13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2843

A METHOD TO ASSESS THE LIQUEFACTION POTENTIAL OF AGED SEDIMENTS

Sarah L. GASSMAN[1], Evangelia LEON[2] and Pradeep TALWANI[3]

SUMMARY

Currently available empirical correlations that are used to determine liquefaction resistance of sand deposits from in-situ soil indices were derived from the liquefaction of young Holocene sand deposits. This is a limitation when using in-situ soil data from paleoliquefaction sites with old sands because the current soil resistance is not necessarily representative of the soil resistance at the time of the prehistoric earthquake. Processes such as cementation at particle contacts and increasing frictional resistance resulting from particle rearrangement and interlocking contribute to an increase in strength and stiffness over time. This work developed a method to correct the in-situ soil indices to account for the effect of aging on the liquefaction resistance of old sand deposits using geotechnical data. Four sites in the South Carolina Coastal Plain (SCCP) where sandblows associated with prehistoric earthquakes have been found were studied. The sand deposits at these sites are currently 200,000 to 450,000 years in age. New boundary curves were developed for the aged SCCP soils and indicate that old soil deposits are more resistant to liquefaction than Holocene age soils. Therefore, accounting for aging when assessing liquefaction resistance yields less conservative results regarding the current liquefaction potential than when age is not considered.

INTRODUCTION

Sand deposits encountered in the South Carolina Coastal Plain (SCCP) are older than 100,000 years. Current empirical correlations that are used to determine liquefaction resistance of sand deposits from in-situ soil indices, such as the one proposed by Youd and Idriss [1], are not valid for the sand deposits in the SCCP because they were derived from relatively young Holocene (<10,000 years) sand deposits. In addition, they are based on historical earthquake events in California, China and Japan, which have different style of faulting and site characteristics than the SCCP. Increase in strength and stiffness of sand with time, a phenomenon known as aging, has been reported by many researchers [2,3,4,5,6].

[1] Associate Professor, Department of Civil and Environmental Engineering, University of South Carolina, Columbia, SC 29208, E-mail: gassman@engr.sc.edu
[2] Florence & Hutcheson, Inc., Consulting Engineers, 2700 Middleburg Drive; Suite 150, Columbia, SC 29204, E-mail: lleon@flohut.com.
[3] Professor, Department of Geological Sciences, University of South Carolina, Columbia, SC 29208, E-mail: talwani@geol.sc.edu

Investigation into the different mechanisms that cause aging in sands has not provided explicit evidence so far but is generally focused on chemical and mechanical mechanisms. Chemical mechanisms involve the formation of cementing bonds at particle contacts mainly due to the precipitation of silica from solution [2,7,8]. Mechanical mechanisms involve increasing frictional resistance due to gradual rearrangement of soil particles to a more stable system during secondary consolidation [5,6,9].

Aging has become evident mainly by penetration resistance measurements, where it is reflected in higher blow counts or tip resistance [2,4,10]. Large increases in penetration resistance with time have also been observed following the use of ground modification techniques [7,5,6] such as vibrocompaction and blast densification. Arango and Migues [9] and Lewis et al. [11] have concluded that the field cyclic strength of sand also increases with geologic age, and that this strength increase is not entirely reflected in the penetration resistance. Provided that the most commonly used methods to assess liquefaction potential relate the field cyclic strength (CRR) with in-situ soil tests such as the standard penetration test (SPT), the cone penetration test (CPT), and the shear wave velocity test (V_s), the liquefaction resistance is likely to be underestimated when using conventional SPT, CPT, or V_s correlations for sand deposits older than Holocene. There are at present no empirical charts relating SPT blow count, CPT tip resistance, or shear wave velocity V_s to the field cyclic strength of soil deposits of different geological ages.

This paper presents a methodology to include the effect of aging in the evaluation of liquefaction potential in the SCCP. The method is based upon the currently existing empirical boundary curves for Holocene age soils and in-situ test results from four sites in the SCCP where sandblows associated with prehistoric earthquakes were discovered.

DATA

The geotechnical data used in this work were first examined by Hu et al. [12]. These included SPT, CPT and V_s tests performed in the vicinity of known sandblows at four paleoliquefaction sites in the SCCP. These tests were also performed where no sandblows were observed. The four sites were: Ten Mile Hill sites A and B with sand deposits of 200,000 years in age, and Sampit and Gapway sites with sand deposits of 450,000 years in age. Talwani and Schaeffer [13] associated the paleoliquefaction features found at these sites with seven prehistoric earthquakes. The geotechnical properties of the source sands calculated from the SPT, CPT and V_s data at the Gapway and Sampit sites are summarized in Table 1.

In-situ tests at the Sampit site were performed along a northwest-southeast-trending drainage ditch approximately 500 m in length (SAM-01 to SAM-06). SAM-02 and SAM-05 were in the vicinity of two small sandblows, and SAM-04 was in the vicinity of four large sandblows. Paleoliquefaction features found in SAM-02, SAM-04 and SAM-05 were associated with three earthquake episodes that occurred 546, 1021 and 1648 years ago, respectively [13]. At the Gapway site, CPT and shear-wave velocity tests were conducted at five locations (GAP-01 to GAP-05). SPT tests were carried out at each location except GAP-04. Paleoliquefaction features found in GAP-02 and GAP-03 were associated with two earthquake episodes that occurred 3548 and 5038 years ago [13]. The sandblow discovered at GAP-04 was not associated with a prehistoric earthquake, therefore it is assumed that it was formed during the same earthquake as the nearby sandblow at location GAP-03, that is 5038 years ago.

