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ABSTRACT

Over the past decade, liquefaction assessments have been performed for many existing and planned critical facilities at the Department of Energy's Savannah River Site (SRS). The assessments incorporated site-specific Cyclic Resistance Ratio (CRR) and K_σ with the use of the cone penetration test (CPT). The SRS-specific CRR and K_σ were developed from laboratory testing of carefully collected samples. Test results show SRS soils have increased liquefaction resistance of two to three times when compared to standard literature for Holocene-age deposits. This increase in strength can be attributed to many factors such as aging and overconsolidation. The purpose of this paper is to discuss liquefaction methodologies used at the SRS. Specifically, 1) use of the CPT and correlations of CPT-derived results with that of high-quality undisturbed samples; 2) aging; and 3) K_σ vertical confining stress factor.

INTRODUCTION

Liquefaction potential evaluations are vital in the in the assessments of critical facilities. At the Savannah River Site (SRS), a Department of Energy facility in South Carolina, the federal dollar is ever shrinking. Each analysis of a facility and each step within each analysis compounds conservatism and can possibly result in unnecessary spending due to over design and construction. In some cases a "routine" liquefaction evaluation using a standard methodology may be overly conservative and an alternative approach is necessary to more accurately determine the hazard. It is also important to quantify the liquefaction hazard in relation to other natural hazards (e.g., flood, tornado, etc.) used in facility designs so that dollars can be allocated to reduce the greatest risk. Thus, it is critical that we utilize the latest engineering methodologies to quantify risk allowing informed decisions regarding the expenditure of precious funds. With that in mind, we thought it useful to present the SRS liquefaction potential assessment methodology in light of the NCEER/NSF Workshops (herein referred to as the Workshop) recommendations published in the recent state-of-the-art paper by Youd et al. (2001).

BACKGROUND

The SRS is located in west central South Carolina, along the Savannah River, see Fig. 1. The SRS is about 160 km from Charleston, SC which is the most significant seismogenic zone affecting the site. The largest earthquake in the region during historical times occurred near Charleston in 1886.

Unconsolidated sediments, ranging from about 700 feet in the northwest to 1,500 feet in the southeast, underlie the site. The near surface soils (up to about 200 feet deep) generally consist of sands, silty sands, and clayey sands of Pleistocene age and older. In general these sediments classify as clayey sands (SC) under the unified soil classification system.

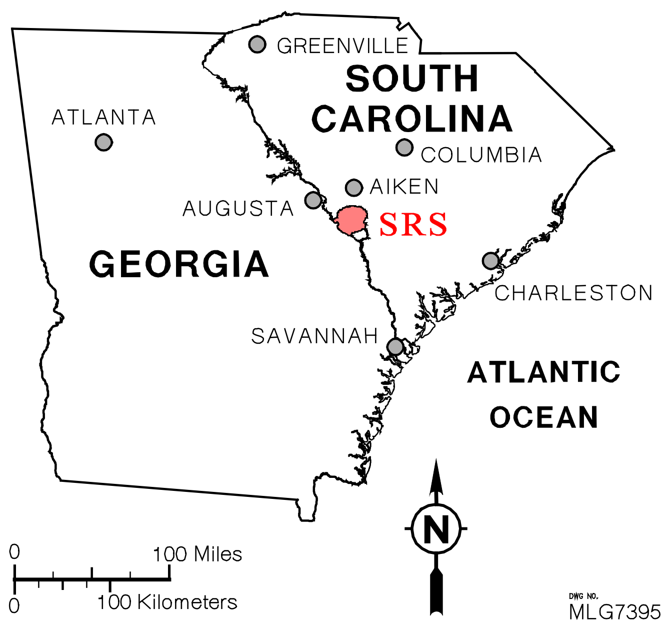

Fig. 1. Savannah River Site and Surrounding Region.

Standard penetration test (SPT) N-values range from less than 5 to over 50 blows/foot, cone penetration test (CPT) tip resistances (q_t) range from 25 to over 400 tons/square foot (tsf), and shear wave velocity (V_s) measurements range from about 850 to over 1,400 feet/second (fps). In terms of International Building Code Classification, SRS would classify as Class D, a "stiff soil site." A typical shallow (upper 200 feet) subsurface profile in the center of the SRS is shown on Fig. 2.

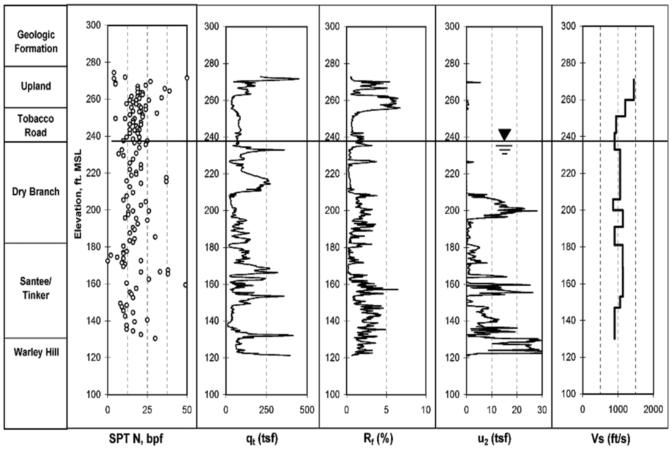

Fig. 2. Typical Shallow Subsurface Soil Profile at the Savannah River Site.

