Calculation Cover Sheet

Project N.A.	Calculation No. K-CLC-F-00059	Project Number NA
Title Liquefaction Settlement for Waste Solidification Building (U)	Functional Classification ~~GS~~ SS 11-6-08 Discipline Geotechnical	Sheet 1 of ~~41~~ 43 MDM 7-26-07 ~~and 1 computer disk~~ 2/4/09

☐ Preliminary ☒ Confirmed

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

Purpose and Objective

This calculation estimates settlement resulting from liquefaction and partial liquefaction for the Waste Solidification Building when subjected to a scaled up Performance Category 3 (PC3) earthquake (i.e., scaled up by multiplying the time history by 1.25). This calculation also estimates settlement due to the Charleston 50th percentile earthquake.

Summary of Conclusion

See Conclusion Section.

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Revision

Rev. No.	Revision Description
0	Original
1	REVISED TO INCORPORATE NEW DATA. SEE SHEET 2 FOR MORE DETAIL.
2	Revised PC3 to PC3+ & updated functional classification

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Rev. No.	Originator (Print) Sign / Date	Verification / Checking Method	Verifier / Checker (Print) Sign / Date	Manager (Print) Sign / Date
0	Michael McHood [signature] 4/23/03	DOCUMENT REVIEW	MICHAEL G. SERRATO [signature] 4/30/03	Michael R. Lewis [signature] 5/1/03
1	MICHAEL MCHOOD [signature] 7/26/07	DOCUMENT REVIEW	Rucker J. Williams [signature] 7/26/07	M. R. LEWIS [signature] 7/30/07
2	Rucker J. Williams [signature] 11/10/08	DOCUMENT REVIEW	LAURA BAGWELL [signature] 10NOV08	William Li [signature] 11-11-08

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JSJ 12/23/08 James T. Salley ~~N/A~~ JSJ 12/23/08	~~N/A~~ [signature] JSJ 12/23/08	12/23/08 ~~N/A~~ JSJ 12/23/08

Security Classification of the Calculation
Unclassified

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Description of Revision 1

Due to the increase in size of the PC3+ portion of the WSB additional SCPTu were analyzed for liquefaction susceptibility and settlement. Four SCPTu locations had been analyzed for Revision 0. For this revision ten additional CPT locations were analyzed, making a total of fourteen. In addition weighted averages were calculated for one additional probabilistic seismic hazard frequency (i.e., 1 hz, see Table 3A). The thickness of the Santee Formation was also revisited and conservatively increased from 30 feet to 40 feet. 1

Description of Revision 2 2

12/23/08

The function classification was revised to reflect the ~~SC~~ SS classification and the revision of PC3 to PC3+.

Purpose

This calculation estimates settlement resulting from liquefaction and partial liquefaction for the Waste Solidification Building (WSB) when subjected to a scaled up Performance Category 3 (PC3) earthquake (i.e., scaled up by multiplying the time history by 1.25), i.e. a PC3+ earthquake. This calculation also estimates settlement due to the Charleston 50th percentile earthquake. 2

Input Data

Two geotechnical investigations have been performed in the vicinity of the WSB. The first was for the general area to the northeast of F-Area (WSRC, 2001) and the second was specifically for the Pit Disassembly and Conversion Facility (PDCF) (LAW, 2001). Seismic Piezocone Penetration Tests (SCPTu) were performed as part of these investigations and are used for this calculation. In addition six SCPTu were performed specifically for the WSB. The SCPTu were pushed to refusal and penetrated to elevations ranging from about 190 to 135 ft msl. The locations of the SCPTu are presented in Figure 1. Seven SCPTu falling within the footprint of the WSB and another seven SCPTu closest to the WSB are used in this calculation to estimate settlement due to liquefaction and partial liquefaction (see Figure 1 and Attachment A). 1

During the geotechnical investigations subsurface stratigraphic layers were delineated (WSRC, 2001; LAW, 2001). The geologic formation nomenclature developed and used for many SRS investigations (e.g., Tobacco Road, Dry Branch, and Santee Formations) correlates to the stratigraphic layers as shown in the table below. The subsurface layers for the WSB, including the new SCPTu are documented in SRS (2007). The subsurface conditions at the WSB are generally consistent with subsurface conditions in the GSA. 1

Soil Strata	SRS Geologic Formations and Units
TR1	Upland Formation
TR1A and TR2A	Tobacco Road Formation
TR2B, TR3/4 and DB1/3	Dry Branch Formation
DB4/5, ST1, and ST2	Santee Formation
GC	Warley Hill Formation
CG	Congaree Formation

Figure 1 shows the location of Well FAC-4 and Figure 3 shows the water levels measured in Well FAC-4 over a ten year period. The initial high water levels (1985) show the influence of the F-Area Acid Caustic Basin (which was abandoned) and do not represent the water table. The water table in the vicinity of the WSB, based on Well FAC-4, varies between 226 and 231 ft-msl (see Figure). The groundwater elevation used for the liquefaction analysis is placed at 230 feet mean sea level. 1 1

The USGS has performed a Probabilistic Seismic Hazard Analysis (PSHA) for the SRS (Frankel, 1999). The SRS rock hazard dissagregation for the 2,500 return period for oscillator frequencies of 1 hz, 2 hz, 3.33 hz and 100 hz (i.e., peak ground acceleration) are presented in Tables 1 through 3A and Figures 4 through 7. The hazard dissagregation is used to establish earthquake magnitude for the liquefaction analyses. 1

Ground response analyses performed for the PDCF site (LAW, 2001) were used to calculate soil shear stress and soil shear strains due to the PC3+ earthquake (i.e., PC3 x 1.25) and the Charleston 50th percentile earthquakes. The soil stresses are shown in Figures 8a and 8b and the soil strains are shown in Figures 9a through 9d.

The Santee Formation is considered too deep for liquefaction. However, dynamic settlement of this layer due to pore pressure buildup may occur. Settlement in the Santee Formation is calculated using the thickness of the Santee Formation as well as the recompression index (C_r), initial void ratio (e_o), and soil shear strain. This information is obtained from the PDCF Geotechnical Report (LAW, 2001).

Calculation Approach

Evaluation of Liquefaction Potential

In this calculation the liquefaction potential at the WSB is evaluated using a modified version of the "Simplified Procedure for Evaluating Soil Liquefaction Potential" (Seed and Idriss, 1971; NCEER, 1997; Youd et al., 2001). The simplified procedure calculates the liquefaction factor of safety as the ratio of Cyclic Resistance Ratio (CRR) to the Cyclic Stress Ratio (CSR) generated by the earthquake.

$$\text{Factor of Safety} = \text{CRR} / \text{CSR} \qquad \text{(Eq. 1)}$$

It should be noted that CRR was previously termed CSR required to induce liquefaction, but has been changed to more clearly distinguish the term from the CSR induced by the earthquake (NCEER, 1997). The CRR and CSR are defined in Equations 2 and 3.

$$\text{CRR} = \tau_{ave} / \sigma'_{vo} \text{ (soil capacity)} \qquad \text{(Eq. 2)}$$

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

$$\text{CSR} = \tau_{ave} / \sigma'_{vo} \text{ (earthquake demand)} \qquad \text{(Eq. 3)}$$

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

Earthquake Demand or Cyclic Stress Ratio (CSR)

The "Simplified Procedure for Evaluating Soil Liquefaction Potential" uses peak ground acceleration (PGA) to estimate shear stress (τ_{ave}) at ground surface and a stress reduction factor (r_d) to calculate τ_{ave} as a function of depth (Seed and Idriss, 1971; NCEER, 1997). However, in this calculation ground response analyses performed for the PDCF site (LAW, 2001) were used to calculate soil stress (see Figures 8 and 9). Soil shear stress (τ_{ave}) as a function of depth is given in Equations 4 and 5 for the PC3+ and Charleston 50th percentile earthquakes respectively.

$$\tau_{ave} = = 0.00000003(D)^4 + 0.000399484(D)^3 - 0.144244209(D)^2 + 19.489960061(D) - 13.849540274 \quad \text{(Eq. 4)}$$

$$\tau_{ave} = -0.000000638(D)^4 + 0.000349465(D)^3 - 0.074120061(D)^2 + 10.613644275(D) - 4.977489286 \quad \text{(Eq. 5)}$$

where:

τ_{ave} = average shear stress induced by the earthquake in psf

D = depth in feet

Note that τ_{eff} given in LAW (2001) is 0.65 time τ_{max}. For this calculation τ_{eff} is judged to be equivalent τ_{ave} as is commonly done in geotechnical practice (NCEER, 1997).

The effective vertical overburden stress is calculated using Equation 6.

$$\sigma'_{vo} = (\text{layer thickness layer above water table})\cdot(\gamma) + (\text{depth below water table})\cdot(\gamma_b) \quad \text{(Eq. 6)}$$

where:

σ'_{vo} = the effective stress

γ = unit weight of soil

γ_b = buoyant unit weight of soil (unit weight of soil - unit weight of water)

Soil Capacity or Cyclic Resistance Ratio (CRR)

The original simplified procedure determines CRR using Standard Penetration Test (SPT) $N1_{60}$ values and a curve delineating liquefaction. The CRR from the curve is further modified by several factors that have been developed over time. These factors correct for: aging, static driving shear stress, overburden pressure and earthquake magnitude. For this calculation curves based on CPTu tip stress data (q_{t1}) are used to determine CRR. The q_{t1} CRR curves were developed from testing of SRS soils in lieu of the standard SPT and CPTu liquefaction methods developed for Holocene and younger deposits. When applying the correction factors Equation 1 becomes Equation 7. Each of the correction factors and the methods for determining CRR are discussed in the following sections.

$$\text{Factor of Safety} = \frac{CRR_{7.5} \cdot K_{\sigma} \cdot K_{age} \cdot K_{\alpha} \cdot MSF}{CSR} \quad \text{(Eq. 7)}$$

Age Correction Factor K_{age}

The q_{tl} curves used for this calculation were developed specifically for SRS using data from investigations in H-Area (BSRI, 1993 and SRS 1994a) after extensive field and laboratory testing programs. Because the SRS q_{tl} CRR curves were developed from testing of the Tobacco Road and Dry Branch sediments, aging is incorporated into the curves. The curves may be directly applied to calculate liquefaction susceptibility without applying an age correction factor (i.e., $K_{age} = 1.0$).

Static Driving Shear Stress Correction Factor, $K\alpha$

Relationships proposed by Seed and Harder (1990) suggest that a static driving shear stress can increase or decrease the soil's resistance to liquefaction, depending on the magnitude of the driving stress and the relative density of the soil. A static driving shear stress correction factor ($K\alpha$) has been proposed by Seed and Harder to correct CRR. However, the proposed chart to estimate $K\alpha$ is preliminary and this correction factor is a subject of current research (NCEER, 1997, pp. 172-176). For this calculation, no $K\alpha$ correction was used (i.e., $K\alpha = 1.0$).

Static Effective Overburden Pressure Correction Factor, $K\sigma$

Most of the case history data used to develop the standard liquefaction curves (Seed and Idriss, 1982 and NRC, 1985) were taken from cases of level ground with relatively small initial effective overburden stresses ($\sigma'_o \leq 1$ tsf). However, at higher effective overburden stresses ($\sigma'_o > 1$ tsf), the liquefaction susceptibility of the soil will increase for a given CSR (Seed and Harder, 1990). Thus, the CRR must be corrected for the influence of the static overburden stresses. This is done by multiplying CRR by the correction factor ($K\sigma$). The soils at SRS are much older than the case history data typically used for liquefaction studies. Therefore testing of soils at SRS has been performed to determine appropriate $K\sigma$ for SRS soils (WSRC, 1995; BSRI, 1993). Figure 10 shows the SRS $K\sigma$ curve along with data used to develop the curve (WSRC, 1995). The NCEER recommended $K\sigma$ curves (Youd et al., 2001) are also shown on Figure 10 for comparison. The polynomial representing the SRS $K\sigma$ curve shown in Figure 10 is given in Equation 9.

$$K\sigma = 1.009376 - 0.18326 \log(\sigma'_{vo}) - 0.08340 \log(\sigma'_{vo})^2 \qquad \text{(Eq. 9)}$$

where: σ'_{vo} is Effective Vertical Overburden Pressure in tsf.

Note that the $K\sigma$ used for this calculation (Eq. 9) is the site-specific relationship developed using data from investigations in H-Area (WSRC, 1995; SRS, 1994b and SRS, 1995) and not the standard $K\sigma$ relationship proposed by NCEER (Youd et al, 2001). The SRS $K\sigma$ used is applicable for the WSB, but less stringent than the $K\sigma$ proposed by NCEER.

Earthquake Magnitude Scaling Factor, MSF

The CRR curves used for liquefaction analysis are only valid for M = 7.5 earthquakes. For earthquakes with differing magnitudes, the CRR values must be multiplied by a magnitude scaling factor (MSF). Values of the MSF and corresponding earthquake magnitude are shown on Figure 11 (NCEER, 1997).

Previous liquefaction evaluations at SRS have used an average magnitude for a given performance category. For example, in the past the PC3 earthquake has been assigned a Mw of 6.1. However, the seismic hazard at a given site is actually comprised of many possible earthquake events and a single magnitude may not be appropriate. The USGS PSHA performed for SRS considers many possible earthquake events having various magnitudes and distances (see Figures 4 through 7). Because a given spectral acceleration (Sa) can occur due to a range of earthquake magnitudes several MSFs are used in this calculation. For comparison, the PC3+ analysis is also performed using a Mw of 6.1. The earthquake magnitudes from the PSHA and the appropriate MSF for each magnitude are given in Table 4. These MSFs represent the middle of the NCEER (1997) recommended values shown on Figure 11.

1 2

CRR from the SRS q_{t1} versus CRR Curves

Figure 12 presents the SRS q_{t1} versus CRR curves (WSRC, 1995). Equations 10 through 14 (SRS, 1995) were developed for computer application of the SRS q_{t1} versus CRR curves. Laboratory testing and development of the curves are discussed in SRS (1994a; 1995) and WSRC (1995).

for fines content 30%:

$$CRR = 0.125721 + 0.002537\,(q_{t1}) + 0.000040\,(q_{t1})^2 \qquad \text{(Eq. 10)}$$

for fines content 22.5%:

$$CRR = 0.093309 + 0.001757\,(q_{t1}) + 0.000029\,(q_{t1})^2 \qquad \text{(Eq. 11)}$$

for fines content 15%:

$$CRR = 0.072666 + 0.001141\,(q_{t1}) + 0.000028\,(q_{t1})^2 \qquad \text{(Eq. 12)}$$

for fines content 10%:

$$CRR = 0.046881 + 0.001190\,(q_{t1}) + 0.000015\,(q_{t1})^2 \qquad \text{(Eq. 13)}$$

for fines content 0%:

$$CRR = 0.021215 + 0.001408\,(q_{t1}) + 0.000007\,(q_{t1})^2 \qquad \text{(Eq. 14)}$$

The CPTu tip stress (q_t) must be normalized (q_{t_1}) for overburden stress prior to applying the curves. This is done using an overburden correction factor C_Q shown in Equations 15 and 16 (Youd et al., 2001).

$$q_{t_1} = C_Q \times q_t \qquad \text{(Eq. 15)}$$

$$C_Q = (P_a / \sigma'_{vo})^n \text{ with } C_Q \leq 1.7 \qquad \text{(Eq. 16)}$$

where: C_Q = CPTu Overburden Normalization Factor
Pa = Atmospheric pressure
σ'_{vo} = Effective vertical overburden pressure at time of testing
P_a and σ'_{vo} must be in the same units
n = exponent ranges between 0.5 for clean sand and 1.0 for clays

For this calculation the n exponent in Equation 16 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%). It is important to note that normalization of q_t to q_{t_1} is performed using effective vertical overburden pressure at the time of data collection.

