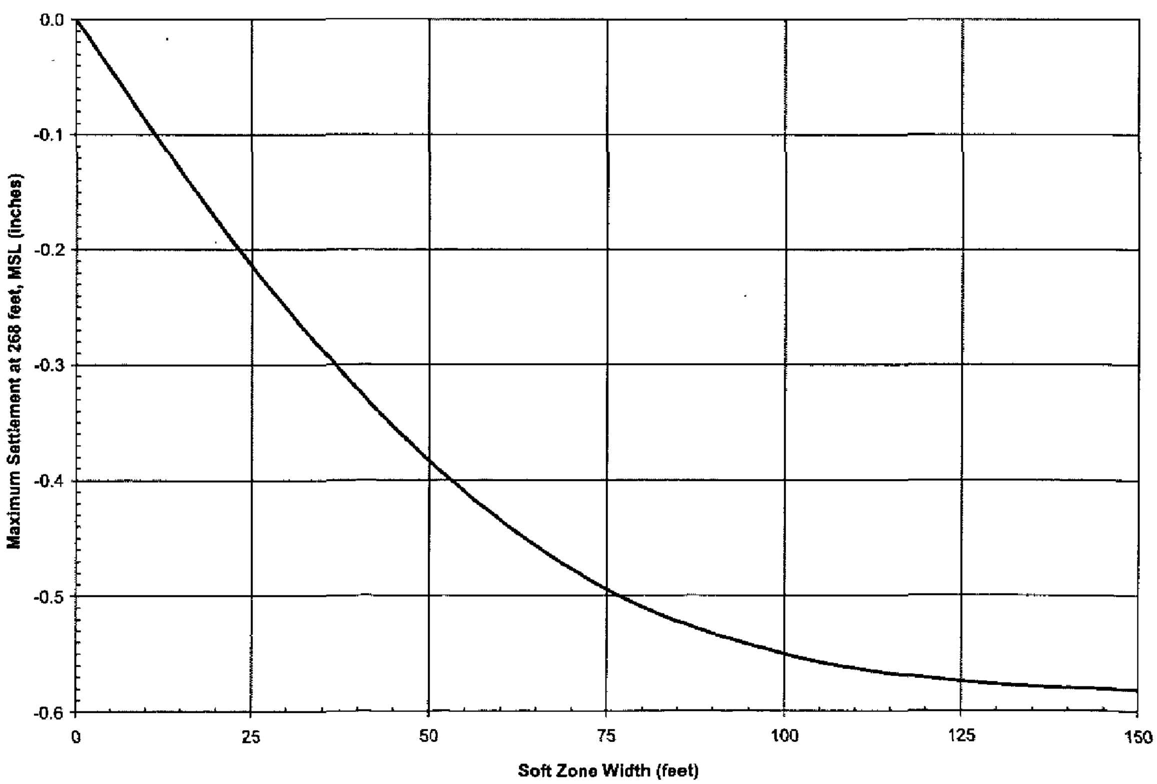

Figure 3 Maximum surface settlement for various widths of soft zone

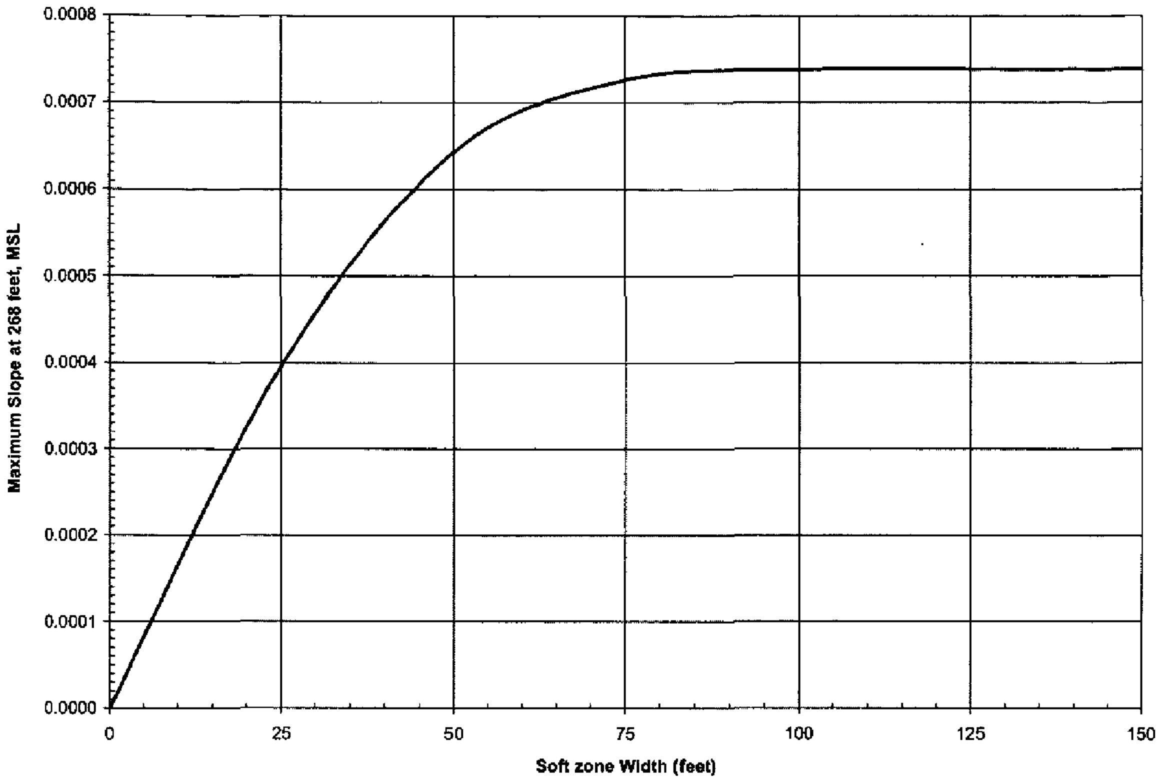

Figure 4 Maximum surface settlement slope for various widths of soft zone

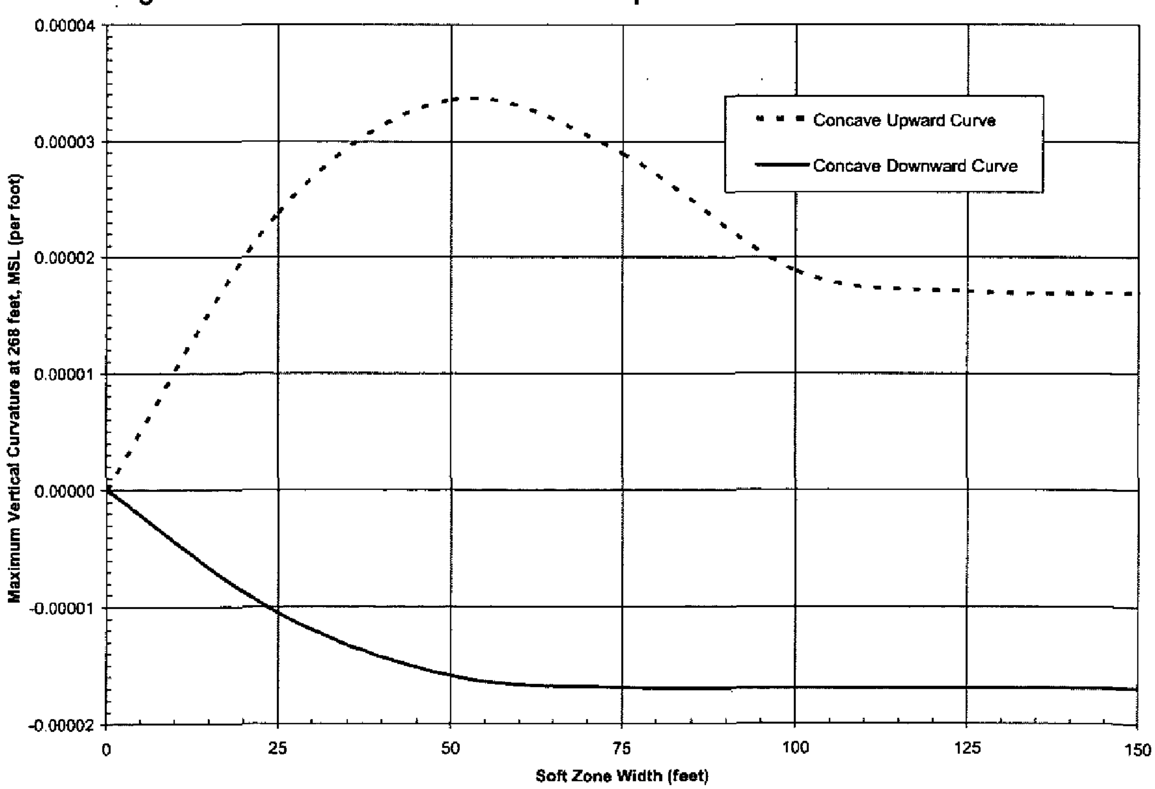

Figure 5 Maximum surface settlement curvature for various widths of soft zone

Appendix G

K-CLC-Z-00002

Slope Stability for the Saltstone Disposal Facility

Rev. 1, September 2003

(57 pages)

Calculation Cover Sheet

Project	Calculation No.	Project Number
N.A.	K-CLC-Z-00002	N.A. MDM 3/24/03

Title	Functional Classification	Sheet 1 of ~~47~~ 56 (one disk containing computer files)
Slope Stability for the Saltstone Disposal Facility (U)	GS	
	Discipline: Geotechnical	

☐ Preliminary ☒ Confirmed

Computer Program No.	Version / Release No.
☐ SLOPW/W	4.24

Purpose and Objective

The purpose of this calculation is to determine global slope stability safety factors for the soil cover over the Saltstone Disposal Facility for static and seismic loading cases.

Summary of Conclusion

See Summary and Conclusions Section on Sheet 7.

FOR REVISION 1 SEE SHEET 7 & 7A.

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ADC & Reviewing Official: Michael D. McHood
(Name and Title)
Date: 3/24/03

Revision

Rev. No.	Revision Description
0	Original
1	ADDITIONAL SLOPE STABILITY RUNS USING TOTAL STRESS SHEAR STRENGTHS AND HIGHER PEAK GROUND ACCELERATIONS.

Sign Off

Rev. No.	Originator (Print) Sign / Date	Verification / Checking Method	Verifier / Checker (Print) Sign / Date	Manager (Print) Sign / Date
0	Michael D. McHood Michael D. McHood	Document Review	William T. Li William T. Li	Michael R. Lewis M.R. Lewis 3/21/03
1	MICHAEL D. MCHOOD Michael D. McHood 3/24/03	Document Review	William T. Li William T. Li	M.R. Lewis 4/3/03

Release to Outside Agency – Design Authority (Print)	Signature	Date
NA	NA	NA

Security Classification of the Calculation

Unclassified

TABLE OF CONTENTS

1.0 PURPOSE

The purpose of this calculation is to determine global slope stability safety factors for the soil cover over the Saltstone Disposal Facility for static and seismic loading cases.

2.0 INPUT DATA

2.1 SLOPE / VAULT GEOMETRY AND DESIGN

This calculation is based on the following design information:

- Vault Dimensions and slope geometry were taken from SRS drawings C-CC-Z-0013 (SRS, 1998), W828992 (SRS, 1989), W780527 (SRS, 1986a), and W780529 (SRS, 1986b) (see Attachment 1).
- For this calculation the concrete vault and contents was modeled as a monolith with a conservatively assumed unit weight of 140 pcf.
- By specification the grout used to fill the Saltstone Vaults has a minimum compressive strength of 200 psi (WSRC, 1992). Actual compressive strength of the grout may be higher. For this calculation the conservative 200 psi value was used to model the disposal vault. No addition strength was attributed to the vault due to the reinforced concrete walls.

It is important to note that several conceptual designs have been considered for closure of the Saltstone Vaults. Both 4H:1V and 8H:1V slopes have been considered in conceptual designs for the final cover system. This calculation assumes the steeper 4H:1V slope. The post closure slope crest was set at 20 feet above the top of the vault. Other conceptual closure designs have much less fill placed over the top of the vault, reducing the slope height. The geometry of the vault and closure cover slope model is shown in Figure 1.

It is also important to note that some of the conceptual designs include geosynthetics and/or designed soil layers (i.e., low permeability capping soil or high permeability drainage layers). This calculation only considers global slope stability. Stability of the interface between geosynthetics and designed soil layers needs to be evaluated and designed for during final design of the cover system.

2.2 GROUNDWATER

The water table in the vicinity of Vault No. 4, based on Well ZBG-2, varies between 213 and 228 ft-msl (see Figure 2 and 3). However, the groundwater elevation used for the slope stability analysis is conservatively placed at 245 feet mean sea level. This conservative assumption is based on water levels observed in boreholes during drilling, (MRCE, 1986a) historical readings (Cook, 1983) and reports of perched water in Z-Area (MRCE, 1986b and Cook, 1986).

2.3 SEISMIC LOADING

Based on the USGS seismic hazard maps (Frankel et al., 1996, see Figure 4) and the International Building Code (ICC, 2000) a PGA of 0.21g would be selected for Z-Area. However, the International Building Code allows a reduction of up to 20% in cases where site specific studies have been performed. Extensive ground response modeling has been performed for the SRS (Lee et. al, 1997; Lee, 1998). Based on the results of ground response modeling at SRS the reduction is warranted and a PGA of 0.17g (i.e., 0.21 x 0.8) is applicable. This PGA is very conservative when compared with the PGA of 0.11g determined for Performance Category 1 structures at SRS (Lee, 1998), which is the performance category of the Saltstone Vaults.

For this calculation a horizontal seismic coefficient (k_h) of 0.17 and vertical seismic coefficients (k_v) of ±0.17 were used. Engineering practice allows seismic coefficients as low as ½ of the PGA and therefore these seismic coefficients are very conservative (Abramson et al., 1996). Combined horizontal and vertical loading cases were run with one component at 100 % and the second concurrent component at 40%, consistent with ASCE Standard 4-98 (ASCE, 1998). Seismic loading cases are summarized below.

- $k_h = 0.17$ and $k_v = 0$
- $k_h = 0$ and $k_v = \pm 0.17$ (+ is down and – is up)
- $k_h = 0.17$ and $k_v = \pm 0.07$ (+ is down and – is up)
- $k_h = 0.07$ and $k_v = \pm 0.17$ (+ is down and – is up)

2.4 SOILS AND VAULT STRENGTH

This calculation considers long-term stability of the soil slopes after closure of the saltstone vaults. Long-term soil shear strength properties are based on effective friction angle (ϕ'). Samples from borings ZB-2, ZB-8, Z-1, Z-2 and Z-4 (see Figure 3) were tested to determine ϕ'. Saturated CU and CIU triaxial shear tests yield ϕ' values between 28° and 34° (BSRI, 1992; Woodward-Clyde, 1985). Selection of the ϕ' values of 28° for the natural soil and 33° for the engineered fill is conservative and allows for some strength loss during seismic loading. Laboratory tests results are presented in Attachment 2. The following effective stress shear strengths were used for the slope stability calculations:

	Effective Stress Properties	
Engineered Fill	$\phi' = 33°$	$c' = 20$ psf
Natural Soil (above water table)	$\phi' = 28°$	$c' = 100$ psf
Natural Soil (below water table)	$\phi' = 28°$	$c' = 0$ psf
Vault Strength *	$\phi' = 0°$	$c' = 4{,}000$ psf

* For this calculation the concrete vault and grout were modeled as a soil having $\phi' = 0$ and $c' = 4{,}000$ psf i.e., constant shear strength corresponding of compressive strength of 200 psi as defined by the American Concrete Institute (shear strength = $2\sqrt{f_c'}$ where f_c' is compressive strength [ACI, 2002]), which is about a third of the shear strength as defined by Mohr-Coulomb failure criteria (shear strength = ½qu where qu is unconfined compression strength).

Engineered fill will be placed around and over the Saltstone vaults. The final design for the Saltstone Disposal Facility has not been performed and engineered fill will not be placed until some future date. The engineered fill must meet the requirements of existing site standards (WSRC, 2001) and be compacted to a minimum density of 95% of maximum dry density, determined in accordance with ASTM D-1557. The shear strength for the engineered fill is based on experience with SRS fill soils and engineering judgement. A soil unit weight of 120 pcf was assumed for this calculation.

For conservatism, a tension crack was assumed for all models (both static and seismic cases). The tension crack is two feet deep along the top of the slope up to the crest, tapering to zero feet at the toe of the slope. The tension crack was filled with water as may occur after rain.

3.0 CALCULATIONS

Slope stability calculations were completed using SLOPE/W version 4.24 software (GEO-SLOPE, 1998). SLOPE/W is commercially available software similar to PC STABL and other slope stability analysis software. Safety factors were calculated using Ordinary, Bishop's, Janbu's, and Spencer's Methods. Results reported in this calculation are from Spencer's method, which considers both moment and force equilibrium. Static and seismic slope stability calculations were completed for a closure concept having a 4H:1V slopes extending to the base of the vault. Several seismic loading cases were run having different vertical and horizontal loading. The results for both static and seismic cases are summarized in Table 1. Computer files are contained on the accompanying compact disk.

The models run allowed slip surfaces to pass through the disposal vault. Failure through the vault was never the critical case (i.e., didn't have the lowest factor of safety). At the request of the South Carolina Department of Health and Environmental Control, (SCDHEC) an additional stability case was run where slip surfaces were forced through the disposal vault to determine factor of safety against a combined vault and slope failure. Factor of safety for "vault failure" case is also summarized in Table 1.

As advised by the consultant, Dr. Marcuson, additional runs were performed where total stress shear strength was used. Total stress shear strength is lower than the effective stress accounting for pore pressure and strength loss due to loading and provides additional conservatism. For the additional runs the natural soil above the water table was given a conservative friction angle of 23° with cohesion of 650 psf. The natural soil below the water table was modeled two ways 1) with a friction angle of 8° and cohesion of 900 psf and 2) with a friction angle of zero and cohesion of 1,600 psf. Attachment 2 discusses the shear strength tests. In addition, the vault strength is the same as the engineered fill, assuming a failure through the engineered fill along the edge of the vault. For the total stress runs the seismic coefficients are $k_h = 0.17$ and $k_v = -0.07$ (i.e., the lowest factor of safety case using effective stress shear strength). Factor of safety for the "total stress" cases are summarized in Table 1.

Table 1. Summary of Slope Stability Safety Factors Computed Using Slope/W and Effective Stress Shear Strengths

	Horizontal Seismic Coefficient (k_h)	Vertical Seismic Coefficient (k_v)	Factor of Safety	Minimum Acceptable Factor of Safety
Case 1 static	0	0	2.6	1.2 to 1.5
Case 2	0.17	0	1.5	1.0 to 1.2
Case 3	0	0.17	2.6	1.0 to 1.2
Case 4	0	-0.17	2.7	1.0 to 1.2
Case 5	0.17	0.07	1.6	1.0 to 1.2
Case 6	0.17	-0.07	1.5	1.0 to 1.2
Case 7	0.07	0.17	2.1	1.0 to 1.2
Case 8	0.07	-0.17	2.0	1.0 to 1.2
Case 9 vault failure	0.17	-0.07	1.8	1.0 to 1.2
Case 10 total stress	0.17	-0.07	1.2	1.0 to 1.2
Case 11 total stress	0.17	-0.07	1.2	1.0 to 1.2

Note: For Cases 1 through 8 effective stress shear strength was used, see Section 2.4.
For Case 9 the failure surface was forced through the vault with k_h and k_v same as Case 6 (i.e., lowest factor of safety case).
For Case 10 and 11 total stress shear strength was used. Natural soil above the water table was given a friction angle of 23° with cohesion of 650 psf. Natural soil below the water table was two ways 1) with a friction angle of 8° with cohesion of 900 psf (Case 10) and 2) with a friction angle of zero with cohesion of 1,600 psf (Case11). See Attachment 2 for shear strength tests.

3.1 ACCEPTABLE SAFETY FACTORS

3.1.1 STATIC

Generally, the recommended safety factor for static slope stability is around 1.5. However, safety factors as low as 1.2 have been recommended. The range of acceptable safety factors is due to many reasons including the consequence of failure, the extent of the subsurface characterization, whether or not site specific strength testing was performed, and the natural variability of the soils.

3.1.2 SEISMIC

For seismic slope stability, generally accepted safety factors range from 1.0 to 1.2 (USACE, 1970; Abramson et al., 1996; Huang, 1983). As with static slope stability, the range of acceptable safety factors is due to many reasons including the consequence of failure, the extent of the subsurface characterization, whether or not site specific strength testing was performed, and the natural variability of the soils.

4.0 RESULTS

A summary of the safety factors computed using SLOPE/W for static and seismic loading cases is contained in Table 1. Figures 5 through 15 contain the plots of the individual SLOPE/W stability runs. In all cases the static and seismic safety factors are much greater than required. Additional details of the analysis can be obtained from computer files listed in Attachment 3 and copied on the accompanying disk.

Figures 16 through 18 show variations of the case 6 seismic loading with a limited number of radius focal points and larger radii. These runs were performed to show other failure surfaces and their factor of safety. These failure surfaces extend down through the natural soil and past the toe of the slope. Factors of safety are greater for these deeper failure surfaces.

5.0 SUMMARY AND CONCLUSIONS

A slope of 4 horizontal to 1 vertical provides factors of safety above the range of acceptable factors of safety (see Table 1). However, for long term stability the following recommendations should be implemented.

- Engineered fill must meet the requirements of existing site standards (WSRC, 2001) and be compacted to a minimum density of 95% of maximum dry density, determined in accordance with ASTM D-1557, to achieve acceptable shear strength.
- Drainage ditches shall be provided so water is conveyed away from the disposal vaults.
- Erosion control on the slopes is required

It is also important to note that final design for cover system has not been performed. Final design may include geosynthetics and/or designed soil layers (i.e., low permeability capping soil or high permeability drainage layers). This calculation only considers global slope stability. Stability of the interface between geosynthetics and designed soil layers needs to be evaluated during final design of the cover system. However, for the range of slopes being considered (4H:1V and 8H:1V), incorporation of geosynthetics and/or designed soil layers is not expected to be an issue.

6.0 PURPOSE AND RESULTS FOR REVISION 1

At the request of SCDHEC additional slope stability analysis was performed for a horizontal PGA of 0.20g and 0.21g. Horizontal seismic coefficients (K_h) of 0.20 and 0.21 were used (i.e., the full PGA). No vertical seismic coefficients (K_v) were used for the additional work.

At the request of SCDHEC, the soil model was extended to include the soil beyond the vault allowing a failure beneath and past the vault structure. For the additional work the "total stress" cases (i.e., case 10 and 11 from the Revision 0 work) were re-evaluated because they result in the lowest Factor of Safety. The "total stress" or "undrained" condition is conservative and assumes significant generation of pore pressure and associated strength loss during seismic loading. Figures 19 and 20 show the case 10 and case 11 soil models used for the previous work. Figures 21 and 22 show the extended models used for the additional work. Note that for the extended soil models (Figures 21 and 22) soil strengths are aligned with the soil stratum as opposed to assigning all soil beneath the water table one strength. Also note that the soil strength for deeper soils has been increased representing average strength as opposed to conservative lower bound strength. Strength test data are presented in Attachment 2. Sheet 42 shows strength data for the C2 stratum and the reinterpreted strength used for the C2 stratum. The strength of the engineered fill for both the effective stress cases and the total stress cases is $\phi = 33°$ & $c = 20$ psf. This strength is conservative and allows for some strength loss during seismic loading. By design the engineered fill will need to meet this strength requirement.