Paleoliquefaction features at Ten Mile Hill site A were associated with an earthquake that occurred 3548 years ago [13]. No sandblows had been discovered at Ten Mile Hill site B, even though widespread liquefaction was reported to have occurred in that location during the 1886 Charleston earthquake. Geotechnical tests were conducted at five locations at site A (TEN-01 to TEN-05) and five locations at site B (TEN-06 to TEN-10).

Table 1. In-situ Geotechnical Data for Source Sands (after Hu et al. [12])

Site	Location	z (m)	h (m)	σ_o (kPa)	σ'_o (kPa)	$(N_1)_{60}$	q_{c1} (MPa)	V_{s1} (m/s)	Fines (%)	D_{50} (mm)
Gapway	GAP-01	2	0.7	36	36	10	3.1	181	N/A	-
	GAP-02	2	0.9	36	36	11*	5.5	220	9	0.15
	GAP-03	2	1.0	36	36	11	8.3	177	6	0.19
	GAP-04	2	1.1	36	36	8	7.9	240	N/A	-
	GAP-05	2	1.3	36	36	16	8.6	154	5	0.20
Sampit	SAM-01	4	5.7	71	55	14*	10.9	277	3	0.17
	SAM-02	6	4.3	108	73	14*	10.4	250	1	0.16
	SAM-03	5	5.2	89	64	14*	7.4	288	0	0.20
	SAM-04	5	5.4	89	61	14	7.7	291	2	0.18
	SAM-05	5	5.8	89	57	16	9.0	334	4	0.20
	SAM-06	5	5.6	89	61	9	7.7	321	4	0.16

z: depth of the middle of source sand layer; h: thickness of source sand layer; σ_o, σ'_o: total overburden stress and effective overburden stress at the middle point of source sand layer; $(N_1)_{60}$: corrected SPT blow count number; q_{c1}: corrected CPT tip resistance; V_{s1}: normalized shear-wave velocity; Fines: percentage by weight passing through US #200 sieve; D_{50}: grain diameter corresponding to 50% passing) the #200 sieve. *The blow count values at SAM-01 to SAM-03 are based on data from SAM-04 and at GAP-02 to GAP-05 on the data from GAP-03.

The work presented herein considers that the in-situ properties of the source sand at the location of a sandblow provide a liquefaction susceptibility index of sand that liquefied as many years ago as the occurrence date of the prehistoric earthquake that is associated with it. Assuming that destruction of the pre-earthquake soil structure takes place during liquefaction [14] the age of the source sand at the sandblow locations is equal to the occurrence date of the associated earthquake. In addition, it is assumed that the in-situ properties of the source sand at the locations where no sandblows were discovered provide a liquefaction susceptibility index of sand that never liquefied in the past therefore its age is equal to the geologic age of the deposit. Consequently, the age of the source sand at all five locations in Ten Mile Hill site A is 3548 years. At Ten Mile Hill site B where no sandblows were discovered the age of the source sand is equal 200,000 years. For the Sampit and Gapway sites, it is believed that the geotechnical test data in the vicinity of the sandblows are representative of those at the location of the sandblows. So, it is assumed that the age of the source sand at SAM-02, SAM-04, SAM-05, GAP-02, GAP-03, and GAP-04 is 546, 1021, 1648, 3548, 5038, and 5038 years, respectively, and 450,000 years at the non-liquefied sites.

EVALUATION OF LIQUEFACTION POTENTIAL

One of the most widely accepted methods for evaluating soil liquefaction resistance is the Seed et al. [15] Cyclic Stress method. The approach is based on field observations of the performance of sand deposits that did or did not liquefy in previous earthquakes worldwide. These data have been used to generate simplified curves, which relate surface phenomena such as sandblows (boils), intrusive dikes, or lateral spreading to subsurface liquefaction. The method is based on comparing the earthquake-induced (horizontal) cyclic shear stress to the cyclic resistance of the soil.

The earthquake-induced cyclic shear stress along with the soil strength and duration of shaking are all incorporated in to the cyclic stress ratio (CSR) as follows:

$$CSR = 0.65 \cdot \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_o}{\sigma_o'} \right) \cdot r_d \tag{1}$$

where a_{max} is the peak horizontal ground surface acceleration; g is the acceleration due to gravity; σ_o is the total overburden stress; σ_o' is the effective overburden stress; and r_d is the depth-related stress reduction factor decreasing from 1 at the ground surface to 0.9 at depth of 10 m.

The cyclic resistance of the soil or its resistance to pore pressure build-up is represented by the cyclic resistance ratio (CRR). The CRR has been correlated with in-situ soil indices such as the SPT blow count, the CPT tip resistance and the shear wave velocity of the soil.

The SPT-based empirical relationship proposed by Seed et al. [15] with minor modification as recommended by Youd and Idriss [1] that is valid for $(N_1)_{60}$ less than 30 and fines content ≤ 5% is as follows:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \tag{2}$$

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes; $x = (N_1)_{60}$; a = 0.048; b = -0.1248; c = -0.004721; d = 0.009578; e = 0.0006136; f = -0.0003285; g = -1.673E-05; and h = 3.714E-06.

