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ESTIMATING LIQUEFACTION POTENTIAL OF A 200,000-YEAR OLD SAND DEPOSIT NEAR GEORGETOWN, SOUTH CAROLINA

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ABSTRACT

A geotechnical experimentation site is being developed near Georgetown, South Carolina, to study the effect of soil age on liquefaction resistance. The site is located in an area called Hobcaw Barony on a 200,000-year-old beach to barrier-island sand deposit. Initial investigations conducted at the site include seismic cone penetration tests with pore pressure measurements, standard penetration tests with energy measurements, seismic crosshole tests, dilatometer testing, and fixed-piston sampling. Shear-wave velocities calculated from seismic cone test results are based on the true-interval method. The near-surface sand deposit extends from the ground surface to a depth of 8.5 m. The groundwater table is located at a depth of 2.4 m. Measured shear-wave velocities from the near-surface sand deposit are, on average, 47% higher than velocities of 10 year-old sand deposits with similar penetration resistances. The sand deposit at the Hobcaw Barony site is found to be susceptible to liquefaction, but ground shaking levels during the 1886 Charleston earthquake were not sufficient to cause liquefaction. This finding supports the observation that no surface manifestations of liquefaction occurred in the area.

INTRODUCTION

State-of-the-practice for evaluating liquefaction potential of soils is based on the simplified procedure originally proposed by Seed and Idriss (1971). This simplified procedure involves computing a variable that represents the earthquake loading on the soil, called cyclic stress ratio (CSR), and a variable that represents the capacity of the soil to resist liquefaction, called cyclic resistance ratio (CRR). Various charts to estimate CRR based on penetration resistance or small-strain shear wave velocity have been proposed using values of CSR calculated for sites that did and sites that did not liquefy during earthquakes (e.g., Seed et al. 1985, Robertson and Wride 1998, Andrus and Stokoe 2000, Youd et al. 2001, Juang et al. 2002, Cetin et al. 2004, Moss et al. 2006, Idriss and Boulanger 2008). The boundary separating CSR values from sites that liquefied and sites that did not is referred to as the CRR curve.

A limitation of most CRR curves is that they are based on cases of liquefaction that occurred in soil deposits with ages of less than a few thousand years (Youd et al. 2001). In fact, many of the cases are from deposits that were less than fifty years old when they liquefied (Hayati and Andrus 2009). Thus, corrections to CRR curves may be needed to obtain accurate estimates of liquefaction potential in older soil deposits.

Relatively few studies have been conducted to evaluate the influence of age (and age related processes) on CRR. Youd and Perkins (1978) and Seed (1979) suggested that liquefaction resistance increases with age. Subsequent studies (Troncoso et al. 1988, Lewis et al. 1999, Arango et al. 2000, Leon et al. 2006, Hayati et al. 2008, Hayati and Andrus 2008b, 2009) have led to the proposal to correct CRR by:

$$CRR_k = CRR * K_{DR} \quad (1)$$

where CRR_k is the deposit resistance-corrected CRR, and K_{DR} is the correction factor to capture the influence of age, cementation, and stress history on CRR.

The primary purpose of this paper is to estimate the liquefaction potential of a 200,000-year old sand deposit near Georgetown, South Carolina, in an area called Hobcaw Barony. Hobcaw Barony is a 17,500-acre outdoor laboratory owned and operated by the Belle W. Baruch Foundation. It is located east of Georgetown off of U.S. Highway 17, which is about 100 km (62 miles) northeast of Charleston, South Carolina. Hobcaw, a word from the Waccamaw Indian language meaning between waters, is located between the Waccamaw River and the Atlantic Ocean. Presented in Fig. 1 is a map of the Hobcaw Barony area.