For all major facilities onsite, detailed subsurface characterization is carried out to assess site-specific subsurface conditions. In some cases this includes the drilling of deep (1,000 feet) boreholes into rock, and detailed laboratory testing. Discussion of the exploration programs carried out for critical facilities at the SRS is beyond the scope of this paper. However, the programs generally include the following attributes:

- Standard penetration test borings, including energy measurements
- Seismic piezocone penetration test soundings
- Laboratory testing, including static and dynamic testing
- Groundwater level determination
- Summary of the results in a geotechnical report

Early in our involvement with the SRS we recognized that soils at the SRS do not fit the database used to construct the so-called Seed and Idriss empirical chart (see Fig. 3). This is principally due to the age of the deposits found at the SRS, which are much older (Pleistocene and older) than the database (Holocene) used to construct the Seed and Idriss empirical chart. Thus, use of the Seed and Idriss empirical chart would, at best, lead to very conservative results in terms of liquefaction susceptibility assessments. It was concluded that critical facilities at the SRS would require an extensive sampling and testing program to determine the capacity of SRS soils to resist liquefaction. The following section reviews the "routine" methodology and points out non-routine methods used at critical SRS facilities.

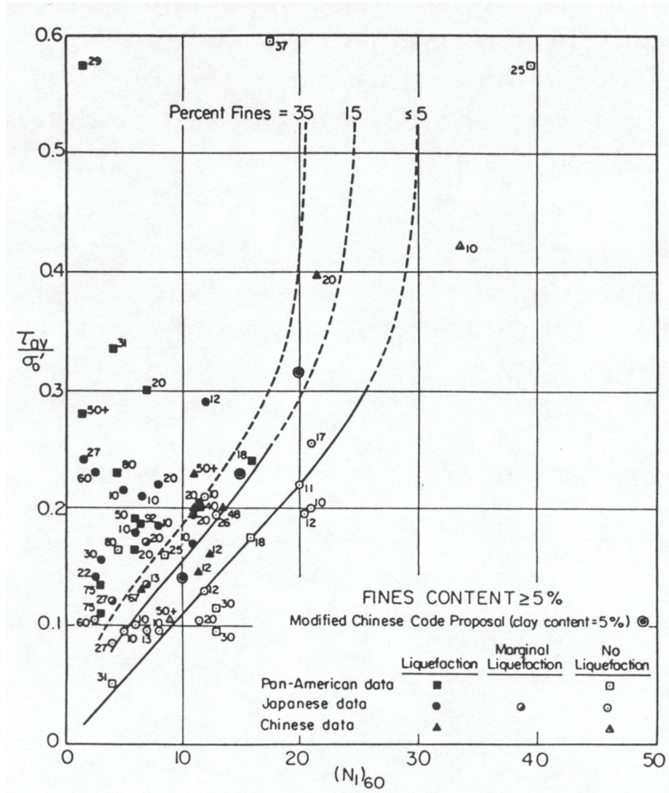

Fig. 3. Cyclic Resistance Ratio versus N-value for Magnitude 7.5 Earthquake with Data from Liquefaction Case Histories (Reproduced from Seed et al., 1985).

ROUTINE METHODOLOGY

Liquefaction, as used herein, is defined as "…the transformation of a granular material from a solid to liquefied state as a consequence of increase pore water pressure and reduced effective stress." (Marcusson, 1978). The factor of safety against liquefaction is defined as the ratio of soil capacity to resist liquefaction, expressed as Cyclic Resistance Ratio (CRR) to seismic demand, expressed as Cyclic Stress Ratio (CSR).

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

The original simplified procedure for determining CRR, often referred to as the Seed and Idriss empirical chart, utilized SPT N-values [$(N_1)_{60}$] to estimate CRR (i.e., τ_{ave}/σ'_{vo}), see Fig. 3. Over time other parameters for determining CRR have been introduced and developed, such as; normalized CPT tip resistance $(q_t)_1$ and shear wave $(V_s)_1$ (see Figs. 4 and 5). These empirical techniques are based on field observations of the performance of soil deposits in previous earthquakes, and they work reasonably well for routine projects founded on Holocene soil deposits. As mentioned previously, we recognized that the use of the empirical chart, at best, would be conservative due to the fact the empirical chart does not account for increased resistance with increased age. Although the use of Fig. 3 is appropriate to derive CRR for most facilities (particularly if the are founded on Holocene soils) it was deemed inappropriate for critical facility evaluations at the SRS. This will be discussed further in the next section.

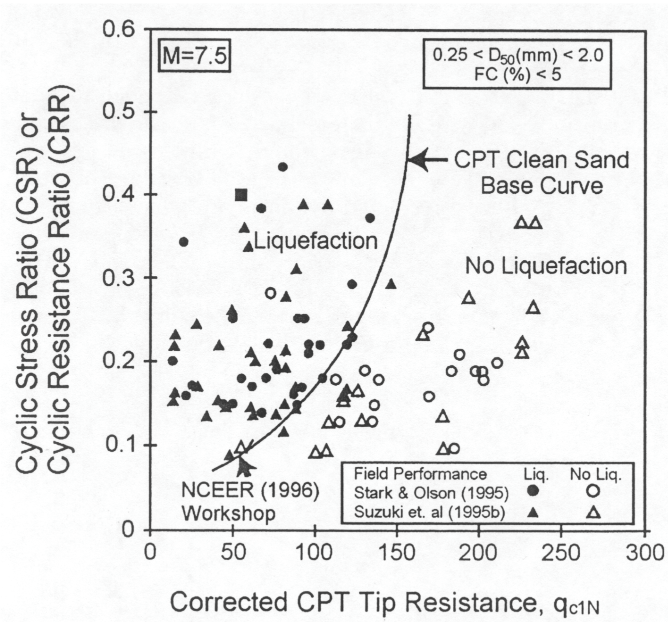

Fig. 4. Cyclic Resistance Ratio versus Tip Resistance for Magnitude 7.5 Earthquake with Data from Liquefaction Case Histories (After Robertson and Wride, 1998; Reproduced from Youd et al., 2001).