Percent Fines
The SRS q_{t1} method used in this calculation requires percent fines to determine CRR. Percent fines is determined using an SRS site specific CPTu method.

The SRS CPTu method for determining percent fines was developed by correlating laboratory determined fines content from borings with nearby CPTu results (SRS, 2001). The SRS method uses a classification index (I_c) to calculate percent fines. The SRS method is given in Equations 17 and 18.

$$\text{Percent Fines} = 29.47(I_c)^{1.21} - 0.09 \qquad \text{(Eq. 17)}$$

$$I_c = [(1.60\text{-}\log Q_t)^2 + (\log Fr + 0.41)^2]^{0.5} \qquad \text{(Eq. 18)}$$

Where: Q_t is normalized tip resistance $Q_t = (q_t\text{-}\sigma_{vo})/\ \sigma'_{vo}$
Fr is stress normalized friction ratio $Fr = [(f_s/q_t\text{-}\sigma_{vo}) \times 100]$
q_t is CPTu tip stress corrected for unequal area effects
f_s is CPTu sleeve friction
σ_{vo} is the total vertical overburden stress (from Eq. 5)
σ'_{vo} is the effective vertical overburden stress (from Eq. 6)

Figure 13 presents the laboratory determined percent fines for boring FB20 (see Table 5) along with the CPTu calculated percent fines for the FNEC-179 SCPTu location. The CPTu calculated percent fines compare reasonably well with the laboratory determined percent fines.

Dynamic Settlement in the Tobacco Road and Dry Branch Formations

Settlement due to liquefaction and partial liquefaction can be calculated using standard techniques from the geotechnical engineering literature or techniques based on site specific testing. Due to the age and increased strength it was necessary to sample and test SRS soils to quantify strain due to cyclic loading. SRS specific volumetric strain curves were developed during sampling and laboratory testing programs from H-Area (SRS, 1994a; 1994c; 1995; BSRI, 1993). The SRS volumetric strain curves (Figure 14) give volumetric strain as a function of CPTu tip resistance and factor of safety. Because the SRS volumetric strain curves were developed from testing of the Tobacco Road and Dry Branch sediments, the curves incorporate strength due to aging.

For conservatism, the SRS volumetric strain curves considered liquefaction triggered in all zones having a factor of safety less than or equal to 1.15 (BSRI, 1993 and WSRC, 1995). Soils having factors of safety between 1.15 and 2.2 are considered to be partially liquefied. 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.

For this calculation, dynamic settlement of unsaturated (i.e., above the water table) sands was ignored, because of their small contribution to the total dynamic settlement.

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. No consideration is given for dilation or bridging effects of interspersed or overlying, non-liquefied layers. This is a conservative assumption and actual settlements may be less, especially if the thickness of the non-liquefied layer(s) is great. Total cumulative settlement resulting from liquefaction and partial liquefaction is estimated for the profile by summing the liquefaction settlement (i.e., $FS \leq 1.15$) and partial liquefaction settlement (i.e., $1.15 < FS \leq 2.2$) for each increment:

$$S_{Total} = \Sigma\, S_{Liq} + \Sigma\, S_{P\,Liq} \qquad \text{(Eq. 19)}$$

where:

S_{Total} = cumulative settlement,
S_{Liq} = settlement of the increment due to liquefaction, and
$S_{P\,Liq}$ = settlement of the increment due to partial liquefaction.

The value of S_{Liq} is calculated from:

$$S_{Liq} = \text{(volumetric strain due to liquefaction) } dz \qquad \text{(Eq. 20)}$$

and the value of $S_{P\,Liq}$ is calculated from:

$$S_{P\,Liq} = \text{(volumetric strain due to partial liquefaction) } dz \qquad \text{(Eq. 21)}$$

where: dz = thickness of the increment.

Volumetric Strain Curves

The volumetric strain curves developed for SRS using H-Area data and used in this calculation are presented in Figure 14 (SRS, 1994a; 1994c; WSRC, 1995). The volumetric strain curves for various values of q_{t_1} are a function of factor or safety. These curves have been fitted with the following regression equations, which were derived and presented in calculation C-CLC-H-00815 (SRS, 1995).

for $q_{t_1} = 160$, $0.4 < FS < 1.15$

$$\text{strain } (\%) = 0.65 \qquad \text{(Eq. 22)}$$

for $q_{t_1} = 130$, $0.4 < FS < 1.15$

$$\text{strain } (\%) = 2.9883 + 10.354(FS)^4 - 30.258(FS)^3 + 30.7(FS)^2 - 13.064(FS) \qquad \text{(Eq. 23)}$$

for $q_{t_1} = 100$, $0.4 < FS < 1.15$

$$\text{strain } (\%) = 2.0308 + 8.3929(FS)^4 - 21.111(FS)^3 + 16.12(FS)^2 - 4.5756(FS) \qquad \text{(Eq. 24)}$$

for $q_{t_1} = 50$, $0.4 < FS < 0.65$

$$\text{strain } (\%) = -41.6495 - 756.666(FS)^4 + 1505.222(FS)^3 - 1123.65(FS)^2 + 371.2387(FS) \qquad \text{(Eq. 25)}$$

for $q_{t_1} = 50$, $0.65 < FS < 1.15$

$$\log \text{strain } (\%) = 1.256225 - 0.21100(FS)^2 - 1.01242(FS) \qquad \text{(Eq. 26)}$$

for $q_{t_1} = 30$, $0.4 < FS < 0.65$

$$\text{strain } (\%) = -45.4815 - 830.0000(FS)^4 + 1651.074(FS)^3 - 1231.64(FS)^2 + 406.5062(FS) \qquad \text{(Eq. 27)}$$

for $q_{t_1} = 30$, $0.65 < FS < 1.15$

$$\log \text{strain } (\%) = 1.181442 - 0.47909(FS)^2 - 0.63184(FS) \qquad \text{(Eq. 28)}$$

for $q_{t_1} = 20$, $0.4 < FS < 0.65$

$$\text{strain } (\%) = -45.2315 - 830.0000(FS)^4 + 1651.074(FS)^3 - 1231.64(FS)^2 + 406.5062(FS) \qquad \text{(Eq. 29)}$$

for $q_{t_1} = 20$, $0.65 < FS < 1.15$

$$\log \text{strain } (\%) = 0.679601 - 1.17026(FS)^2 + 0.616392(FS) \qquad \text{(Eq. 30)}$$

for $q_{t_1} = 10$, $0.4 < FS < 0.65$

$$\text{strain } (\%) = -29.6577 - 560.0000(FS)^4 + 1114.074(FS)^3 - 836.066(FS)^2 + 278.6576(FS) \quad \text{(Eq. 31)}$$

for $q_{t_1} = 10$, $0.65 < FS < 1.15$

$$\log \text{strain } (\%) = 0.454166 - 1.56185(FS)^2 + 1.272068(FS) \quad \text{(Eq. 32)}$$

for $q_{t_1} = 5$, $0.4 < FS < 0.65$

$$\text{strain } (\%) = -29.7775 - 566.666(FS)^4 + 1127.333(FS)^3 - 845.883(FS)^2 + 281.8638(FS) \quad \text{(Eq. 33)}$$

for $q_{t_1} = 5$, $0.65 < FS < 1.15$

$$\log \text{strain } (\%) = 0.367762 - 1.73636(FS)^2 + 1.555255(FS) \quad \text{(Eq. 34)}$$

for partial liquefaction $1.15 < FS < 1.6$ and all q_{t_1} values

$$\log \text{strain } (\%) = 1.256225 - 0.21100(FS)^2 - 1.01242(FS) \quad \text{(Eq. 35)}$$

for partial liquefaction $1.6 < FS < 2.2$ and all q_{t_1} values

$$\text{strain } (\%) = 0.728794 + 0.100221(FS)^2 - 0.54090(FS) \quad \text{(Eq. 36)}$$

Dynamic Settlement in the Santee Formation

The liquefaction and settlement analyses were performed for the Tobacco Road and Dry Branch formations as discussed above. The Santee Formation is considered too deep for liquefaction. However, the dynamic settlement of this layer due to soil straining, pore pressure buildup and eventual dissipation may occur.

Dynamic settlement of the Santee Formation is calculated based on Equation 37 (WSRC, 1995).

$$S = H\,[C_r/1+e_o)] \times \log[1/(1-r_u)] \quad \text{(Eq. 37)}$$

where: S = settlement in feet
H = the thickness of the Santee Formation in feet
C_r = the recompression index
e_o = initial void ratio
r_u = pore water pressure ratio

The thickness of the Santee formation is conservatively estimated at about 40 feet from SCPTu soundings (SRS, 2007). The Santee Formation corresponds to the DB4/5, ST1 and ST2 layers. The recompression index and void ratio for the Santee formation are obtained from the lab tests documented in the PDCF Geotechnical Report (LAW, 2001). The recompression index and initial void ratio are summarized in Table 6. The pore pressure ratio is determined based on the 1

maximum strain obtained from SHAKE output (see Figures 9a through 9d) and Figure 15. The upper portion of the Santee (the DB4/5 layer) experiences higher strains than the lower portion. A shear strain of 0.04% was selected for the PC3+ earthquake, roughly the average for the DB4/5 portion of the Santee (see Figure 9a). A shear strain of 0.02% was selected for Charleston 50th percentile earthquake, roughly the average for the DB4/5 portion of the Santee (see Figure 9c). Using these strains, pore water pressure ratio can be obtained from Figure 15 and settlement can be calculated.

Assumptions

1. The CRR and volumetric strain relationships developed for SRS (Figures 12 and 14) using data from investigations in H-Area (BSRI, 1993 and SRS, 1994a) are applicable to F-Area soils. These curves were developed from samples of the Tobacco Road and Dry Branch Formations. In applying these curves in F-Area, it is assumed the liquefaction resistance of the Tobacco Road and Dry Branch Formations in F-Area is similar to the Tobacco Road and Dry Branch Formations in H-Area.
2. Because the SRS CRR and volumetric strain relationships (Figures 12 and 14) incorporate aging of the Tobacco Road and Dry Branch Formations, the curves may be directly applied to calculate liquefaction susceptibility without applying an age correction factor.
3. The static effective overburden correction factor (K_σ) developed for SRS (Figure 10) using data from investigations in H-Area (WSRC, 1995; SRS, 1994b and SRS, 1995) is applicable for F-Area soils.
4. In order to calculate percent fines to be used with the SRS CRR curve (Figures 12) the SRS site specific relationship (Equations 17 and 18) developed using data gathered from several locations at the SRS was used. It is assumed that this relationship is valid for F-Area.
5. These calculations were done for free field conditions. The effects of static driving shear stresses were ignored. No K_α correction was made (i.e., $K_\alpha = 1$). The factor, K_α, has been proposed to account for static driving stresses. However, this factor is a subject of current research (NCEER, 1997).
6. For conservatism, liquefaction was considered triggered in all zones having a factor of safety less than or equal to 1.15. Soils having factors of safety between 1.15 and 2.2 were considered to be partially liquefiable. Soils with a factor of safety > 2.2 are considered to be non-liquefiable with no dynamic settlement.
7. The water level elevation near the WSB ranges between 226 and 231 ft-msl (see Figure 5). The groundwater elevation used for the liquefaction analysis is 230 ft-msl. Soils above 230 ft-msl were excluded from the analyses because they are not saturated and therefore not susceptible to liquefaction. Dynamic settlement occurring in dry sands is small in comparison to liquefaction and partial liquefaction settlement and can be neglected. (Volumetric strains in dry sands are about an order of magnitude smaller than those in liquefied saturated sands at equal relative densities and under the same intensity of ground shaking [Soydemir, 1993]).
8. For the liquefaction and partial liquefaction settlement calculations, 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. No consideration is given for dilation or bridging effects of interspersed or overlying, non-liquefied layers. This is a conservative

assumption and actual settlements may be less, especially if the thickness of the non-liquefied layer(s) is great.

9. The middle of the Magnitude Scaling Factor (MSF) range recommended by NCEER workshop (NCEER, 1997) is used in this calculation (see Figure 11). It is assumed that these MSFs best account for differences in earthquake energy or magnitude.
10. Dynamic settlement is calculated for the Santee Formation even though it is judged to be too deep for liquefaction. The dynamic settlement of the Santee is due to shear strain, pore pressure buildup and eventual dissipation of pore pressure. Geologic conditions in F-, E-, H-, S-, and Z-Areas are similar. The pore water pressure ratio versus cyclic shear strain relationship (Figure 15) developed for the Santee Formation using laboratory test data from H-Area (WSRC, 1995) are applicable to the Santee Formation in F-Area.
11. Earthquake shear stress and strain developed by LAW (2001) for the PC3+ and Charleston 50th percentile earthquake (see Figures 8a, 8b, 9a and 9b) represent the design basis for the WSB and this liquefaction calculation.

Calculation

Liquefaction susceptibility and estimated settlement calculations for the Dry Branch and Tobacco Road Formations due to the PC3+ and Charleston 50th percentile earthquakes are not provided as hard copy due to the volume of data, but are provided on the disk accompanying this calculation (see Attachment B).

Sheet 15 shows the calculation of dynamic settlement for the Santee Formation for the PC3+ and Charleston 50th ground motion events. Note that several magnitudes have been used in conjunction with the PC3+ earthquake.

Settlements calculated for the PC3+ and Charleston 50th percentile earthquakes using the relationships developed from testing of SRS soils, the four SCPTu locations and each PSHA magnitude are summarized on Sheet 16. Settlement due to the PC3+ earthquake for each SCPTu location varies according to the magnitude. The appropriate settlement is determined by taking a weighted average with weighting based on the hazard contribution from each magnitude. There is some uncertainty regarding what ground motion frequency most strongly influences liquefaction. One study performed for H-Area (WSRC, 1995) suggests that liquefaction is most strongly influenced by the 1 to 3 hz frequency range. Traditionally liquefaction shear stresses have been calculated based on peak ground acceleration (PGA). For this calculation weighting was performed using the SRS hazard contribution for 1 hz, 2 hz, 3.33 hz and PGA (100 hz). 1

An example of how the weighted average is performed is as follows; settlements are calculated for SCPTu location FPCDA-39 and each magnitude bin in the SRS PSHA (i.e., Mw of 4.75, 5.25, 5.75, 6.25, 6.75 and 7.5, see Sheet 16). The resulting settlement for each magnitude is then multiplied by the percent contribution of that magnitude from the SRS PSHA (see Table 3 100 hz for this example). These results are summed and divided by 100 to give the weighted average.