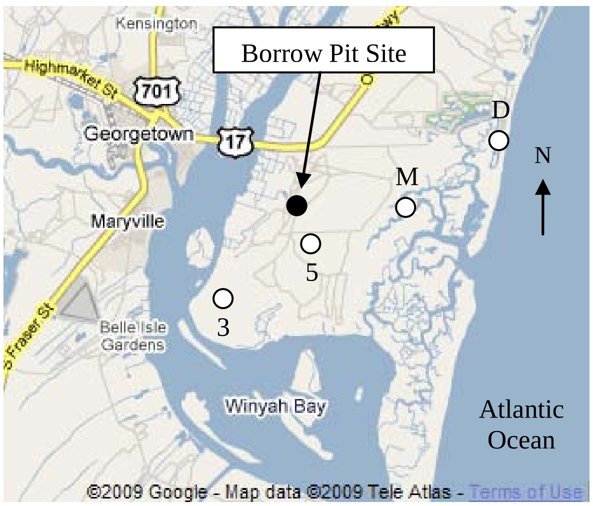

Fig. 1. Map of Hobcaw Barony and surrounding area showing locations of the borrow pit site and selected boreholes from the geologic investigation by May (1978)

The Belle W. Baruch Foundation's primary research and educational activities include forestry, wildlife, and marine science. To support these activities, Clemson University and the University of South Carolina have established research facilities located on Hobcaw Barony. In addition, the Baruch Foundation maintains multiple historic homes and a 19th Century slave village. The foundation has made available a site near an active borrow area to conduct field geotechnical investigations. This geotechnical investigation site is herein called the Hobcaw borrow pit site, or just borrow pit site (see Fig. 1).

In 2007 and 2008, initial geotechnical investigations were conducted at the Hobcaw borrow pit site with funding from the National Science Foundation. Methods of investigations included the seismic cone penetration test (SCPT), flat plate dilatometer test (DMT), standard penetration test (SPT), fixed-piston sampling, and seismic crosshole testing. Results of these investigations are presented in the thesis reports by Boller (2008) and Geiger (2009, 2010). This paper presents for the first time the SCPT and SPT results, and uses these results to estimate liquefaction potential of the aged sand deposit at the borrow pit site.

GEOLOGY

Hobcaw Barony is formed on the east by modern beach barriers and tidal flats (see Fig. 1). Inland, the area is covered by beach ridges that were formed by the deposition of sand from waves, which resulted from regression of the Atlantic Ocean or a seaward growth of the coastline. May (1978) estimated the sandy surficial beach deposits around the borrow pit area to be 100,000 to 200,000 years old. McCartan et al. (1984) also estimated the age of these beach deposits to be about 200,000 years old.

May (1978) developed general geologic cross-sections of the Hobcaw area based on available borehole information. Presented in Fig. 2 is the cross-section for the alignment containing boreholes 3, 5, M, and D (see Fig. 1). The Pleistocene age deposits range in thickness from 9 to 15 m (29 to 50 ft). Underlying the Pleistocene deposits is the Tertiary-age Black Mingo Formation. The Black Mingo Formation is about 58 m (190 ft) thick. The Paleocene-age Peedee Formation concludes the layering in the cross-section, beginning at an average depth below sea level of about 66 m (217 ft). Stiple (1957) characterized the Black Mingo Formation as sand to sandstone with possible interbedded clay layers, and the Peedee Formation as a black to gray sand with interbedded clay layers.

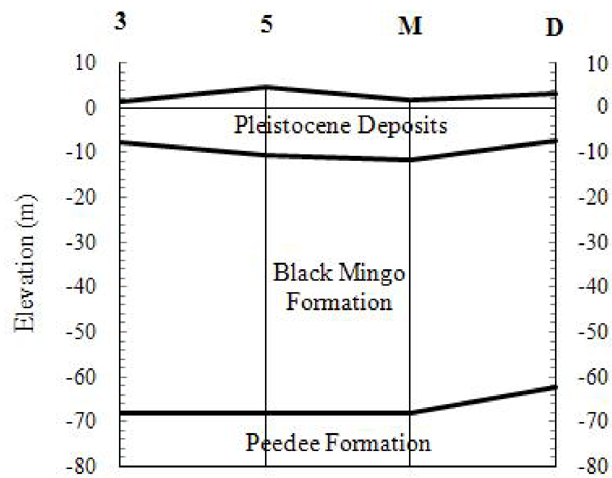

Fig. 2. Geologic cross-section for boreholes 3, 5, M, and D shown in Fig. 1 (adapted from May 1978)

Estimates of earthquake moment magnitude for the 1886 Charleston earthquake range from 6.6 to 7.6 (Bollinger 1986, Johnston 1996, Bakun and Hopper 2004), with the most likely value around 6.9. Silva et al. (2003) estimated a peak horizontal ground surface acceleration of about 0.15 g for the Hobcaw area, based on a ground motion simulation of the 1886 event.