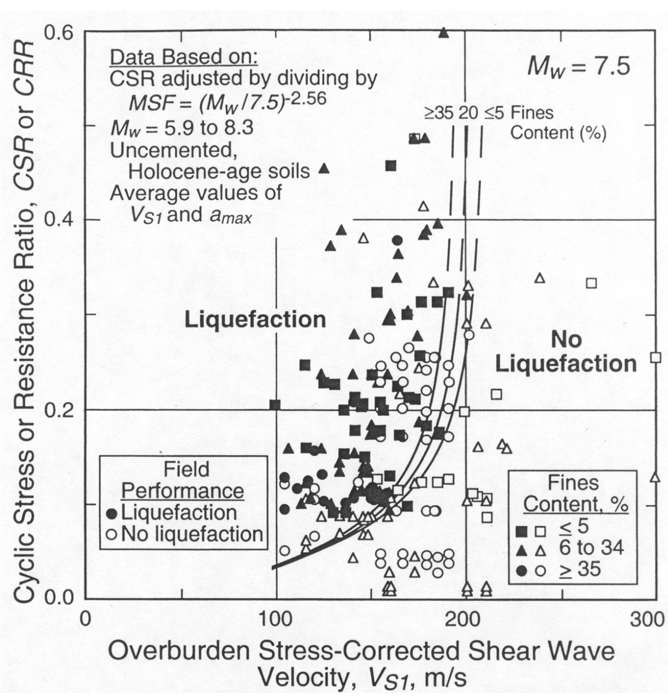

Fig. 5. Cyclic Resistance Ratio versus Shear Wave Velocity for Magnitude 7.5 Earthquake with Data from Liquefaction Case Histories (Reproduced from Andrus and Stokoe, 2000).

The CRR derived using SPT, CPT or V_s may still need to be modified to account for earthquake magnitude or other factors that influence liquefaction. Over time Equation 1 has evolved to account for these factors.

$$\text{Factor of Safety} = \frac{CRR_{7.5} \cdot MSF \cdot K_s \cdot K_a}{CSR} \qquad (2)$$

Where:

- $CRR_{7.5}$ is cyclic resistance ratio (τ_{ave}/σ'_{vo}) normalized for earthquake magnitude 7.5 (obtained from Figs. 3, 4 or 5),
- *MSF* is magnitude scaling factor accounting for magnitudes other than 7.5,
- K_s accounts for confining stress greater than 1 atmosphere (atm) (˜ 1 tsf or 100 kPa),
- K_a accounts for static driving shear stress due to sloping ground, and
- CSR is cyclic stress ratio or average shear stress induced by the earthquake divided by effective vertical overburden stress (τ_{ave}/σ'_{vo}).

Fines content also influences liquefaction susceptibility, but the empirical techniques discussed herein account for fines. A fines content correction term could be included in Equation 2.

The empirical techniques are developed (normalized) for earthquake magnitudes of 7.5. The CRR must be then be corrected for the appropriate earthquake magnitude under consideration using a magnitude scaling factor (MSF). Figure 6 presents several MSFs that have been proposed by various investigators (Youd, et al., 2001). Also presented in Fig. 6 is the range of MSFs recommended from the Workshop. Over the past decade SRS liquefaction evaluations have used MSFs developed by Arango, (1994; 1996) which fall in the recommended range for most earthquake magnitudes. Thus, SRS practice does not differ from standard or routine practice in this respect.

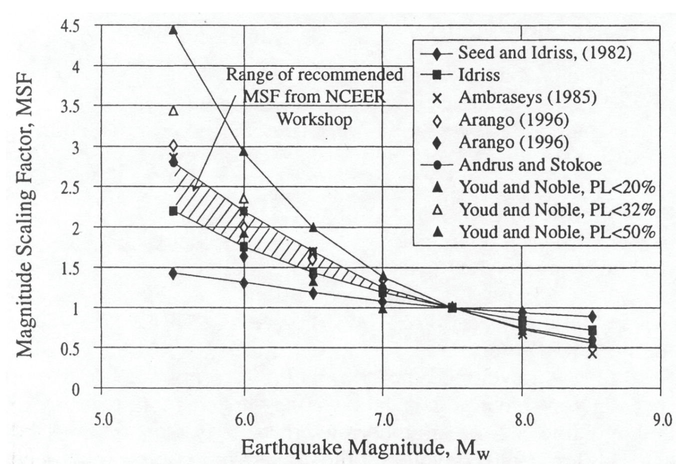

Fig. 6. Magnitude Scaling Factors Derived by Various Investigators (Reproduced from Youd et al., 2001).

Most of the case history data used to develop the standard liquefaction curves (Seed and Idriss, 1982) were taken from cases of level ground with relatively small initial effective overburden stresses ($\sigma'_{vo} \leq 1$ tsf). However, at higher effective overburden stresses ($\sigma'_{vo} > 1$ tsf), the liquefaction susceptibility of the soil will increase for a given CSR (Seed and Harder, 1990). For routine projects use of the Workshop recommended K_σ is appropriate. However, at the SRS, laboratory testing has been performed on carefully collected samples to determine a site specific K_σ. The details will be discussed in the next section.