The CPT-based relationship as recommended by Youd and Idriss [1] for clean sands and earthquake of magnitude M7.5 is approximated by the following simplified equations:

$$CRR = 0.833\left[\frac{(q_{c1})_{cs}}{1000}\right] + 0.05 \qquad (q_{cl})_{cs} < 50 \tag{3}$$

$$CRR = 93\left[\frac{(q_{c1})_{cs}}{1000}\right]^3 + 0.08 \qquad 50 \le (q_{cl})_{cs} < 160 \tag{4}$$

The index value employed in this criterion is the clean sand cone penetration resistance normalized to 100 kPa (q_{c1}).

Another useful index of liquefaction potential, even though the database is quite limited so far, is the shear wave velocity. The V_s-based empirical relationship proposed by Andrus and Stokoe [16] is described by the following equation:

$$CRR = a(V_{s1}/100)^2 + b\left[1/\left(V_{s1}^* - V_{s1}\right) - 1/V_{s1}^*\right] \tag{5}$$

where a = 0.022 and b = 2.8 are the curve fitting parameters; V_{s1} is the corrected (to 100 KPa) shear wave velocity accounting for overburden pressure; V_{s1}^* is the limiting upper value of V_{s1} for cyclic liquefaction occurrence and is equal to 215 m/s for sands with fines content ≤ 5% .

Each of these empirical curves relates a resistance parameter of the individual test [$(N_1)_{60}$, q_{c1}, V_{s1}] to the soil's resistance to cyclic loading (CRR). Points along the CRR curve are considered as the capacity of the soil to resist the cyclic shear stresses induced by an earthquake M7.5 for a given $(N_1)_{60}$, q_{c1}, or V_{s1} measurement. All curves divide sites that liquefied (CSR>CRR) from those that did not (CSR<CRR) on the basis of $(N_1)_{60}$, q_{c1}, and V_{s1}.

The SPT, CPT, and V_s tests can be easily and economically performed today leading to a routine use of these empirical relationships in the assessment of the liquefaction potential of sand deposits. However these correlations pertain to Holocene-age (<10,000 years) soil deposits, derived predominantly from post-earthquake field investigations, and are not strictly valid for evaluating the CRR for older than Holocene soils.

PROPOSED METHODOLOGY

The empirical correlations for liquefaction evaluation applicable for young or "freshly deposited" soils can be used for the older soil deposits as long as the parameters involved (SPT, CPT, V_s, and CRR) are modified appropriately to account for the effect of aging. To assess the liquefaction potential of the aged soil deposits in the SCCP using the existing empirical correlations, the post-earthquake penetration resistance or shear wave velocity of the soil should be employed. In this work the "post-earthquake" and "freshly deposited" terms are used interchangeably. A schematic explanation is demonstrated in Figure 1 for two different scenarios: sites with evidence of liquefaction and sites with no evidence of liquefaction. For sites that liquefied and thus associated with a prehistoric earthquake (Figure 1a) the freshly-deposited condition coincides with the post-liquefaction condition. In this case aging covers the period of time from the date the field tests were performed to the date of occurrence of the associated earthquake that caused liquefaction. It is assumed that no more liquefaction events have disrupted the soil structure during this period of time. For sites that did not liquefy (Figure 1b) the freshly-deposited condition describes the condition after the deposition of the soil. In this case aging covers as much time as the geologic age of the deposit assuming that no liquefaction event has ever disrupted the soil structure.

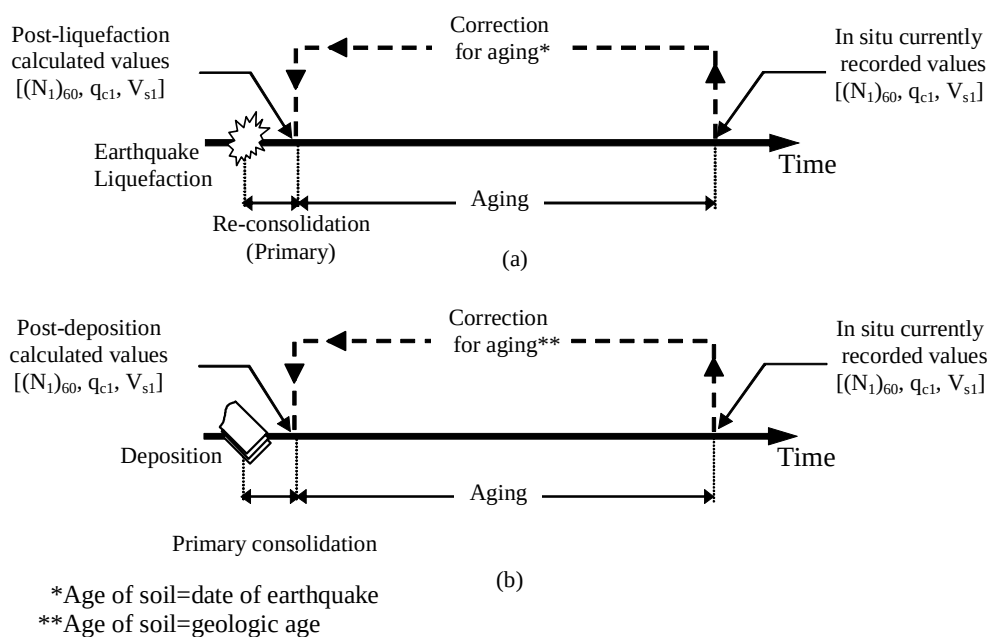

Figure 1. Correction of In-Situ Currently Recorded Values of $(N_1)_{60}$, q_{c1} or V_{s1} for the Effect of Aging for (a) Liquefied Sites and (b) Non-liquefied Sites

Provided that aging effects reestablish themselves with time after the occurrence of liquefaction (or deposition), their effect on the SPT, CPT, V_s, and CRR is taken into account with a four-step procedure as shown in Figure 2. As a first step (Figure 2a), the in-situ currently recorded values of $(N_1)_{60}$, q_{c1} or V_{s1}, are corrected for the effect of aging using the Kulhawy and Mayne [10] and/or the Mesri et al. [6] method. The Kulhawy and Mayne [10] method specifies that the $(N_1)_{60}$ and q_{c1} values are corrected by the factor c_A described by:

$$c_A = 1.2 + 0.05\,log\left(t/100\right) \tag{6}$$

where t is time in years.