Martin and Clough (1990) and Lewis et al. (1999) reviewed several reports of the 1886 earthquake and found no evidence (e.g., sand boils, fissures) that liquefaction occurred in the 200,000-year old beach deposits in the Hobcaw area.

Although it is possible for liquefaction to have occurred and not have been manifested at the ground surface because of a relatively thick capping layer (Ishihara 1985), the test results and the liquefaction potential evaluation presented later in this paper support the conclusion of no liquefaction at the borrow pit site.

TEST METHODS

A map showing the locations of field tests at the Hobcaw borrow pit site is presented in Fig. 3. The field tests included three SCPT soundings at HB-1, HB-2, and HB-3; one DMT sounding at D-1; one SPT boring at B-1; and fixed-piston sampling at B-2 and B-3. Inclinometer casings for seismic crosshole testing were installed to 11 m (36 ft) in B-1, B-2, and B-3. A standpipe for monitoring the groundwater table was installed in borehole B-4. Test procedures followed during SCPT soundings and SPT boring are described in this section.

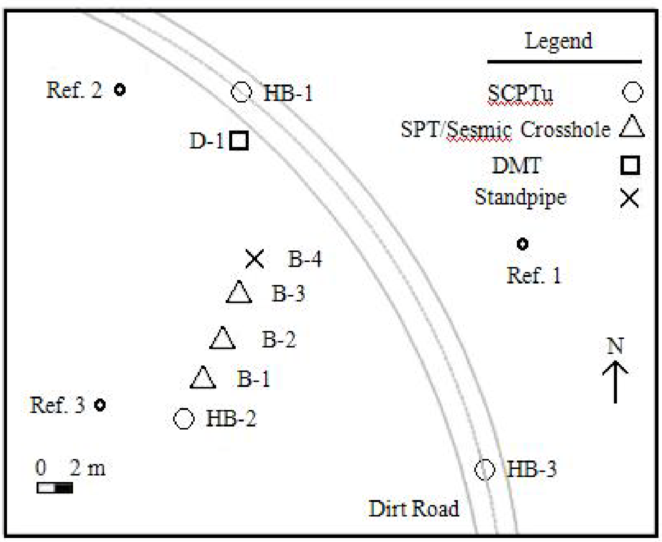

Fig. 3. Map showing locations of tests at the Hobcaw borrow pit site.

SCPTs with pore pressure measurements were performed as per ASTM D5778 using a track-mounted rig and a 15 cm^2 electric piezocone penetrometer. The penetrometer was hydraulically pushed at a rate of 2 cm/sec (0.79 in./sec). At HB-1, the penetrometer was pushed until refusal, which occurred at 15.7 m (51.6 ft). At HB-2 and HB-3, the penetrometer was pushed to depths of about 12.2 m (40 ft).

During each cone sounding, load cell recordings were made every 370 mm (1.5 in.) to determine cone tip resistance (q_t) and sleeve resistance (f_s). Recordings were also made with a pore pressure transducer located directly behind the cone tip (u_2). Values of q_t were corrected for pore pressures acting behind the cone tip. A new filter saturated with silicon oil was installed around the pore pressure transducer at the beginning of cone testing. Pushing was stopped every 1 m (3.3 ft), long enough to add another rod and to make shear-wave measurements. Shear waves were generated by striking a wood block source located on the ground surface in the horizontal direction. Waveform time histories were recorded simultaneously by two geophones located above the cone sleeve and spaced 1.00 m (3.28 ft) apart. Waveforms from both forward and reverse hits were recorded.