Results of the additional analyses are presented in Table 2. Figures 23 through 26 contain the plots of the additional slope stability runs. In all cases the static and seismic safety factors are acceptable. Additional details of the analysis can be obtained from computer files listed in Attachment 3 and copied on the accompanying disk.

Table 2. Additional Slope Stability Safety Factors Computed Using Slope/W and Total Stress Shear Strengths

	Soil Strength Condition	Soil Strength [1]	Vault Strength [2]	Horizontal Seismic Coefficient (k_h)	Vertical Seismic Coefficient (k_v)	Factor of Safety
Case 10 a	total stress	Figure 21	$\phi = 33°$ & $c = 20$ psf	0.20	0	1.3
Case 10 b	total stress	Figure 21	$\phi = 33°$ & $c = 20$ psf	0.21	0	1.3
Case 11 a	total stress	Figure 22	$\phi = 0°$ & $c = 4000$ psf	0.20	0	1.2
Case 11 b	total stress	Figure 22	$\phi = 0°$ & $c = 4000$ psf	0.21	0	1.2

1 Shear strengths of the soil layers are shown on the referenced Figure.

2 Shear strength of the vault was modeled both as a weak grout ($\phi = 0°$ & $c = 4000$ psf) and as engineered fill ($\phi = 33°$ & $c = 20$ psf). Results reported here are those that produced the lowest factor of safety.

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FIGURES

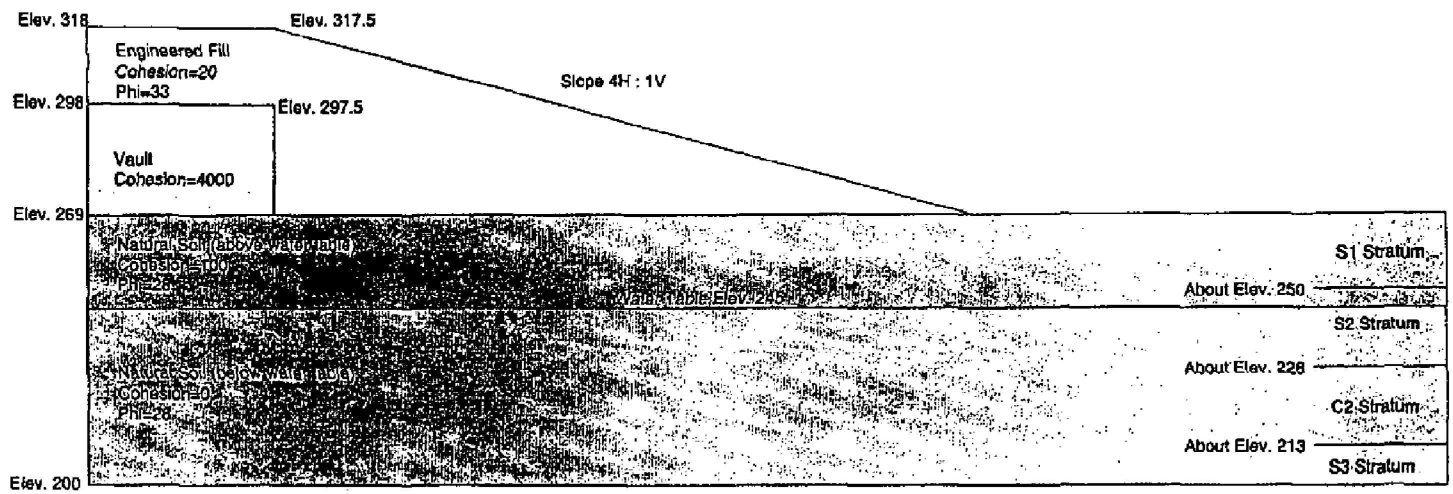

Figure 1. Vault and Closure Cover Slope Geometry Used to Calculate Slope Stability

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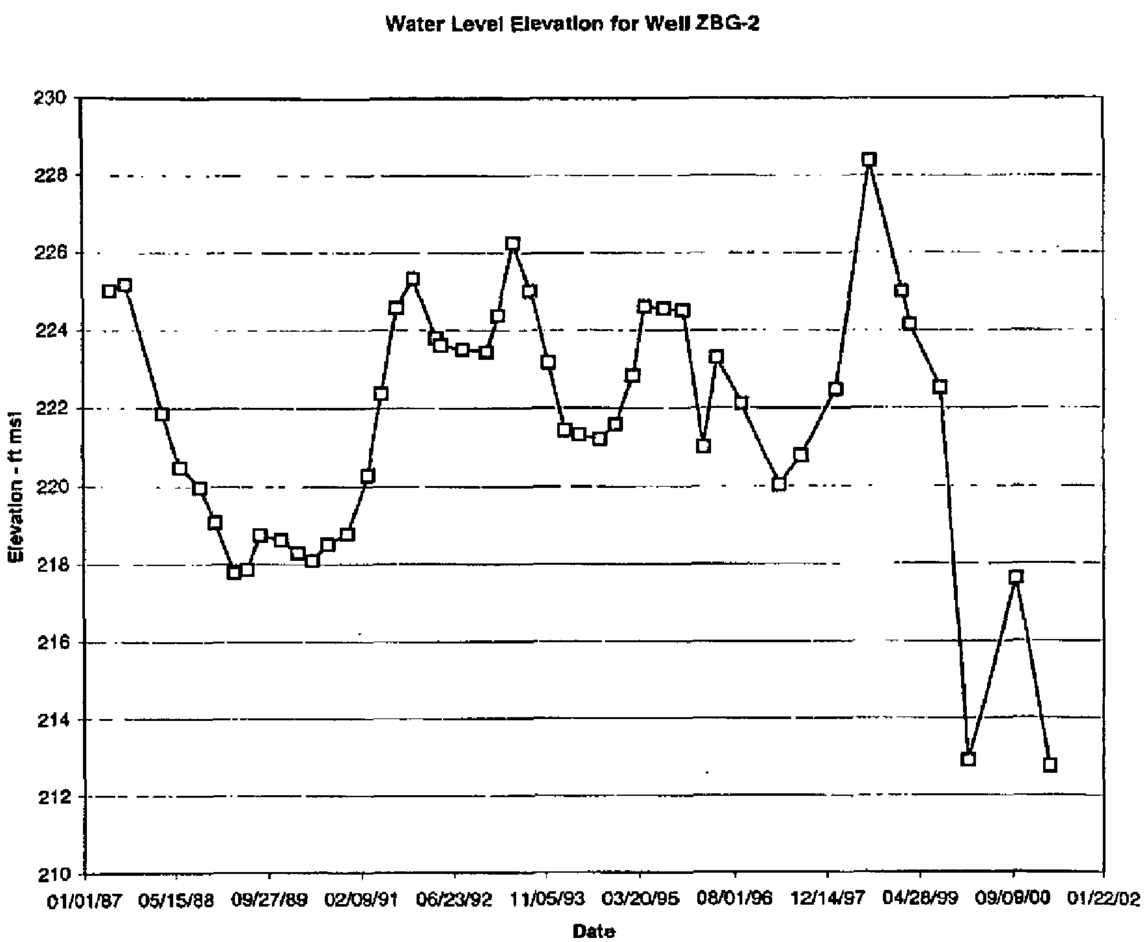
Sheet No. 11	
Rev. 0	

Calculation No. K-CLC-Z-00002
Sheet No. 12
Rev. 0



well-zbg-2.xls

Figure 2 - Water Table Elevation Measured in Well ZBG-2
(SRS Groundwater Information Management System)

Calculation No. K-CLC-Z-00002
Sheet No. 13
Rev. 0

Z-Area Vault No. 4

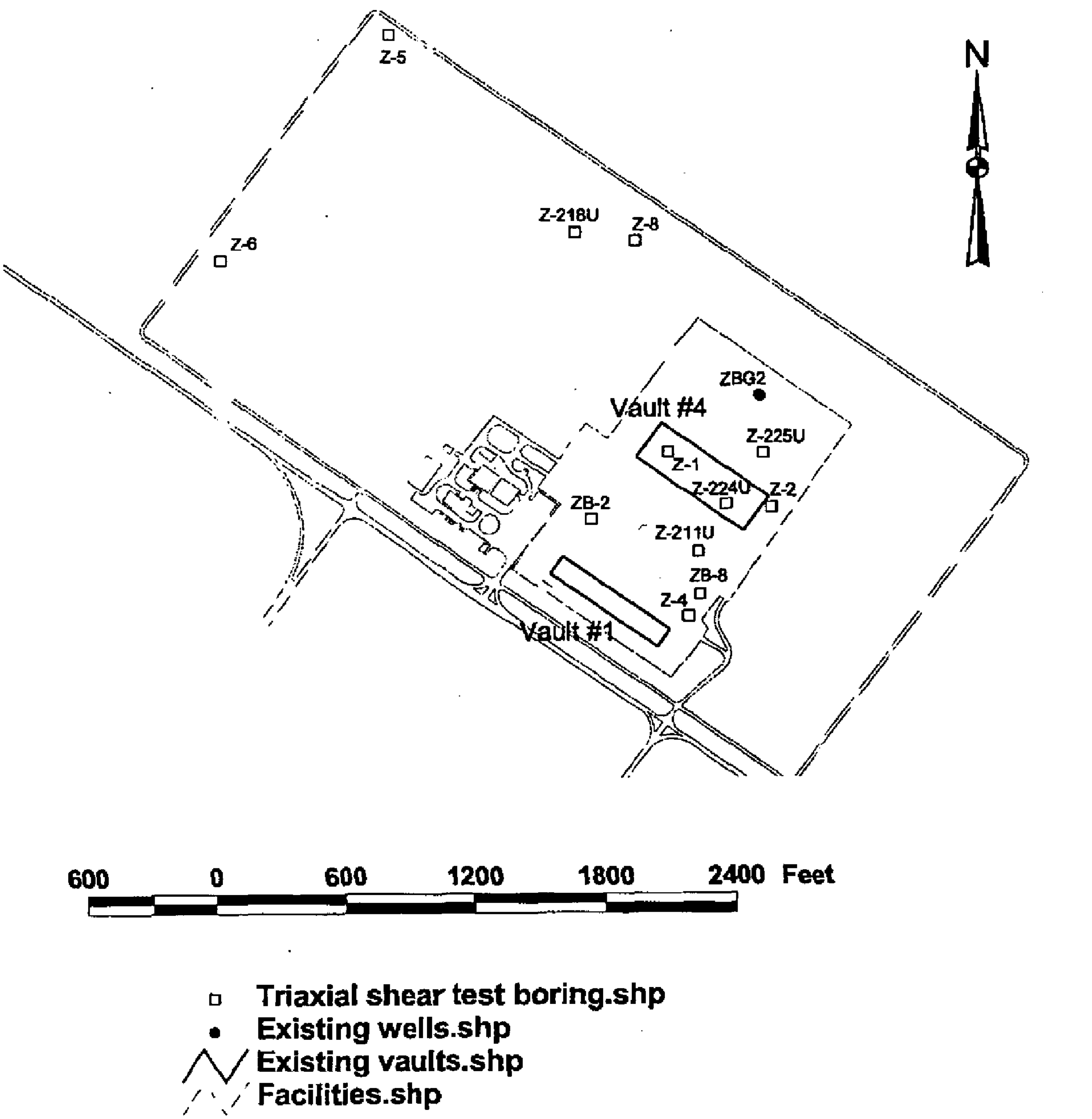


Figure 3 - Location of Vault No. 4, Triaxial Shear Test Borings and Well ZBG-2

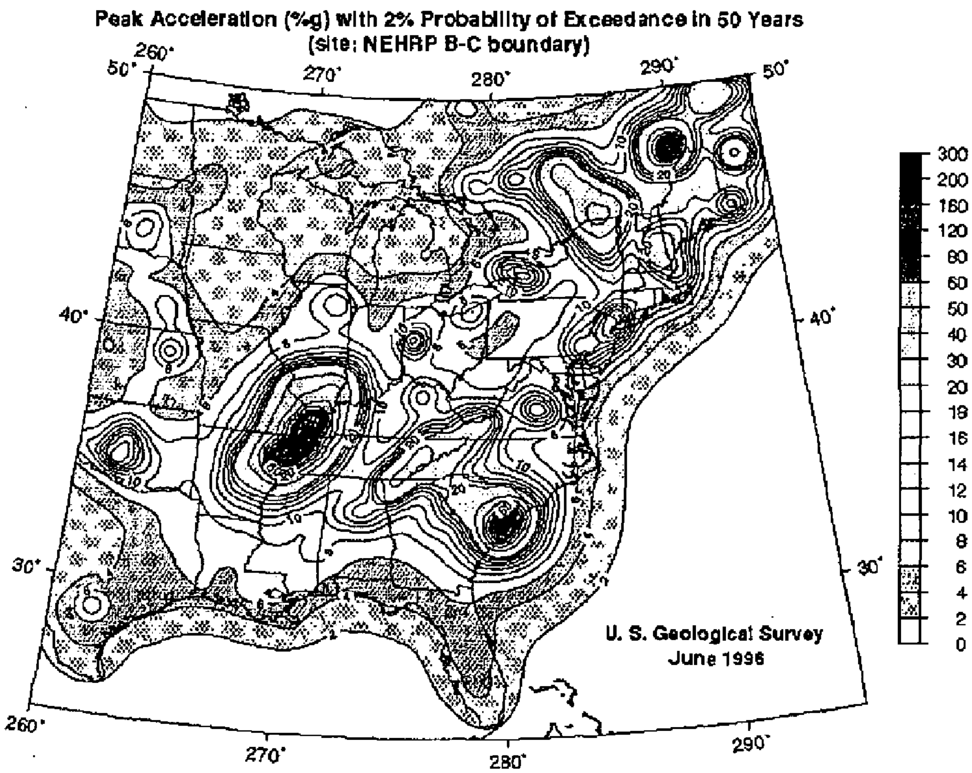


Figure 4 - USGS Hazard Map for Peak Ground Acceleration
2% Probability of Exceedance in 50 Years
(Frankel et al., 1996)

Calculation No. K-CLC-Z-00002	Sheet No. 15	Rev. 0

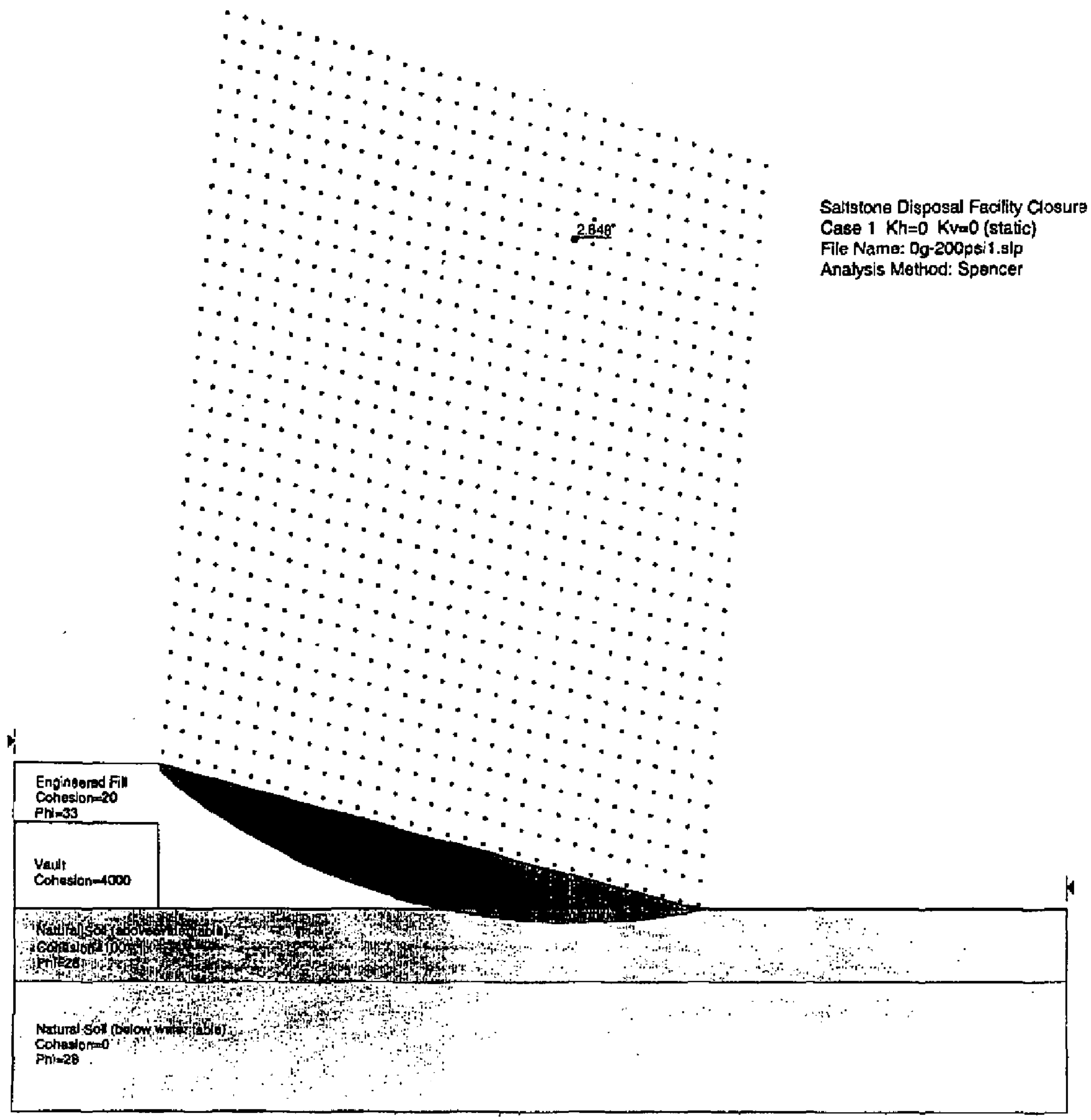


Figure 5. Safety Factor Calculated Using Slope/W for Case 1 (Static Case k_h=0 and k_v=0)

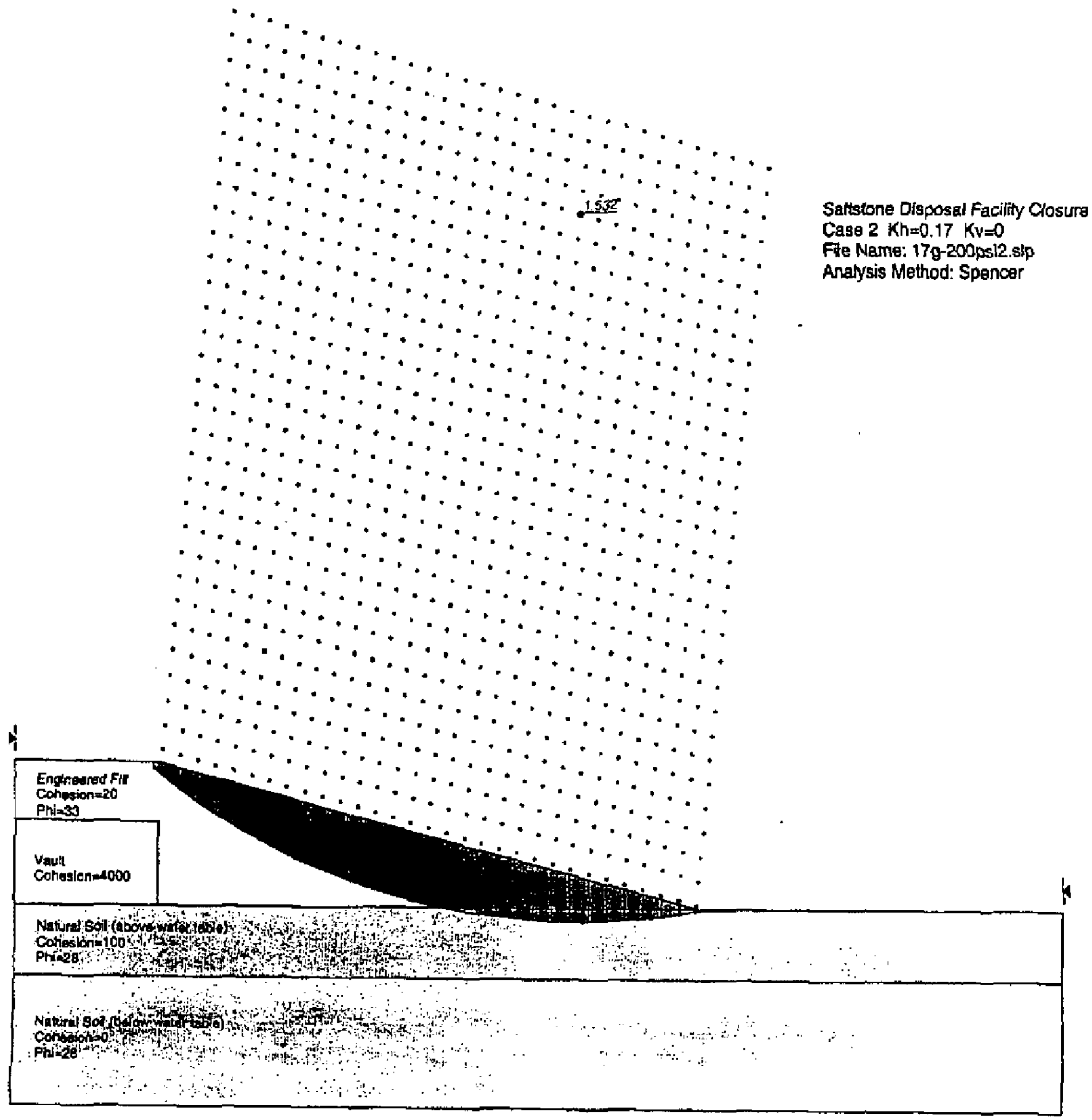


Figure 6. Safety Factor Calculated Using Slope/W for Case 2 (k_h=0.17 and k_v=0)