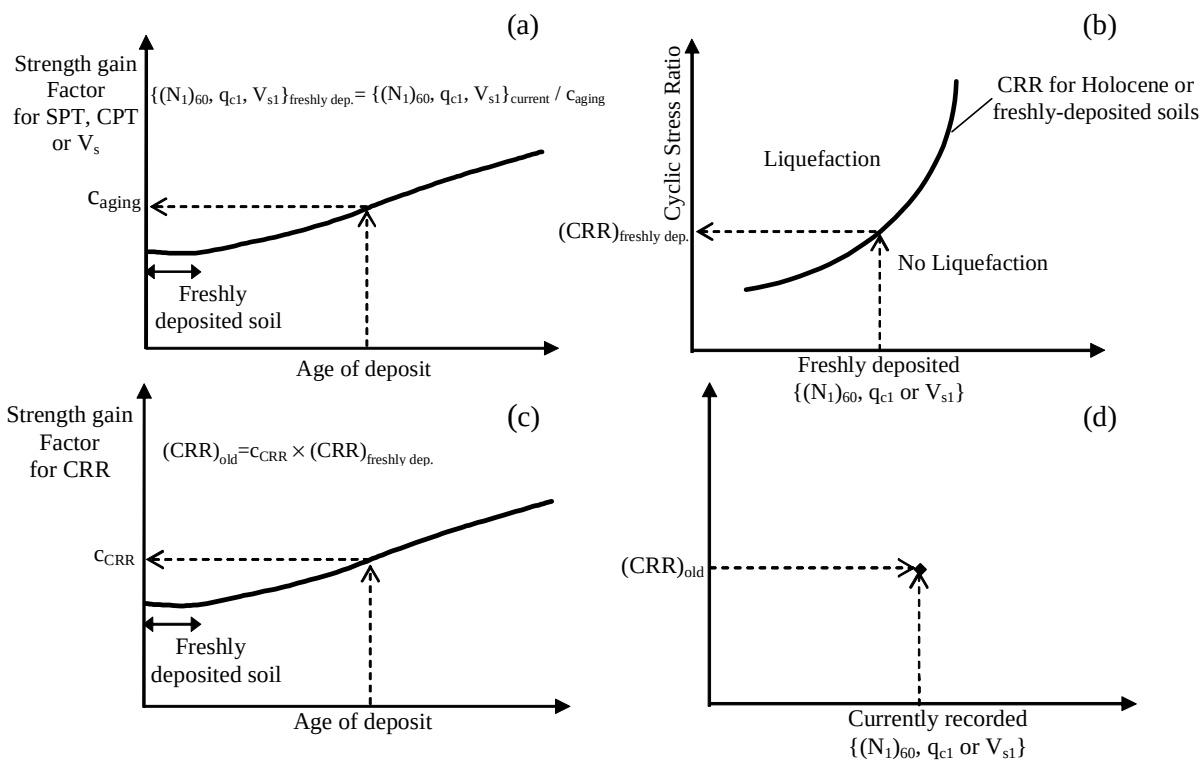

Figure 2. Proposed Methodology: (a) Step 1-Correct the in-situ currently recorded data for aging, (b) Step 2-Determine CRR for freshly deposited soil, (c) Step 3-Determine CRR for old/aged soil deposit, (d) Step 4-Associate in-situ currently recorded data with CRR for old/aged soil deposit

The Mesri et al. (1990) method specifies a different correction factor for the $(N_1)_{60}$ and q_{c1} values given by the right-hand-side of the following:

$$\frac{q_{c1}}{\left(q_{c1}\right)_R} = \frac{\left(N_1\right)_{60}}{\left[\left(N_1\right)_{60}\right]_R} = \left(\frac{t}{t_R}\right)^{C_D C_a / C_C} \tag{7}$$

where $[(N_1)_{60}]_R$ and $(N_1)_{60}$ are the SPT blow count at some reference time t_R and at any time $t>t_R$, respectively. For both methods the correction factor that is applied on the V_{s1} values is derived by converting the V_{s1} to equivalent q_{c1} values where the correction factor c_A (for the Kulhawy and Mayne [10] method) or $(\frac{t}{t_R})^{C_D C_a/C_C}$ (for the Mesri et al. [6] method) can be applied and combined with the correlation proposed by Andrus et al. (2003). Thus, the increase in $(N_1)_{60}$, q_{c1}, and V_{s1} with age for the Kulhawy and Mayne [10] method can be described by the following equation:

$$\frac{(N_1)_{60}}{[(N_1)_{60}]_R} = \frac{q_{c1}}{(q_{c1})_R} = \left[\frac{(V_{s1})}{(V_{s1})_R}\right]^{1/0.178} = c_A \tag{8}$$

and for the Mesri et al. [6] method:

$$\frac{(N_1)_{60}}{[(N_1)_{60}]_R} = \frac{q_{c1}}{(q_{c1})_R} = \left[\frac{(V_{s1})}{(V_{s1})_R}\right]^{1/0.178} = \left(\frac{t}{t_R}\right)^{C_D C_a/C_C} \tag{9}$$

where $[(N_1)_{60}]_R$, $(q_{c1})_R$, $(V_{s1})_R$ are the SPT, CPT, and shear wave velocity values respectively at a reference time t_R=0 which chronologically coincides with the freshly deposited state of the soil (after liquefaction or deposition), and; $(N_1)_{60}$, q_{c1}, V_{s1} are the SPT, CPT, and shear wave velocity values respectively at any time t>0 or the currently recorded values. By incorporating correction factors c_A and $\left(\frac{t}{t_R}\right)^{C_D C_a/C_C}$ into c_{aging} the following general equation describes the evaluation of the $(N_1)_{60}$, q_{c1}, and V_{s1} values of the soil at its freshly-deposited state for both the Kulhawy and Mayne [10] and Mesri et al. [6] methods:

$$\left\{(N_1)_{60}, q_{c1}, V_{s1}^{1/0.178}\right\}_{freshly-dep} = \frac{\left\{(N_1)_{60}, q_{c1}, V_{s1}^{1/0.178}\right\}_{current}}{c_{aging}} \tag{10}$$

Once the post-earthquake penetration resistance and shear wave velocity of the soil have been estimated, the corresponding CRR of the freshly deposited soil (immediately after liquefaction for liquefied or deposition for non-liquefied sites) can be evaluated from the existing empirical correlations as a second step (Figure 2b). Equation 2 is employed for the SPT-based evaluation, Equations 3 and 4 for the CPT-based evaluation, and Equation 5 for the V_s-based evaluation.

The third step is to evaluate the current CRR value for the aged soil deposit considering that over the hundreds or thousands of years that follow the earthquake (for liquefied sites) or deposition of the soil (non-liquefied sites), as aging effects re-establish themselves, resistance to liquefaction generally continues to increase (Figure 2c). To this end the correlation for the strength gain factor proposed by Arango et al. [18] is employed. In this case by "strength" the CRR of the soil is implied and the strength gain factor, c_{CRR}, is defined as the ratio of the strength of the soil after a period under "aging" over the strength of the soil at its freshly-deposited state:

$$c_{CRR} = \frac{(CRR)_{aged/current}}{(CRR)_{freshly-dep.}} \tag{11}$$

where c_{CRR} is obtained from Figure 3 for the specific age of the soil. Similarly to the first step, the age of the soil is equal to the date of the earthquake for liquefied sites associated with a specific prehistoric earthquake and equal to the geologic age of the deposit for non-liquefied sites.

Finally, the currently recorded penetration resistance or shear wave velocity values are plotted with $(CRR)_{aged/current}$ values determined in the third step for each one of the locations where liquefaction evidence was or was not found (Figure 2d). Using all the locations the best-fit curve is plotted which constitutes the liquefaction resistance curve for the old sand deposits encountered in the SCCP.

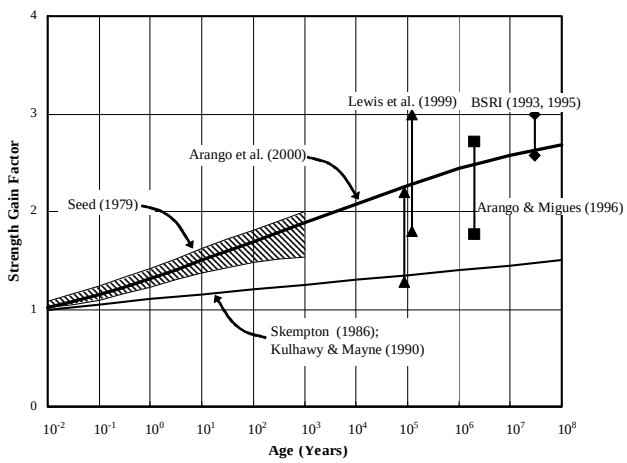

Figure 3. Field Cyclic Strength of Aged Sand Deposits (after Arango et al. [18])

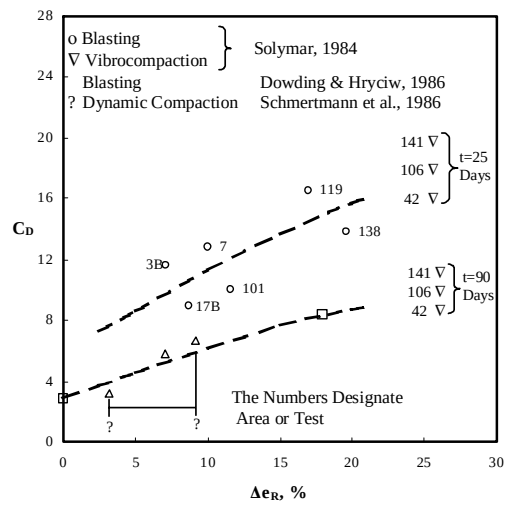

Figure 4. Summary of C_D Data for Ground-Modification Projects (after Mesri et al. [6])