Various index properties were computed from the q_t, f_s, and u_2 measurements. These index properties included: stress-corrected normalized cone tip resistance (q_{t1N}), friction ratio (FR), normalized friction ratio (F_N), normalized cone tip resistance (Q_t), normalized cone pore pressure ratio (B_q), and soil behavior type index (I_c). The equations for these properties are as follows (Robertson and Wride 1998, Youd et al. 2001):

$$q_{t1N} = (q_t/P_a)\,(P_a/\sigma'_v)^{n} \tag{2}$$

$$FR = (f_s/q_t) * 100\% \tag{3}$$

$$F_N = (f_s/(q_t - \sigma_v)) * 100\% \tag{4}$$

$$Q_t = [(q_t - \sigma_v)/P_a]\,(P_a/\sigma'_v)^{n} \tag{5}$$

$$B_q = (u_2 - u_0)/(q_t - \sigma_v) \tag{6}$$

$$I_c = [(3.47 - \log Q_t)^2 + (1.22 + \log F_N)^2]^{0.5} \tag{7}$$

where P_a is a reference stress of 100 kPa (2000 lb/ft^2), σ'_v is the vertical effective stress, σ_v is the vertical total stress, n is an exponent that ranges from 0.5 for sand to 1.0 for clay, and u_0 is the hydrostatic pressure determined by multiplying the depth below the groundwater table by the unit weight of water. Values of q_{t1N}, FR, F_N, Q_t, B_q and I_c are all dimensionless.

Shear-wave velocities were determined by the true interval method. The true interval method involves one hit and two time history recordings and is different from the pseudo interval method, which involves two hits and two time history recordings. The true interval method is more accurate because the two recordings are based on the same wave front, and problems with trigger times and different wave fronts associated with the pseudo interval are avoided. For this study, the difference in shear-wave travel times to the two geophones (Δt) was determined from offsets of first peak and first crossover points. Shear-wave velocity (V_s) was calculated by:

$$V_s = (d_2 - d_1)/\Delta t \tag{8}$$

where d_1 is the straight line distance from the source at the ground surface to the top geophone at depth, and d_2 is the straight line distance from the source at the ground surface to the bottom geophone.

Values of V_s were corrected for overburden stress by (Robertson and Wride 1998, Youd et al. 2001):

$$V_{s1} = V_s \, (P_a/\sigma'_v)^{0.25} \qquad (9)$$

where V_{s1} is the overburden stress-corrected shear-wave velocity.

SPTs were conducted in borehole B-1 to a depth of 11.6 m (38 ft) following ASTM D1586. The borehole was established by rotary drilling with a high viscosity bentonite mud. A CME 550X automatic trip hammer system was used to drive the split-spoon sampler 457 mm (18 in.) into the ground. The sum of the blows for the last 305 mm (12 in.) is called the measured blowcount. For each blow, hammer system energy efficiency was determined using an instrumented SPT rod section. Hammer efficiencies varied between 71% and 105%.

The measured blowcount (N_m) was corrected for hammer efficiency (C_E), borehole diameter (C_B), rod length (C_R), type of sampler (C_S), and overburden stress (C_N). The corrections were applied as follows (Youd et al. 2001):

$$(N_1)_{60} = N_m C_E C_B C_R C_S C_N \qquad (10)$$

where $(N_1)_{60}$ is the corrected blowcount. C_N was calculated by:

$$C_N = 2.2/(1.2 + \sigma'_v/P_a) \qquad (11)$$

Values of C_B, C_R, and C_S were all 1 for this investigation because standard SPT equipment with energy measurements was used.

RESULTS

CPT Stratigraphy

Figure 4 displays the q_t and FR profiles for the three cone soundings at the Hobcaw Barony borrow pit site. The figure indicates three different soil layers (i.e., A, B, C) were encountered in the test depths. The groundwater table is located at a depth of 2.7 m (8.9 ft). Layer A is a sand layer that extends from the ground surface to a depth of 8.8 m (29 ft). Layer A exhibits an average q_t of 6.9 MPa (72 tons/ft^2) and an average FR of 0.27%.

Layer B lies between the depths of 8.8 and 9.6 m (29 and 32 ft). Layer B is characterized by an average q_t of 0.71 MPa (7.4 tons/ft^2) and an average FR of 0.64%. Measured cone pore pressures in this layer were greater than u_0. These results suggest significant fines content in Layer B soils.