Sloping ground induces static shear stress on horizontal planes within a soil mass. Relationships proposed by Seed and Harder (1990) suggest that a static 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_α) has been proposed by Seed and Harder to correct CRR. However, the proposed chart to estimate K_α is preliminary and this correction factor is a subject of current research (NCEER, 1997; Youd et al., 2001). For work at SRS, no K_α correction has been used. Therefore, SRS practice is routine with respect to K_α.

The "Simplified Procedure for Evaluating Soil Liquefaction Potential" uses peak ground acceleration (PGA or a_{max}) to calculate average 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; Youd et al., 2001). For routine projects equation 3 is appropriate for determining CSR. At the SRS a more rigorous approach to determining CSR is used. The details will be discussed in the next section.

$$CSR = (\tau_{ave}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo} / \sigma'_{vo})r_d \qquad (3)$$

NON-ROUTINE METHODOLOGY

Liquefaction assessments at the SRS follow the accepted practice recommended in the Workshop (Youd et al., 2001). However due to the critical nature of SRS facilities the liquefaction assessments are considered "non-routine" and more detailed assessments of liquefaction susceptibility are carried out. The details are given in the following sections.

Evaluation of Cyclic Resistance Ratio (CRR)

The standard empirical techniques for determining CRR (Figs. 3, 4 and 5) correlate field measured properties (i.e., $(N_1)_{60}$, $(q_t)_1$, and $(V_s)_1$) with field observations of the performance of soil deposits in previous earthquakes. Alternatively, CRR determined from laboratory samples can be correlated with field measured properties (i.e., $(N_1)_{60}$, $(q_t)_1$, and $(V_s)_1$). This requires a well thought out and carefully implemented sampling and testing program.

It has been the experience of the authors that the empirical based methods are more widely used in the profession than the laboratory approach. This is due in part to the difficulty in obtaining good, high-quality, undisturbed soil samples in sands and, unfortunately, to the time and expense necessary to perform a quality field sampling and laboratory testing program. For many applications the empirical approach is acceptable. However, the empirical methods may be too conservative particularly in the case of geologically older deposits and the more expensive laboratory approach may be warranted.

The influence that age has on the properties of a soil deposit has long been recognized by several researchers, such as Youd and Hoose (1977), Seed (1979), Mitchell and Solymar (1984), Mitchell (1986), Skempton (1986), and Schmertmann (1991 and 1993). Many processes occur during soil aging, such as cementation, weathering, and increased exposure to low-level seismic shaking, cold bonding and consolidation. All of these processes tend to increase the liquefaction resistance of sands.

Presently there is no verified or "standard K_{age}" correction term available in the literature. Because of the critical nature of the facilities at the SRS, a detailed exploration program including very careful undisturbed sampling and dynamic testing was carried out to determine if the soils at SRS were in fact more resistant to liquefaction. The program was developed and implemented by the authors and the dynamic laboratory testing was carried out at the University of California at Berkeley (UCB) laboratory.

The samples were obtained by careful sampling techniques using a fixed-piston sampler. The detailed sampling procedure and sampling process have been used at the SRS for quite some time. The samples were handled in such a way to minimize disturbance. Careful measurements, including X-ray, were performed on each sample tube prior to sealing and transporting and after being received in the UCB laboratory. If a sample was deemed "disturbed" it was not tested. Thus, only the highest quality samples were retained for testing. Even so, it is recognized that there truly is no such thing as an "undisturbed" sample. However, in general it's the belief of the authors that disturbance would tend to reduce the dynamic strength not increase it. Thus, the results obtained should be on the conservative side. The testing included the determination of dynamic strength, volumetric strain after liquefaction and an evaluation of confining pressure, leading to site-specific recommendations for K_σ. All of the test results were correlated back to a sample specific $(N_1)_{60}$, or $(q_t)_1$. The results are summarized in a series of plots discussed in the following sections.

Standard Penetration Test (SPT). Site-specific sampling and testing was performed resulting in a series of 17 stress-controlled, isotropically consolidated undrained triaxial tests. The samples were obtained adjacent to (within 5 to 10 feet) SPT boreholes at locations exhibiting low N-values. Note, the samples were not obtained within an SPT boring in order to reduce possible disturbance from the "driving" of SPT samples above the location of the undisturbed sample. To aid in the evaluation, SPT energy measurements were obtained and used later to correct the field N-values to N_{60}. An example of the results, for material with an $(N_1)_{60}$ of about 6 and a fines content of about 15%, in terms of field cyclic stress ratio versus number of cycles to liquefaction (N_L) is shown on Fig. 7.

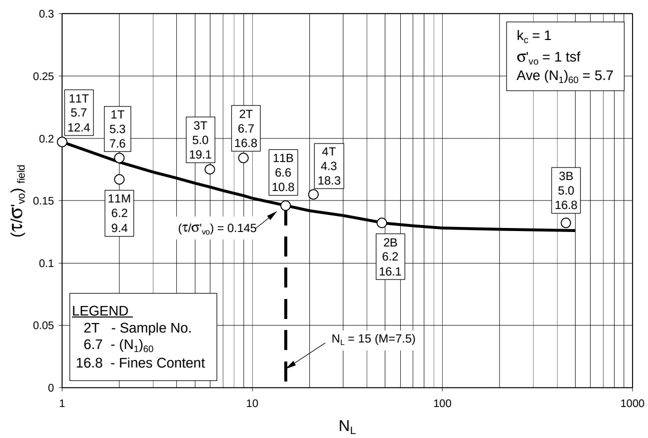

Fig. 7. Number of Cycles Causing Initial Liquefaction versus Stress Ratio, $(N_1)_{60}$ = 5.7.