	Settlement Mw =4.75	**Settlement Mw =5.25**	**Settlement Mw =5.75**	**Settlement Mw =6.25**	**Settlement Mw =6.75**	**Settlement Mw =7.5**
FPDCA-39	0.0	0.04	0.11	0.17	0.35	1.10
PGA Weight	15.52	12.16	10.76	10.25	3.92	47.40

Weighted Average = 0.57 inch

Weighted Average = [(0.0x15.52)+(0.04x12.16)+(0.11x10.76)+(0.17x10.25)+(0.35x3.92)+(1.10x47.40)] / 100

Settlement at each SCPTu location and for each of the frequencies is summarized on Sheet 17. Also given are the settlements due to the Charleston 50th percentile earthquake having a Mw of 7.3. The settlements calculated for PC3+ earthquake with Mw = 6.1, as has been done in the past, are also given.

Calculation No. K-CLC-F-00059
Sheet No. 15
Rev. 2

Santee Compression Calculation

Settlement = H [C_r / (1+e_o)] x log[1/(1-r_u)]

H = 40	ft (Santee thickness based on CPTs FPDCA20 and FPDCA24, see K-CLC-F-00075)
C_r = 0.0188	recompression index from PDCF data (see Table 6)
e_o = 1.1348	initial void ratio from PDCF data (see Table 6)
M_w = 7.5	magnitude varies for the PC3 analysis
γ = 0.04	% Soil Shear Strain from SHAKE analysis (see Figure 9a) (γ = 0.04 for PC3+ earthquake)
r_u = 0.16	pore water pressure ratio from (see Figure 15) (for PC3+ earthquake r_u varies between 0 for M=4.75 and 0.16 for M=7.5)

Settlement = 0.32 inch

PC3+ Results for Various Magnitudes

Mw = 4.75	**Mw = 5.25**	**Mw = 5.75**	**Mw = 6.25**	**Mw = 6.75**	**Mw = 7.5**	**Mw = 6.1**
r_u = 0	**r_u = 0.03**	**r_u = 0.06**	**r_u = 0.09**	**r_u = 0.12**	**r_u = 0.16**	**r_u = 0.08**
0.00	0.04	0.11	0.17	0.23	0.32	0.15

Settlement = H [C_r / (1+e_o)] x log[1/(1-r_u)]

H = 40	ft (Santee thickness based on CPTs FPDCA20 and FPDCA24, see K-CLC-F-00075)
C_r = 0.0188	recompression index from PDCF data (see Table 6)
e_o = 1.13475	initial void ratio from PDCF data (see Table 6)
M_w = 7.3	for the Charleston earthquake Mw = 7.3 (WSRC, 1994; Johnston, 1996)
γ = 0.02	% Soil Shear Strain from SHAKE analysis (see Figure 9b) (γ = 0.02 for Charleston 50th percentile earthquake)
r_u = 0.09	pore water pressure ratio from (see Figure 15, 14 load cycles) (for Charleston 50th percentile earthquake r_u =0.09 corresponding to M=7.3)

Charleston 50th percentile Results Mw=7.3

Settlement = 0.18 inch

Calculation No. K-CLC-F-00059
Sheet No. 16
Rev. 2

Settlement Due to Pore Pressures in Santee Formation
Calculated Using SRS Strain Methodology

Settlement in Santee Formation for PC3 x 1.25 Earthquake Settlement Due to Partial Liquefaction (inch)						PC3 x 1.25 PGA = 0.2g	Charleston 50th PGA = 0.11g
Mw = 4.75	Mw = 5.25	Mw = 5.75	Mw = 6.25	Mw = 6.75	Mw = 7.5	Mw = 6.1	Mw = 7.3
r_u = 0	r_u = 0.03	r_u = 0.06	r_u = 0.09	r_u = 0.12	r_u = 0.16	r_u = 0.08	r_u = 0.09
0.00	0.04	0.11	0.17	0.23	0.32	0.15	0.18

1

Settlement Due to Liquefaction and Partial Liquefaction in Tobacco Rd and Dry Branch Formations
Calculated Using SRS Stress Methodology

	PC3 x 1.25 Earthquake PGA of about 0.2g (2,500 year event) Settlement Due to Liquefaction and Partial Liquefaction (inch)						PC3 x 1.25 PGA = 0.2g	Charleston 50th PGA = 0.11g
	Mw = 4.75	Mw = 5.25	Mw = 5.75	Mw = 6.25	Mw = 6.75	Mw = 7.5	Mw = 6.1	Mw = 7.3
SCPTu ID	MSF=3.4	MSF=2.8	MSF=2.2	MSF=1.7	MSF=1.4	MSF=1.0	MSF=1.9	MSF=1.1
FNEC-179	0.00	0.00	0.00	0.03	0.15	0.90	0.02	0.04
FNEC-180	0.00	0.00	0.00	0.00	0.07	0.45	0.00	0.00
FNEC-182	0.00	0.00	0.00	0.00	0.08	0.54	0.00	0.01
FNEC-183	0.00	0.00	0.00	0.00	0.04	0.30	0.00	0.00
FNEC-186	0.00	0.00	0.00	0.00	0.08	0.56	0.00	0.01
FPDC-A11	0.00	0.00	0.00	0.01	0.14	0.79	0.00	0.02
FPDC-A15	0.00	0.00	0.00	0.01	0.10	0.67	0.00	0.00
FPDC-A39	0.00	0.00	0.00	0.01	0.13	0.78	0.00	0.01
FWSBC1	0.00	0.00	0.00	0.00	0.04	0.24	0.00	0.00
FWSBC2	0.00	0.00	0.00	0.00	0.07	0.43	0.00	0.00
FWSBC3	0.00	0.00	0.00	0.00	0.08	0.53	0.00	0.00
FWSBC4	0.00	0.00	0.00	0.05	0.21	1.22	0.02	0.07
FWSBC5	0.00	0.00	0.00	0.00	0.10	0.63	0.00	0.01
FWSBC6	0.00	0.00	0.00	0.02	0.10	0.62	0.00	0.04

1

Sum of Settlement Due to Liquefaction and Partial Liquefaction
Tobacco Rd, Dry Branch, and Santee Formations Combined

	PC3 x 1.25 Earthquake PGA of about 0.2g (2,500 year event) Settlement Due to Liquefaction and Partial Liquefaction (inch)						PC3 x 1.25 PGA = 0.2g	Charleston 50th PGA = 0.11g
	Mw = 4.75	Mw = 5.25	Mw = 5.75	Mw = 6.25	Mw = 6.75	Mw = 7.5	Mw = 6.1	Mw = 7.3
SCPTu ID	MSF=3.4	MSF=2.8	MSF=2.2	MSF=1.7	MSF=1.4	MSF=1.0	MSF=1.9	MSF=1.1
FNEC-179	0.00	0.04	0.11	0.20	0.38	1.22	0.17	0.22
FNEC-180	0.00	0.04	0.11	0.17	0.30	0.77	0.15	0.18
FNEC-182	0.00	0.04	0.11	0.17	0.30	0.86	0.15	0.19
FNEC-183	0.00	0.04	0.11	0.17	0.27	0.62	0.15	0.18
FNEC-186	0.00	0.04	0.11	0.17	0.31	0.88	0.15	0.18
FPDC-A11	0.00	0.04	0.11	0.18	0.36	1.11	0.15	0.20
FPDC-A15	0.00	0.04	0.11	0.18	0.33	0.99	0.15	0.18
FPDC-A39	0.00	0.04	0.11	0.17	0.35	1.10	0.15	0.19
FWSBC1	0.00	0.04	0.11	0.17	0.26	0.56	0.15	0.18
FWSBC2	0.00	0.04	0.11	0.17	0.29	0.75	0.15	0.18
FWSBC3	0.00	0.04	0.11	0.17	0.31	0.85	0.15	0.18
FWSBC4	0.00	0.04	0.11	0.22	0.44	1.54	0.17	0.25
FWSBC5	0.00	0.04	0.11	0.17	0.33	0.95	0.15	0.18
FWSBC6	0.00	0.04	0.11	0.18	0.32	0.94	0.15	0.22

1

1

Sum of Settlement Due to Liquefaction and Partial Liquefaction
Tobacco Rd, Dry Branch, and Santee Formations Combined

SCPTu ID	PC3 x 1.25 Average all Magnitudes Using 100hz Weighting (inch)	PC3 x 1.25 Average all Magnitudes Using 3.33hz Weighting (inch)	PC3 x 1.25 Average all Magnitudes Using 2 hz Weighting (inch)	PC3 x 1.25 Average all Magnitudes Using 1 hz Weighting (inch)	PC3 x 1.25 PGA = 0.2g Mw = 6.1 MSF=1.9 (inch)	Charleston 50th PGA = 0.11g Mw = 7.3 MSF=1.1 (inch)
FNEC-179	0.63	0.89	0.98	1.05	0.17	0.22
FNEC-180	0.41	0.57	0.62	0.67	0.15	0.18
FNEC-182	0.45	0.63	0.69	0.75	0.15	0.19
FNEC-183	0.34	0.47	0.51	0.55	0.15	0.18
FNEC-186	0.46	0.64	0.71	0.76	0.15	0.18
FPDC-A11	0.57	0.80	0.89	0.95	0.15	0.20
FPDC-A15	0.52	0.72	0.79	0.85	0.15	0.18
FPDC-A39	0.57	0.80	0.88	0.95	0.15	0.19
FWSBC1	0.31	0.42	0.46	0.49	0.15	0.18
FWSBC2	0.40	0.55	0.61	0.65	0.15	0.18
FWSBC3	0.45	0.63	0.69	0.74	0.15	0.18
FWSBC4	0.79	1.11	1.23	1.32	0.17	0.25
FWSBC5	0.50	0.69	0.76	0.82	0.15	0.18
FWSBC6	0.50	0.69	0.76	0.82	0.15	0.22

Conclusion

Settlement due to liquefaction and partial liquefaction for the PC3+ earthquake ranges from ¼ inch to 1¼ inches with the majority of the locations at less than an inch. Settlement due to liquefaction and partial liquefaction for the Charleston 50th percentile earthquake is less than ¼ inch. Using several different magnitudes to represent the PC3+ earthquake and weighting each magnitude appropriately results in greater settlement than using one magnitude to represent the PC3+ earthquake. Appropriate settlement for design would be less than 1 inch with little differential settlement as the settlements are relatively small and are expected to distribute due to the depth at which the settlements occur. 1

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Table 1 – USGS Rock Seismic Hazard for SRS at Oscillator Frequency of 2 hz for Return Period 2,500 years and Sa = 0.15g

(Frankel, 1999)

Distance	Mw = 4.5 to 5.0	Mw = 5.0 to 5.5	Mw = 5.5 to 6.0	Mw = 6.0 to 6.5	Mw = 6.5 to 7.0	Mw = 7.0 to 8.0
7.5 km	0.74	1.35	1.21	0.70	0.41	0.00
20 km	0.21	0.76	1.15	0.95	0.72	0.00
37.5 km	0.10	0.67	1.70	2.18	2.42	0.00
75 km	0.02	0.24	1.22	2.85	0.00	17.77
150 km	0.00	0.08	0.71	2.75	0.00	50.89
250 km	0.00	0.00	0.04	0.26	0.00	6.45
550 km	0.00	0.00	0.01	0.07	0.00	1.38
Mw Bin Sum =	1.07	3.11	6.02	9.76	3.55	76.49

Note: 1/3 wt Frankel et al. attenuation model, 1/3 wt Toro et al. attenuation model and 1/3 wt AB95 attenuation model

Table 2 – USGS Rock Seismic Hazard for SRS at Oscillator Frequency of 3.33 hz for Return Period 2,500 years and Sa = 0.21g

(Frankel, 1999)

Distance	Mw = 4.5 to 5.0	Mw = 5.0 to 5.5	Mw = 5.5 to 6.0	Mw = 6.0 to 6.5	Mw = 6.5 to 7.0	Mw = 7.0 to 8.0
7.5 km	2.16	2.14	1.42	0.73	0.42	0.00
20 km	0.83	1.48	1.53	1.08	0.74	0.00
37.5 km	0.46	1.40	2.37	2.54	2.54	0.00
75 km	0.08	0.49	1.62	3.18	0.00	17.69
150 km	0.01	0.15	0.87	2.79	0.00	45.62
250 km	0.00	0.00	0.03	0.19	0.00	4.81
550 km	0.00	0.00	0.00	0.03	0.00	0.59
Mw Bin Sum =	3.54	5.68	7.84	10.55	3.70	68.70

Note: 1/3 wt Frankel et al. attenuation model, 1/3 wt Toro et al. attenuation model and 1/3 wt AB95 attenuation model

Table 3 – USGS Rock Seismic Hazard for SRS at Oscillator Frequency of 100 hz (PGA) for Return Period 2,500 years and Sa = 0.15g

(Frankel, 1999)

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

Note: 1/3 wt Frankel et al. attenuation model, 1/3 wt Toro et al. attenuation model and 1/3 wt AB95 attenuation model

Table 3A – USGS Rock Seismic Hazard for SRS at Oscillator Frequency of 1 hz for Return Period 2,500 years and Sa = 0.10g

1

(Frankel, 1999)

Distance	Mw = 4.5 to 5.0	Mw = 5.0 to 5.5	Mw = 5.5 to 6.0	Mw = 6.0 to 6.5	Mw = 6.5 to 7.0	Mw = 7.0 to 8.0
7.5 km	0.12	0.48	0.75	0.57	0.39	0.00
20 km	0.03	0.24	0.63	0.71	0.64	0.00
37.5 km	0.02	0.22	0.93	1.57	2.07	0.00
75 km	0.00	0.10	0.81	2.36	0.00	16.07
150 km	0.00	0.03	0.57	2.74	0.00	54.07
250 km	0.00	0.00	0.05	0.41	0.00	8.75
550 km	0.00	0.00	0.02	0.23	0.00	4.42
Mw Bin Sum =	0.16	1.07	3.77	8.59	3.10	83.30

Note: 1/3 wt Frankel et al. attenuation model, 1/3 wt Toro et al. attenuation model and 1/3 wt AB95 attenuation model

Table 4 – Earthquake Magnitude Scaling Factors

Earthquake Magnitude (Mw)	Magnitude Scaling Factor
4.75	3.4
5.25	2.8
5.75	2.2
6.1 †	1.9
6.25	1.7
6.75	1.4
7.3 ‡	1.1
7.5	1.0

† Mw of 6.1 used for previous PC3 analyses

‡ Magnitude for the Charleston earthquake is estimated at Mw = 7.3 (WSRC, 1994; Johnston, 1996).