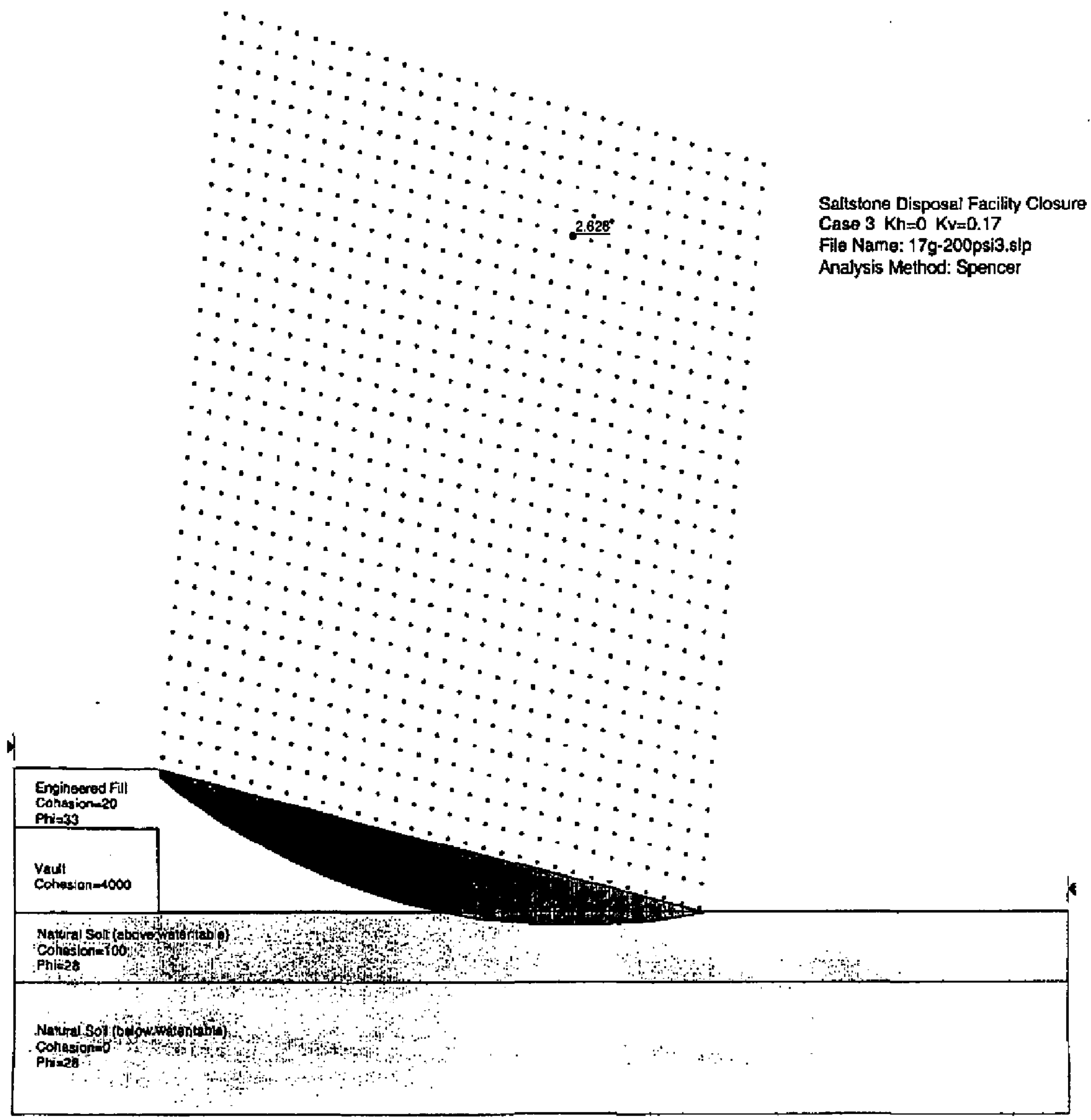


Figure 7. Safety Factor Calculated Using Slope/W for Case 3 (k_h=0 and k_v=0.17)

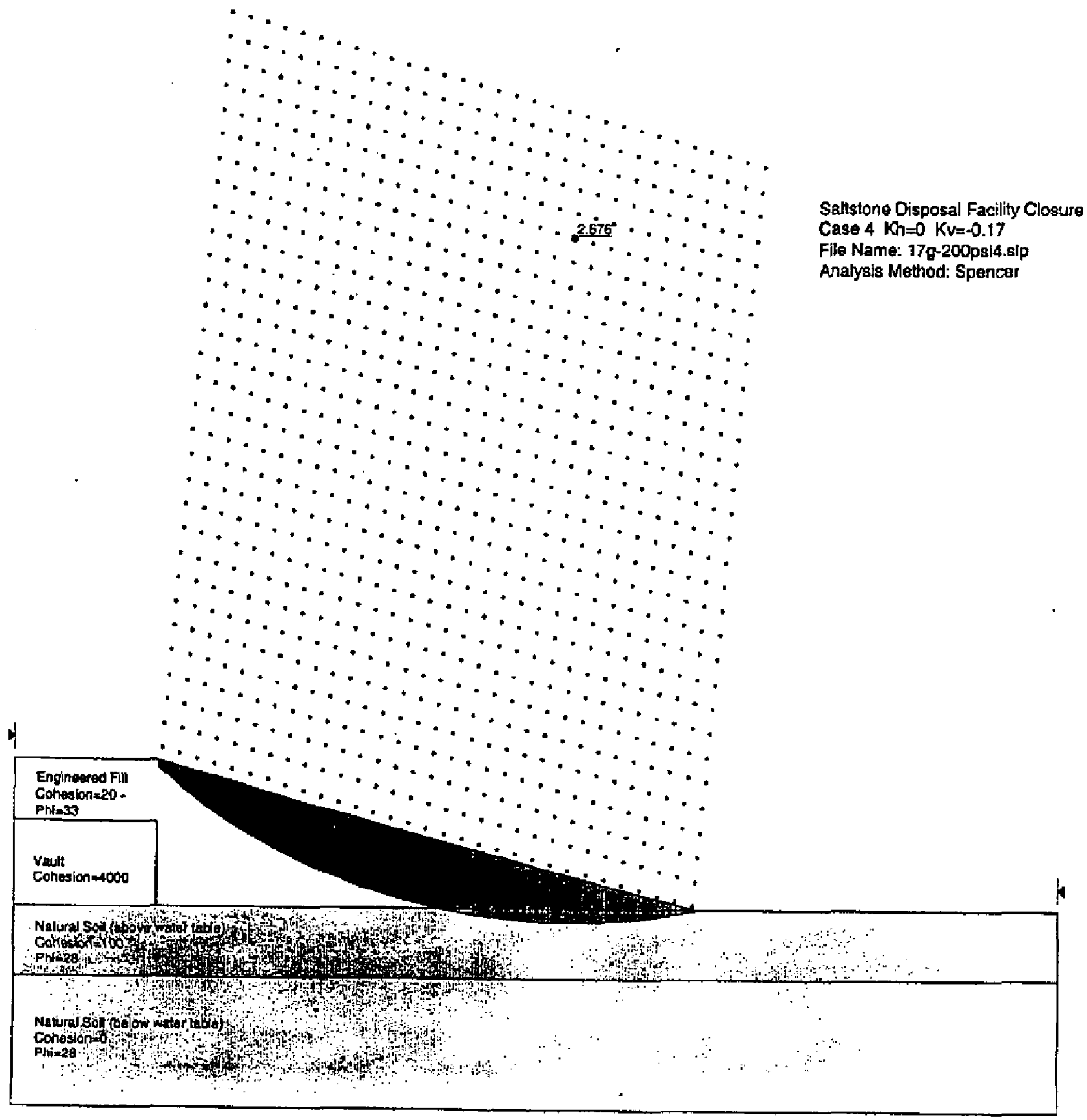


Figure 8. Safety Factor Calculated Using Slope/W for Case 4 (k_h=0 and k_v=-0.17)

Calculation No. K-CLC-Z-00002
Sheet No. 19
Rev. 0

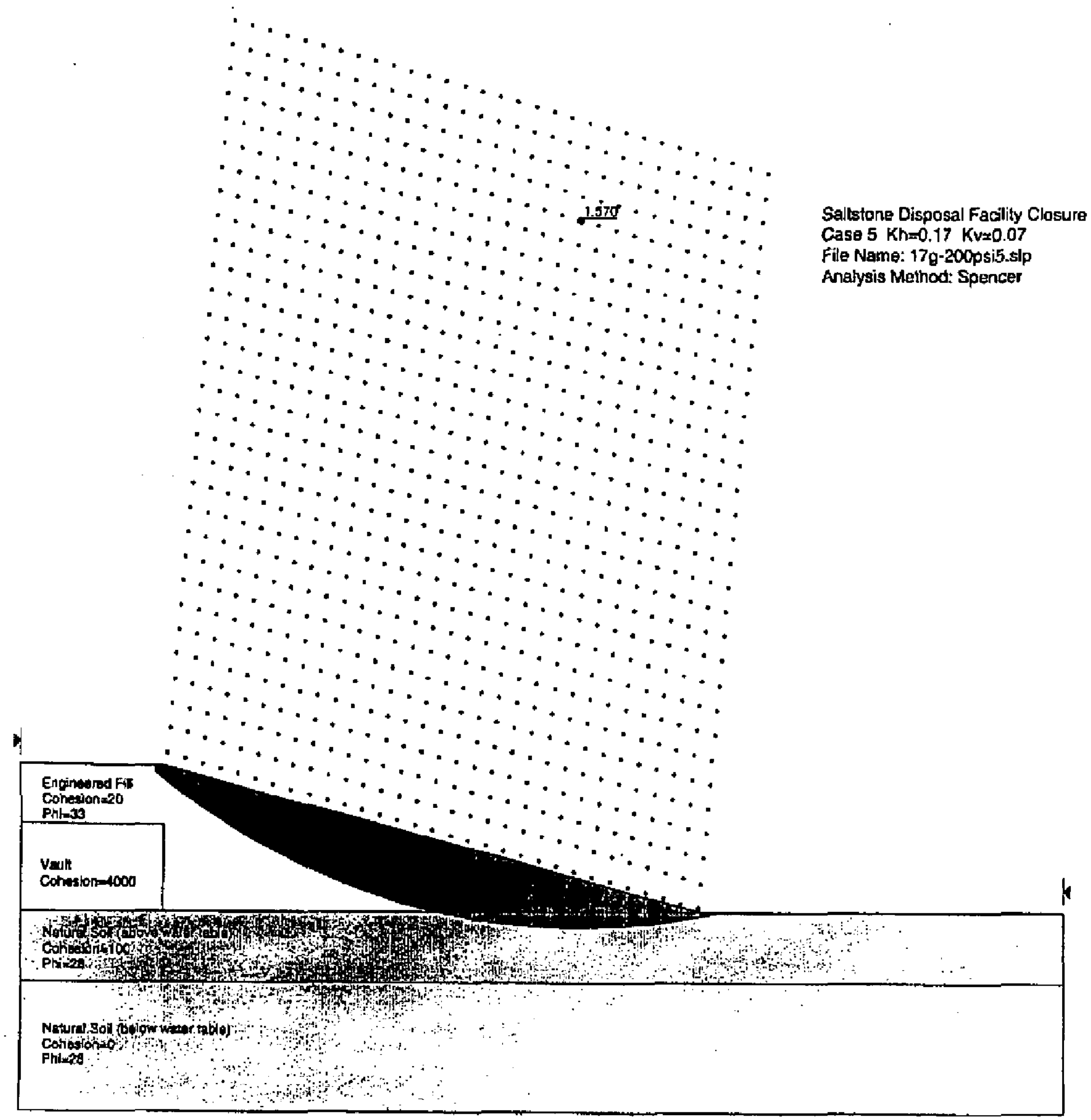


Figure 9. Safety Factor Calculated Using Slope/W for Case 5 (k_h=0.17 and k_v=0.07)

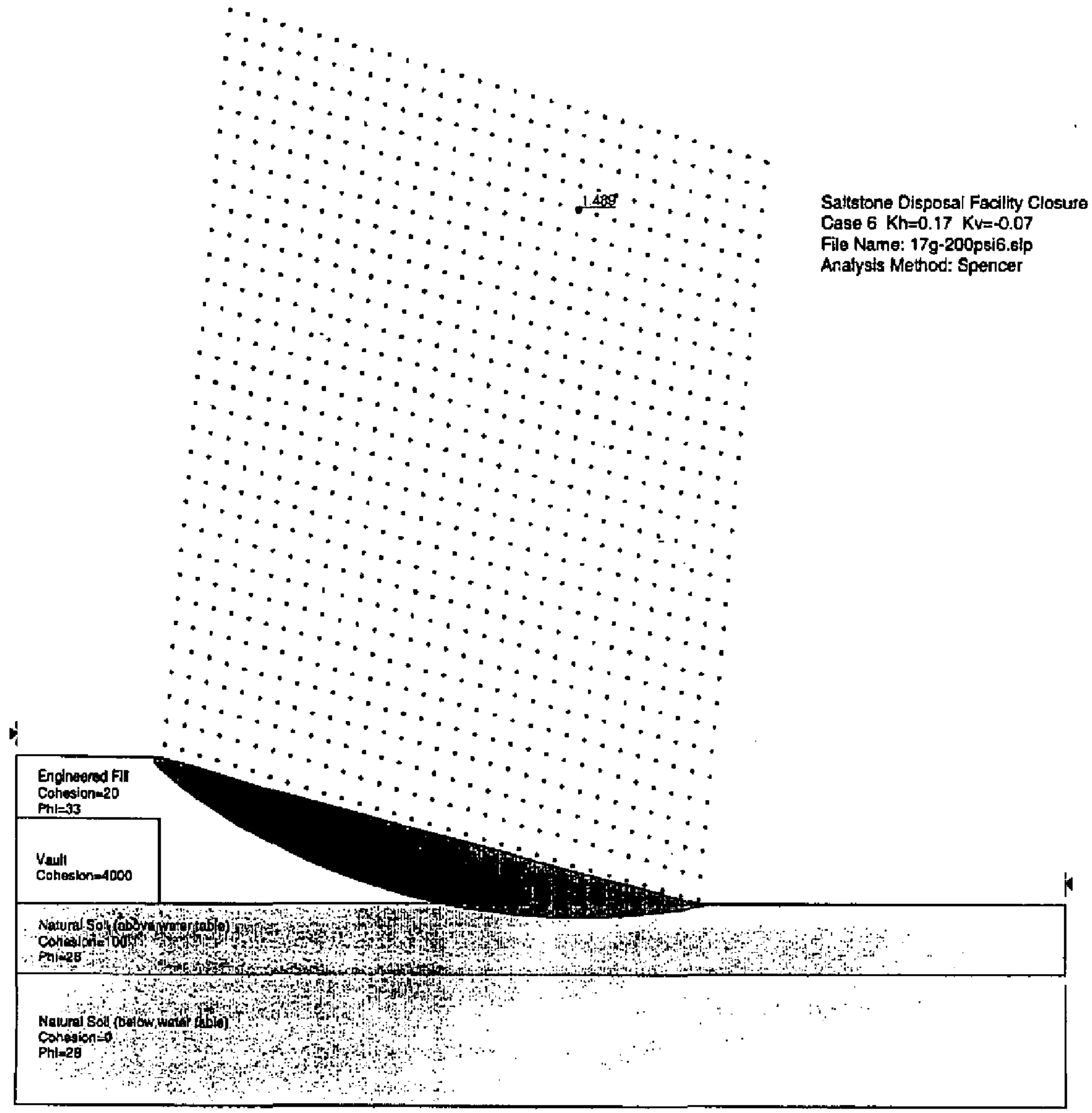


Figure 10. Safety Factor Calculated Using Slope/W for Case 6 (k_h=0.17 and k_v=-0.07)

Calculation No. K-CLC-Z-00002
Sheet No. 21
Rev. 0

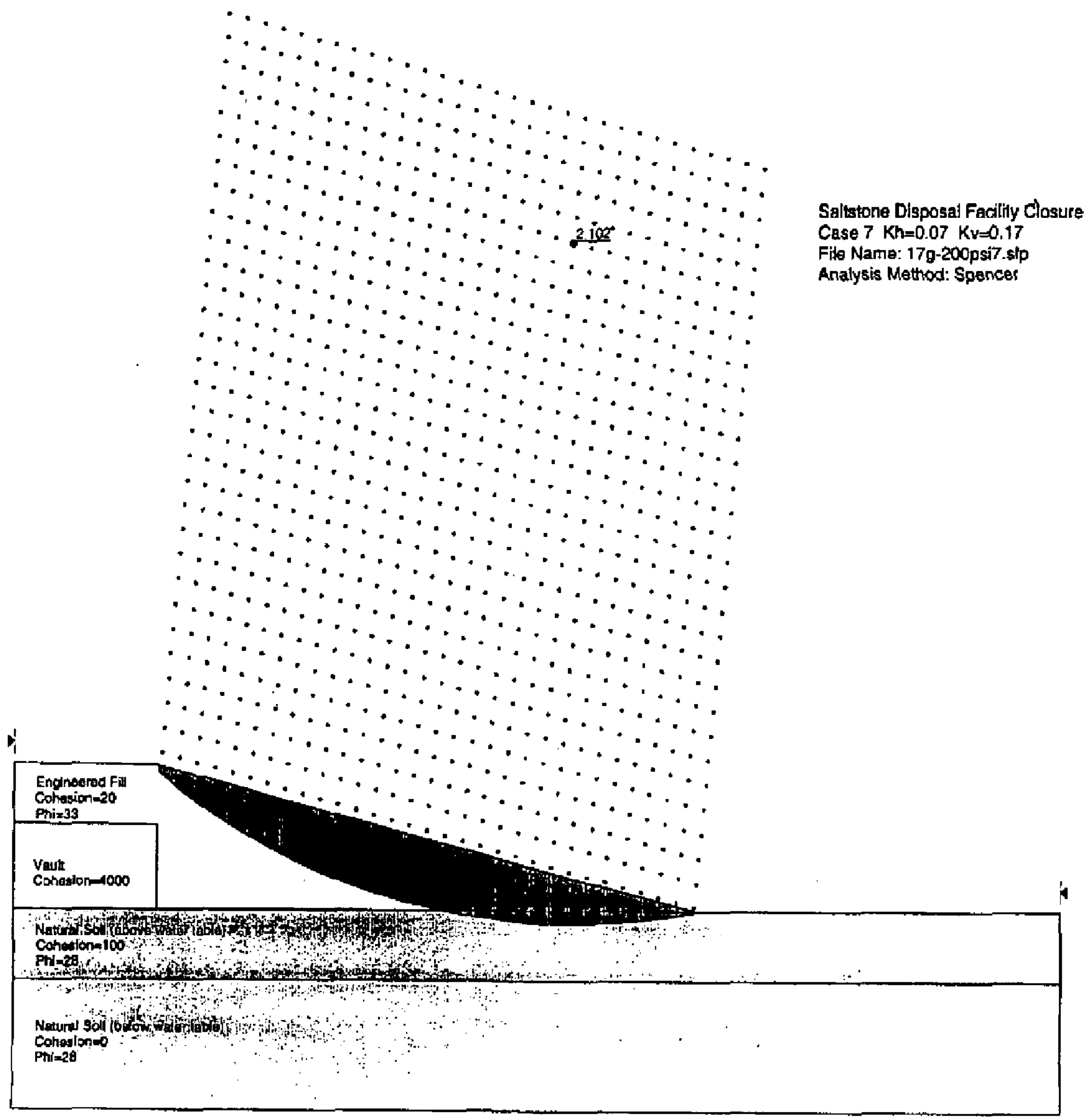


Figure 11. Safety Factor Calculated Using Slope/W for Case 7 (k_h=0.07 and k_v=0.17)

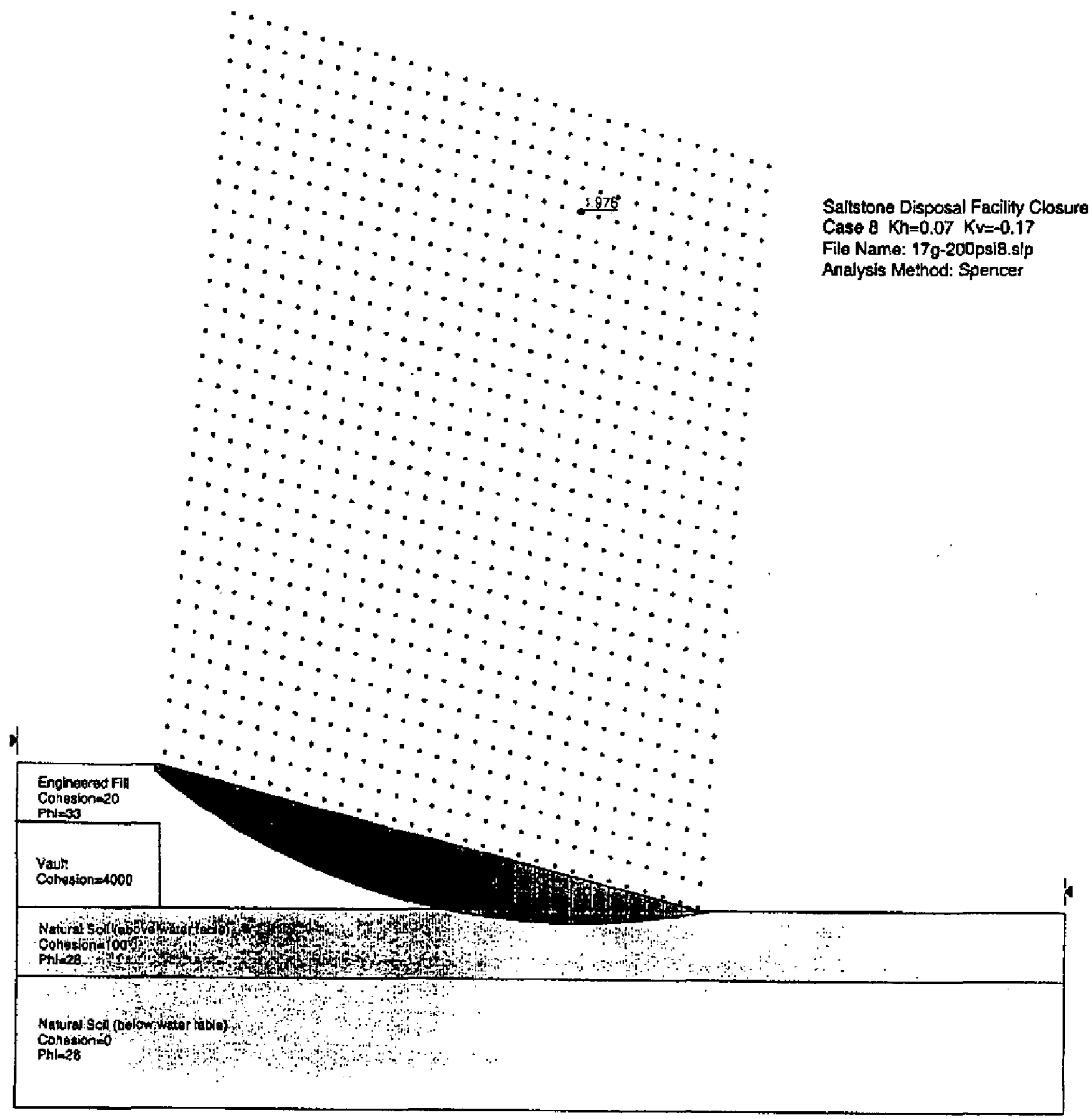


Figure 12. Safety Factor Calculated Using Slope/W for Case 8 (k_h=0.07 and k_v=-0.17)