Layer C extends from a depth of 9.6 m (32 ft) to a depth of about 12 m (39 ft). Layer C exhibits an average q_t of 8.7 MPa (91 tons/ft^2) and an average FR of 0.60%. These values are similar to those of Layer A.

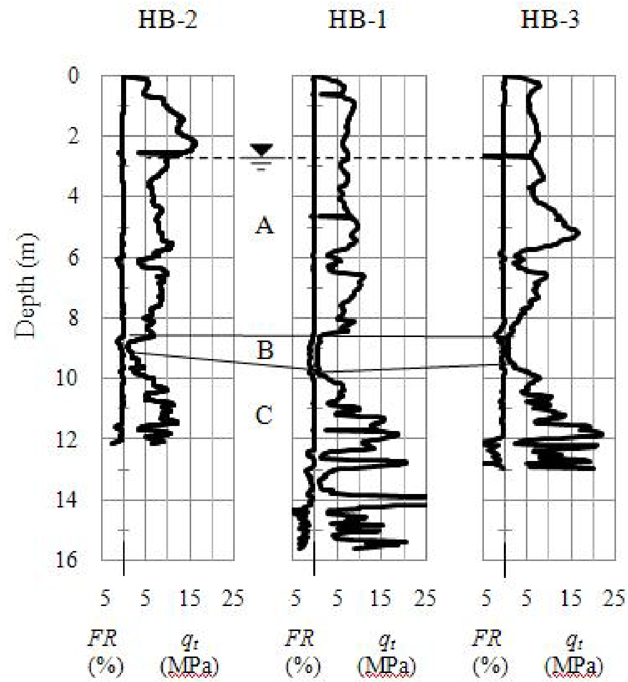

Fig. 4. CPT cross-section of the Hobcaw Barony borrow pit site.

Below 12 m (39 ft), values of q_t exhibit greater fluctuations and FR increases significantly, suggesting denser sands with interbedded fine-grained materials. The top of the Black Mingo Formation is believed to be at or just below the depth of 16 m (52 ft).

Table 1 presents a summary of average CPT results for Layers A (below the groundwater table), B and C. Equivalent clean sand values of normalized cone tip resistance ($(q_{t1N})_{cs}$) were determined following the procedure of Robertson and Wride (1998).

Table 1. Average CPT Results

Site	Depth (m)	q_{t1N}	I_c	B_q	$(q_{t1N})_{cs}$
Layer A					
HB-1	3.2-8.9	80.1	1.66	0.008	83.3
HB-2	2.7-8.9	85.5	1.64	0.012	88.7
HB-3	2.4-8.7	89.9	1.58	0.039	91.4
Layer B					
HB-1	8.9-9.8	6.7	3.08	0.501	50.2
HB-2	8.9-9.3	8.1	2.98	0.509	52.1
HB-3	8.7-9.6	8.4	2.95	0.477	50.8
Layer C					
HB-1	9.8-13.0	83.0	1.76	0.015	93.3
HB-2	9.3-12.2	84.4	1.65	0.017	88.1
HB-3	9.6-12.0	87.3	1.76	0.022	97.4

Shear-Wave Velocity Profiles

Profiles of V_s for the three SCPTs are presented in Fig. 5. All three profiles exhibit higher V_s values near a depth of 6 m (20 ft), where values of q_t peak (see Fig. 4). Also similar to the q_t profiles, values of V_s increase with depth in Layer C. Average V_s and V_{s1} values for each layer are given in Table 2.