Note: MSFs represent the middle of the NCEER (1997) recommended values, see Figure 11.

Table 5 – Fines Content Determined from Laboratory Testing for Boring FB-20
(LAW, 1998)

Boring ID	Sample Mid Depth (feet)	Mid Sample Elevation (ft msl)	Fines Content (%)
FB20	7.0	294.4	20.8
FB20	10.0	291.4	44.3
FB20	13.0	288.4	37.8
FB20	16.0	285.4	39.7
FB20	19.0	282.4	26.4
FB20	25.0	276.4	26.6
FB20	31.0	270.4	18
FB20	37.0	264.4	16.4
FB20	44.5	256.9	24.8
FB20	50.5	250.9	5.3
FB20	53.5	247.9	16.7
FB20	56.5	244.9	9.5
FB20	59.5	241.9	17.5
FB20	62.5	238.9	5.6
FB20	65.5	235.9	9.2
FB20	71.5	229.9	9.9
FB20	74.5	226.9	11.9
FB20	77.5	223.9	36
FB20	79.0	222.4	40.3
FB20	80.5	220.9	36.2
FB20	83.5	217.9	18.3
FB20	86.5	214.9	9.7
FB20	89.5	211.9	7.8
FB20	94.0	207.4	11.8
FB20	97.0	204.4	11.1
FB20	100.1	201.3	10.7
FB20	104.6	196.8	5.3
FB20	107.6	193.8	7.9
FB20	110.6	190.8	24.5

Table 6 – Summary of Recompression Index (C_r) and Initial Void Ratio (e_o) for the Santee Formation
(LAW, 2001)

Boring ID	Sample No.	Top Depth	Layer	C_r	e_o
BA-34 UD	UD-10	115	DB-4/5	0.0404	0.929
BA-34 UD	UD-11	123	DB-4/5	0.0122	0.874
BA-35 UDA	UD-10	90	DB-4/5	0.0045	1.516
BA-35 UDA	UD-11	96	DB-4/5	0.0116	0.929
BA-7A	UD-8	114	DB-4/5	0.0253	1.030
BA-34 UD	UD-12	129	ST-1	0.0108	0.852
BA-35 UDA	UD-12	106	ST-1	0.0127	0.872
BA-7A	UD-9	122	ST-1	0.0149	0.917
BA-34 UD	UD-13	143.5	ST-2	0.0177	1.264
BA-34 UD	UD-14	151	ST-2	0.0284	1.791
BA-35 UDA	UD-13	118	ST-2	0.0266	1.310
BA-35 UDA	UD-14	126	ST-2	0.0205	1.333
			Average =	0.0188	1.1348

The values reported in this table come from Table D-2 of LAW (2001). The samples listed as being from boring ID BA-35 UDA may actually be from boring BA-35 UD as these sample numbers and top depths appear on the BA-35 UD boring log and not on the BA-35 UDA log. These two borings (i.e., BA-35 UDA and BA-35 UD) are relatively close to each other and this discrepancy does not affect the results. 1

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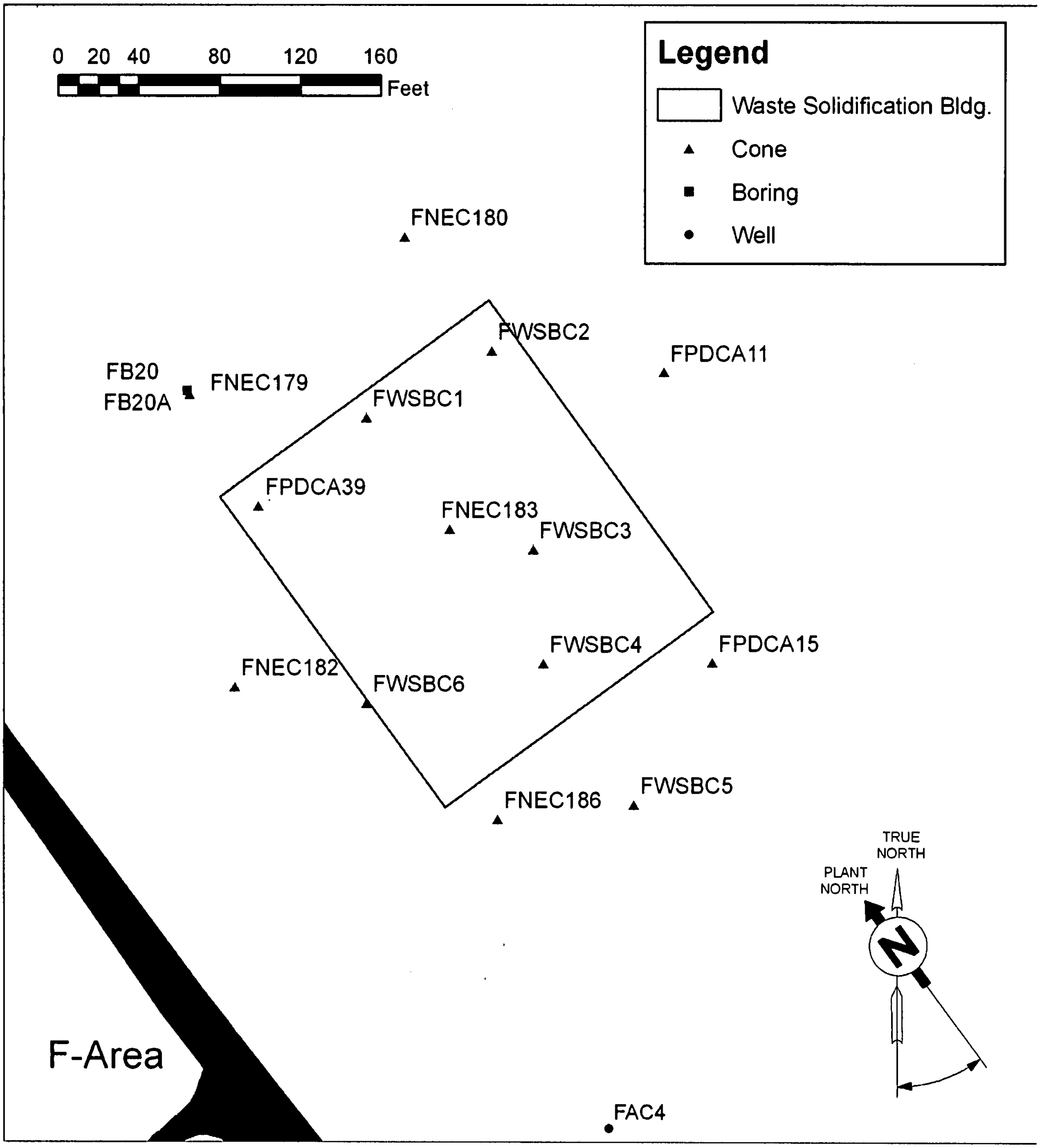

Figure 1 - Location of Waste Solidification Building, SCPTu Soundings, SPT Borings and Well FAC-4

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Note there is no Figure 2. 1

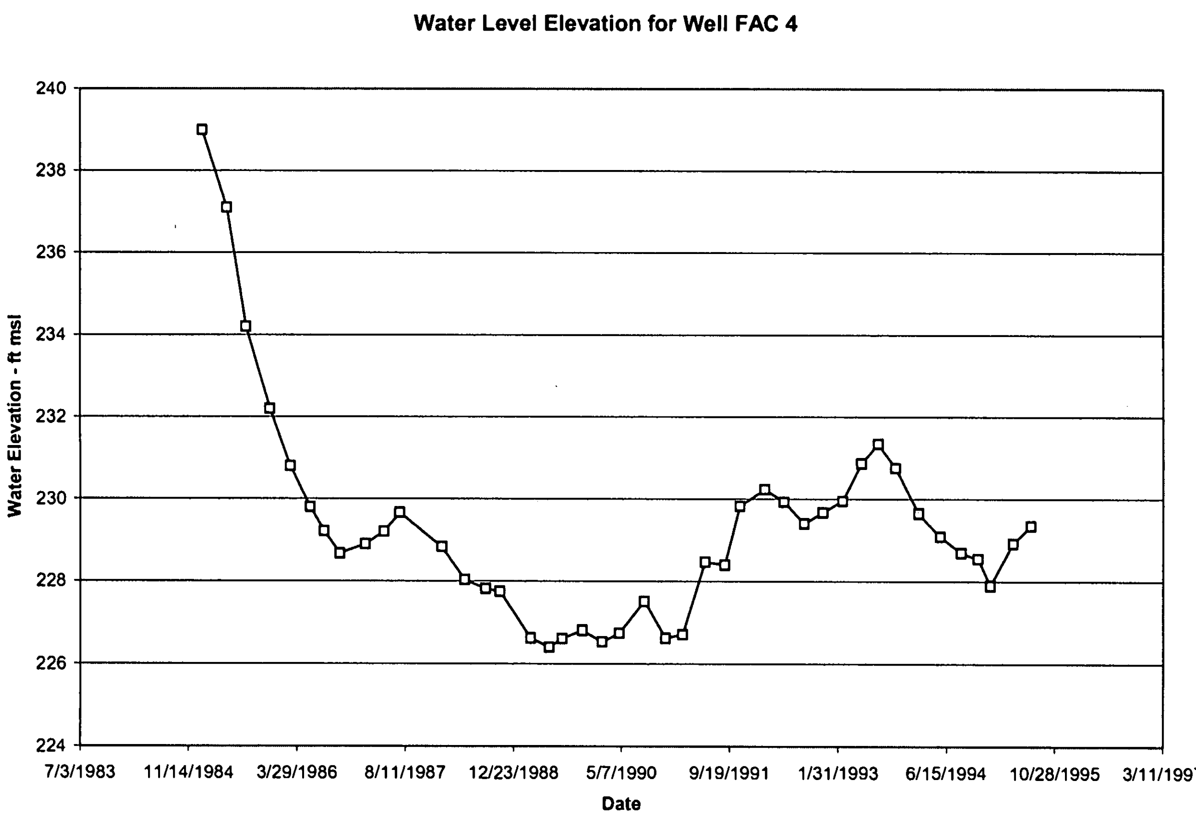

Figure 3 - Water Elevation Measured in Well FAC-4 1
(SRS Groundwater Information Management System)

Note: Early 1985 data discounted due to influence from the F-Area Acid Caustic Basin. Average ground water elevation in the vicinity of the Waste Solidification Building is expected to be at elevation 230 ft msl or lower most of the time.

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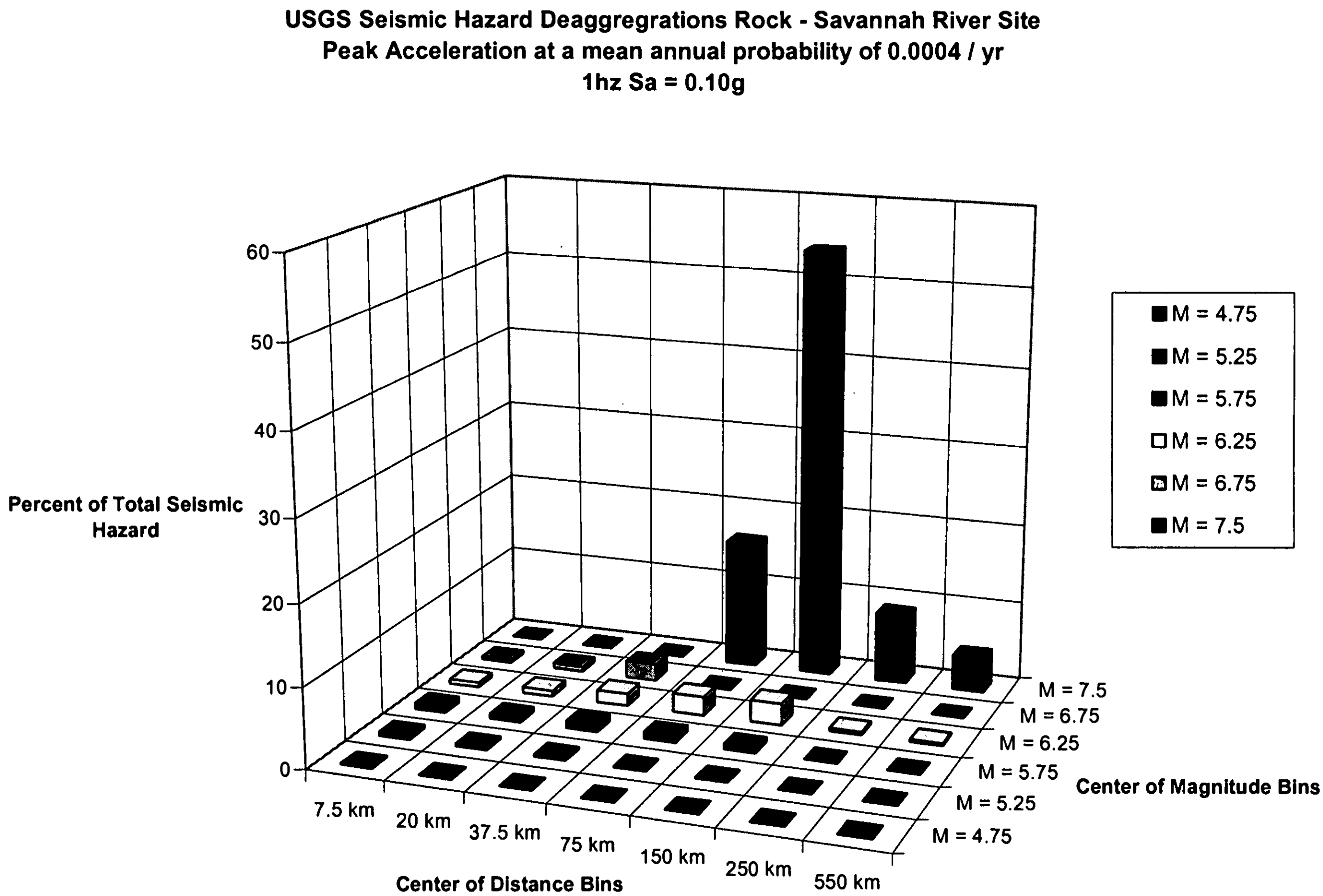

1

Figure 4 – USGS Rock Seismic Hazard Deaggregation for SRS at Frequency of 1 hz for Return Period of 2,500 years and Sa = 0.10g
(Frankel, 1999)