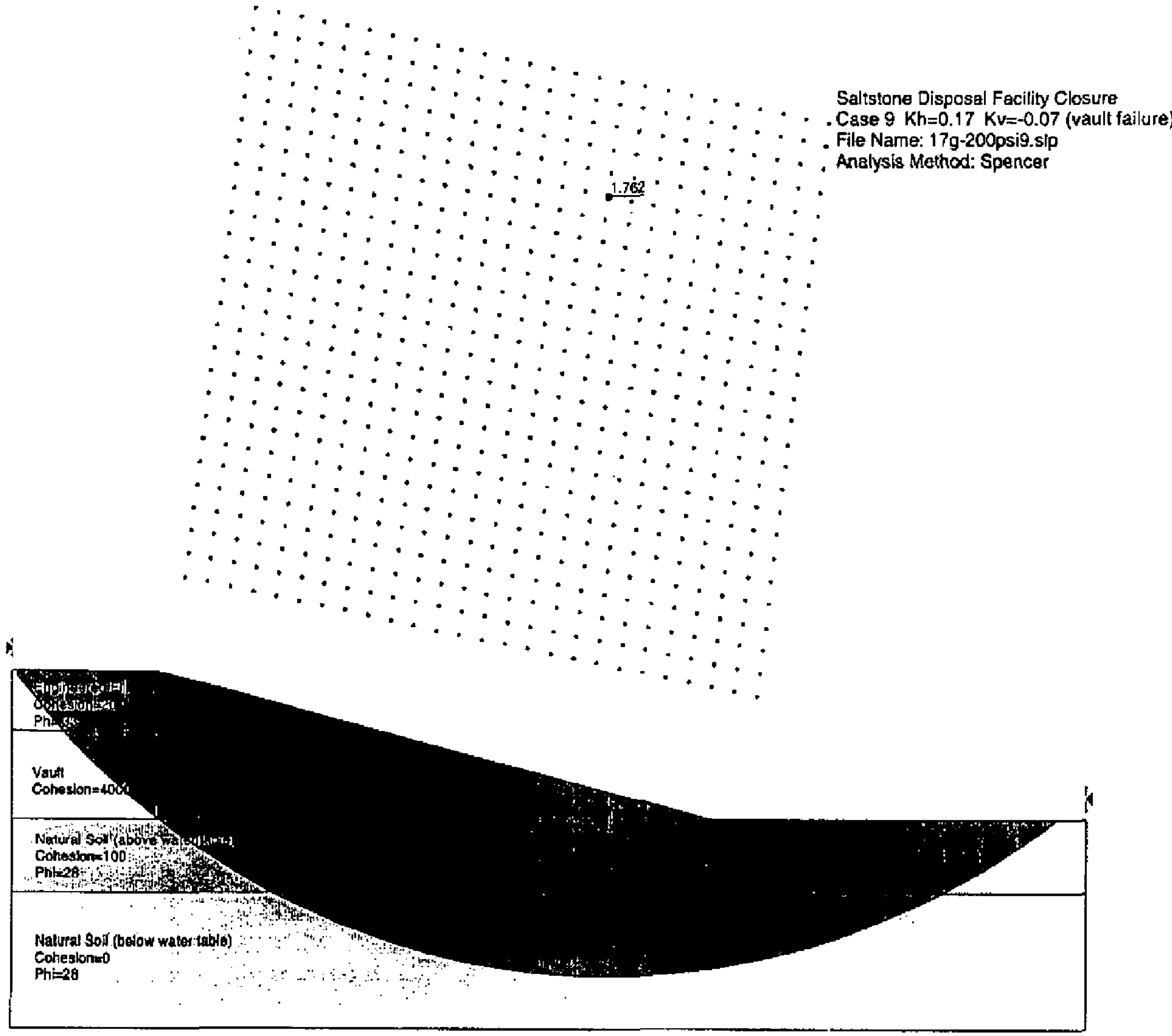


Figure 13. Safety Factor Calculated Using Slope/W for Case 9 (Vault Failure – corner k_h=0.17 and k_v=-0.07)

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Sheet No. 24
Rev. 0

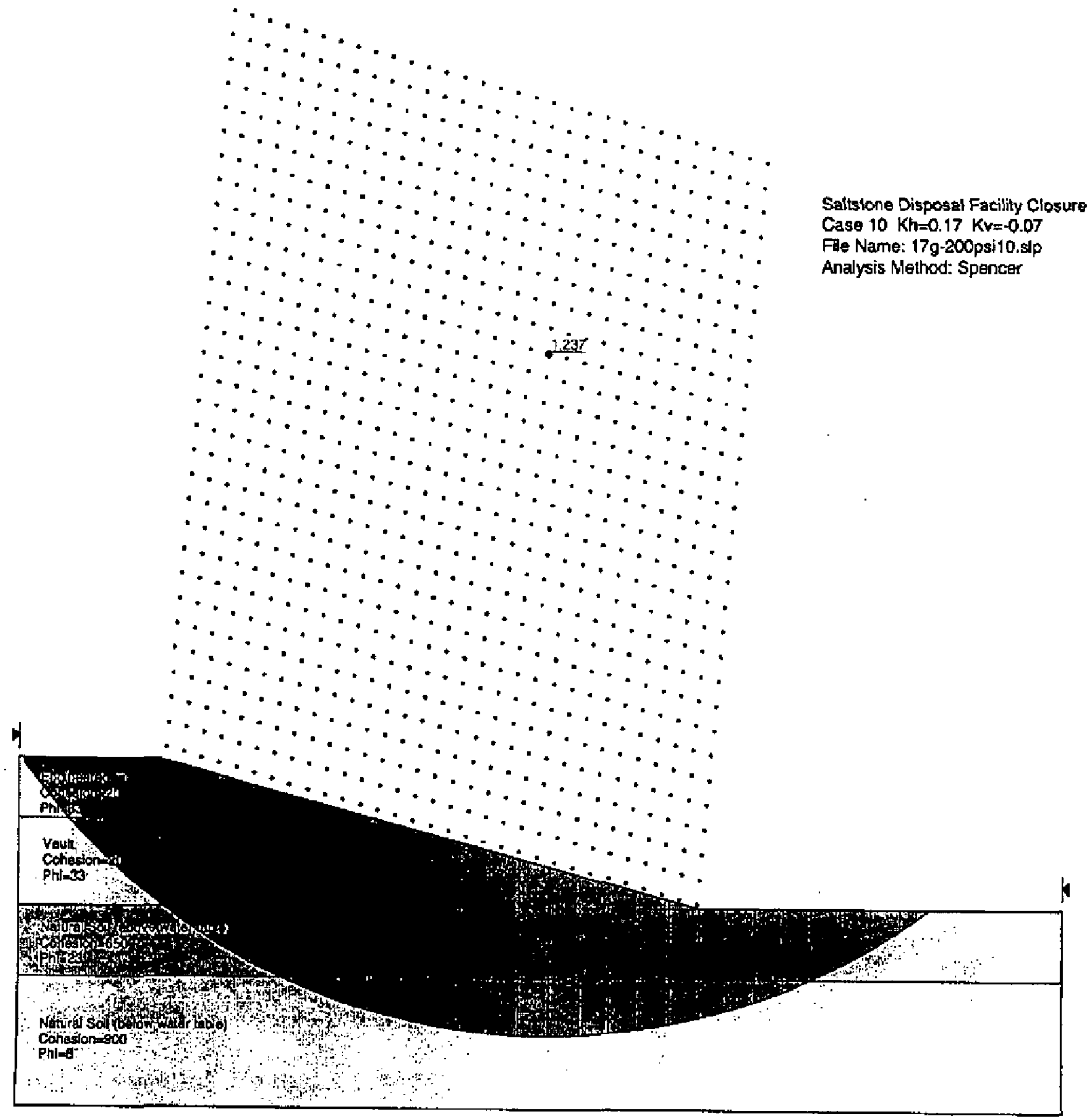

Figure 14. Safety Factor Calculated Using Slope/W for Case 10 (total stress k_h=0.17 and k_v=-0.07)

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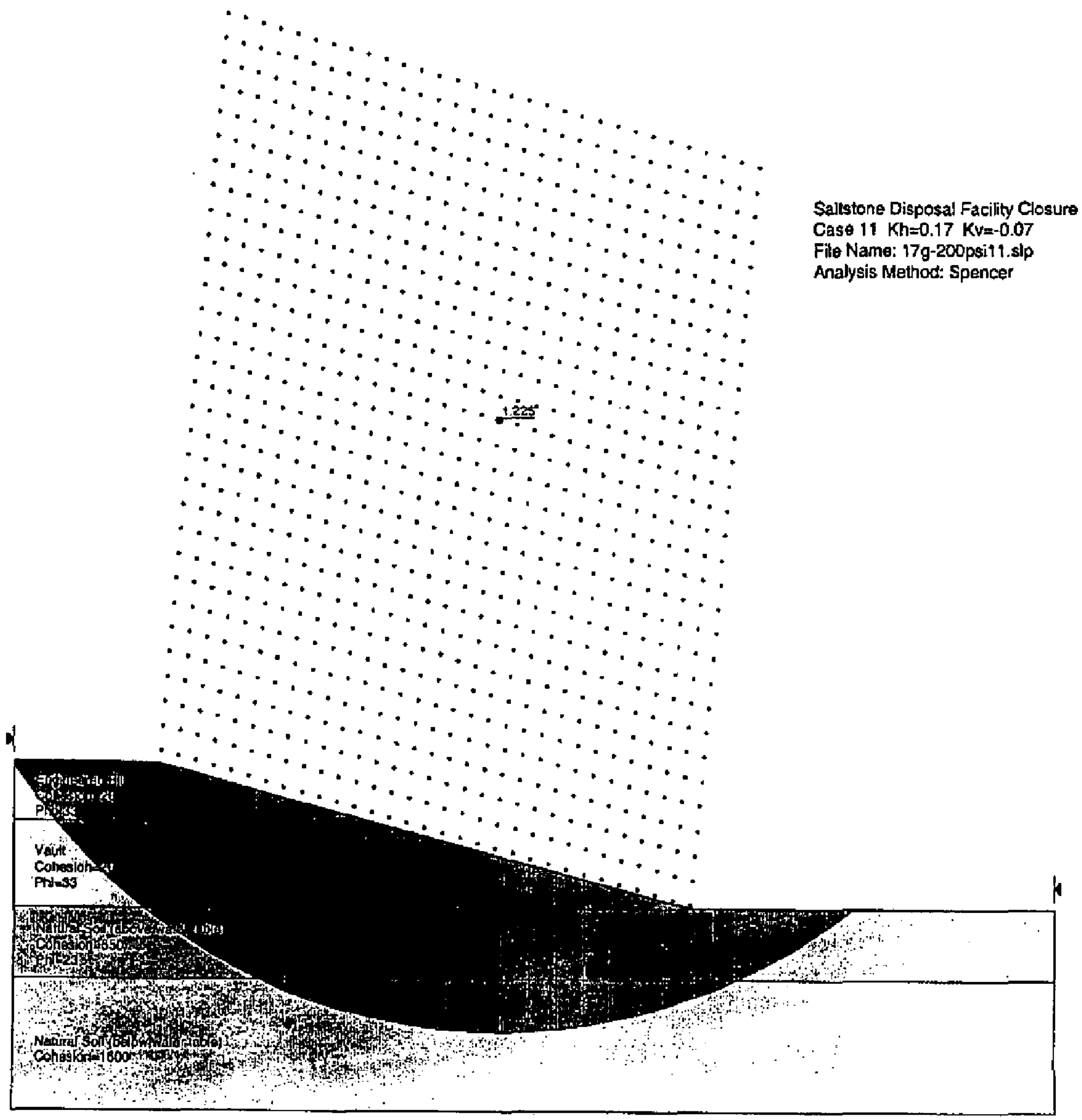

Figure 15. Safety Factor Calculated Using Slope/W for Case 11 (total stress k_h=0.17 and k_v=-0.07)

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Rev. 0

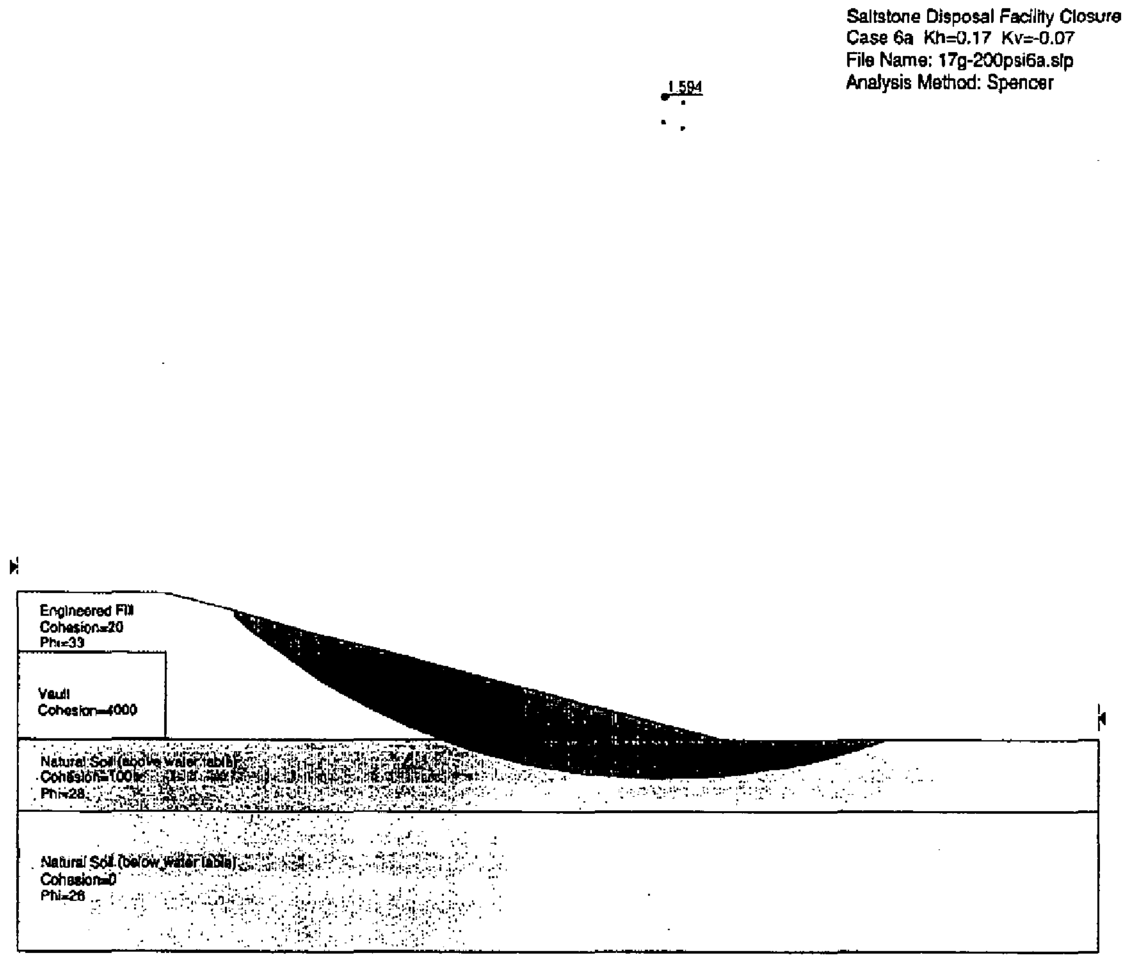

Figure 16. Safety Factor Calculated Using Slope/W for Case 6a (k_h=0.17 and k_v=-0.07)
(limited number of radius focal points and larger radii)

Calculation No. K-CLC-Z-00002
Sheet No. 27
Rev. 0

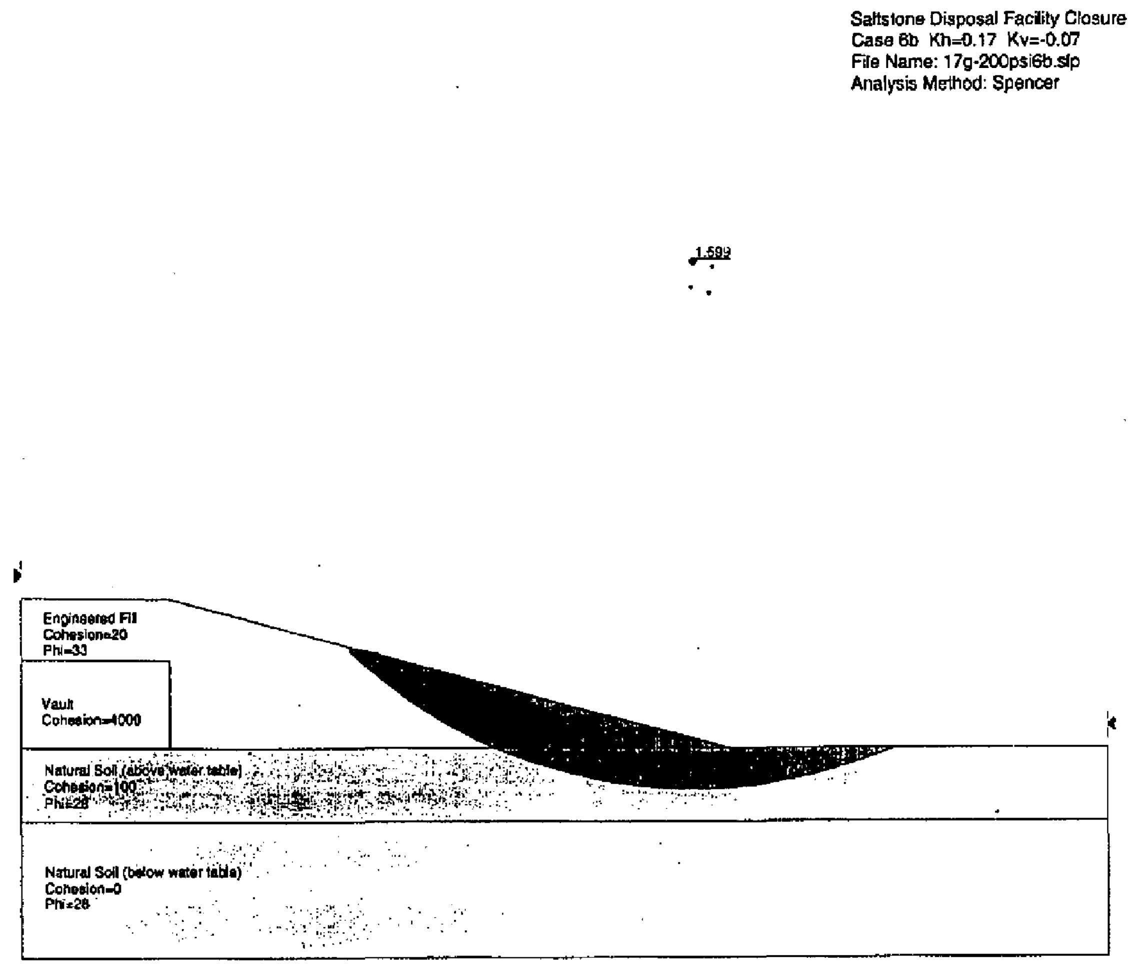

Figure 17. Safety Factor Calculated Using Slope/W for Case 6b (k_h=0.17 and k_v=-0.07)
(limited number of radius focal points and larger radii)

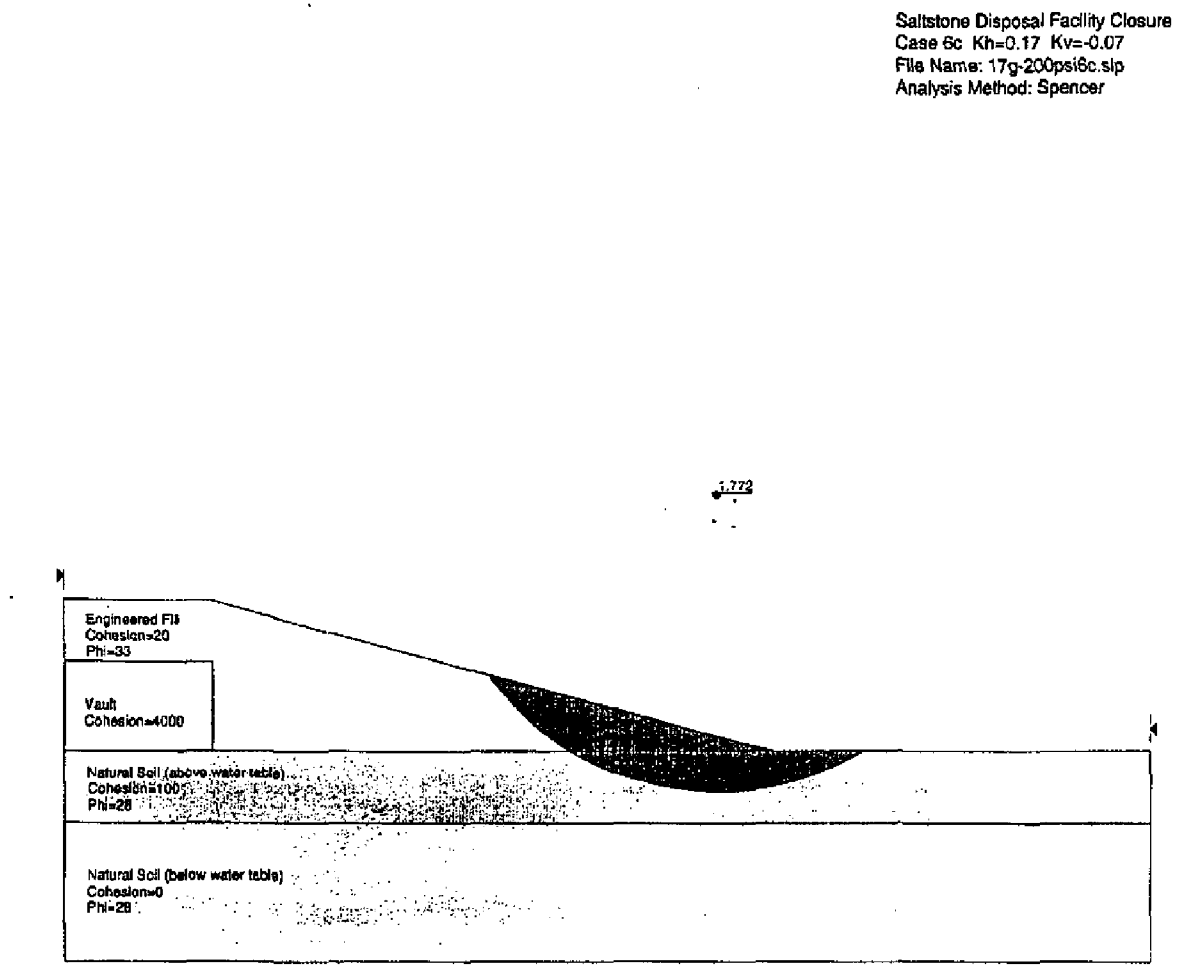

Figure 18. Safety Factor Calculated Using Slope/W for Case 6c (k_h=0.17 and k_v=-0.07)
(limited number of radius focal points and larger radii)