RESULTS

The first step in utilizing the proposed methodology for the SCCP was to correct the in-situ currently recorded penetration resistance data for aging using the Kulhawy and Mayne [10] and the Mesri et al. [6] methods. For this work, t_R, in the Mesri et al. [6] method, was selected equal to 30 days (=0.082 years) and C_α/C_c equal to 0.02. Two values of Δe_R, 5% and 10%, were selected to represent a range of the change in relative density due to post-liquefaction densification. For these values the corresponding C_D values were graphically determined from Figure 4. In the same figure it is observed that C_D is also affected by the disturbance mechanism that induces liquefaction to the soil. In the absence of an earthquake as one of the disturbance mechanisms that induce liquefaction, the smaller values of C_D are hypothesized to be the most applicable for the case of an earthquake-induced liquefaction. So, from Figure 4 and the lower one of the two curves: C_D=5.5 for Δe_R = 5% and C_D=7.0 for Δe_R = 10%. For locations that did not liquefy, the Mesri et al. [6] method cannot be used since it is assumed that a disturbance mechanism as expressed by C_D never existed.

The in-situ geotechnical data that were corrected for aging $[(N_1)_{60}]_R$, and $(q_{c1})_R$ represent the soil at its freshly deposited state and are presented for both methods in Table 2 for the Gapway and Sampit sites. In the same table aging as described by the parameter t as well as the corresponding correction factor c_{aging} as represented by c_A (Equation 6) for the Kulhawy and Mayne [10] method and by the right hand side of

Equation 9 for the Mesri et al. [6] method are also presented. A significant difference is observed between the estimated SPT and CPT values of the soil at its freshly deposited state with the two different methods. The Mesri et al. [6] method suggests the highest c_{aging} values thereby estimating the lowest SPT and CPT values for the freshly deposited soil. This is possibly because the method is based on disturbances generated from a ground improvement technique rather than an earthquake event.

Table 2. Aging Correction Applied to Penetration Resistance Data

		Kulhawy & Mayne [10] Method			Mesri et al. [6] Method					
					Δe_R=5%			Δe_R=10%		
Location	**t (years)**	c_{aging}	$[(N_1)_{60}]_R$	$(q_{c1})_R$ (MPa)	c_{aging}	$[(N_1)_{60}]_R$	$(q_{c1})_R$ (MPa)	c_{aging}	$[(N_1)_{60}]_R$	$(q_{c1})_R$ (MPa)
GAP-01	450000	1.38	7	2.3	--	--	--	--	--	--
GAP-02	3548	1.28	9	4.3	3.24	3	1.7	4	2	1.2
GAP-03	5038	1.29	9	6.4	3.36	3	2.5	5	2	1.8
GAP-04	5038*	1.29	6	6.1	3.36	2	2.4	5	2	1.7
GAP-05	450000	1.38	12	6.3	--	--	--	--	--	--
SAM-01	450000	1.38	10	7.9	--	--	--	--	--	--
SAM-02	546	1.24	11	8.4	2.63	5	3.9	3	4	3.0
SAM-03	450000	1.38	10	5.3	--	--	--	--	--	--
SAM-04	1021	1.25	11	6.1	2.82	5	2.7	4	4	2.0
SAM-05	1648	1.26	13	7.2	2.97	5	3.0	4	4	2.3
SAM-06	450000	1.38	7	5.6	--	--	--	--	--	--

The next three steps take into consideration only the results obtained from the Kulhawy and Mayne [10] method where more confidence is shown. So, as a second step, the $[(N_1)_{60}]_R$ and $(q_{c1})_R$ values for the soil at its freshly deposited state are employed to access the existing SPT- and CPT-based empirical relationships, respectively. The intersection points give the CRR values of the freshly deposited soil for the SPT- and CPT-based procedure. The results are presented in Table 3 for the Gapway and Sampit sites. The percentage of fines present at all locations is small enough (see Table 1) for the source sand to be approximated as clean sand. This assumption allows the empirical relationship proposed by Youd and Idriss [1] for clean sands to be employed in the CPT-based procedure. For the SPT procedure the fines content as given in Table 1 is used to interpolate between the curves.

Table 3. Cyclic Resistance Ratios for the Freshly Deposited and the Current (Aged) State of the Soil

				CRR for Freshly Deposited State of Soil		CRR for Current (Aged) State of Soil	
Site	**Location**	**t (years)**	c_{CRR}	**SPT-based**	**CPT-based**	**SPT-based**	**CPT-based**
Gapway	GAP-01	450000	2.38	0.080	0.070	0.190	0.166
	GAP-02	3548	1.98	0.101	0.087	0.200	0.173
	GAP-03	5038	2.01	0.095	0.108	0.190	0.218
	GAP-04	5038*	2.01	0.074	0.105	0.149	0.211
	GAP-05	450000	2.38	0.130	0.107	0.310	0.255
Sampit	SAM-01	450000	2.38	0.109	0.134	0.258	0.319
	SAM-02	546	1.85	0.123	0.143	0.227	0.264
	SAM-03	450000	2.38	0.119	0.097	0.284	0.230
	SAM-04	1021	1.89	0.121	0.104	0.229	0.197
	SAM-05	1648	1.92	0.137	0.119	0.264	0.229
	SAM-06	450000	2.38	0.080	0.099	0.190	0.235

*The sandblow at GAP-04 was not associated with a prehistoric earthquake therefore its age is based on the age of the adjacent sandblow at GAP-03.

CRR values for the aged sand deposits which represent the liquefaction resistance of the soil at its current state are calculated from Equation 11 and presented in Table 3. The age of the source sand for each location and the corresponding strength gain factor c_{CRR} are also presented.