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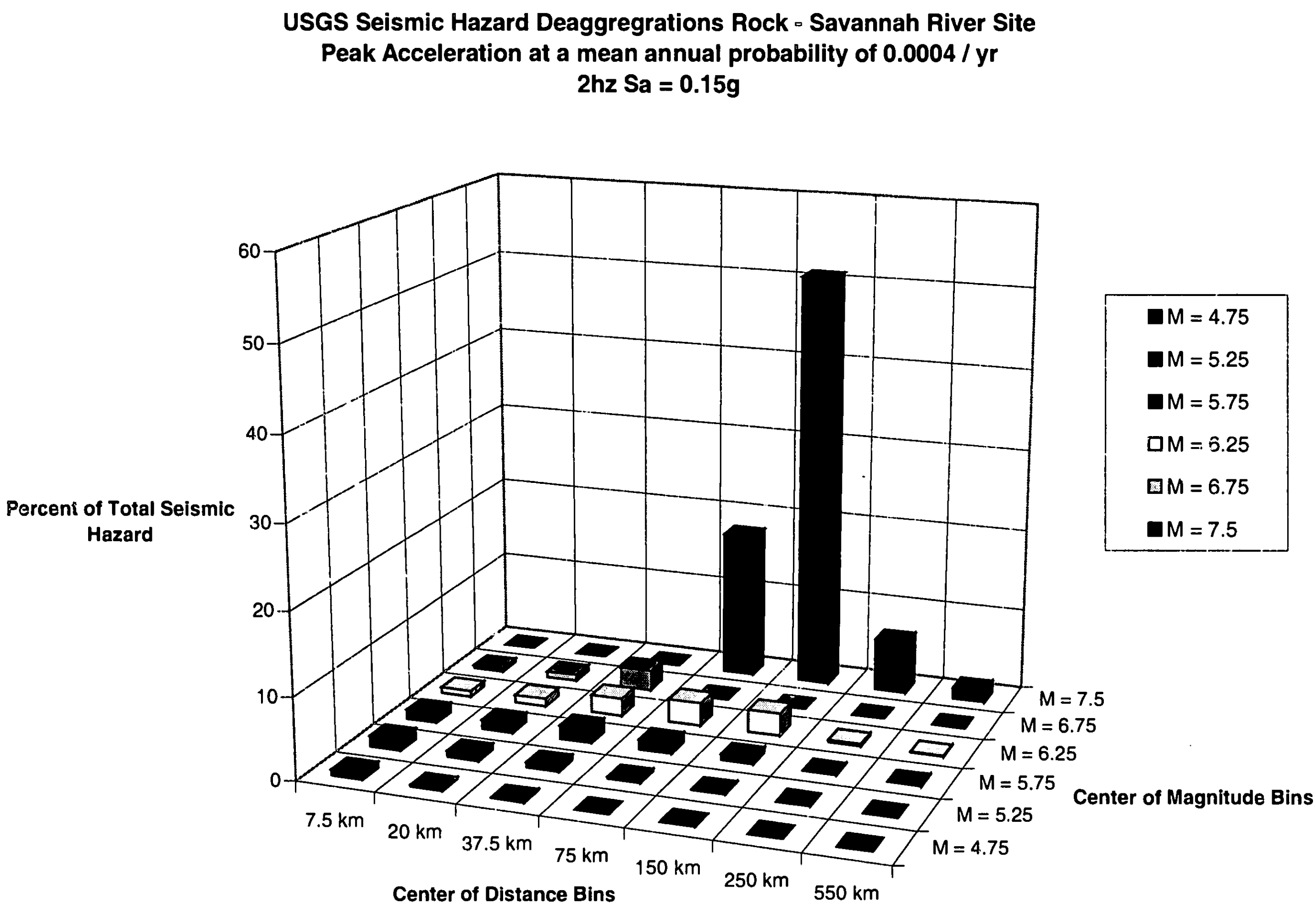

Figure 5 – USGS Rock Seismic Hazard Deaggregation for SRS at Frequency of 2 hz for Return Period of 2,500 years and Sa = 0.15g

(Frankel, 1999)

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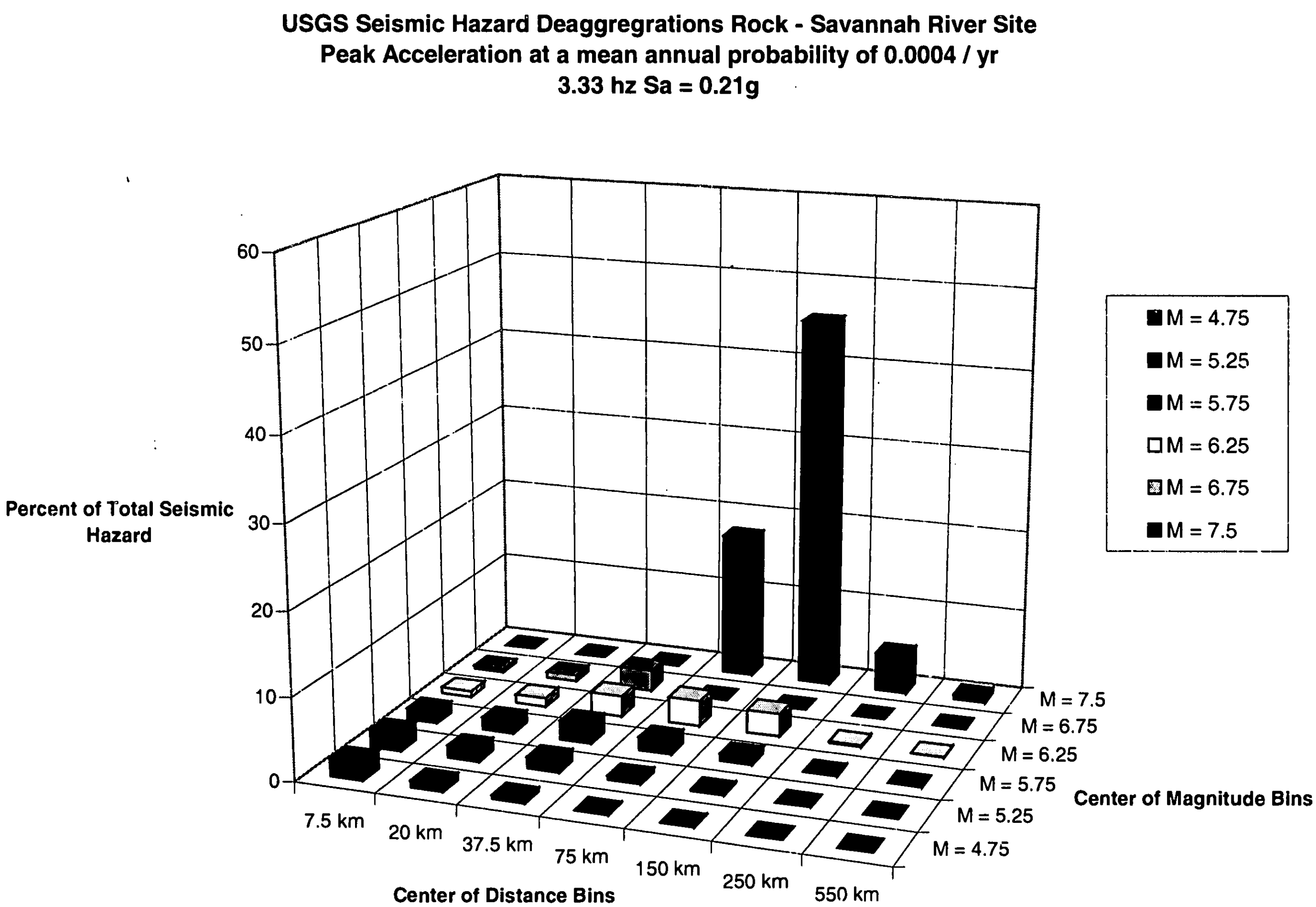

Figure 6 – USGS Rock Seismic Hazard Deaggregation for SRS at Frequency of 3.33 hz for Return Period of 2,500 years and Sa = 0.21g
(Frankel, 1999)

Calculation No. K-CLC-F-00059
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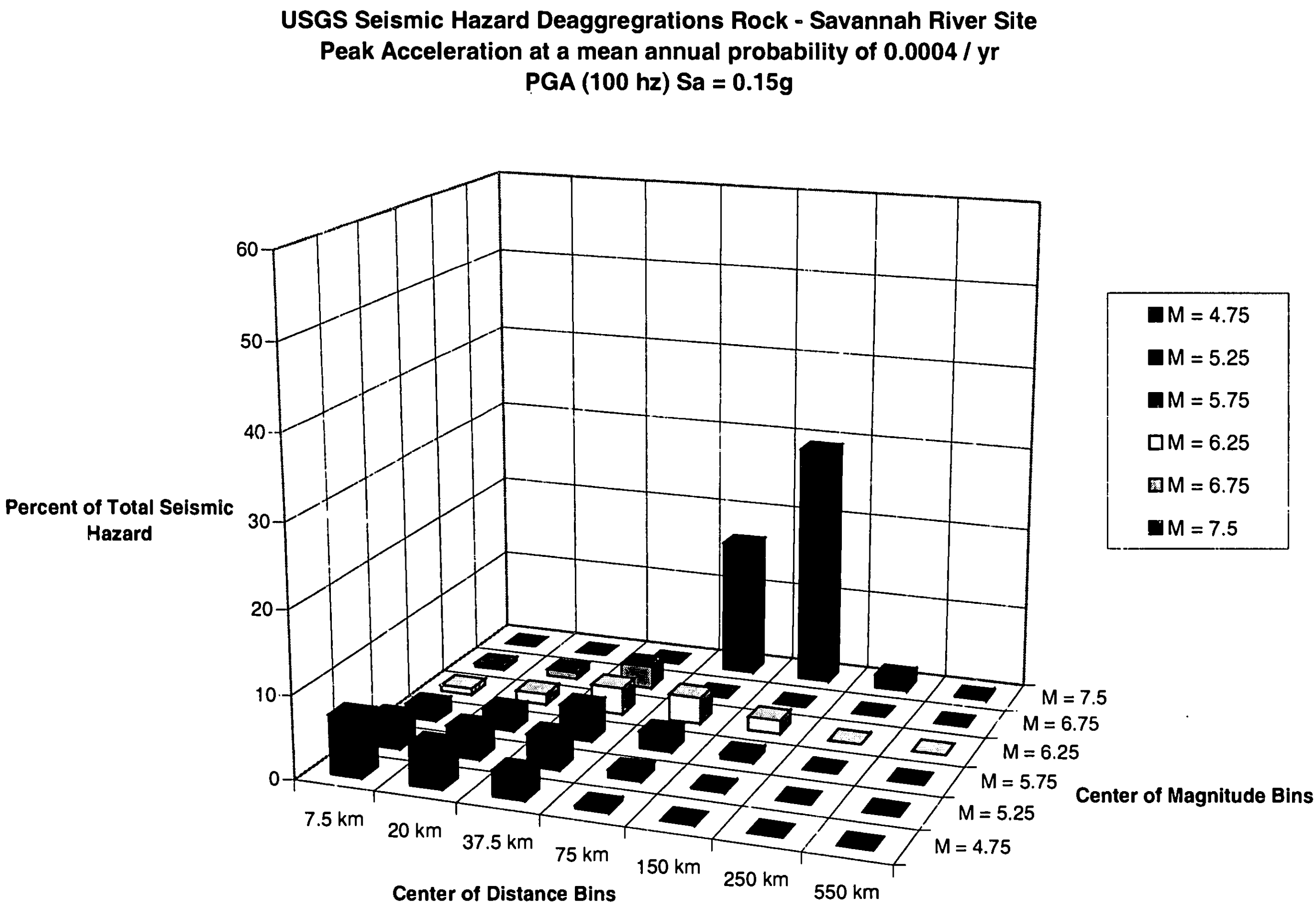
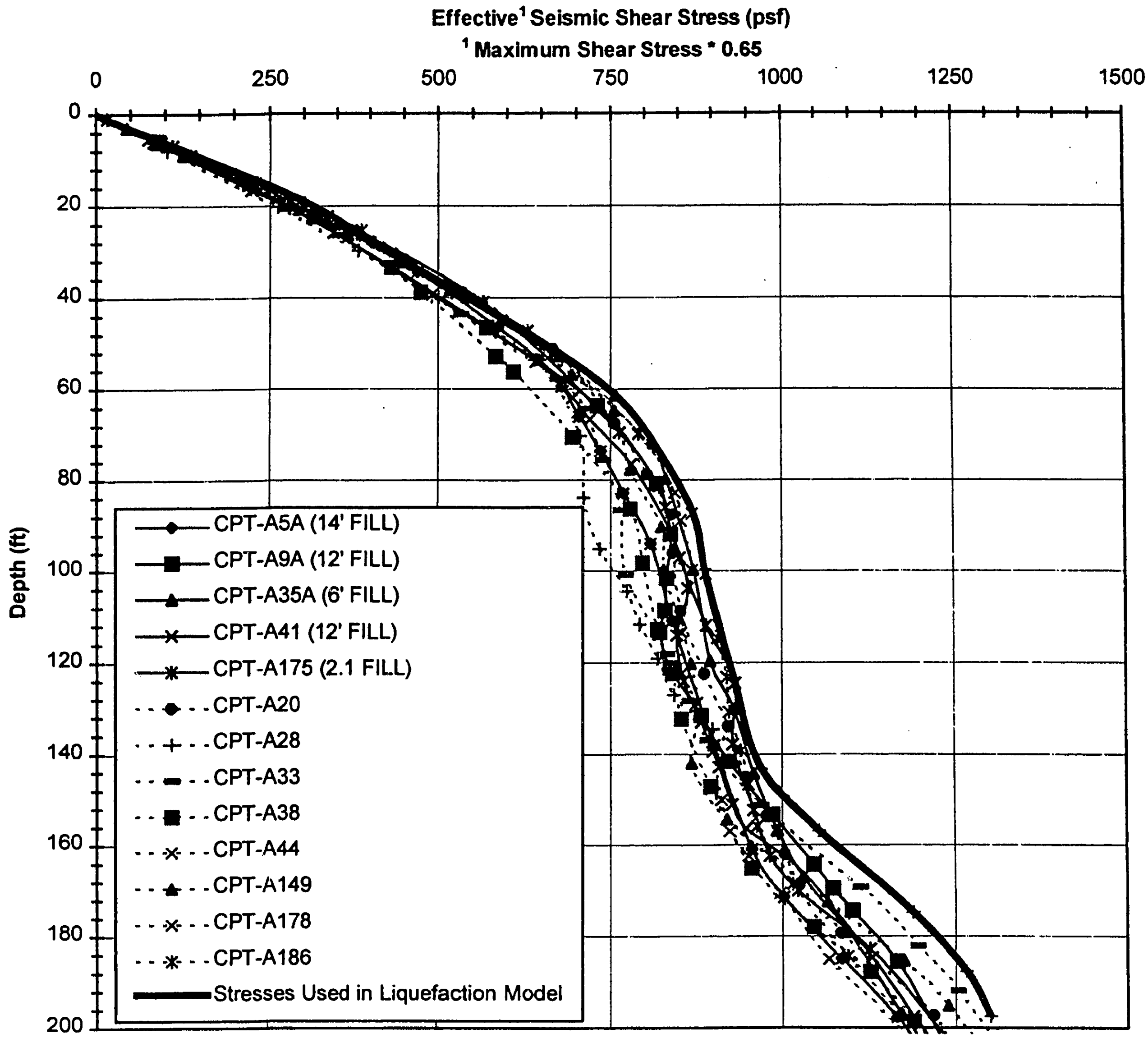