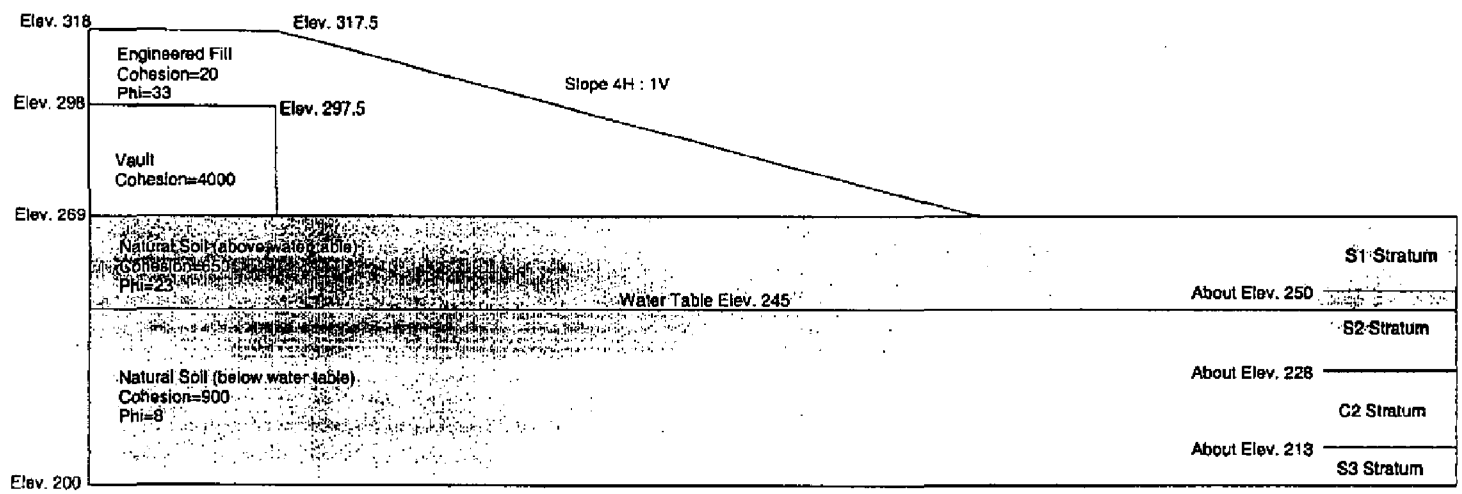

Figure 19. Vault and Closure Cover Soil Model Used for Revision 0 Case 10

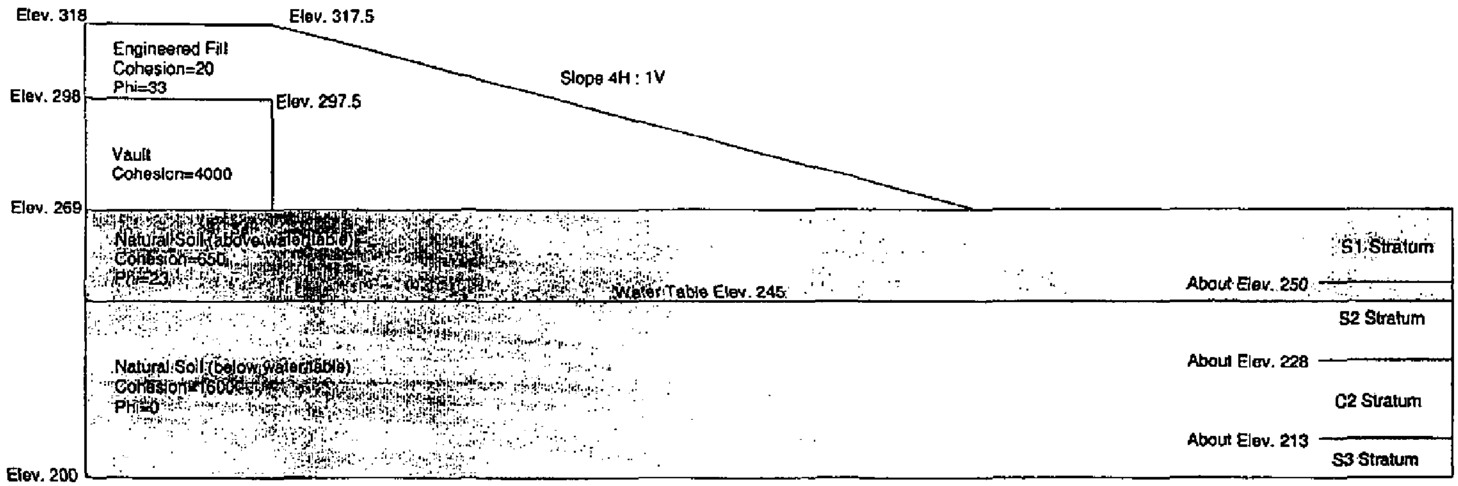

Figure 20. Vault and Closure Cover Soil Model Used for Revision 0 Case 11

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Rev. 1	

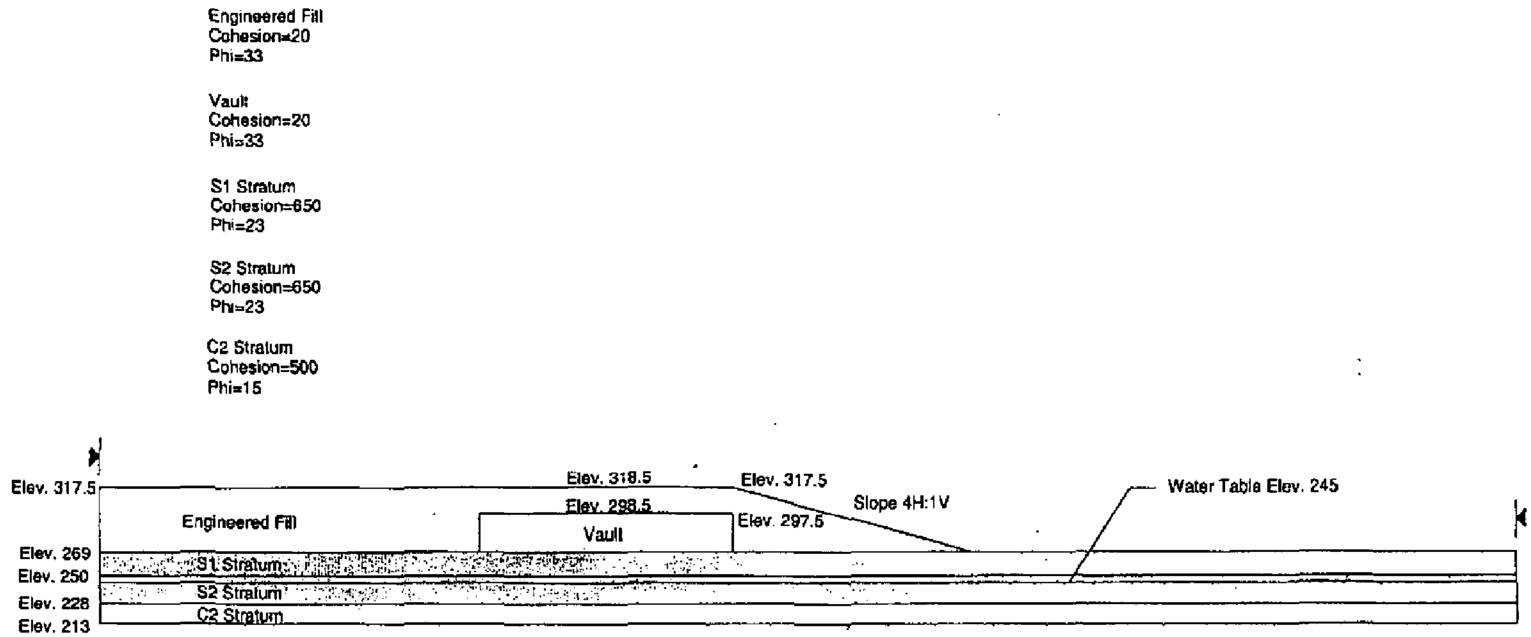

Figure 21. Vault and Closure Cover Soil Model Used for Case 10 Extended (Revision 1)

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Rev.	1

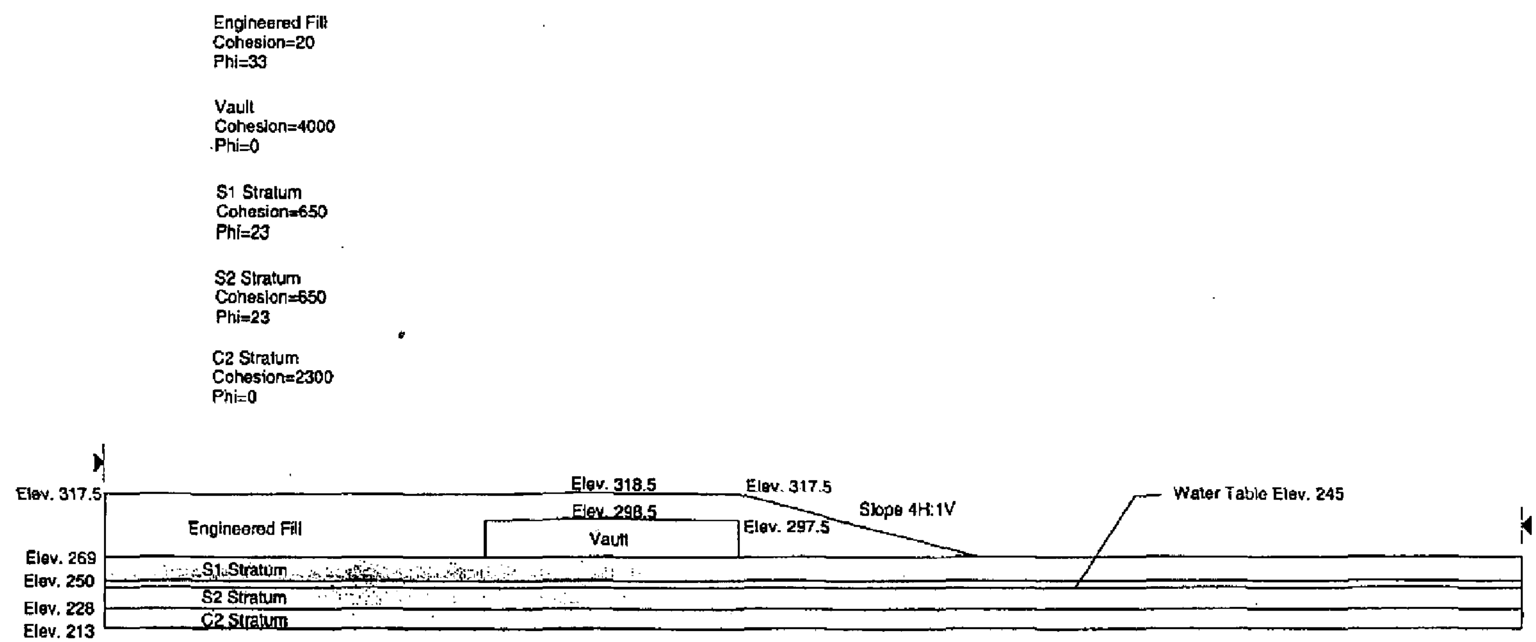


Figure 22. Vault and Closure Cover Soil Model Used for Case 11 Extended (Revision 1)

Calculation No.	Sheet No.	Rev.
K-CLC-Z-00002	28E	1

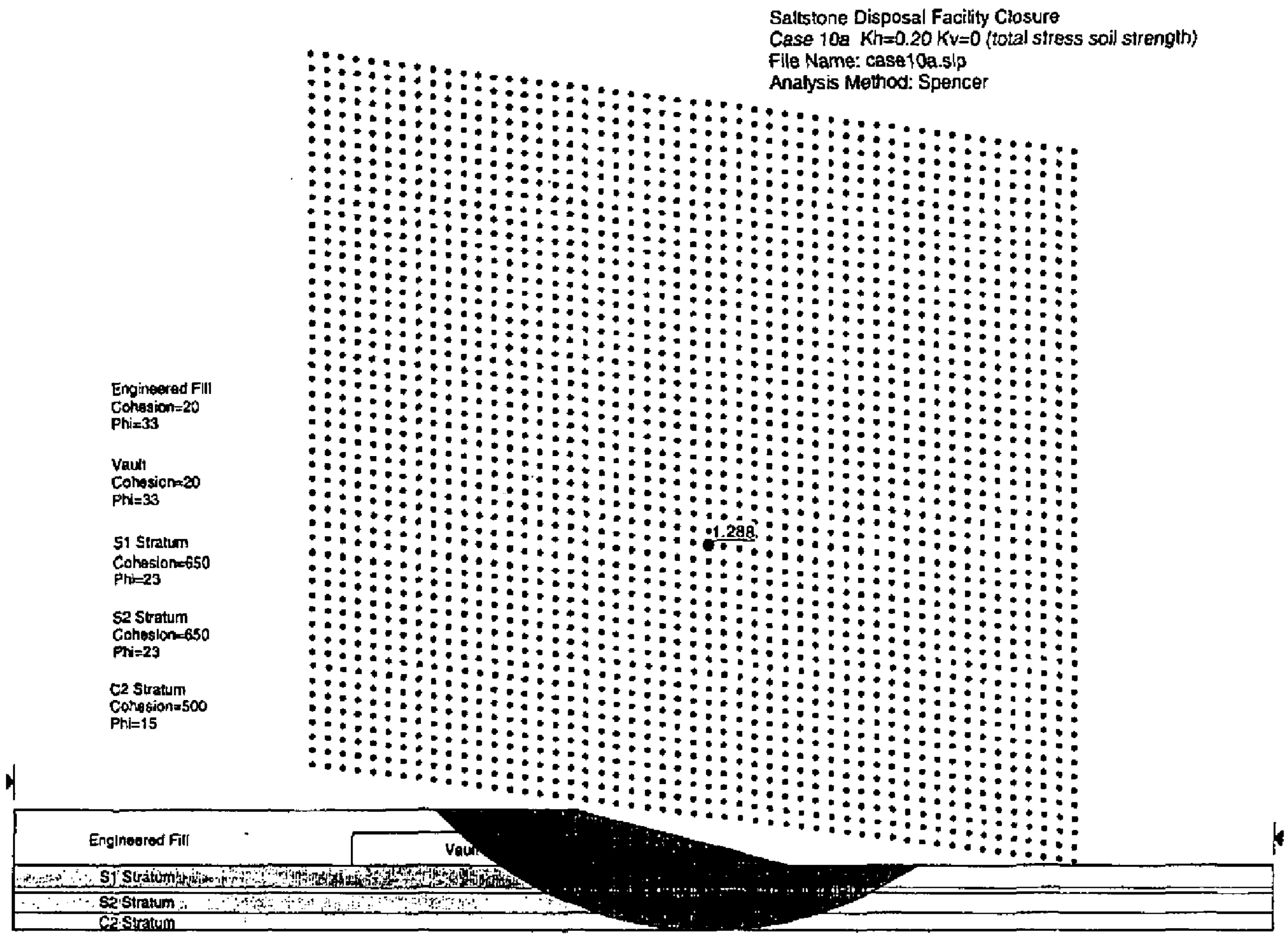


Figure 23. Safety Factor Calculated Using Slope/W for Case 10a Extended (total stress k_h=0.20 and k_v=0)

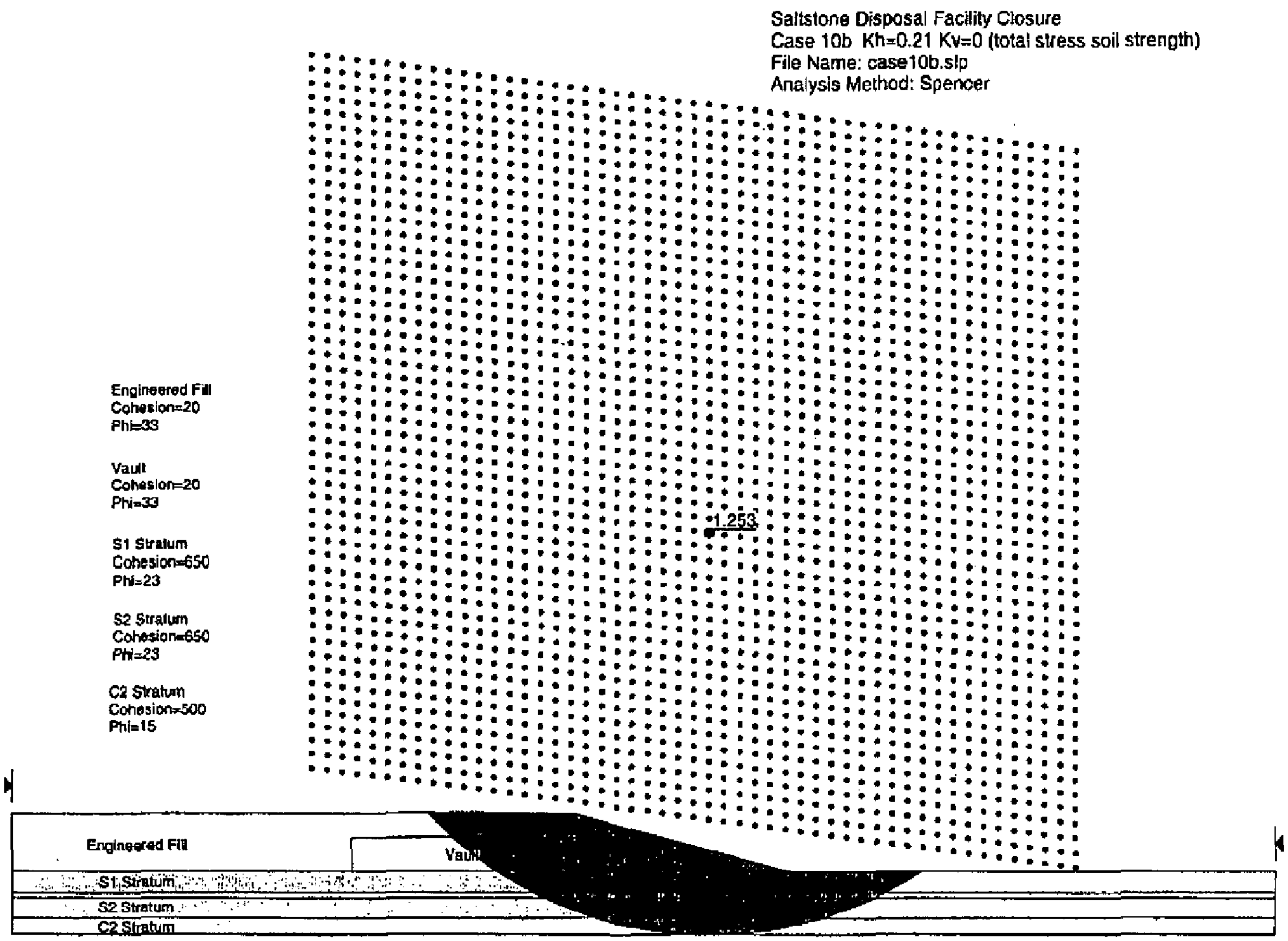


Figure 24. Safety Factor Calculated Using Slope/W for Case 10b Extended (total stress k_h=0.21 and k_v=0)

Calculation No.	Sheet No.	Rev.
K-CLC-Z-00002	28G	1

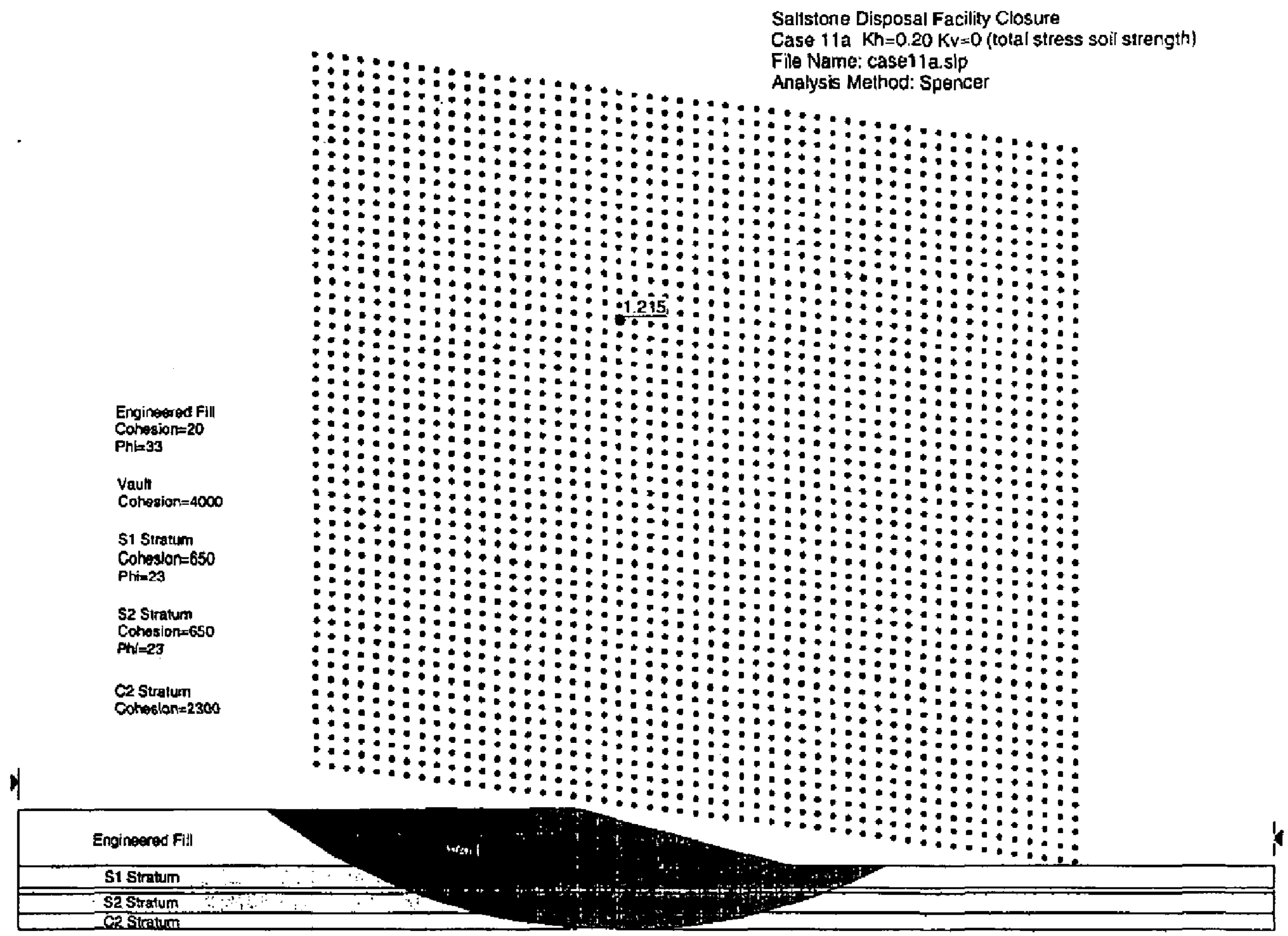


Figure 25. Safety Factor Calculated Using Slope/W for Case 11a Extended (total stress k_h=0.20 and k_v=0)