For these initial evaluations using laboratory-derived strengths, the samples tested were all from the same geologic formation, known as the Tobacco Road Formation. Thus, even though the fines content of the samples varied, a single "CRR design curve" for these soils was established, and is shown on Fig. 8. The overall assessment resulted in three data points relating $(N_1)_{60}$ to CRR, as shown on Fig. 8. The samples tested to develop the SRS curve had fines content ranging from about 8 to 33%, with an average of about 15%. For comparison, the Seed and Idriss empirical curves are also shown in Fig. 8. At a $(N_1)_{60}$ of about 5 the strength increase is about 40 to 50%, which is attributed to aging.

<u>Cone Penetrometer Sounding (CPT)</u>. Subsequent to the relationship developed for the SPT, a similar relationship was developed for the CPT. First of all, CPT soundings were pushed adjacent to the borings where the samples described above (SPT) were obtained in order to correlate CPT $(q_t)_1$ with CRR. In addition, 18 high quality fixed-piston samples were obtained at other site locations in the same way described above from boreholes adjacent to 18 CPT soundings and sent to the UCB laboratory for dynamic testing (stress-controlled, isotropically consolidated undrained triaxial tests). The laboratory results were evaluated in the same way described above except that the CPT $(q_t)_1$ was utilized instead of $(N_1)_{60}$. Figure 9 shows the result of that testing in terms of CRR versus $(q_t)_1$ for the shallow soils at SRS. Note that although 35 tests were performed only 10 data points are shown. This is due to grouping of like material in terms of fines content and $(q_t)_1$. However, unlike the initial SPT evaluation, a suite of curves, based on fines content, was established as shown on Fig. 9. For comparison the CPT curve recommended in the Workshop for 15% fines is also shown. At a $(q_t)_1$ of 25 tsf the strength increase is about 50%. Once again, the increase in strength is attributed to aging.

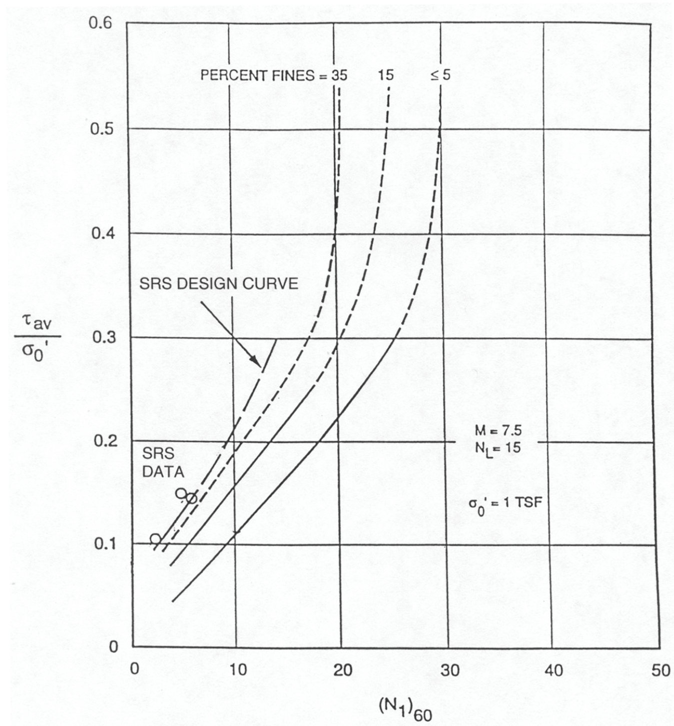

Fig. 8. Savannah River Site Cyclic Resistance Ratio versus $(N_1)_{60}$ for Magnitude 7.5 Earthquake Compared with Seed and Idriss (1985) Empirical Curves.

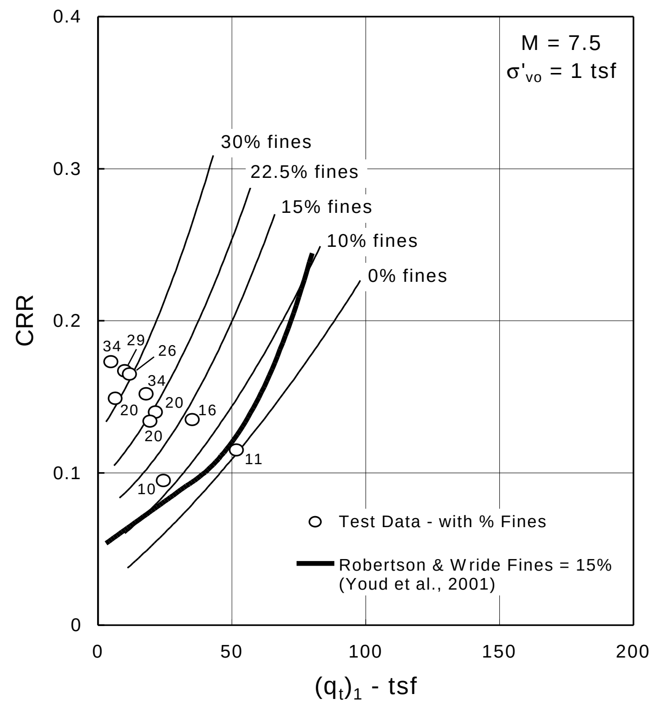

Fig. 9. Savannah River Site Cyclic Resistance Ratio versus $(q_t)_1$ for Magnitude 7.5 Earthquake Compared With Robertson and Wride.

We have found the CPT sounding to be particularly useful for liquefaction assessments, both in terms of technical attributes and cost. When combined with soil sampling (SPT) and V_s determinations, the CPT is a very valuable tool.