Figure 7 – USGS Rock Seismic Hazard Deaggregation for SRS at Frequency of 100 hz for Return Period of 2,500 years and Sa = 0.15g

(Frankel, 1999)

Calculation No. K-CLC-F-00059
Sheet No. 30
Rev. 0



Upper-bound Stress Equation:

$$\tau_{eff} = 0.000000030(\text{Depth})^4 + 0.000399484(\text{Depth})^3 - 0.144244209(\text{Depth})^2 + 19.489960061(\text{Depth}) - 13.849540274$$

Figure 8a – Range of Equivalent Uniform Cyclic Shear Stress from SHAKE'91 Response Analysis for PC3 Time History Factored by 1.25

(LAW, 2001 Figure 23)

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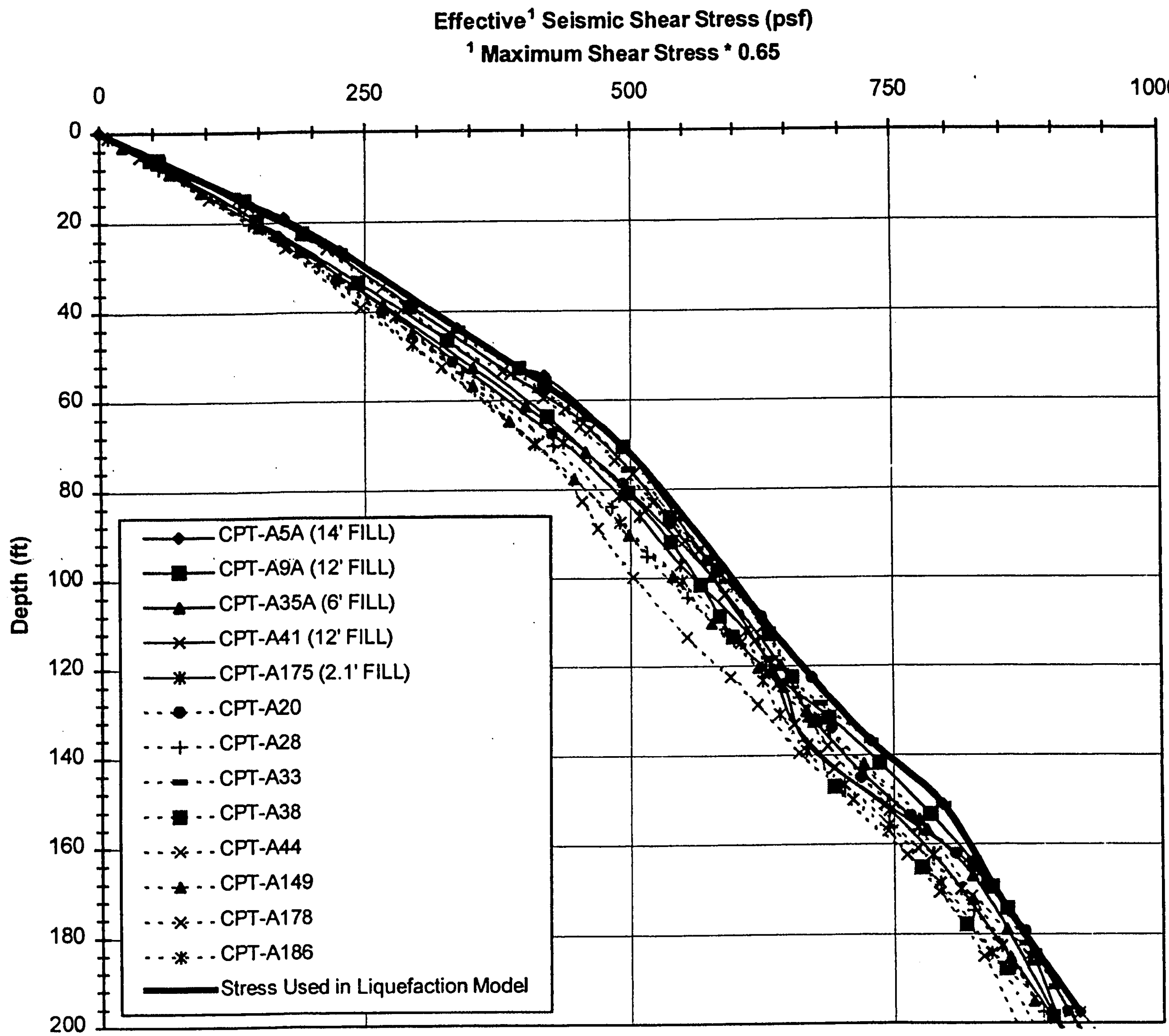

Upper-bound Stress Equation:

$$\tau_{eff} = -0.000000638(\text{Depth})^4 + 0.000349465(\text{Depth})^3 - 0.074120061(\text{Depth})^3 + 10.613644275(\text{Depth}) - 4.977489286$$

Figure 8b – Range of Equivalent Uniform Cyclic Shear Stress from SHAKE'91 Response Analysis for Charleston 50% Time History

(LAW, 2001 Figure 24)

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Rev. 0

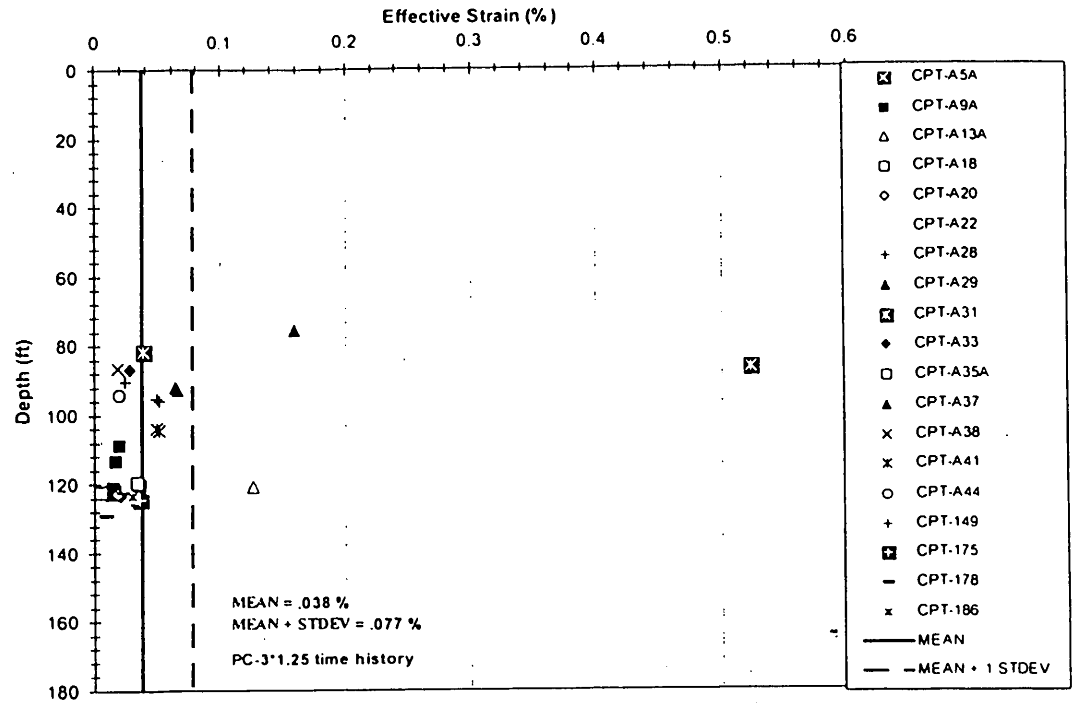

Figure 4.1-3a. Range of Equivalent Uniform Cyclic Shear Strains from SHAKE '91 response analysis within DB 4/5 based on PC-3 time history factored by 1.25

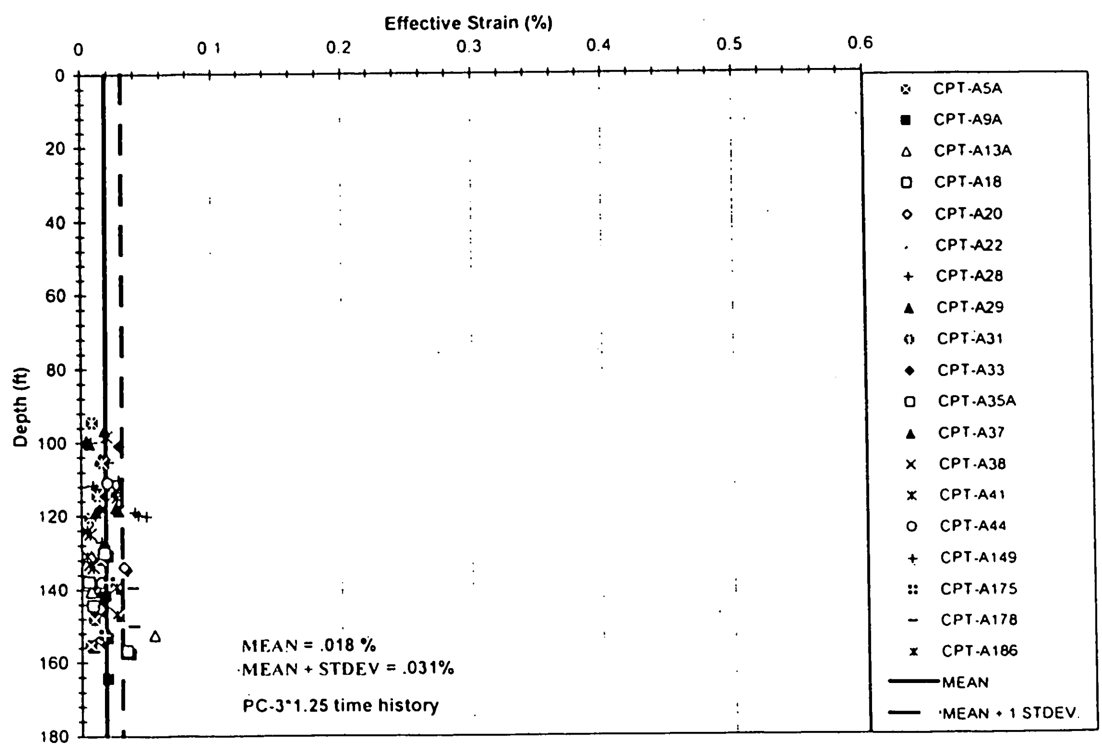

Figure 4.1-3b. Range of Equivalent Uniform Cyclic Shear Strains from SHAKE '91 response analysis within ST 1 and ST 2 based on PC-3 time history factored by 1.25

Figure 9a – Range of Equivalent Uniform Cyclic Shear Strains from SHAKE'91 Response Analysis for PC3 Time History Factored by 1.25

(LAW, 2001 Figure 4.1-3a and 4.1-3b)

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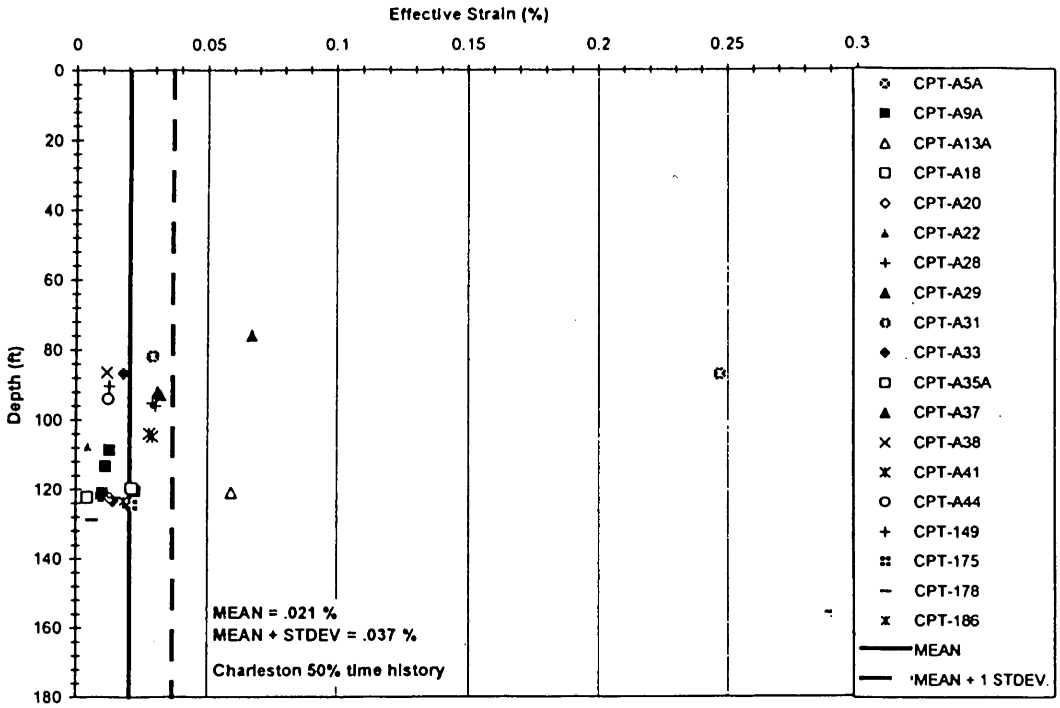

Figure 4.1-4a. Range of Equivalent Uniform Cyclic Shear Strains from SHAKE '91 response analysis within DB 4/5 based on Charleston 50% time history

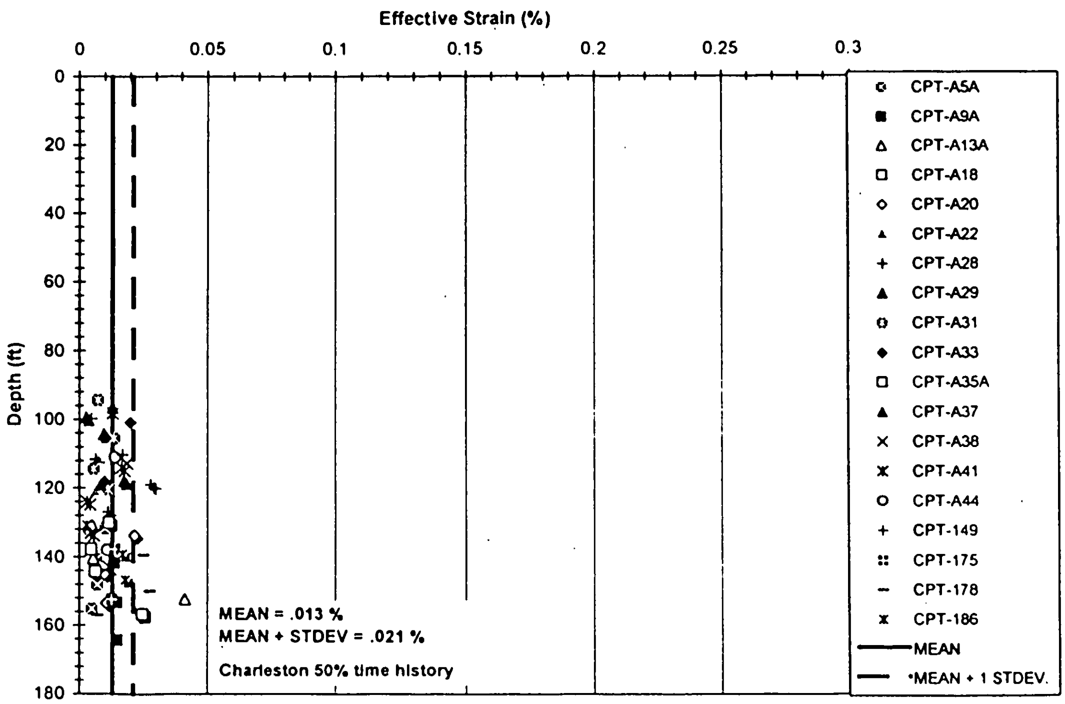

Figure 4.1-4b. Range of Equivalent Uniform Cyclic Shear Strains from SHAKE '91 response analysis within ST 1 and ST 2 based on Charleston 50% time history