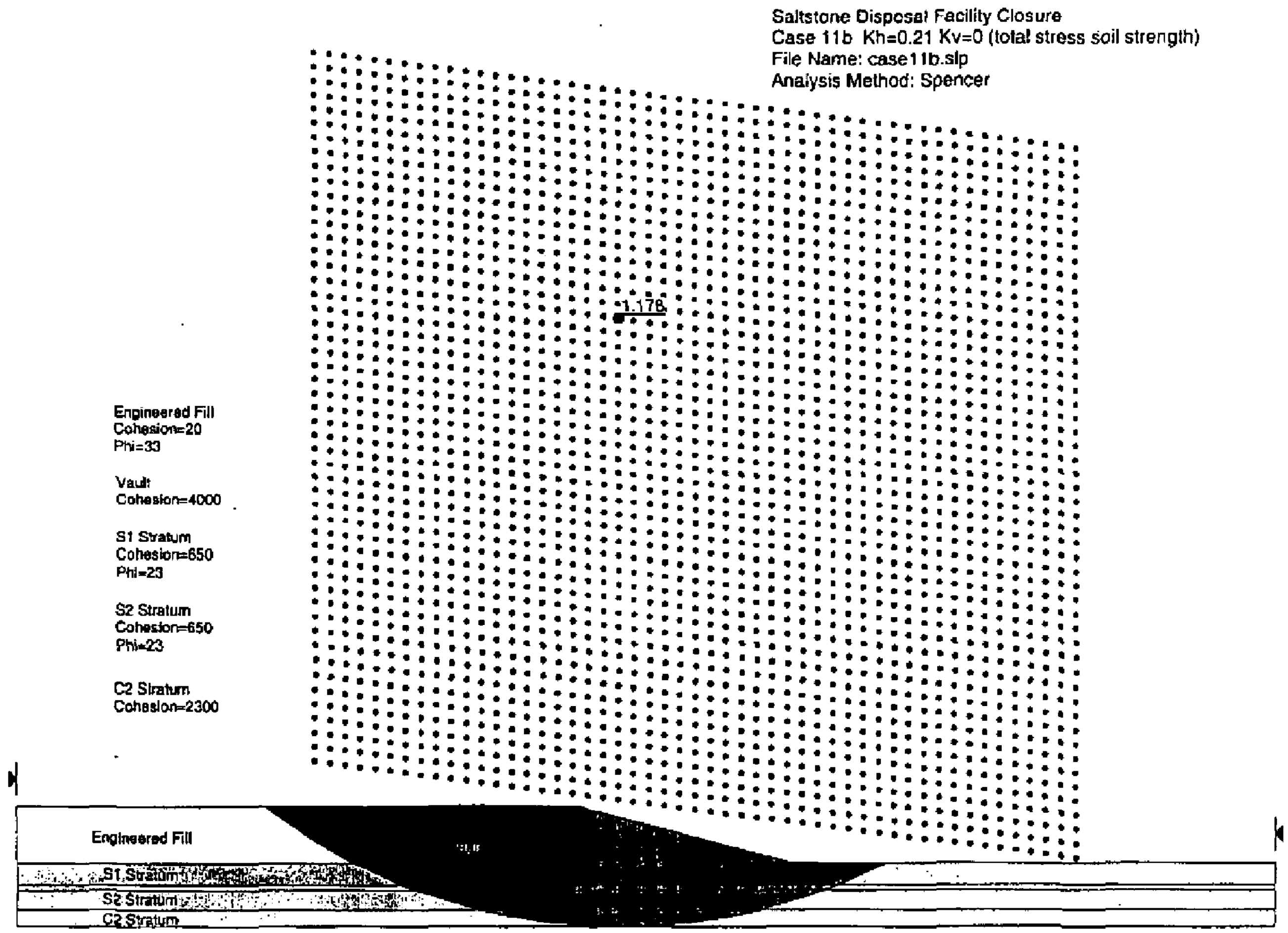
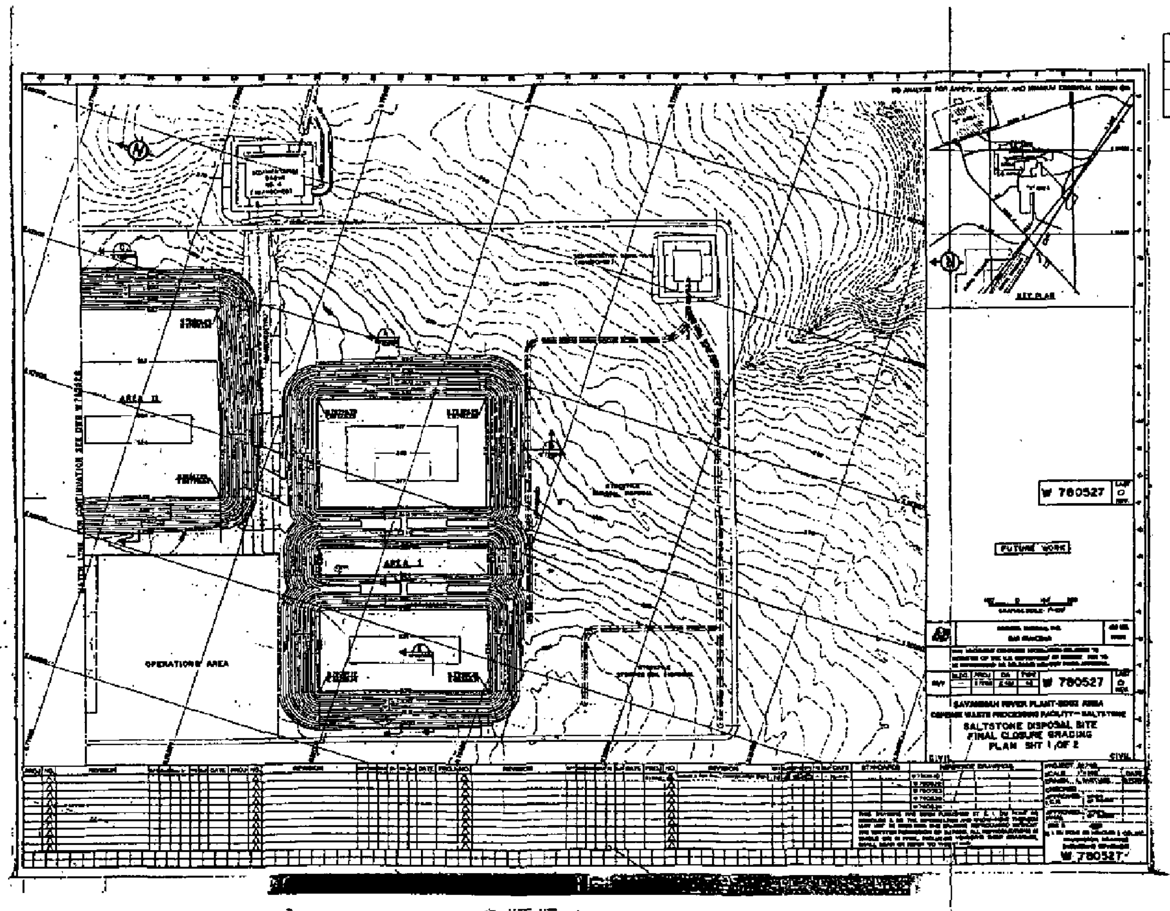


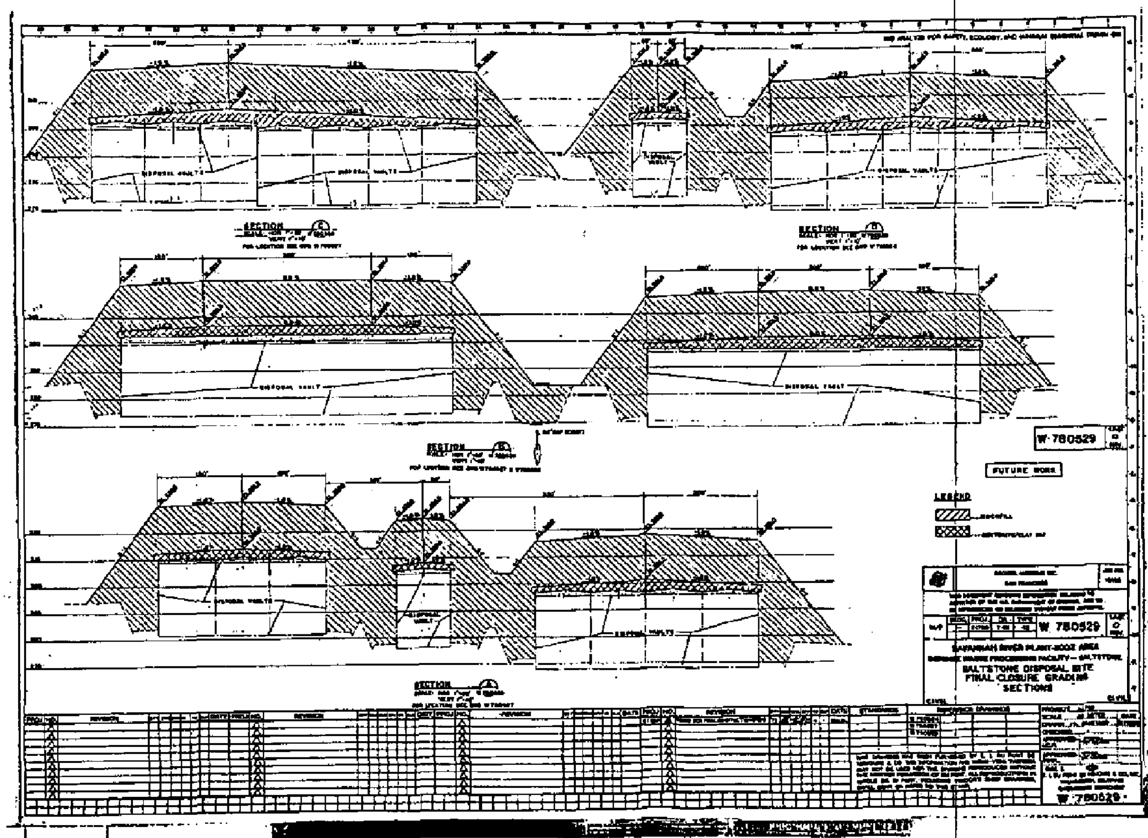
Figure 26. Safety Factor Calculated Using Slope/W for Case 11b Extended (total stress k_h=0.21 and k_v=0)

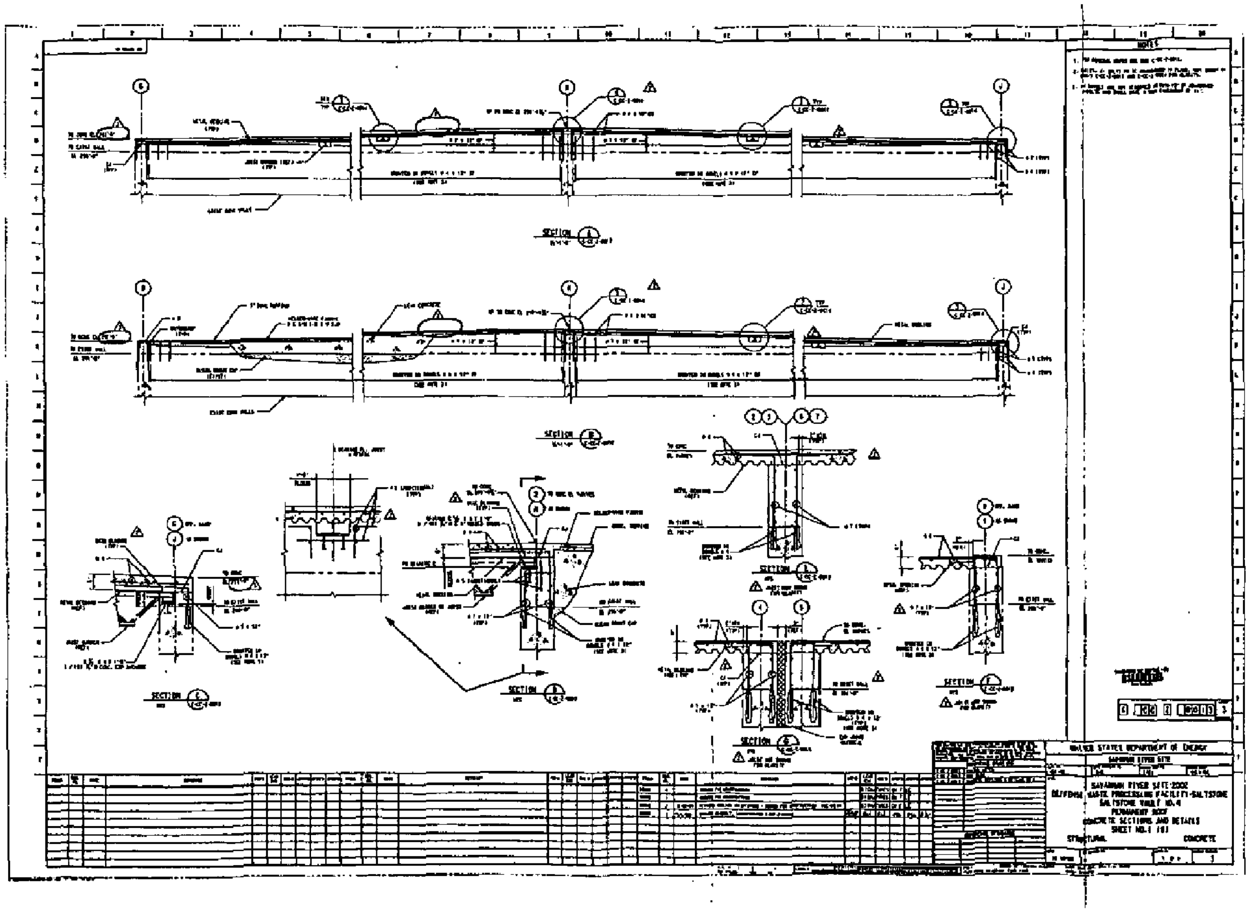
ATTACHMENT 1: DRAWINGS

Calculation No. K-CLC-Z-00002
Sheet No. 30
Rev. 0



Calculation No. K-CLC-Z-00002
Sheet No. 31
Rev. 0



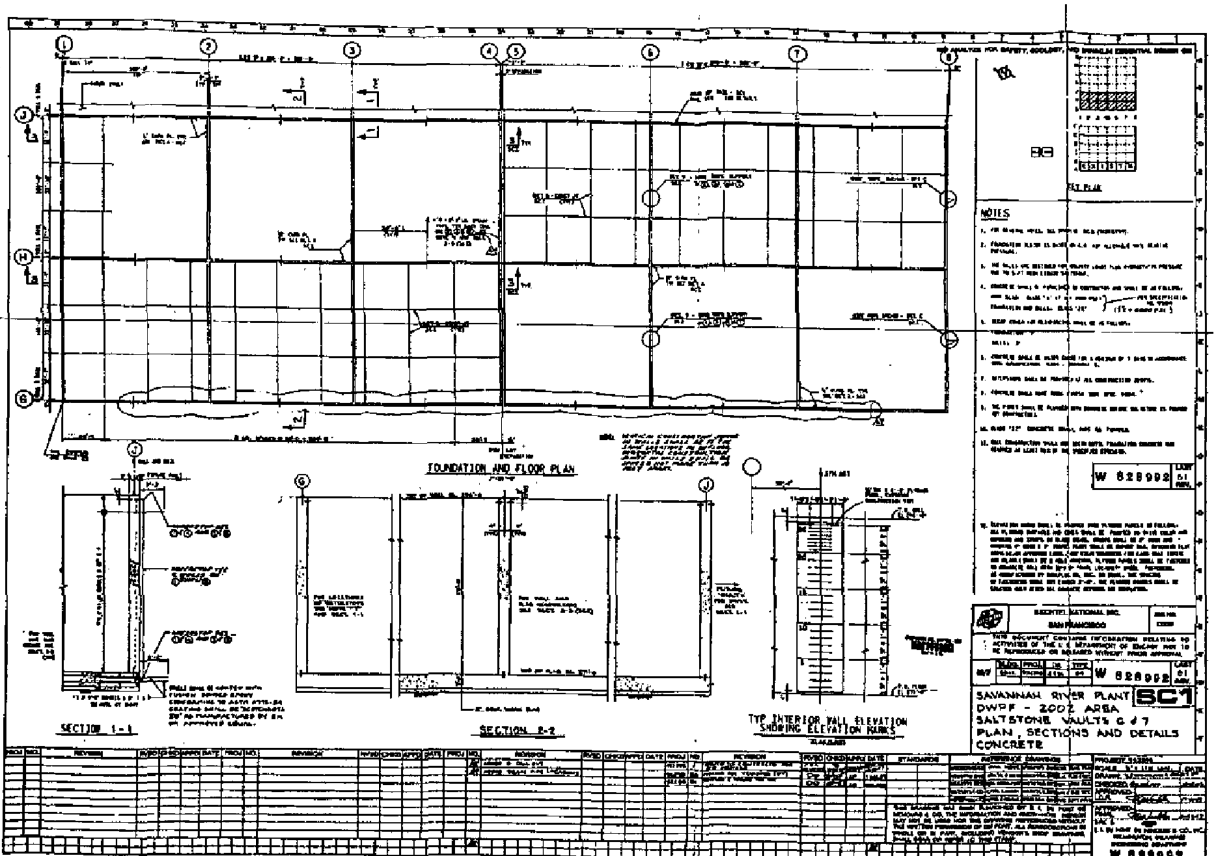


Calculation No.
K-CLC-Z-00402

Sheet No.
33

Rev.
0

Calculation No. K-CLC-Z-00002
Sheet No. 33
Rev. 0



ATTACHMENT 2: SOIL SHEAR STRENGTH DATA

This attachment contains effective and total stress shear strength test results performed in the vicinity of the Saltstone Vaults. More detail regarding the testing can be obtained from *Investigations of Slope Stability Savannah River Plant Aiken, South Carolina*, by Woodward-Clyde Consultants (1985), *Stability of Trenches 200 Z Area Saltstone Landfill Savannah River Plant Aiken, South Carolina*, by Woodward-Clyde Consultants (1986), *Savannah River Site Z-Area Vault No. 2 Geotechnical Investigations Report (U)*, by Bechtel Savannah River, Inc. (1992), and *Saltstone Disposal Z-Area Savannah River Plant*, by Mueser Rutledge Consulting Engineers (1986).

Figure 1, on sheet 11 of this calculation, shows approximate elevations for the various soil strata beneath Vault No. 4. Detailed descriptions of the strata are provided in, *Saltstone Disposal Z-Area Savannah River Plant*, by Mueser Rutledge Consulting Engineers (1986). The majority of the strength tests are above the water table in the S1 stratum (see sheet 35). The S2 and S3 strata have very limited data. The S2 stratum is believed to behave like the S1 stratum when above the water table. Cone Penetrometer and Standard Penetration tests indicate that the S2 stratum is as strong or stronger than the S1 stratum. Assuming that the S2 stratum behaves as the S1 stratum is conservative. Five tests are available for the C2 stratum below the water table. The S3 stratum is interspersed and underlies the C2 stratum and is stronger than the C2 stratum. Assuming that the S3 stratum behaves as the C2 stratum is conservative. Boring ID, type of test, test moisture, and stratum for the triaxial shear tests are provided on sheet 35.

Effective stress results show sandy behavior with little cohesion for both saturated and unsaturated tests. Total stress results from unsaturated samples also tend to behave as sandy soils with little cohesion. However, total stress results from saturated samples exhibit strength due to cohesion and behave somewhat like clayey soils.

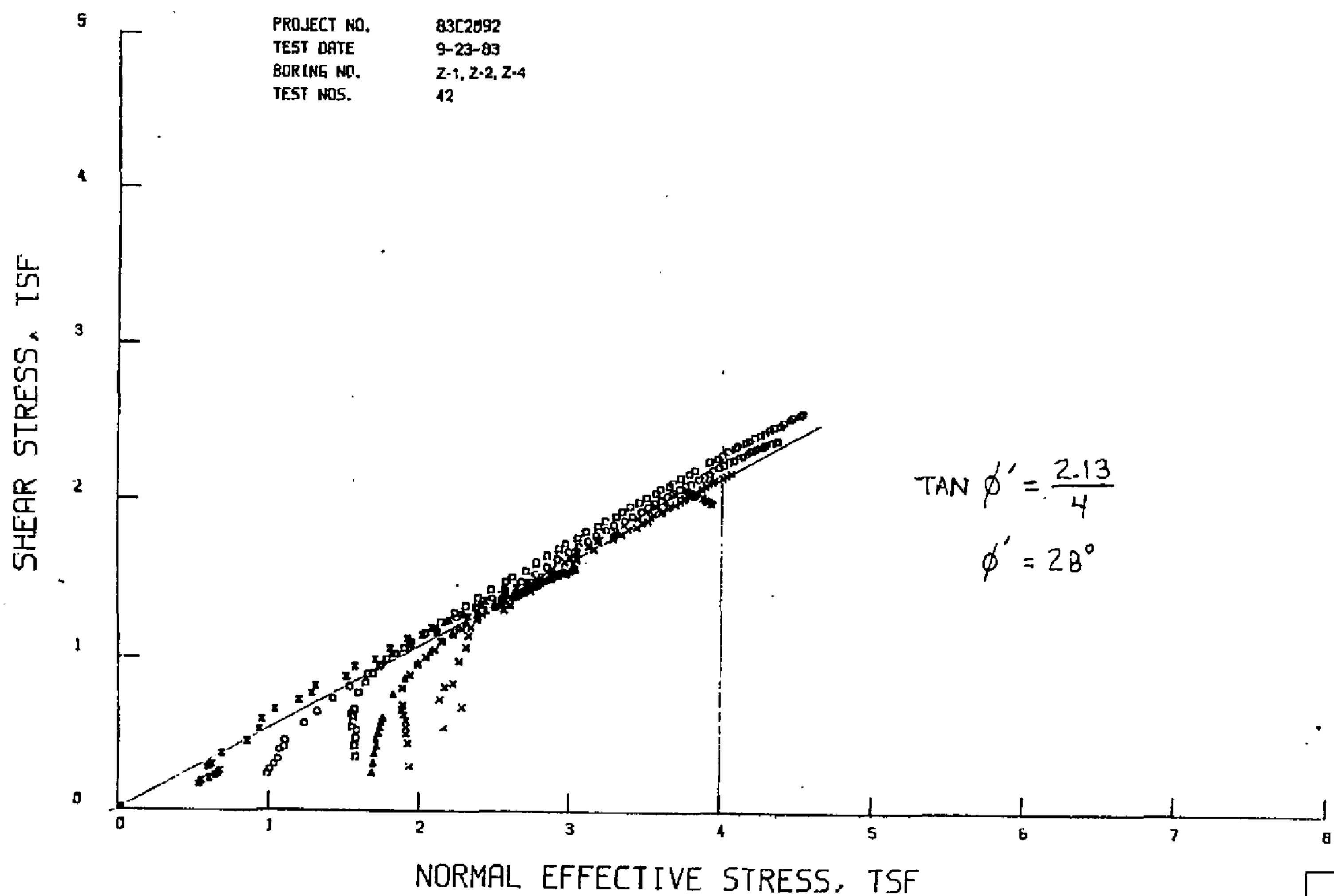
For "effective stress" cases the S1, S2, C2 and S3 soils above and below the water table were modeled as with a conservative friction angle 28° and cohesion of 100 psf above the water and no cohesion below the water table (see sheets 36 through 40 for supporting data).