<u>Shear Wave Velocity (V_s).</u> At the SRS investigations for critical facilities include measurement of shear-wave velocities. However, laboratory test results have not been correlated back to a sample specific V_s as has been done for $(N_1)_{60}$ and $(q_t)_1$. The Andrus and Stokoe (2000) shear-wave method (see Fig. 5) was developed for Holocene and younger sands, but should show increased liquefaction resistance for older soils, since shear wave velocity increases with time.

An average V_s at the SRS is about 1,000 fps (see Fig. 2) with a typical water table depth of about 30 feet. At depths between 30 and 90 feet the effective vertical stress (σ'_{vo}) is between 1.8 and 3.5 tsf. The resulting normalized shear velocity $(V_s)_1$ falls between ~ 725 and 860 fps, or ~ 220 and 260 meters/second (mps). According to Andrus and Stokoe, a $(V_s)_1$ above 210 mps indicates liquefaction is not expected. The average $(V_s)_1$ at the SRS is typically greater than the threshold value given by Andrus and Stokoe, thus liquefaction although not quantitative in terms of safety factor, is not expected. This is consistent with the results obtained using the SPT and CPT correlations for the SRS soils. It should be pointed out that, site-specific V_s measurements made using the CPT are key to liquefaction assessments at the SRS, even in the absence of a quantifiable safety factor. Measurement of V_s also facilitates ground response analysis and evaluation of CSR, which will be discussed later.

Absence of a quantifiable CRR using V_s would indicate that the increase in V_s over time is more significant than the increase in N-value or q_t with time. The use of V_s to determine CRR may actually account for the influence of aging. However, Andrus and Stokoe advise caution when site conditions are not well understood as weak interparticle bonding can increase V_s while not necessarily increasing CRR.

<u>Correction for Age</u>

Aging has been addressed through the laboratory testing of high quality samples described above. These results are inherently incorporated into the CRR relationships for the SRS soils (Figs. 8 and 9) thus there is no specific "K_{age}" factor to be applied to Equation 2. We believe the dynamic strengths utilized for the SRS soils are more realistic (not less conservative, but more representative) than strengths derived from the Seed and Idriss empirical chart. The results show that the soils at the SRS have significantly higher cyclic shear strengths than similar soils of the Holocene period, as discussed previously. Figure 10 from Arango, et al., (2000) presents these results in a slightly different way demonstrating the role aging plays in dynamic strength. As pointed out by Youd et al., 2001, when accounting for age "engineering judgement is required to estimate the liquefaction resistance of sediments more than a few thousand years old." Thus, caution must be used when using charts such as Fig. 10.

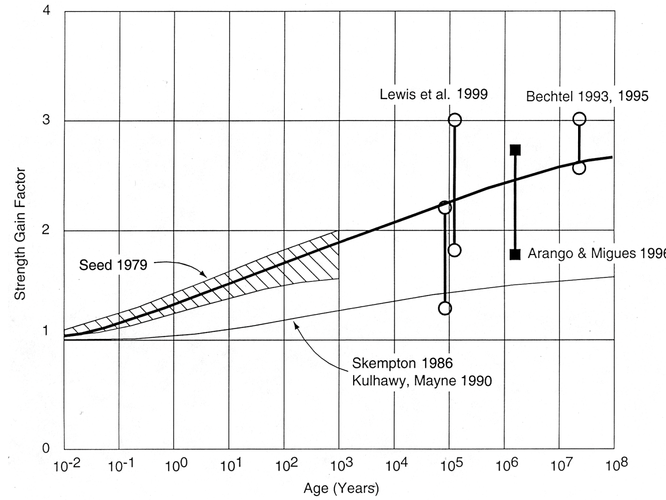

Fig. 10. Influence of Age on Relative Strength Against Liquefaction (Arango et al., 2000).

<u>Correction for High Overburden Stresses, K_σ</u>

For critical facilities it may be necessary to evaluate liquefaction potential to depths with σ'_{vo} much greater than 1 atm. In addition, the available literature K_σ curves are considered to be "minimal or conservative" (Youd et al., 2001). Therefore laboratory testing of SRS soils have been utilized to determine appropriate K_σ for SRS (BSRI, 1993; 1995).

The methodology utilized to determine the site-specific K_σ is the same as that proposed by Harder (1988), which is the ratio between the cyclic stress ratio required for initial liquefaction under an initial confining pressure (σ'_{3c}) relative to that under a confining pressure of 1 tsf. Figure 11 shows the SRS site-specific K_σ relationship, along with the data used to develop the curve (BSRI, 1995). The "minimal or conservative" K_σ curves from the Workshop are also shown on Fig. 11 for comparison. The SRS results are coincident with an f of 0.9 and have the same trend as the other Workshop relationships.

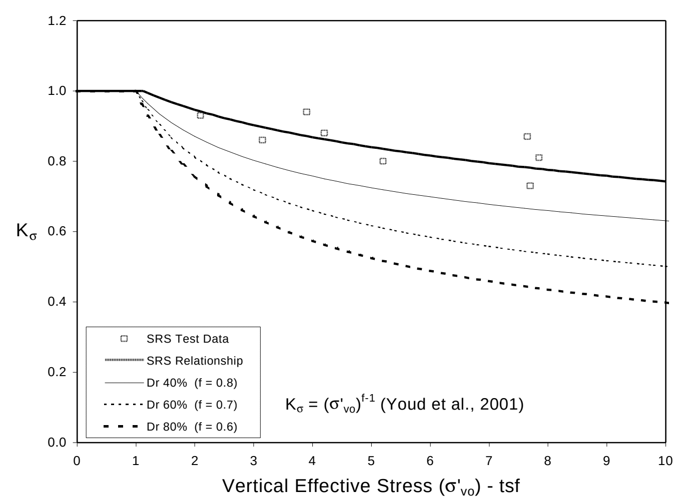

Fig. 11. Comparison of SRS and Workshop K_s relationships.