Figure 9b – Range of Equivalent Uniform Cyclic Shear Strain from SHAKE'91 Response Analysis for Charleston 50% Time History

(LAW, 2001 Figure 4.1-4a and 4.1-4b)

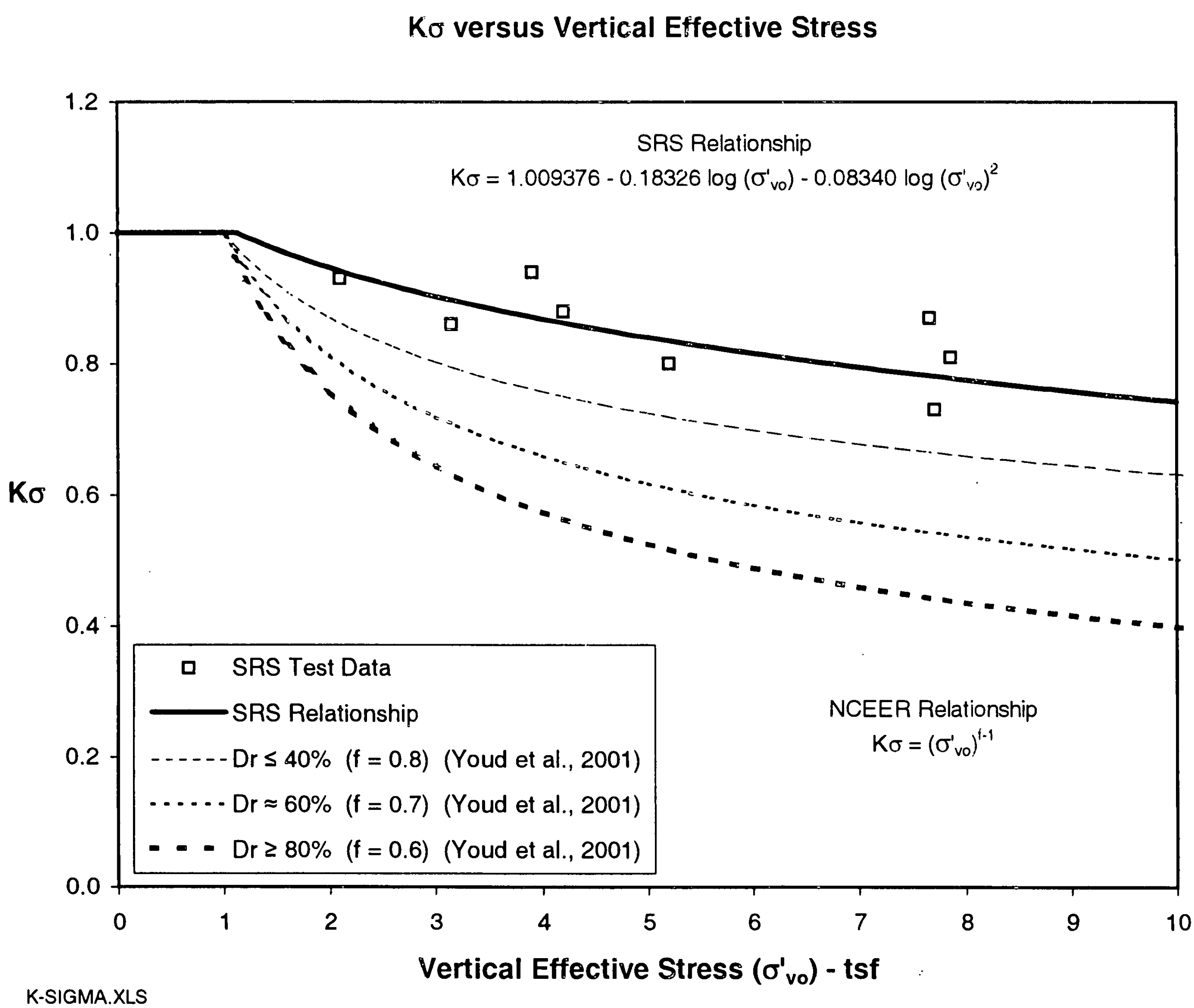

Figure 10 – Comparison of Effective Overburden Correction Factors (Kσ)
(SRS, 1994b; WSRC, 1995; Youd et al., 2001)

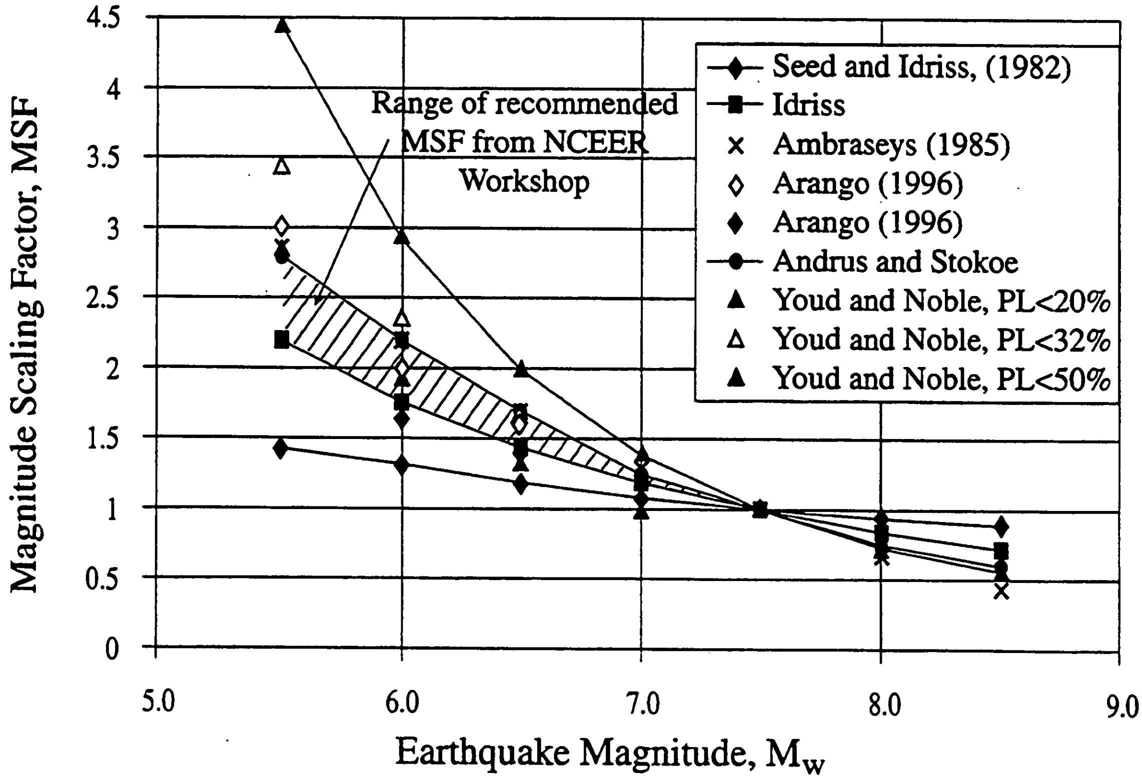

Figure 11 - Magnitude Scaling Factors Proposed by Various Investigators With Range Recommended by NCEER Workshop
(NCEER, 1997)

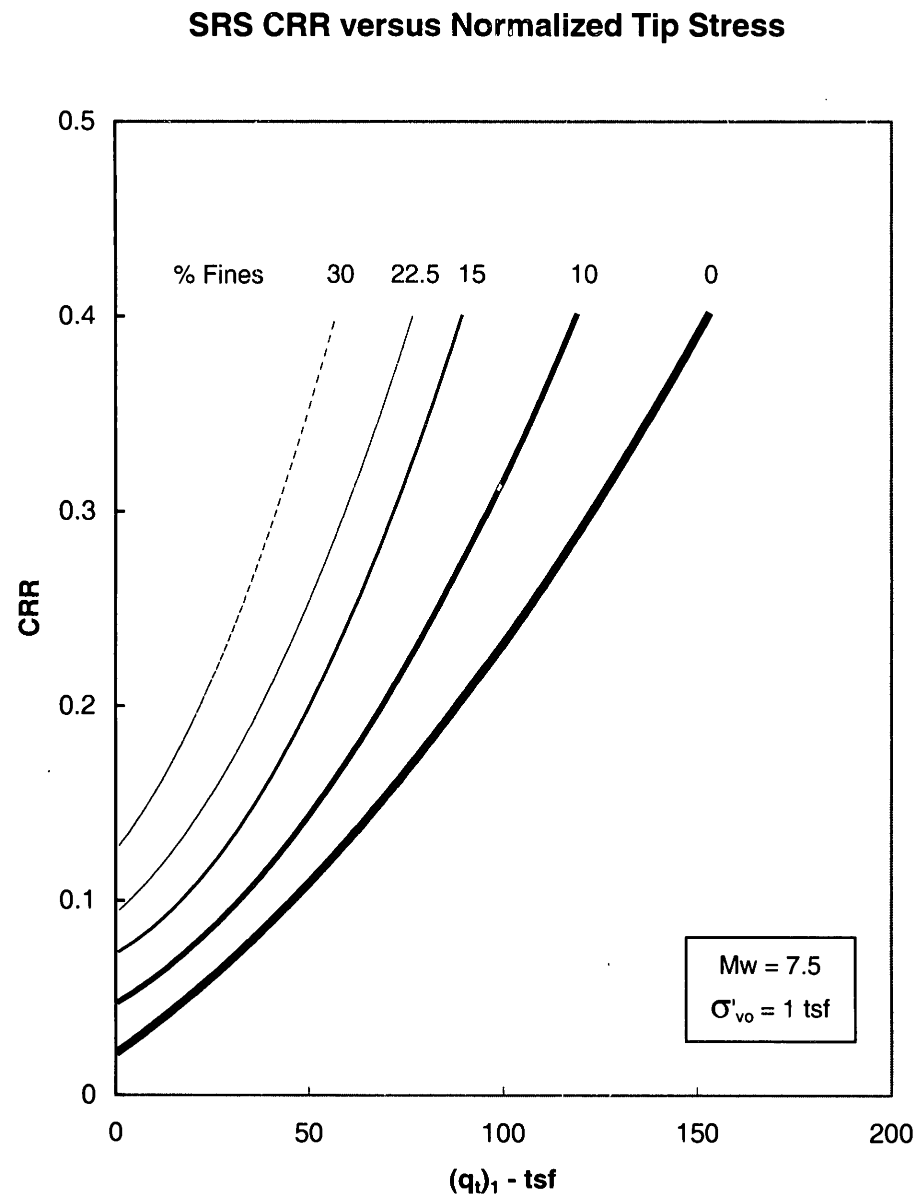

Figure 12 - Relationship Between Cyclic Resistance Ratio (CRR) Normalized CPTu Tip Resistance (q_{t1}) and Fines Content for SRS

(SRS, 1994a; 1995 and WSRC, 1995)