For the "total stress" cases the S1 and S2 soils above the water table were modeled with a conservative friction angle of 23° with cohesion of 650 psf as interpreted by Woodward-Clyde Consultants (1986) (see sheet 41 for plot of data). The S2, C2 and S3 soils below the water table were modeled two ways to insure the most conservative interpretation. The soils below the water table were modeled with 1) a friction angle of 8° and cohesion of 900 psf and 2) a friction angle of zero and cohesion of 1,600 psf (see sheet 42 for plot of saturated total stress data).

Calculation No. K-CLC-Z-00002
Sheet No. 35
Rev. 0

SGS File No.	Boring ID	SRS Northing	SRS Easting	Type of Test	Test Moisture	Stratum	Mid. Elev. (ft-msl)	Sheet No.
Z-SDF-9	ZB-2	76156	66598	CU	saturated	S1	261.0	37, 38, 42
Z-SDF-9	ZB-8	75575	66689	CU	saturated	S1	270.8	39, 40, 42
G-SRS-17 and Z-SDF-8	Z-1	76200	66975	CIU	natural	S1	270.9	41
G-SRS-17 and Z-SDF-8	Z-1	76200	66975	CIU	saturated	S1	270.4	36
G-SRS-17 and Z-SDF-8	Z-1	76200	66975	CIU	natural	S1	260.9	41
G-SRS-17 and Z-SDF-8	Z-1	76200	66975	CIU	saturated	S1	260.4	36
G-SRS-17 and Z-SDF-8	Z-2	75710	67200	CIU	saturated	S1	271.2	36
G-SRS-17 and Z-SDF-8	Z-2	75710	67200	CIU	natural	S1	270.7	41
G-SRS-17 and Z-SDF-8	Z-2	75710	67200	CIU	saturated	S1	256.7	36
G-SRS-17 and Z-SDF-8	Z-2	75710	67200	CIU	natural	S1	256.2	41
G-SRS-17 and Z-SDF-8	Z-4	75520	66585	CIU	natural	S1	273.3	41
G-SRS-17 and Z-SDF-8	Z-4	75520	66585	CIU	saturated	S1	272.8	36
G-SRS-17 and Z-SDF-8	Z-4	75520	66585	CIU	natural	S1	263.25	41
G-SRS-17 and Z-SDF-8	Z-4	75520	66585	CIU	saturated	S1	262.8	36
G-SRS-17 and Z-SDF-8	Z-1	76200	66975	UU	natural	S1	250.6	41
G-SRS-17 and Z-SDF-8	Z-1	76200	66975	UU	natural	S2	240.9	41
G-SRS-17 and Z-SDF-8	Z-5	78560	67120	UU	natural	S2	240.3	41
G-SRS-17 and Z-SDF-8	Z-6	78150	65850	UU	natural	S1	260.1	41
G-SRS-17 and Z-SDF-8	Z-8	77100	67450	UU	natural	S1	259.6	41
Z-SDF-2	Z-211U	75741	66805	CU	natural	S3	192	42, 43, 44
Z-SDF-2	Z-218U	77297	67251	CU	natural	C2	214.9	42, 43, 44
Z-SDF-2	Z-224U	75847	67041	CU	natural	C2	221.2	42, 43, 44
Z-SDF-2	Z-224U	75847	67041	CU	natural	C2	217.7	42, 43, 44
Z-SDF-2	Z-225U	75939	67325	CU	natural	C2	216	42, 43, 44
Z-SDF-2	Z-225U	75939	67325	CU	natural	C2	216	42, 43, 44

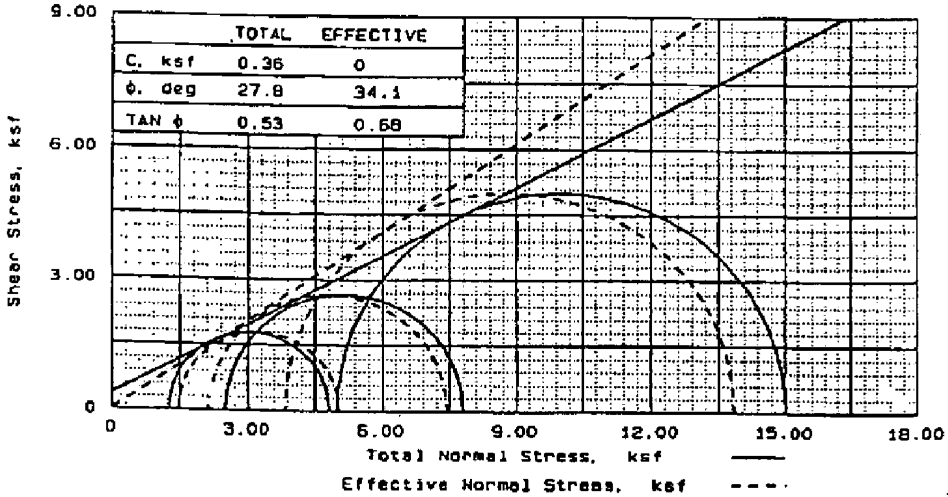
CU – Consolidated Undrained
CIU – Isotropically Consolidated Undrained
UU – Unconsolidated Undrained



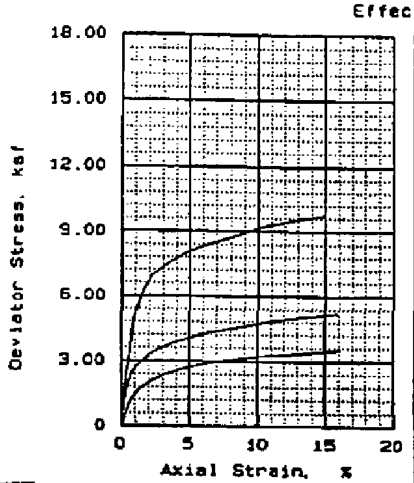
$$\tan \phi' = \frac{2.13}{4}$$

$$\phi' = 28^\circ$$

Calculation No. K-CLC-Z-00002
Sheet No. 37
Rev. 0



	TOTAL	EFFECTIVE
C, ksf	0.36	0
ϕ, deg	27.8	34.1
TAN ϕ	0.53	0.68



SAMPLE NO.		1	2	3	
INITIAL	WATER CONTENT, %	14.5	12.8	12.9	[illegible]
	DRY DENSITY, pcf	109.3	107.3	105.8	[illegible]
	SATURATION, %	72.2	60.4	58.6	[illegible]
	VOID RATIO	0.543	0.571	0.593	[illegible]
	DIAMETER, in	2.87	2.87	2.86	
	HEIGHT, in	5.60	5.60	5.60	
AT TEST	WATER CONTENT, %	17.4	19.8	21.2	
	DRY DENSITY, pcf	114.7	109.8	107.2	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.470	0.536	0.572	
	DIAMETER, in	2.81	2.84	2.85	
	HEIGHT, in	5.54	5.56	5.56	
BACK PRESSURE, ksf		2.75	2.68	2.00	
CELL PRESSURE, ksf		7.75	5.18	3.25	
FAILURE STRESS, ksf		10.01	5.30	3.56	
PORE PRESSURE, ksf		3.90	3.04	1.79	
STRAIN RATE, %/min.		0.100	0.100	0.100	
ULTIMATE STRESS, ksf					
PORE PRESSURE, ksf					
$\bar{\sigma}_1$ FAILURE, ksf		13.86	7.44	5.02	
$\bar{\sigma}_3$ FAILURE, ksf		3.85	2.14	1.47	

TYPE OF TEST:
CU with pore pressures
SAMPLE TYPE: UNDISTURBED
DESCRIPTION: RED BROWN & PURPLE CLAYEY SILTY MED. TO FN. SAND
LL= PL= PI=
SPECIFIC GRAVITY= 2.70
REMARKS: TESTED BY

REVIEWED BY

FIG. NO. 1

CLIENT: BECHTEL SAVANNAH RIVER, INC.

PROJECT: Z-AREA TASK ORDER 4

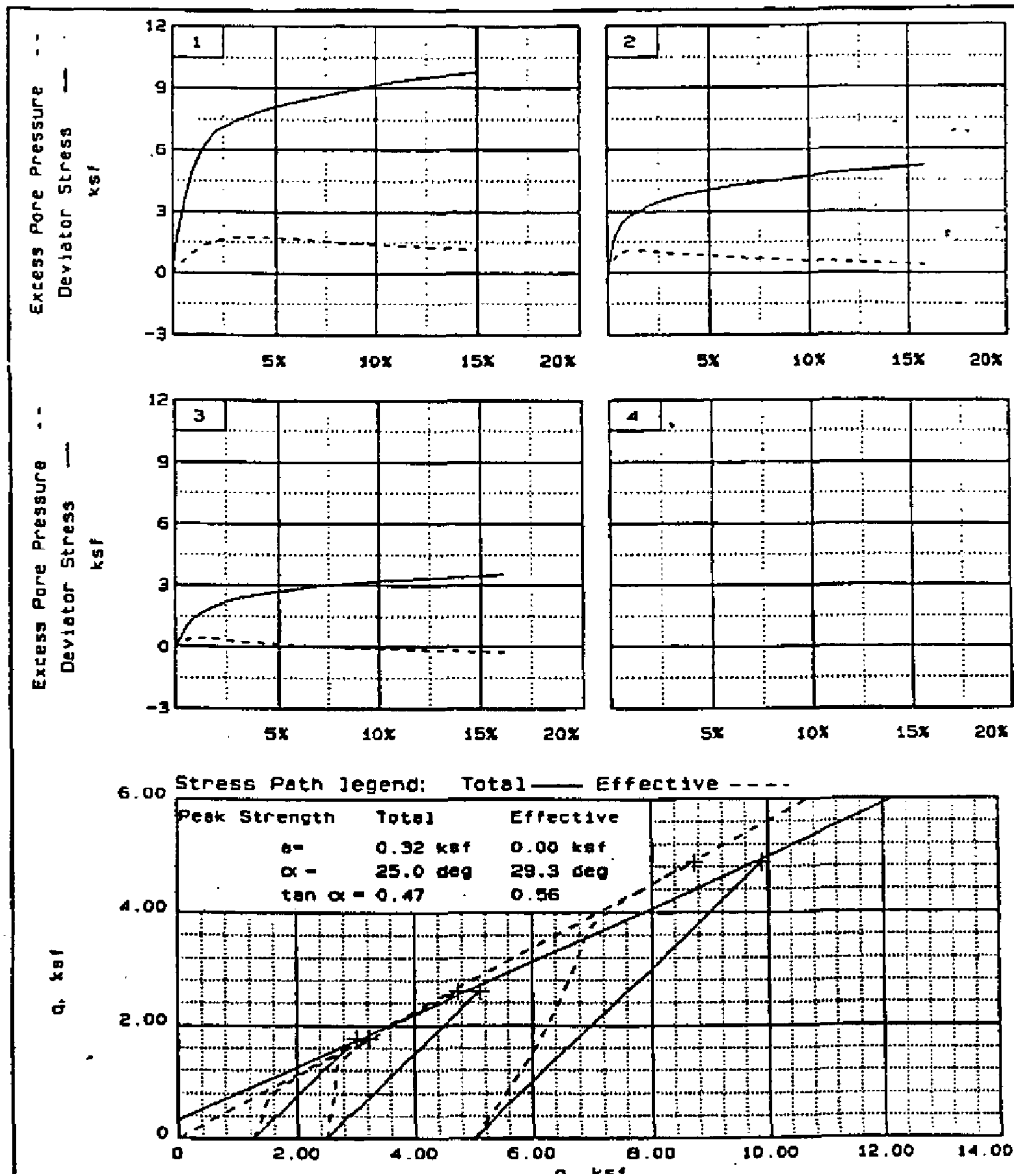
SAMPLE LOCATION: B-2 UD @ 20-22 FT.

PROJ. NO.: 1451010594 DATE: JUNE 12, 1992

TRIAXIAL COMPRESSION TEST

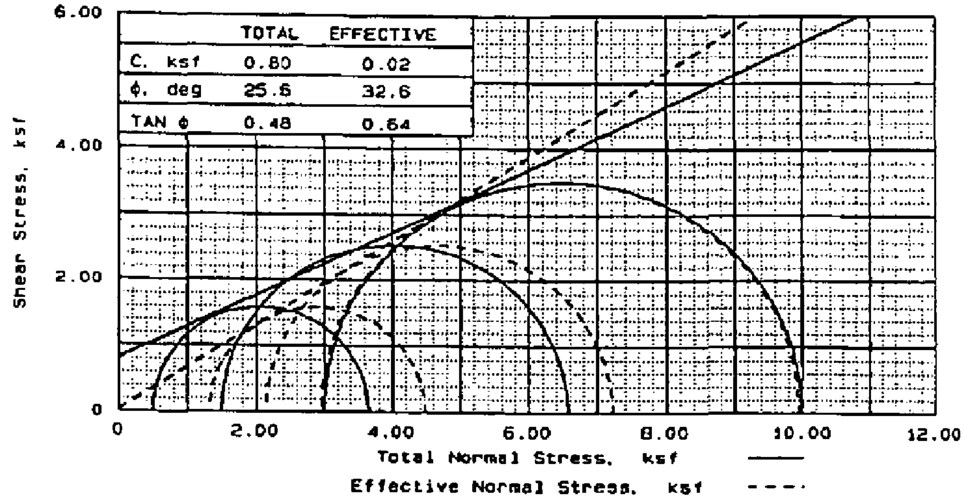
LAW ENGINEERING, INC.

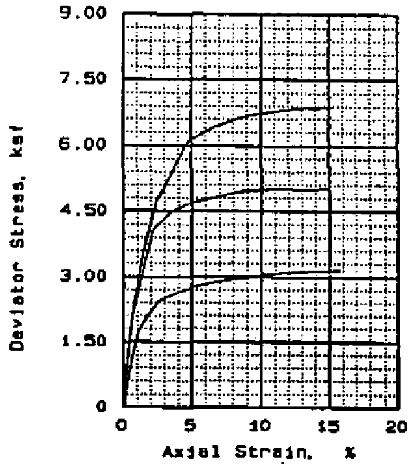
Calculation No. K-CLC-Z-00002
Sheet No. 38
Rev. 0



Client: BECHTEL SAVANNAH RIVER, INC.
Project: Z-AREA TASK ORDER 4
Location: B-2 UD @ 20-22 FT.
File: 61010594 Project No.: 1461010594 Page 2/2 Fig. No. 1

Calculation No.	Sheet No.	Rev.
K-CLC-Z-00002	39	0





SAMPLE NO.		1	2	3
INITIAL	WATER CONTENT, %	15.7	16.6	15.0
	DRY DENSITY, pcf	109.5	109.5	119.5
	SATURATION, %	78.5	83.2	98.9
	VOID RATIO	0.539	0.540	0.410
	DIAMETER, in	2.86	2.86	2.87
	HEIGHT, in	5.60	5.60	5.60
AT TEST	WATER CONTENT, %	19.5	19.2	13.8
	DRY DENSITY, pcf	110.4	110.9	122.8
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.527	0.519	0.373
	DIAMETER, in	2.85	2.85	2.84
	HEIGHT, in	5.59	5.58	5.56
BACK PRESSURE, ksf		2.85	2.82	2.81
CELL PRESSURE, ksf		3.35	4.32	5.81
FAILURE STRESS, ksf		3.16	5.07	6.99
PORE PRESSURE, ksf		2.03	2.17	2.85
STRAIN RATE, %/min.		0.100	0.100	0.100
ULTIMATE STRESS, ksf				
PORE PRESSURE, ksf				
$\bar{\sigma}_1$ FAILURE, ksf		4.48	7.22	9.94
$\bar{\sigma}_3$ FAILURE, ksf		1.32	2.15	2.96

TYPE OF TEST:
CU with pore pressures
SAMPLE TYPE: UNDISTURBED
DESCRIPTION: PURPLE BROWN
CLAYEY SILTY MED. TO FN SAND
LL= PL= PI=
SPECIFIC GRAVITY= 2.70
REMARKS: TESTED BY

REVIEWED BY

FIG. NO. 2

CLIENT: BECHTEL SAVANNAH RIVER, INC.

PROJECT: Z-AREA TASK ORDER 4

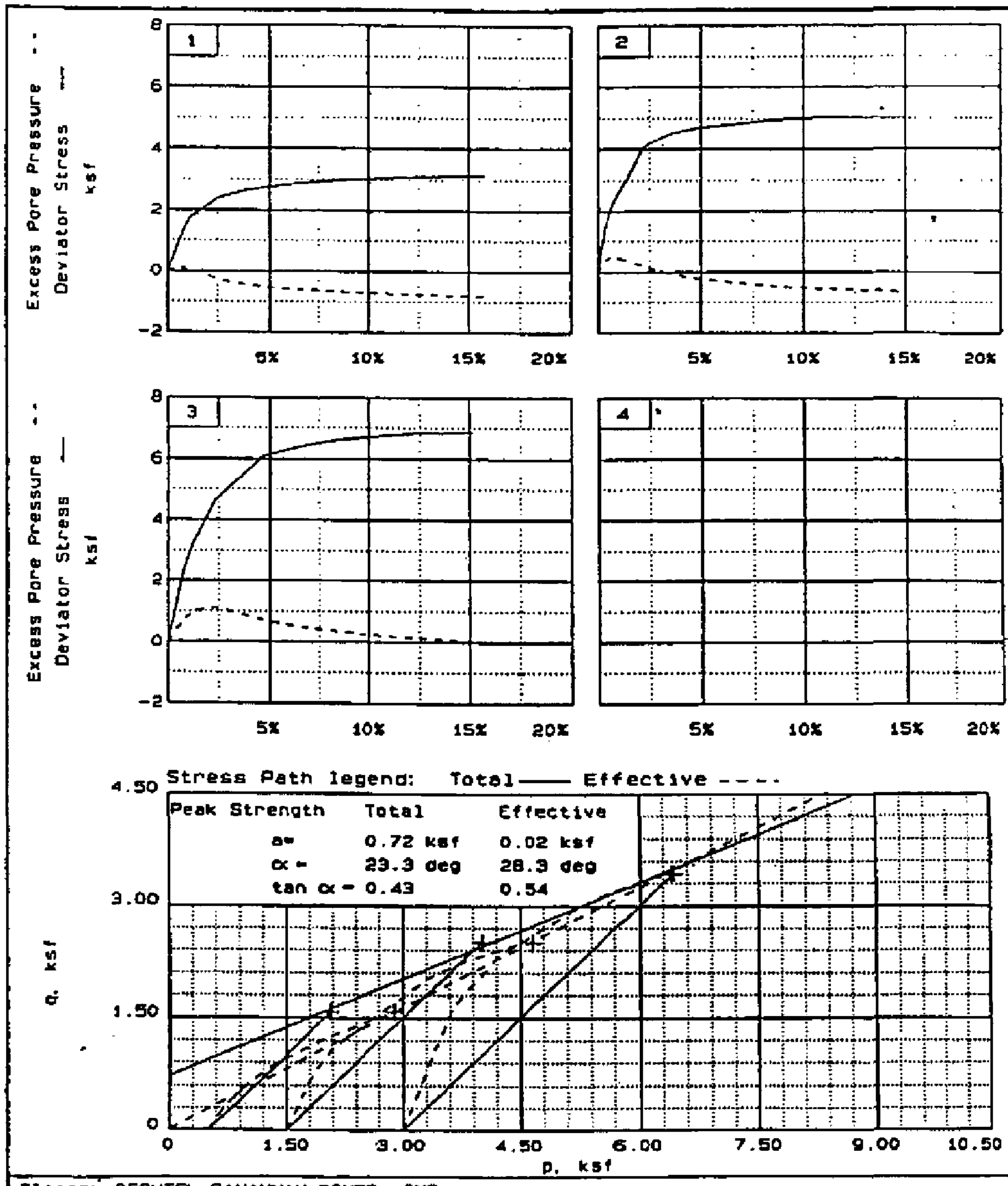
SAMPLE LOCATION: B-8 UD @ 10-12.5 FT.

PROJ. NO.: 1461010594 DATE: JUNE 12, 1992

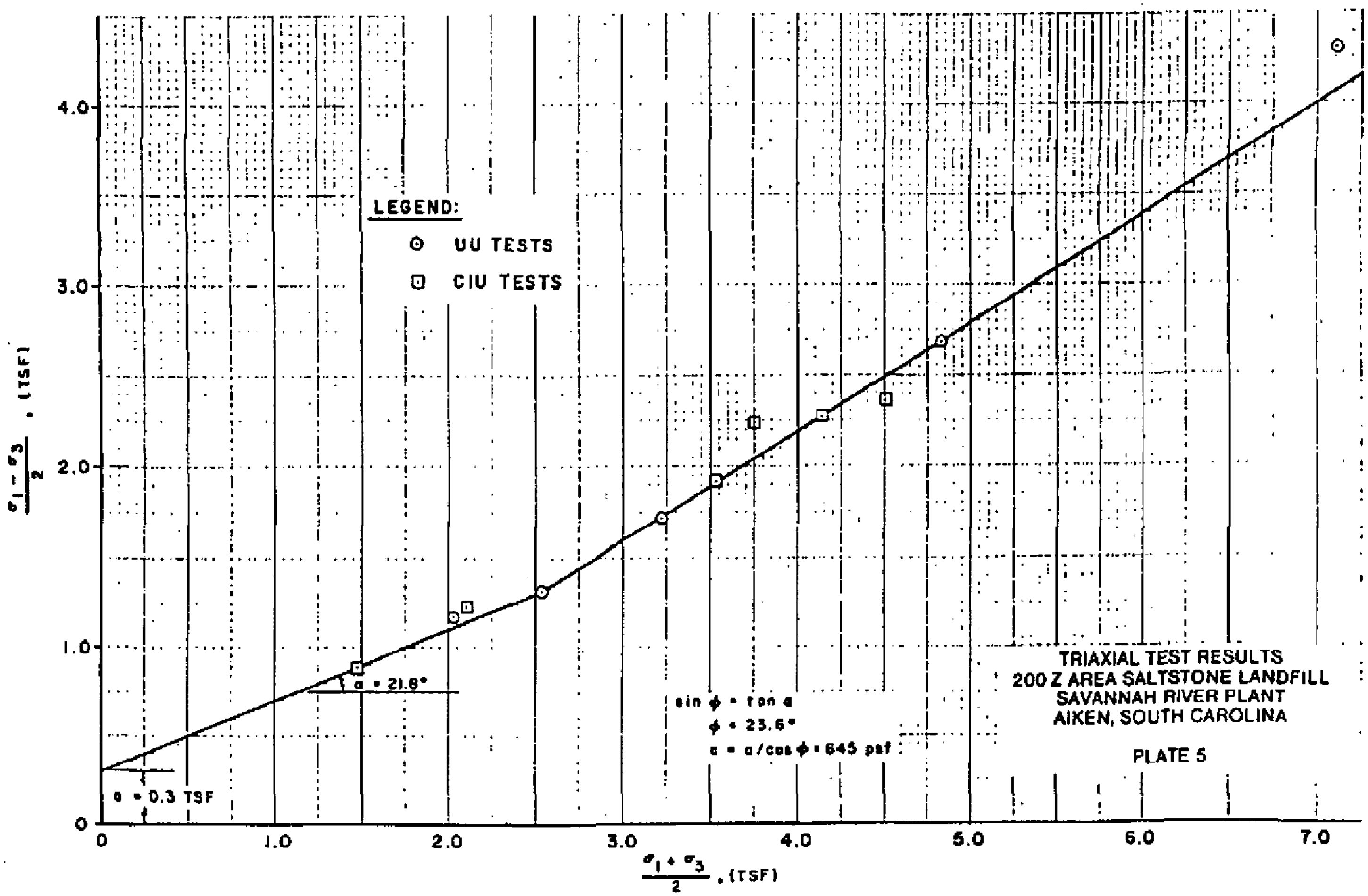
TRIAXIAL COMPRESSION TEST

LAW ENGINEERING, INC.