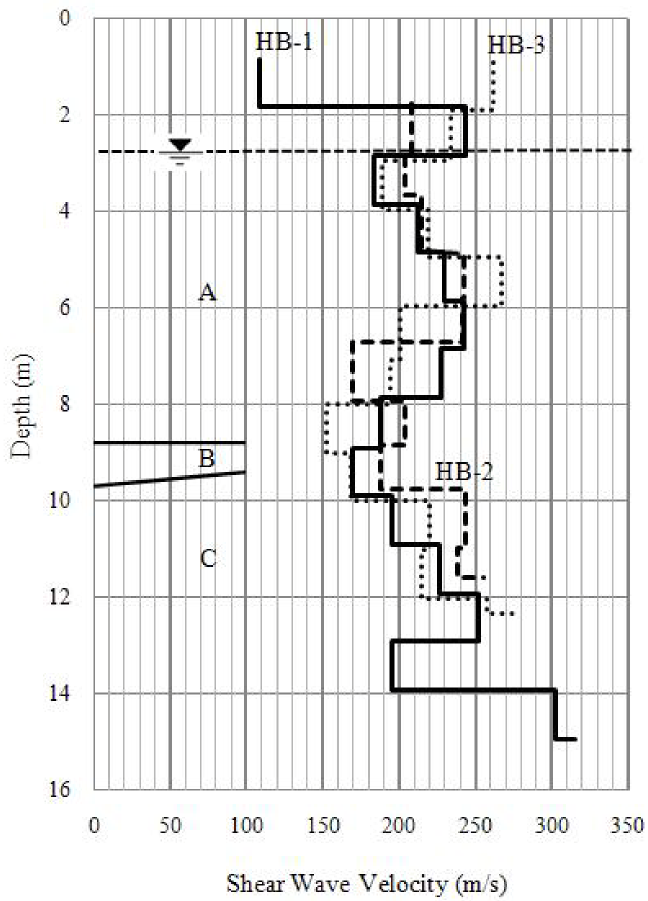

Fig. 5. SCPT shear-wave velocity profiles

Table 2. Average Shear-Wave Velocity Results

Site	Depth (m)	V_s (m/s)	V_{s1} (m/s)	$(V_{s1})_{cs}$ (m/s)	MEVR
Layer A					
HB-1	3.2-8.9	215	231	233	1.35
HB-2	2.7-8.9	211	236	237	1.36
HB-3	2.4-8.7	204	231	232	1.30
Layer B					
HB-1	8.9-9.8	183	182	189	1.22
HB-2	8.9-9.3	---	---	---	---
HB-3	8.7-9.6	177	183	193	1.24
Layer C					
HB-1	9.8-13.0	223	214	216	1.24
HB-2	9.3-12.2	210	235	237	1.36
HB-3	9.6-12.0	237	235	239	1.36

Also given in Table 2 are equivalent clean sand values of normalized shear-wave velocity ($(V_{s1})_{cs}$) and measure-to-estimated velocity ratio (MEVR). Values of $(V_{s1})_{cs}$ were obtained following the procedure of Juang et al. (2002).

MEVR is a promising new index property to represent the influence of age, cementation, and stress history on soils (Andrus et al. 2009, Hayati and Andrus 2009). Based on a study of various penetration resistance-V_s relationships, Andrus et al. (2009) recommended that estimated velocity be obtained using the following equation (Andrus et al. 2004b):

$$(V_{s1})_{cs} = 62.6[(q_{t1N})_{cs}]^{0.231} \qquad (12)$$

Equation (12) provides estimated $(V_{s1})_{cs}$ for sand deposits that are about 10 years old. MEVR is calculated by dividing $(V_{s1})_{cs}$ calculated from travel-time measurements by $(V_{s1})_{cs}$ estimated using Eq. (12).

Presented in Fig. 6 are profiles of MEVR for each SCPT. Higher MEVR indicates greater aging, cementation, and/or stress history effects in the soil. The relationship by Andrus et al. (2009) suggests MEVR of about 1.4 for 200,000-year-old sands.