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Percent Finest vs Elevation

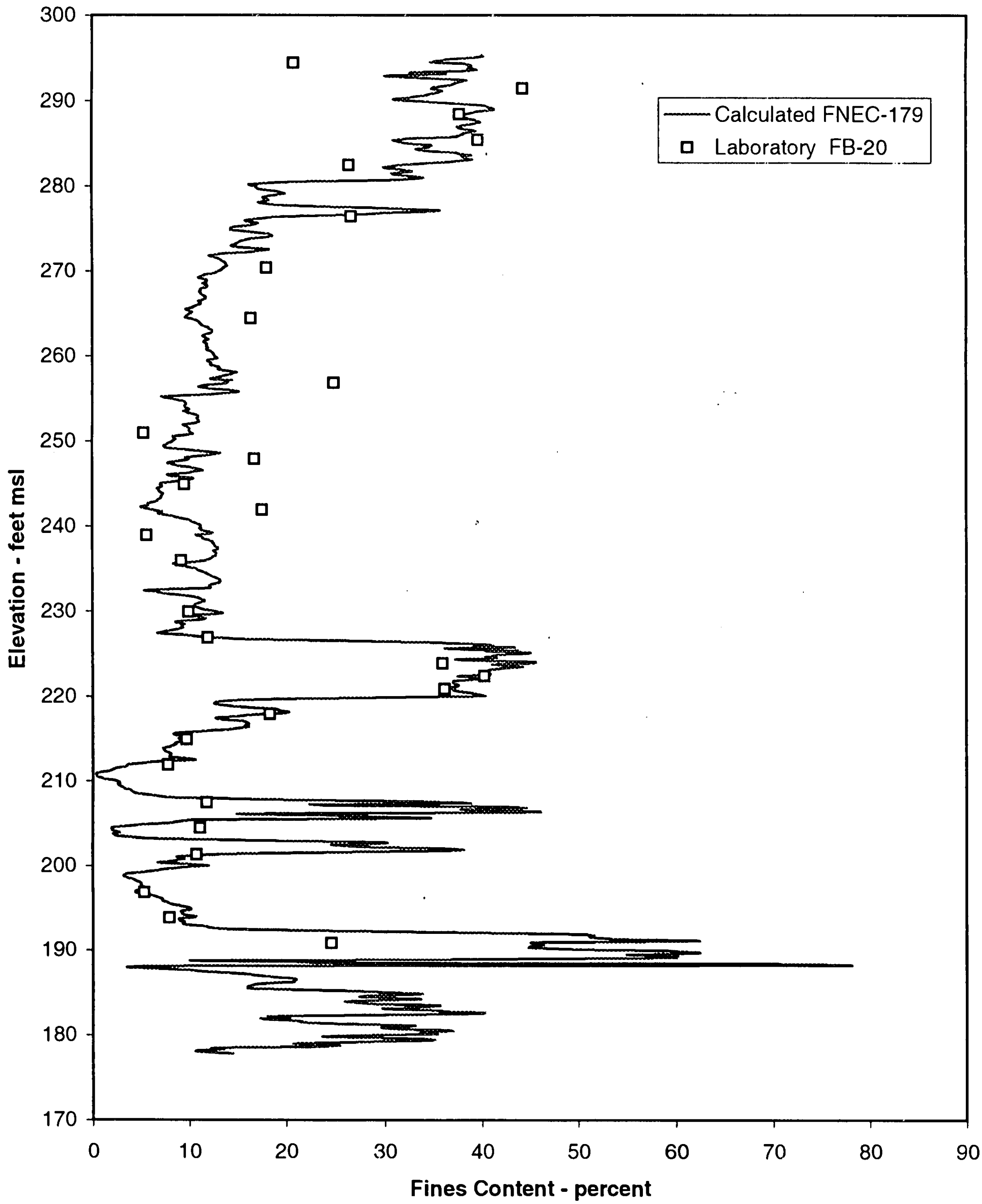
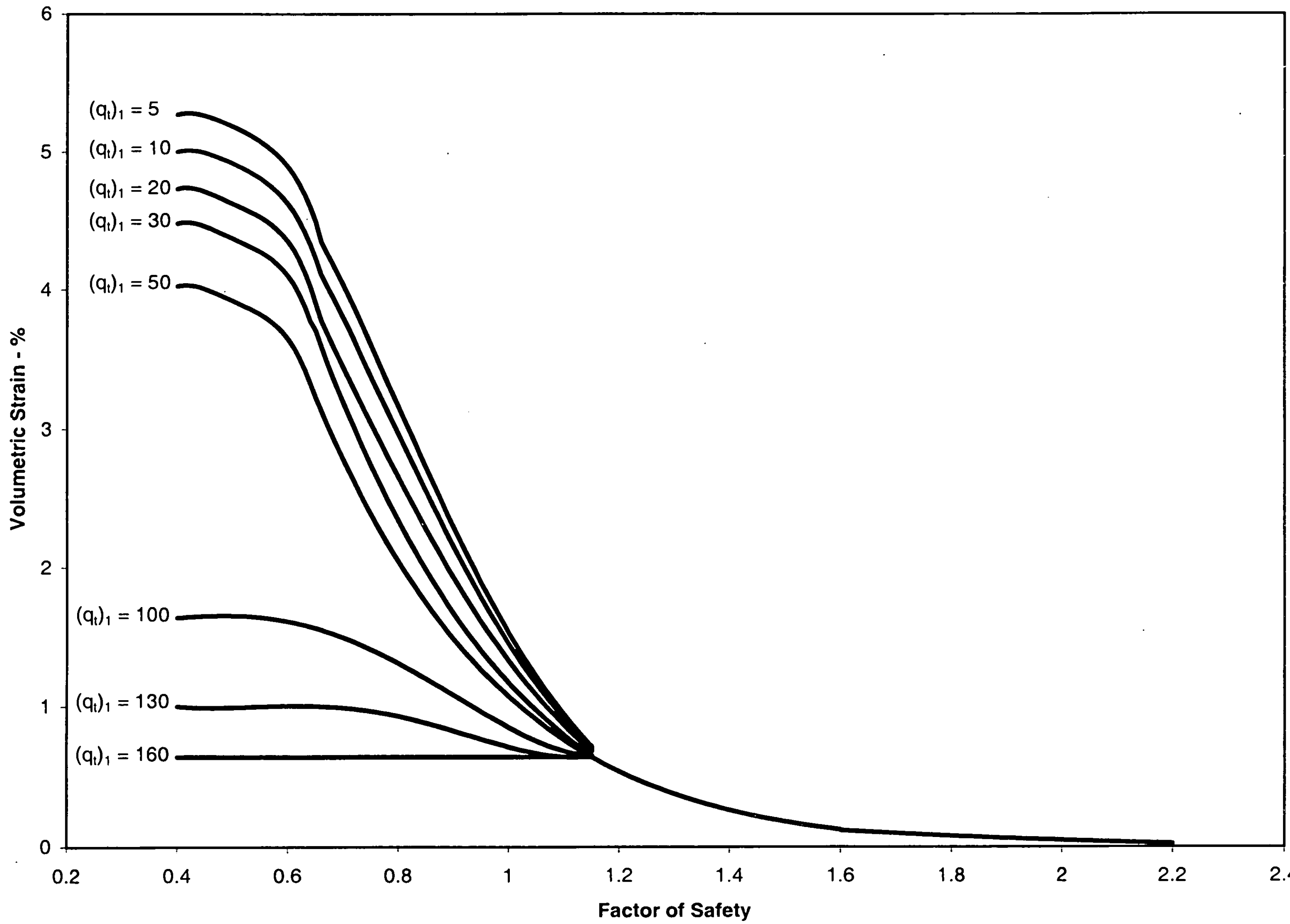


Figure 13 – Percent Fines Calculated Using CPTu Data from FNEC-179 Compared with Laboratory Determined Percent Fines for FB-20
(Laboratory data from Table 5, CPTu fines calculated using Eq. 17 and 18)

SRS Volumetric Strain versus Factor of Safety Relationship



figures.XLS

Figure 14 – SRS Relationship Between Volumetric Strain Normalized CPTu Tip Resistance $(q_t)_1$ and Factor of Safety Against Liquefaction

(SRS, 1994a; 1994c and WSRC, 1995)

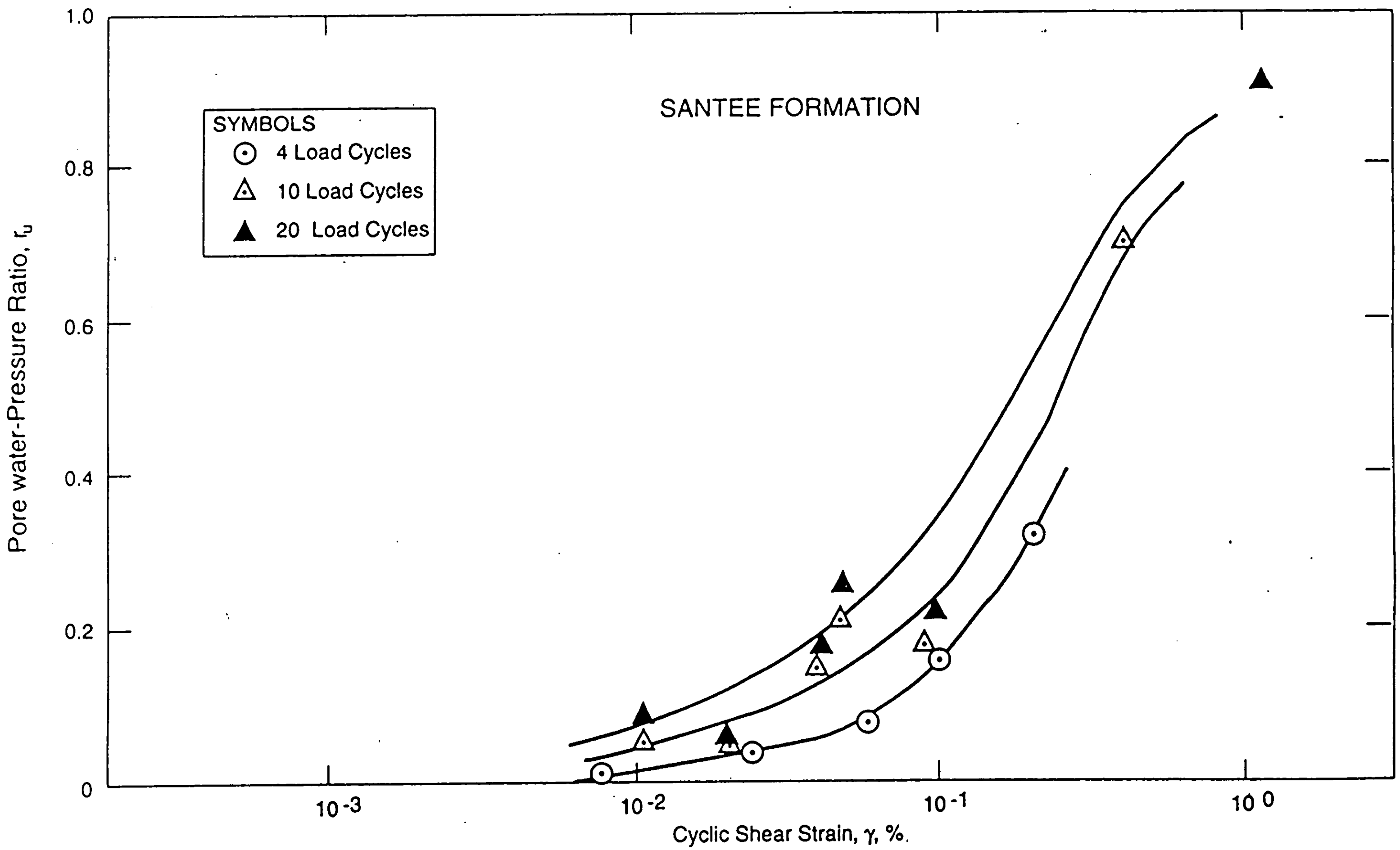


Figure 15 – SRS Relationship Between Cyclic Shear Strain Amplitude, Number of Cycles, and Pore Water Pressure Increase for Santee Formation

(WSRC, 1995 Figure 6.2-29)

Attachment A

Listing of Seismic Piezocone Penetration Test Data Files at locations FWSBC1, FWSBC2, FWSBC3, FWSBC4, FWSBC5, FWSBC6, FPDCA-11, FPDCA-15, FPDCA-39, FNEC-179, FNEC-180, FNEC-182, FNEC-183 and FNEC-186

(WSRC, 2001; LAW, 2001; ARA, 2004)

Listing of computer files containing CPTu data

Date	Time	File Size	File Name
4/23/1998	04:11a	347,536	CPT_179.ECP
4/17/1998	10:05 AM	345,306	CPT_180.ECP
4/23/1998	08:32a	387,230	CPT_182.ECP
4/17/1998	11:08a	311,856	CPT_183.ECP
4/22/1998	10:52a	442,757	CPT_186.ECP
1/2/2001	7:17 AM	489,915	CPTA_11.ECP
1/2/2001	7:18 AM	406,398	CPTA_15.ECP
3/27/2001	07:50a	421,320	CPT-A-39.ECP
11/13/2003	1:39 PM	318,993	FWSBC1
11/13/2003	1:38 PM	421,431	FWSBC2
11/13/2003	1:35 PM	467,506	FWSBC3
11/13/2003	1:34 PM	520,620	FWSBC4
11/13/2003	1:32 PM	441,759	FWSBC5
11/13/2003	1:41 PM	420,889	FWSBC6

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Attachment B

Listing of computer files containing liquefaction susceptibility and estimated settlement calculations using CPTu sounding data

The file name gives the CPTu location, the earthquake, and earthquake magnitude. The first 7 characters indicate SCPTu location, the PC3+ or CH50 indicate the earthquake and the last 3 or four characters indicate the earthquake magnitude.

Date	Time	File Size	File Name
4/23/2003	03:14p	4,489,216	FNEC182-CH50M73.xls
4/23/2003	03:11p	4,274,688	FNEC182-PC3+M475.xls
4/23/2003	03:10p	4,488,704	FNEC182-PC3+M525.xls
4/23/2003	03:09p	4,488,704	FNEC182-PC3+M575.xls
4/23/2003	03:12p	4,274,688	FNEC182-PC3+M61.xls
4/23/2003	03:09p	4,488,704	FNEC182-PC3+M625.xls
4/23/2003	03:07p	4,492,288	FNEC182-PC3+M675.xls
4/23/2003	03:07p	4,493,312	FNEC182-PC3+M75.xls
4/23/2003	03:28p	4,018,688	FNEC183-CH50M73.xls
4/23/2003	03:24p	3,898,880	FNEC183-PC3+M475.xls
4/23/2003	03:24p	4,020,224	FNEC183-PC3+M525.xls
4/23/2003	03:23p	4,020,224	FNEC183-PC3+M575.xls
4/23/2003	03:25p	4,020,224	FNEC183-PC3+M61.xls
4/23/2003	03:23p	3,848,704	FNEC183-PC3+M625.xls
4/23/2003	03:21p	4,020,736	FNEC183-PC3+M675.xls
4/23/2003	03:19p	3,850,240	FNEC183-PC3+M75.xls
4/23/2003	03:36p	6,255,104	FNEC186-CH50M73.xls
4/23/2003	03:34p	6,009,856	FNEC186-PC3+M475.xls
4/23/2003	03:33p	6,076,928	FNEC186-PC3+M525.xls
4/23/2003	03:32p	6,009,856	FNEC186-PC3+M575.xls
4/23/2003	03:35p	6,009,856	FNEC186-PC3+M61.xls
4/23/2003	03:32p	6,009,856	FNEC186-PC3+M625.xls
4/23/2003	03:30p	6,257,152	FNEC186-PC3+M675.xls
4/23/2003	03:30p	6,078,976	FNEC186-PC3+M75.xls
4/23/2003	03:03p	3,951,616	FPDCA39-CH50M73.xls
4/23/2003	02:59p	4,133,376	FPDCA39-PC3+M475.xls
4/23/2003	02:58p	4,133,376	FPDCA39-PC3+M525.xls
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4/23/2003	02:57p	3,954,176	FPDCA39-PC3+M625.xls
4/23/2003	02:56p	3,955,200	FPDCA39-PC3+M675.xls
4/23/2003	03:05p	4,029,440	FPDCA39-PC3+M75.xls
7/18/2007	11:39 AM	3,815,424	FNEC179-C50M73.xls
7/19/2007	2:07 PM	3,777,536	FNEC179-PC3+M475.xls
7/19/2007	2:06 PM	3,777,536	FNEC179-PC3+M525.xls
7/19/2007	2:06 PM	3,973,120	FNEC179-PC3+M575.xls
7/19/2007	2:08 PM	3,778,048	FNEC179-PC3+M61.xls
7/19/2007	2:05 PM	3,973,120	FNEC179-PC3+M625.xls
7/19/2007	2:05 PM	3,974,144	FNEC179-PC3+M675.xls

1

1

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7/19/2007	2:10 PM	4,163,584	FNEC180-PC3+M575.xls
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7/19/2007	2:13 PM	4,725,248	FPDCA11-PC3+M625.xls
7/19/2007	2:13 PM	4,726,272	FPDCA11-PC3+M675.xls
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7/19/2007	1:41 PM	3,260,928	FWSBC1-PC3+M61.xls
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1

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7/19/2007	2:03 PM	3,995,136	FWSBC6-PC3+M525.xls
7/19/2007	2:03 PM	3,995,136	FWSBC6-PC3+M575.xls
7/19/2007	2:04 PM	3,995,136	FWSBC6-PC3+M61.xls
7/19/2007	2:02 PM	3,995,648	FWSBC6-PC3+M625.xls
7/19/2007	2:01 PM	3,996,160	FWSBC6-PC3+M675.xls
7/18/2007	3:52 PM	4,191,232	FWSBC6-PC3+M75.xls