Calculation No. K-CLC-Z-00002
Sheet No. 40
Rev. 0

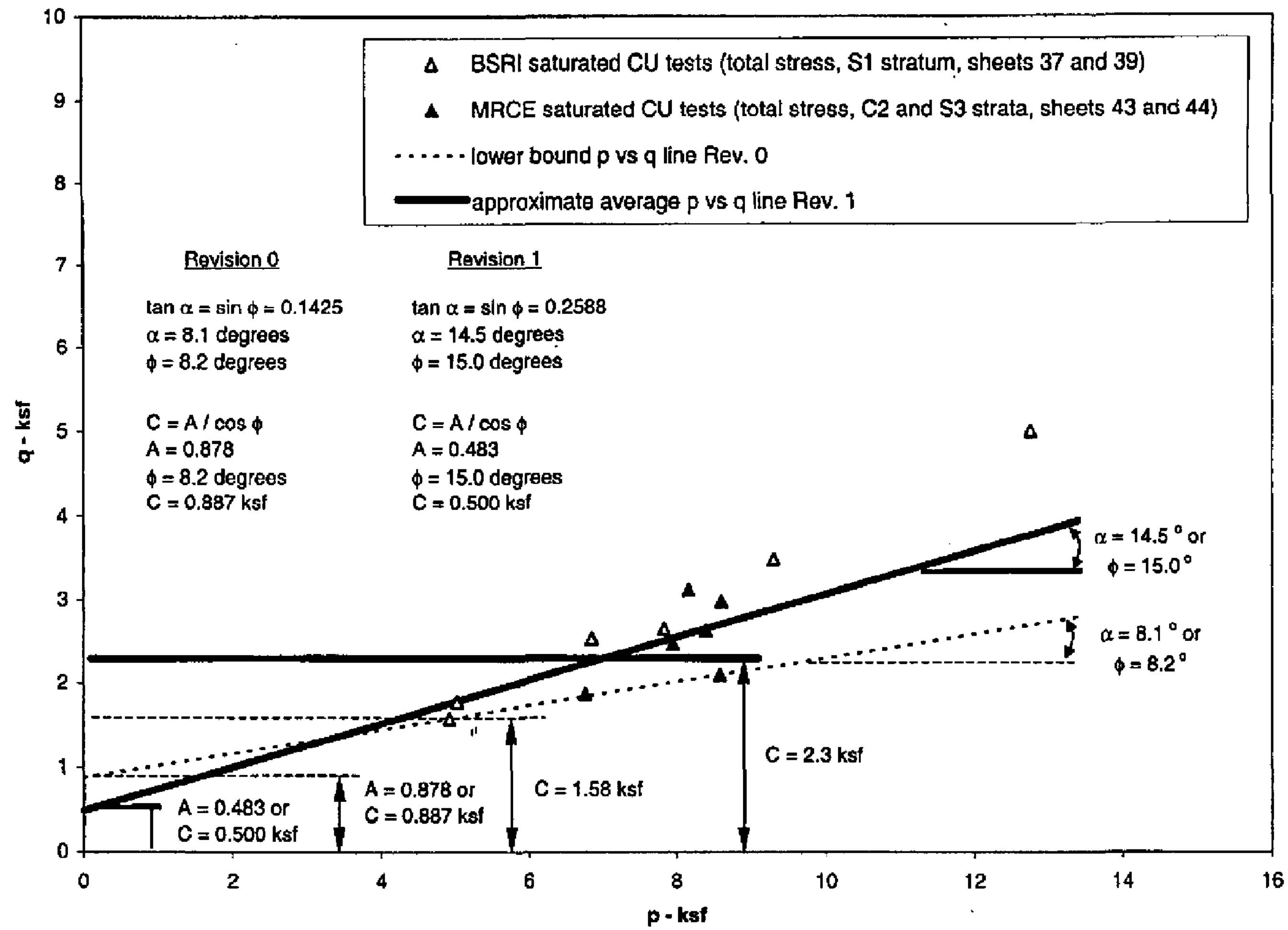


Client: BECHTEL SAVANNAH RIVER, INC.
Project: Z-AREA TASK ORDER 4
Location: B-8 UD @ 10-12.5 FT.
File: 1010594A Project No.: 1461010594 Page 2/2 Fig. No. 2



Calculation No. K-CLC-Z-00002	
Sheet No. 41	
Rev. 0	

p vs q plot for Saturated Total Stress Tests



Calculation No. K-CLC-Z-00002
Sheet No. 43
Rev. 0

TABLE NO. 1

SUMMARY OF LABORATORY TEST DATA

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES												
										UNIFIED SOIL CLASSIFICATION SYSTEM			STRENGTH						CONSOLIDATION						
BORING NUMBER	SAMPLE NUMBER	ELEVATION, FT.	STRATUM DESIGNATION	SOIL TYPE	AVERAGE NATURAL WATER CONTENT, w_n, %	LIQUID LIMIT, w_L, %	PLASTICITY INDEX, I_p, %	NATURAL WATER CONTENT OF LIMIT SAMPLE, w_n, %	SPECIFIC GRAVITY OF SOLIDS, G_s	GROUP SYMBOL	% SAND (<#4 >#200 SIEVE)	% FINES (<#200 SIEVE)	TYPE OF TEST	COMPRESSIVE STRENGTH $(\sigma_1-\sigma_3)$, TSF	CONFINING PRESSURE σ_3, TSF	STRAIN AT FAILURE, %	NATURAL WATER CONTENT, w_n, %	WATER CONTENT AT END OF TEST, w_f, %	NATURAL WATER CONTENT, w_n, %	INITIAL VOID RATIO, e_o	EXISTING OVERBURDEN STRESS, P_o, TSF	ESTIMATED PRECONSOLIDATION STRESS, P_c, TSF	COMPRESSION INDEX, C_c	SWELLING INDEX, C_s	VOID RATIO AT START OF SWELL, e_r
Z-201U	18S	220.3	C2		31	42	17	25		SC	73	27													
										SC	82	18													
PZ-202U	1UD	263.8	S1		19	49	26	19		SC&CL	69	31													
	2UD	258.8	S1		16	44	21	17		SC	68	32													
	3UD	253.8	S1		16	42	15	16		SC	82	18													
PZ-210U	1UD	270.6	S1		17	45	24	18		SC	72	28													
	2UD	265.6	S1		17	35	12	17		SC	77	23													
	3U	258.6	S1		15	49	26	15		SC	74	26													
Z-211U	25S	192.0	S3a		53	60	36	47	2.77	SC			CU	2.10	3.24	3.0	52.9	46.9	56.3	1.616	4.8	2.7	0.600	0.060	0.909
PZ-216U	1U	275.0	S1		13	51	24	14		SC	79	21													
	2U	270.5	S1		21	31	7	21		SC	68	32													
	3UD	265.2	S1		17	38	14	17		SC	55	45													
Z-218U	18U	215.1	C2		49	136	35	100		MH		98	CU	2.63	2.88	3.3	65.5	87.6							
			S3a		46					SC	68	32													
Z-219U	19U	210.0	C2		30	62	43	20	2.75	SC&CH									34.3	1.017	4.6	5.3	0.429	0.063	0.516
					30					SC	67	33													
	22U	211.5	C2		33	68	47	30		SC	87	13													
	23S	209.5	S3a		28	66	44	20		SC	88	12													
Z-220U	17U	219.3	S3b		27					SP-SC	91	9													
	18U	217.8	S3b		30					SP-SC	90	10													
						127	98	104		CH															

SOIL DESCRIPTION

S1 RED-BROWN AND GRAY CLAYEY FINE TO MEDIUM SAND; TO FINE TO MEDIUM SAND, SOME CLAY OCCASIONALLY INTERLAYERED WITH SANDY CLAY.

C2 YELLOW-BROWN AND LIGHT GREEN CLAYEY FINE SAND:TO FINE TO MEDIUM SAND, SOME CLAY, INTERLAYERED WITH STIFF YELLOW-BROWN SILTY CLAY, TRACE LIGNITE, OCCASIONAL LAYERS OF LIGHT GRAY-GREEN CALCAREOUS FINE TO MEDIUM SAND, SOME CLAY, CLAY LAYERS, TRACE SHELLS.

S3a LIGHT BROWN TO GRAY FINE TO MEDIUM SAND, SOME CLAY, TRACE LIGNITE, OCCASIONAL SANDY CLAY LAYERS.

S3b LIGHT BROWN, AND YELLOW-BROWN FINE TO MEDIUM SAND, TRACE CLAY.

NOTES

1. All tests summarized were performed in the soils laboratory of Mueser Rutledge Consulting Engineers.
2. The sample elevation is the average of the sampling interval.
3. GROUND SURFACE ELEVATIONS AT BORINGS ARE:

Boring No.	Elevation Ft.	Boring No.	Elevation Ft.
Z-201U	279.3	Z-218U	278.9
PZ-202U	278.3	Z-219U	290.5
PZ-210U	288.1	Z-220U	288.3
Z-211U	289.5	Z-224U	281.2
PZ-216U	294.5	Z-225U	277.0

4. "Average natural water content" is a weighted average of all material types recovered.
5. Compression tests performed were: CU - Consolidated Undrained Triaxial Compression.
6. Strength tests were performed on samples approximately 2.8 inches in diameter with a height-to-diameter ratio of 2. .
7. Compression Index, C_c = the slope of the virgin curve (straight line portion of the consolidation test e-log p plot).

$$e_2 = e_1 - C_c \times \log (P_2/P_1)$$

8. Swelling Index, C_s = the slope of the rebound curve of the consolidation test...

$$e_2 = e_1 + C_s \times \log (P_1/P_2)$$

MUESER RUTLEDGE CONSULTING ENGINEERS

708 THIRD AVENUE, NEW YORK, N.Y. 10017

SALTSTONE DISPOSAL
SAVANNAH RIVER PLANT - Z AREA
E.I. DUPONT DE NEMOURS & CO., INC.

DATE: 7-14-86 FILE NO. 5178 SHEET 1 OF 2 TABLE NO. 1

Calculation No. K-CLC-Z-00002
Sheet No. 44
Rev. 0

TABLE NO. 1

SUMMARY OF LABORATORY TEST DATA

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES												
										UNIFIED SOIL CLASSIFICATION SYSTEM			STRENGTH						CONSOLIDATION						
BORING NUMBER	SAMPLE NUMBER	ELEVATION, FT.	STRATUM DESIGNATION	SOIL TYPE	AVERAGE NATURAL WATER CONTENT, w_n, %	LIQUID LIMIT, w_L, %	PLASTICITY INDEX, I_p, %	NATURAL WATER CONTENT OF LIMIT SAMPLE, w_n, %	SPECIFIC GRAVITY OF SOLIDS, G_s	GROUP SYMBOL	% SAND (<#4 >#200 SIEVE)	% FINES (< #200 SIEVE)	TYPE OF TEST	COMPRESSIVE STRENGTH $(\sigma_1-\sigma_3)$, TSF	CONFINING PRESSURE σ_3, TSF	STRAIN AT FAILURE, %	NATURAL WATER CONTENT, w_n, %	WATER CONTENT AT END OF TEST, w_f, %	NATURAL WATER CONTENT, w_n, %	INITIAL VOID RATIO, e_0	EXISTING OVERBURDEN STRESS, P_o, TSF	ESTIMATED PRECONSOLIDATION STRESS, P_c, TSF	COMPRESSION INDEX, C_c	SWELLING INDEX, C_s	VOID RATIO AT START OF SWELL, e_s
Z-224U	6U	224.7	C2		22					SC	81	19													
					40	84	38	43	2.86	SC	62	38							36.7	1.213	3.6	7.5	0.478	0.032	0.678
	8U	221.3	C2		58	114	84	58		CH	34	66	CU	1.87	2.45	4.3	54.8	54.8							
	10U	218.2	C2		80	134	83	63		CH		81	CU	3.12	2.52	2.6	76.8	77.9							
	13U	213.2	C2			62	36	35		CH		54													
					31					*SC	65	35													
	15U	210.5	C2		31	104	83	33		SC	86	14													
Z-225U	4U	219.5	S3a		31	56	36	29		SC	74	26													
	6U	216.0	C2		61	131	100	58	2.79	CH		77	CU	2.47	2.74	2.9	62.3	62.6	58.9	1.675	3.6	6.6	1.118	0.214	0.700
	7U	214.0	C2			94	77	54		CH		59													
					49					SC	56	44	CU	2.98	2.81	4.1	43.5	43.8							

SOIL DESCRIPTION

(S3) RED-BROWN AND GRAY CLAYEY FINE TO MEDIUM SAND: TO FINE TO MEDIUM SAND, SOME CLAY OCCASIONALLY INTERLAYERED WITH SANDY CLAY.

(C2) YELLOW-BROWN AND LIGHT GREEN CLAYEY FINE SAND: TO FINE TO MEDIUM SAND, SOME CLAY. INTERLAYERED WITH STIFF YELLOW-BROWN SILTY CLAY. TRACE LIGNITE, OCCASIONAL LAYERS OF LIGHT GRAY-GREEN CALCAREOUS FINE TO MEDIUM SAND, SOME CLAY. CLAY LAYERS, TRACE SHELLS.

(S3a) LIGHT BROWN TO GRAY FINE TO MEDIUM SAND, SOME CLAY, TRACE LIGNITE. OCCASIONAL SANDY CLAY LAYERS.

(S3b) LIGHT BROWN, AND YELLOW-BROWN FINE TO MEDIUM SAND, TRACE CLAY.

NOTES

1. All tests summarized were performed in the soils laboratory of Mueser Rutledge Consulting Engineers.
2. The sample elevation is the average of the sampling interval.
3. GROUND SURFACE ELEVATIONS AT BORINGS ARE:

Boring No.	Elevation Ft.	Boring No.	Elevation Ft.
Z-201U	279.3	Z-218U	278.9
PZ-202U	278.3	Z-219U	290.5
PZ-210U	288.1	Z-220U	286.3
Z-211U	289.5	Z-224U	281.2
PZ-216U	294.5	Z-225U	277.0

4. "Average natural water content" is a weighted average of all material types recovered.
5. Compression tests performed were: CU - Consolidated Undrained Triaxial Compression.
6. Strength tests were performed on samples approximately 2.8 inches in diameter with a height-to-diameter ratio of 2.
7. Compression Index, C_c = the slope of the virgin curve (straight line portion of the consolidation test e-log p plot).

$$e_2 = e_1 - C_c \times \log (P_2/P_1)$$

8. Swelling Index, C_s = the slope of the rebound curve of the consolidation test...

$$e_2 = e_1 + C_s \times \log (P_1/P_2)$$

9. * - Sample showed a positive reaction with HCL.

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SALTSTONE DISPOSAL

SAVANNAH RIVER PLANT - Z AREA

E.I. DUPONT DE NEMOURS & CO., INC.

ATTACHMENT 3: SLOPE/W FILES

The five types of files were created and used by SLOPE/W for this calculation.
The **SLP** file contains the data required for the factor of safety calculations (slope dimensions, soil layering, loads, etc.). The **SL2** file contains information relating to the graphical layout or presentation of the problem (e.g. page size and units, engineering units and scale, sketch lines and text). The **FAC** or factor of safety file contains the computed factors of safety for each slip surface. The **FRC** or slice forces file stores the slice forces for the critical slip surface. The **SL3** file contains the current graphical layout information for the contour drawing information.

The file names provide some information about the problem. The first part of the name indicates the seismic force, 17g indicate the peak ground acceleration of 0.17g and 0g indicates no seismic force or the static case (see Section 2.3 Seismic Loading for horizontal and vertical loading combinations). The 200psi indicates compressive strength of the vault grout (see Section 2.1 Slope /Vault Geometry and Design). The last number (1 through 11) is the seismic loading case (see Table 1). The files created for this calculation are listed on sheets 46 and 47. The attached CD contains a copy of the computer files.

Calculation No. K-CLC-Z-00002
Sheet No. 46
Rev. 0

SLOPE/W File Listing

Date	Time	File Size	File Name
7/29/02	04:44p	3,658,435	0g-200psi1.fac
7/29/02	04:44p	23,694	0g-200psi1.frc
7/29/02	04:43p	2,157	0g-200psi1.sl2
7/29/02	04:51p	2,116	0g-200psi1.sl3
7/29/02	04:43p	4,650	0g-200psi1.slp
7/29/02	04:52p	3,658,435	17g-200psi2.fac
7/29/02	04:52p	23,694	17g-200psi2.frc
7/29/02	05:18p	2,157	17g-200psi2.sl2
7/29/02	05:18p	2,116	17g-200psi2.sl3
7/29/02	05:18p	4,650	17g-200psi2.slp
7/29/02	05:20p	3,658,435	17g-200psi3.fac
7/29/02	05:20p	23,694	17g-200psi3.frc
7/29/02	05:20p	2,157	17g-200psi3.sl2
7/29/02	05:22p	2,116	17g-200psi3.sl3
7/29/02	05:20p	4,650	17g-200psi3.slp
7/29/02	05:24p	3,658,435	17g-200psi4.fac
7/29/02	05:24p	23,694	17g-200psi4.frc
7/29/02	05:24p	2,157	17g-200psi4.sl2
7/29/02	05:26p	2,116	17g-200psi4.sl3
7/29/02	05:24p	4,650	17g-200psi4.slp
7/29/02	05:27p	3,658,435	17g-200psi5.fac
7/29/02	05:27p	23,694	17g-200psi5.frc
7/29/02	05:27p	2,157	17g-200psi5.sl2
7/29/02	05:32p	2,116	17g-200psi5.sl3
7/29/02	05:27p	4,650	17g-200psi5.slp
7/29/02	05:33p	3,658,435	17g-200psi6.fac
7/29/02	05:33p	23,694	17g-200psi6.frc
7/29/02	05:33p	2,157	17g-200psi6.sl2
7/29/02	05:35p	2,116	17g-200psi6.sl3
7/29/02	05:33p	4,650	17g-200psi6.slp
7/30/02	01:12p	16,164	17g-200psi6a.fac
7/30/02	01:12p	23,694	17g-200psi6a.frc
7/30/02	01:12p	2,157	17g-200psi6a.sl2
7/30/02	01:13p	2,116	17g-200psi6a.sl3
7/30/02	01:12p	4,650	17g-200psi6a.slp
7/30/02	01:16p	16,164	17g-200psi6b.fac
7/30/02	01:16p	22,995	17g-200psi6b.frc
7/30/02	01:15p	2,157	17g-200psi6b.sl2
7/30/02	11:57a	2,116	17g-200psi6b.sl3
7/30/02	01:15p	4,650	17g-200psi6b.slp
7/30/02	01:19p	16,164	17g-200psi6c.fac
7/30/02	01:19p	22,296	17g-200psi6c.frc
7/30/02	01:19p	2,157	17g-200psi6c.sl2
7/30/02	01:20p	2,116	17g-200psi6c.sl3
7/30/02	01:19p	4,650	17g-200psi6c.slp

Calculation No. K-CLC-Z-00002
Sheet No. 47
Rev. 1

Date	Time	File Size	File Name
7/29/02	05:36p	3,658,435	17g-200psi7.fac
7/29/02	05:36p	23,694	17g-200psi7.frc
7/29/02	05:36p	2,157	17g-200psi7.sl2
7/29/02	05:48p	2,116	17g-200psi7.sl3
7/29/02	05:36p	4,650	17g-200psi7.slp
7/29/02	05:50p	3,658,435	17g-200psi8.fac
7/29/02	05:50p	23,694	17g-200psi8.frc
7/29/02	05:50p	2,157	17g-200psi8.sl2
7/29/02	05:52p	2,116	17g-200psi8.sl3
7/29/02	05:50p	4,650	17g-200psi8.slp
7/30/02	08:51a	478,827	17g-200psi9.fac
7/30/02	08:51a	23,694	17g-200psi9.frc
7/30/02	08:50a	2,157	17g-200psi9.sl2
7/30/02	08:56a	2,116	17g-200psi9.sl3
7/30/02	08:50a	4,684	17g-200psi9.slp
8/6/02	04:34p	3,658,435	17g-200psi10.fac
8/6/02	04:34p	23,694	17g-200psi10.frc
8/6/02	04:34p	2,157	17g-200psi10.sl2
8/6/02	04:36p	2,116	17g-200psi10.sl3
8/6/02	04:34p	4,650	17g-200psi10.slp
8/6/02	04:38p	3,658,435	17g-200psi11.fac
8/6/02	04:38p	23,694	17g-200psi11.frc
8/6/02	04:38p	2,157	17g-200psi11.sl2
8/6/02	04:39p	2,116	17g-200psi11.sl3
8/6/02	04:38p	4,650	17g-200psi11.slp

Additional SLOPE/W File Listing for Revision 1

Date	Time	File Size	File Name
3/21/2003	01:53p	7,459,075	case10a.fac
3/21/2003	01:53p	22,995	case10a.frc
3/21/2003	01:55p	3,071	case10a.sl2
3/21/2003	01:55p	3,030	case10a.sl3
3/21/2003	01:55p	5,380	case10a.slp
3/21/2003	01:56p	7,459,075	case10b.fac
3/21/2003	01:56p	22,995	case10b.frc
3/21/2003	01:58p	3,071	case10b.sl2
3/21/2003	01:58p	3,030	case10b.sl3
3/21/2003	01:58p	5,380	case10b.slp
3/21/2003	01:33p	7,459,075	case11a.fac
3/21/2003	01:33p	23,694	case11a.frc
3/21/2003	01:35p	3,071	case11a.sl2
3/21/2003	01:34p	3,030	case11a.sl3
3/21/2003	01:35p	5,380	case11a.slp
3/21/2003	01:29p	7,459,075	case11b.fac
3/21/2003	01:29p	23,694	case11b.frc
3/21/2003	01:31p	3,071	case11b.sl2
3/21/2003	01:31p	3,030	case11b.sl3
3/21/2003	01:31p	5,380	case11b.slp