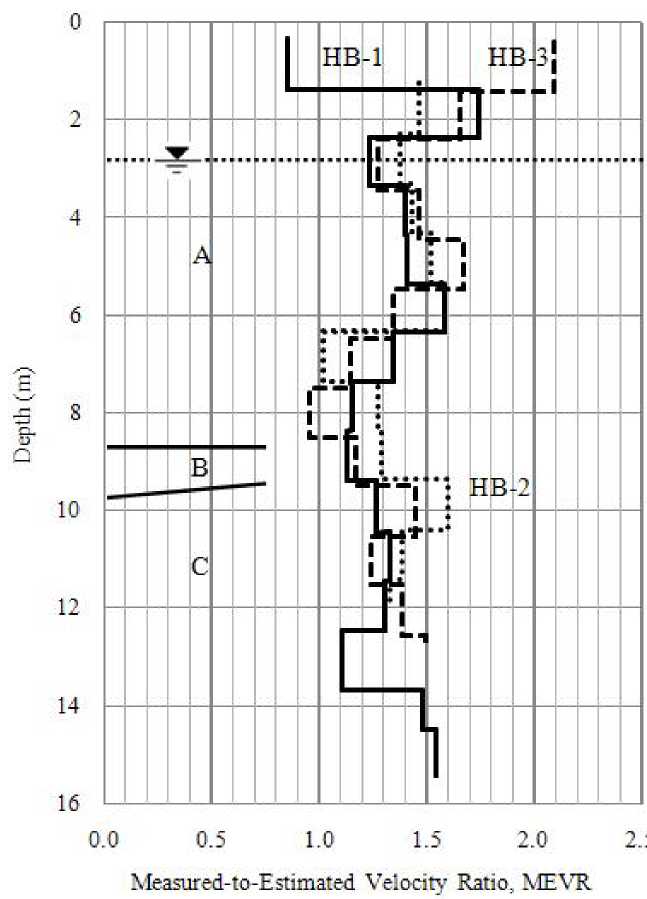

Fig. 6. Profiles of measured-to-estimated velocity ratio

SPT Blowcount Profile

Presented in Fig. 7 is a profile of the fifteen SPTs conducted in boring B-1. The highest blowcount occurs just below a depth of 5 m (16 ft), similar to the higher resistances in the CPT profiles (see Fig. 4) and the higher velocities in the V_s profiles (see Fig. 5). Also similar with the CPT and V_s profiles are the increasing blowcounts with depth in Layer C.

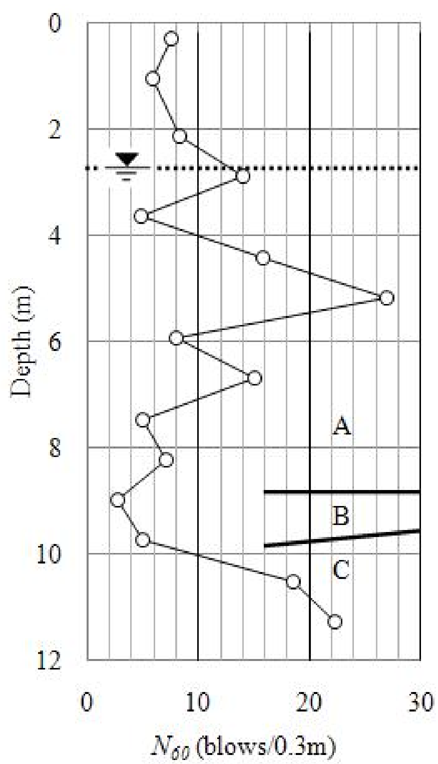

Fig. 7. SPT blowcount profile for borehole B-1

Given in Table 3 are average $(N_1)_{60}$ values for each layer. Also given in Table 3 are average values of fines contents determined from the split-spoon samples and equivalent clean sand values of corrected SPT blowcounts ($(N_1)_{60cs}$). Values of $(N_1)_{60cs}$ were obtained following the procedure of Youd et al. (2001).

Table 3. Average SPT Results for Borehole B-1

Layer	Depth (m)	$(N_1)_{60}$ (blows/0.3m)	Fines Content (%)	$(N_1)_{60cs}$ (blows/0.3m)
A	3.2-8.9	13	6.0	14
B	8.9-9.3	4	21.3	7
C	9.3-12.2	16	7.8	16

Six grain-size distribution curves for SPT split-spoon samples collected from Layer A are plotted in Fig. 8. Materials in Layers A and C classify as poorly graded sand with silt (SP-SM). Materials in Layer B classify as clayey sand or silty sand (SC or SM).