Evaluation of Cyclic Stress Ratio (CSR)

For routine projects use of the simplified equation (Equation 3) is appropriate for determining CSR. However, for critical facilities a more rigorous approach may be warranted. At the SRS, shear stress (τ_{ave}) is calculated using the one-dimensional wave propagation (e.g., the computer program SHAKE). We believe using one-dimensional wave propagation more accurately accounts for site soil conditions. Thus, for every major liquefaction assessment at the SRS, convolution studies are carried out to determine the CSR (or seismic demand). The effort consists of defining the earthquake motion, in terms of an acceleration time history (preferably a few), at the top of rock and convolving that motion up through the subsurface profile (accounting for site variation) to the ground surface. The resulting shear stresses and strains are then used in subsequent analyses.

Using one-dimensional wave propagation and equivalent-linear computer programs requires modulus reduction and damping curves. At the SRS, a site-wide sampling and dynamic soil testing program was developed and performed to determine the non-linear behavior of shear modulus and hysteretic damping ratio as function of shear strain. The details of which are beyond the scope of this paper. Suffice to say that the testing program was extensive enough to include statistics describing expected variation for the various SRS soil formations (Stokoe et al., 1995; Lee, 1996). The result is a series of site-specific shear modulus reduction and material damping versus shear strain relationships (Figs. 12 and 13) for the onsite soils, grouped by geologic layer and material type.

Using one-dimensional wave propagation and equivalent-linear computer programs allows for variation of soil parameters (e.g., shear wave velocity, modulus and damping) and earthquake input (e.g., distance, magnitude and PGA) making it possible to quantify uncertainty. Analyzing variation and uncertainty is necessary for probabilistic assessments.

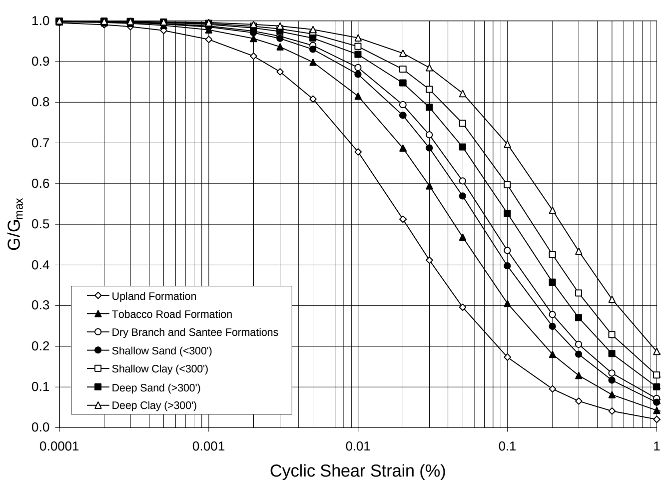

Fig. 12. Savannah River Site Dynamic Shear Modulus.

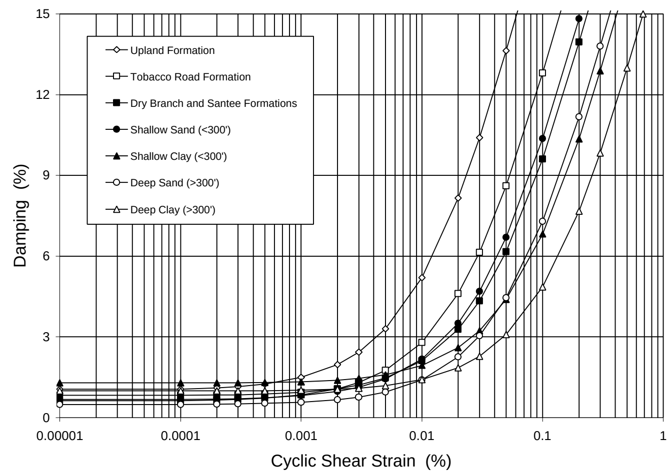

Fig. 13. Savannah River Site Dynamic Damping Ratio.

Evaluation of Dynamic Settlement

Determination of the factor of safety against liquefaction is only half of the problem. Whether a site experiences liquefaction or not, it will undergo some amount of distortion as a result of the dissipation of excess pore water pressures generated from the seismic shaking. That distortion can occur at safety factors well in excess of unity. At the SRS we also estimate the dynamic settlement that may occur as a result of seismic shaking. The amount of dynamic settlement is a function of many of the factors already discussed.

As mentioned previously, volumetric strain determinations were made in the laboratory on many of the samples tested. The results were correlated to a sample-specific $(q_t)_1$ and the computed factor of safety for that particular sample. The results are summarized in a plot between volumetric strain and factor of safety for various values of $(q_t)_1$, as shown on Fig. 14. The results follow a similar trend for clean sands developed by Ishihara and Yoshimine (1992). For comparison, some of the curves developed Ishihara and Yoshimine at various values of $(q_t)_1$ and factor of safety are also shown on Fig. 14.