Finally, the currently recorded penetration resistances (Table 1) are correlated with the calculated CRR values for old sand deposits encountered at the investigated locations in the SCCP. SPT- and CPT-based boundary curves are developed representative of the SCCP conditions and are compared with the existing empirical correlations for Holocene age soils in Figures 5 and 6, respectively. Three boundary curves for each in-situ method are recommended for the SCCP and correspond to sand deposits of 546 to 5038, 200,000, and 450,000 years in age.

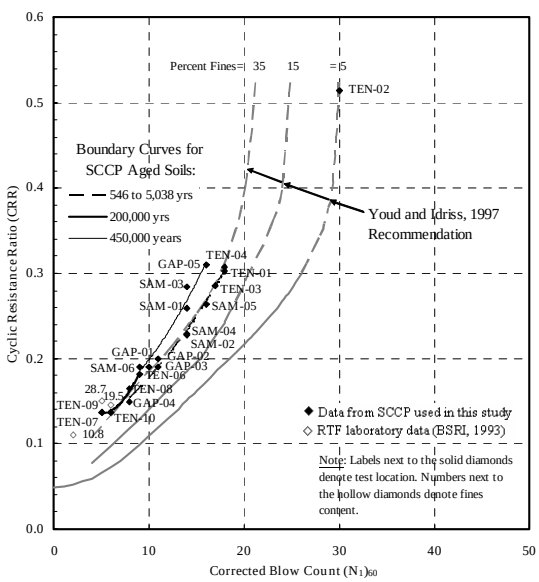

Figure 5. SPT-Based Boundary Curves for the Aged Soil Deposits in SCCP

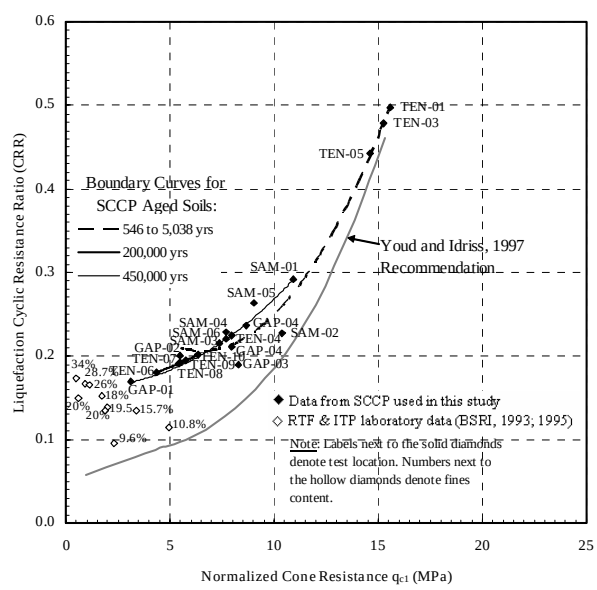

Figure 6. CPT-Based Boundary Curves for the Aged Soil Deposits in SCCP

The recommended boundary curves lay to the left of the existing curves recommended by Youd and Idriss [1] indicating that the aged soil deposits in SCCP posess more resistance to liquefaction than specified by the currently existing correlations. The boundary curves are also in agreement with data from cyclic triaxial tests performed on undisturbed samples of 30 million year old sand (10 to 35% fines content) obtained from two sites (RTF & ITP) in the location of the Savannah River Site [19,20].

Using the new boundary curves for aged soils developed for an earthquake magnitude M7.5, and the Cyclic Stress Method (Equation 1), the current threshold acceleration required to trigger liquefaction at each one of the four investigated sites is calculated based on the currently recorded SPT and CPT data. The estimated acceleration levels are listed in Table 4 where they are compared with the ones obtained by Hu et al. [21] for the same sites using the same data but not accounting for soil aging. Accounting for soil aging in the Cyclic Stress method yields higher values of the peak ground acceleration required to trigger liquefaction than when aging is not taken into account. On the average for the sites investigated in the SCCP the estimated acceleration in this study and the one by Hu et al. [21] differ by a factor of 1.6

suggesting that the sand deposits in the SCCP are 60% more resistant to liquefaction induced by an earthquake M7.5 than indicated by the existing liquefaction resistance empirical correlations for young soil deposits.

Table 4. Estimated Peak Ground Accelerations Capable of Triggering Liquefaction Using the Cyclic Stress Method for Earthquake Magnitude M7.5

		Threshold Peak Ground Acceleration (g)			
		Accounting for Soil Aging (This study)		Not Accounting for Soil Aging (Hu et al. [21])	
Site	**Location**	**SPT-based**	**CPT-based**	**SPT-based**	**CPT-based**
Gapway	GAP-01	0.31	0.26	0.17	0.13
	GAP-02	0.29	0.32	0.19	0.15
	GAP-03	0.29	0.31	0.19	0.22
	GAP-04	0.24	0.31	0.14	0.21
	GAP-05	0.48	0.33	0.27	0.23
Sampit	SAM-01	0.33	0.31	0.20	0.28
	SAM-02	0.24	0.25	0.17	0.22
	SAM-03	0.31	0.23	0.18	0.14
	SAM-04	0.25	0.22	0.17	0.14
	SAM-05	0.26	0.22	0.18	0.17
	SAM-06	0.20	0.22	0.11	0.14

CONCLUSIONS

A methodology has been developed to incorporate the age of the soil deposit when evaluating the liquefaction potential of the old sand deposits encountered in the SCCP. The study found that accounting for aging of the old sand deposits in the SCCP yields less conservative results regarding the liquefaction resistance than when not accounting for aging. The modified boundary curves are shifted to the left of the currently existing curves for Holocene age soils indicating that old soil deposits are more resistant to liquefaction. Minimum peak ground acceleration required to cause liquefaction of the old sand deposits in the SCCP was estimated to differ by a factor of 1.6 from the case were soil aging is not taken into consideration. Thus it is suggested that the liquefaction resistance of the old sand deposits in the SCCP is 60% higher than indicated by the existing liquefaction resistance empirical correlations for young soil deposits.

ACKNOWLEDGEMENTS

The writers wish to thank Bechtel Savannah River Company for providing the CPT and shear-wave velocity data. The SPT were collected with funding provided by the Nuclear Regulatory Commission. Deep appreciation is extended to Mike Lewis from Westinghouse Savannah River Company for his valuable suggestions.

REFERENCES

1. Youd, T. L., and Idriss, I. M. (1997). Summary Report, *Proc. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Tech. Rep. NCEER-97-0022, Youd, T.L. and Idriss, I.M. eds., National Center for Earthquake Engineering Research, Buffalo, pp. 1-40.
2. Mitchell, J. K., and Solymar, Z. V. (1984). "Time dependent strength gain in freshly deposited or densified sand," *J. Geotechnical Engrg.*, ASCE, 110(11), 1559-1576.

3. Dowding, C. H., and Hryciw, R. D. (1986). "A laboratory study of blast densification of saturated sand," *J. Geotechnical Engrg.*, ASCE, 112(2), 187-199.
4. Skempton A.W. (1986). "Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, aging and overconsolidation," *Geotechnique*, London, 36(3), 425-447.
5. Schmertmann, J. H. (1987). "Discussion of 'Time-dependent strength gain in freshly deposited or densified sand,'" by J. K. Mitchell and Z. V. Solymar, *J. Geotechnical Engrg*, ASCE, 113(2), 173-175.
6. Mesri, G., Feng, T. W., and Benak, J. M. (1990). "Postdensification penetration resistance of clean sands," *J. Geotechnical Engrg.*, ASCE, 116(7), 1095-1115.
7. Mitchell, J. K. (1986). "Practical problems from surprising soil behavior," The 20th Terzaghi Lecture, *J. Geotechnical Engrg.*, ASCE, 112(3), 259-289.
8. Joshi R. C., Achari, G., Kaniraj, S. R., and Wijeweera, H. (1995). "Effect of aging on the penetration resistance of sands," *Canadian Geotechnical Journal*, 32(5), 767-782.
9. Kulhawy, F. H. and Mayne, P. W. (1990) "Manual on estimating soil properties for foundation design," *Final Report 1493-6, EL-6800, Electric Power Research Institute*, Palo Alto, CA.
10. Arango I., and Migues, R. E. (1996). "Investigation of the seismic liquefaction of old sand deposits," *Report on Research, Bechtel Corporation, NSF Grant No. CMS-94-16169*, San Francisco, CA.
11. Lewis M. R., Arango, I., Kimball, J. K., and Ross, T. E. (1999). "Liquefaction resistance of old sand deposits." *Proc., 11th Panamerican Conference on Soil Mechanics and Geotechnical Engineering*, Foz do Iguassu, Brasil, 821-829.
12. Hu, K., Gassman, S. L., and Talwani, P. (2002). "In-situ properties of soils at paleoliquefaction sites in the South Carolina Coastal Plain," *Seismological Research Letters*, 73(6), 964-978.
13. Talwani, P. and Schaeffer, W. T. (2001). "Recurrence rates of large earthquakes in the South Carolina Coastal Plain based on paleoliquefaction data," *J. Geophysical Research*, 106(B4), 6621-6642.
14. Olson, S. M., Obermeier, S. F. and Stark, T. D. (2001). "Interpretation of penetration resistance for back-analysis at sites of previous liquefaction," *Seismological Research Letters*, 72(1), 46-59.
15. Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M. (1985). "The Influence of SPT procedures in soil liquefaction resistance evaluations," *J. Geotech. Eng, ASCE*, 111(12), 1425-1445.
16. Andrus, R. D. and Stokoe, K. H. II (2000). "Liquefaction resistance of soils from shear wave velocity," *J. Geotechnical and Geoenvironmental Engrg.*, 126(11), 1015-1025.
17. Andrus, R. D., Piratheepan, P., and Juang, C. H. (2003). "Shear wave velocity- penetration resistance correlations for ground shaking and liquefaction hazards assessment," *http://erp-web.er.usgs.gov/reports/annsum/vol43/pt/01g0007.pdf, (2/08/03)*.
18. Arango I., Lewis, M. R., and Kramer, C. (2000). "Updated Liquefaction Potential Analysis Eliminates Foundation Retrofitting of Two Critical Structures," *Soil Dynamics and Earthquake Engineering*, Vol. 20, 17-25.
19. BSRI, (1993). "Savannah River Site replacement tritium facility (233H) geotechnical investigation (U)," WSRC-RP-93-606, Vol. 3.
20. BSRI, (1995). "In tank precipitation facility (ITP) and H-tank farm (HTF) geotechnical report (U)," WSRC-TR-95-0057, Rev. 0, Vol. 6.
21. Hu, K., Gassman, S. L., and Talwani, P. (2002). "Magnitudes of prehistoric earthquakes in the South Carolina Coastal Plain from geotechnical data," *Seismological Research Letters*, 73(6), 979-991.