The results of volumetric strain measurements from laboratory tests are dependent on many factors including type of test (cyclic triaxial vs. simple shear) number of stress applications after initial liquefaction takes place, volume measurement technique, stress level, etc. For these reasons, comparisons of volumetric strains reported in published studies should examine trends rather than specific numerical values. However, as noted on Figure 14, the trend for the SRS soils is almost identical to that of Ishihara and Yoshimine at factors of safety greater than unity. For values less than unity the trend is similar, however less severe at corresponding values of $(q_t)_1$. Both sets of data indicate that maximum volumetric strain is independent of safety factor at low factors of safety, and both sets of data indicate an upper limit of volumetric strain of approximately 4 to 6%. The differences can be attributed to the soils themselves, age and fines content and to the type of tests carried out. The SRS soils were tested in cyclic triaxial while the Ishihara and Yoshimine tests were cyclic simple shear. In any event, the SRS relationships are utilized for dynamic settlement estimates at the SRS.

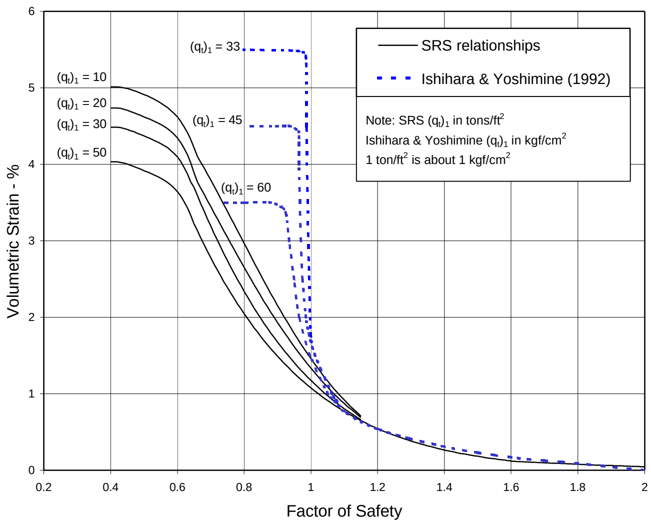

Fig. 14. Savannah River Site liquefaction volumetric strain relationship versus Ishihara & Yoshimine (1992).

Probabilistic Assessment

The Workshop considered other topics, one of which was probabilistic analysis. Although the Workshop acknowledged that some limited probabilistic liquefaction analysis has been performed, they fell short of recommending this type of analysis because probabilistic procedures are still evolving. However, we have used probabilistic assessments as a tool to help us bridge the gap between deterministic analysis (factor of safety) and risk.

In 1996 the United States Geologic Survey (USGS) made available deaggregated seismic hazard for many major cities in the US (Frankel et al., 1996 and 1997). Therefore, the probability of a given ground motion and the earthquake magnitudes contributing to that ground motion are available for many US cities. Thus, the probability of liquefaction (POL) at a particular site can be calculated utilizing the USGS results.

The details of the SRS probabilistic assessments are beyond the scope of this paper. However the general methodology employed is as follows: 1) establish the probability of occurrence of bedrock motion based on a probabilistic seismic hazard assessment, for example, USGS; 2) define the critical layer that may be susceptible to liquefaction; 3) estimate distributions of CSR (i.e., seismic demand) for the critical layer using site-specific soil properties; 4) estimate capacity (e.g., $(N_1)_{60}$) of the critical layer; and 5) sum the probability of liquefaction for each range of bedrock motion using empirical data correlating demand and capacity with liquefaction.

The POL is obtained by evaluating the probability of occurrence of specific earthquakes and the probability of liquefaction given the occurrence of a specific earthquake. Given those evaluations, the probability of liquefaction for a specific earthquake is:

$$P_E(L) = P[L \mid E]\, P[E] \qquad (4)$$

Where $P_E(L)$ is the probability of liquefaction as a result of an earthquake; $P[L \mid E]$ is the conditional probability of liquefaction given that the earthquake occurs; and P[E] is the probability that the earthquake occurs. The total overall POL is obtained by summing over all possible earthquakes, as follows:

$$P[L] = \sum_E P[L \mid E]\, P[E] \qquad (5)$$

The model for conditional probability of liquefaction developed by Liao, et al. (1988) or Youd and Noble (Youd et al., 2001) can be used. In the cases that we have analyzed the results indicate the annual POL for the shallow sediments at the SRS is around 1×10^{-5}.

The Workshop mentions two SPT-based probabilistic methodologies, Liao et al., 1988 and Youd and Noble (Youd et al., 2001). Methods for evaluating probability of liquefaction based on CPT were not presented in the Workshop. However, other investigators have been developing probabilistic methods utilizing CPT data (e.g., Toprak et al., 1999 and Juang, 2000).

CONCLUSIONS

In general, the liquefaction assessment methodology employed at the SRS is the same as outlined in the Workshop. The main difference being the recognition that aging plays a major role in liquefaction susceptibility assessments. Due to the critical facilities located at the SRS, extensive sampling and laboratory testing programs have been conducted to better quantify soil capacity and seismic demand. These differences become critical to the analysis as safety factors approach unity. Our experience indicates the following:

1. Aging plays a significant role in the dynamic strength of soils,
2. For critical facilities, site-specific laboratory testing, may have to be performed, particularly when the foundation soils are older than Holocene,
3. The seismic piezocone penetrometer test, coupled with soil sampling, is a very valuable tool for liquefaction assessments, and
4. Detailed site-specific ground response analysis, coupled with deaggregated seismic hazard can be an important tool for quantifying seismic demand and assessing probability of liquefaction. Thus shedding more light on risk.

In terms of liquefaction assessments we still have much to learn. However, the recent summary by Youd et al. (2001) presents the latest framework with which to proceed. When combining these recommendations with site-specific testing, we believe the liquefaction potential of the soils at the SRS is very low. Thus, the design of critical facilities at the SRS can concentrate on other higher risk accident scenarios allowing informed decisions regarding cost and benefit of proposed remediation or alternate designs.

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