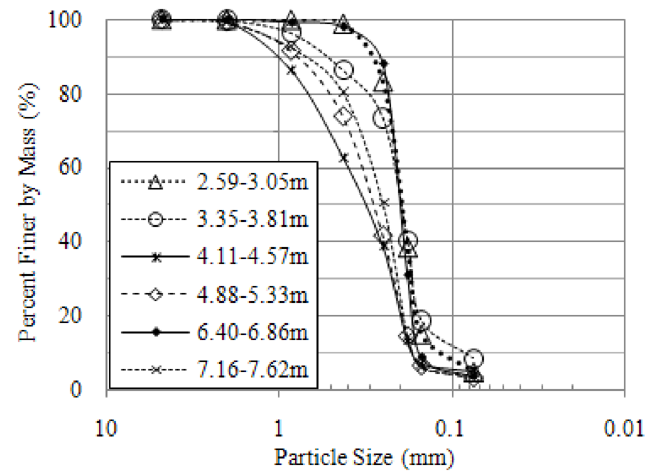

Fig. 8. Grain size distribution curves for Layer A samples

LIQUEFACTION SUSCEPTIBILITY

Typically, soils with high plastic clay content are not susceptible to liquefaction (Seed and Idriss 1982, Robertson and Wride 1998, Youd et al. 2001, Bray and Sancio 2006). Robertson and Wride (1998) proposed that clayey soils with I_c greater than 2.6 are unlikely to liquefy. Youd et al. (2001) suggested that this cutoff is too conservative for some soils and soils with I_c values between 2.4 and 2.6 should have additional testing or analysis for liquefaction susceptibility. Hayati and Andrus (2008a) proposed that clayey soils with B_q values greater than 0.5 are unlikely to liquefy and B_q values between 0.4 and 0.5 should have additional testing. These CPT-based liquefaction susceptibility criteria are illustrated in Fig. 9.

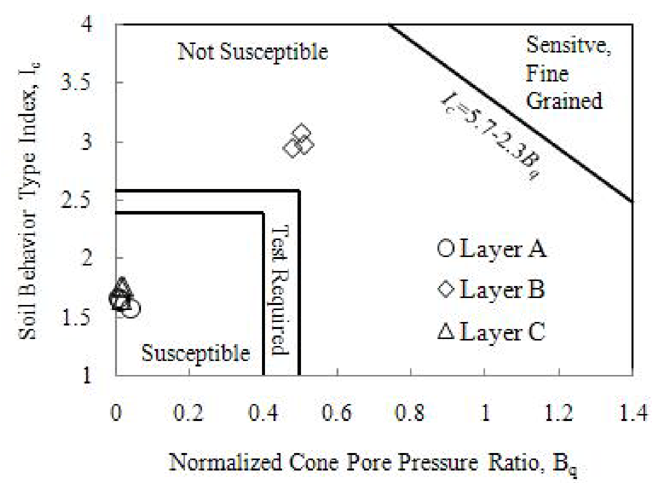

Fig. 9. CPT-based liquefaction susceptibility chart by Hayati and Andrus (2008a) with data from Hobcaw

Plotted on the CPT-based liquefaction susceptibility chart shown in Fig. 9 are data from the Hobcaw borrow pit site. Data from Layers A and C both plot within the susceptible-to-liquefaction region. Data from Layer B plot in the non-susceptible region. These results agree with criteria based on soil composition and consistency (e.g., grain size, Atterberg limits).

LIQUEFACTION POTENTIAL

Soil liquefaction potential can be expressed as the factor of safety against liquefaction (FS), defined as CRR divided by CSR. CSR is calculated using the following relationship (after Seed and Idriss 1971, Youd et al. 2001):

$$\mathrm{CSR} = 0.65\ (a_{max}/g)\ (\sigma_v/\sigma'_v)\ (r_d/MSF) \tag{13}$$

where a_{max} is the peak horizontal acceleration at the ground surface, g is the acceleration of gravity, r_d is the stress reduction coefficient, and MSF is the magnitude scaling factor that accounts for effects of shaking duration. Values of r_d are estimated using the relationship by Liao and Whitman (1986). Values of MSF are calculated using the more conservative relationship recommended by Youd et al. (2001), which is expressed as $MSF = (M_w/7.5)^{-2.56}$.

Two different earthquake scenarios are assumed in the calculation of CSR values. The first scenario is the 1886 Charleston earthquake, with M_w = 6.9 and a_{max} = 0.15 g. The second scenario is based on the 2008 United States Geological Survey (USGS) seismic hazard map for 2% probability of exceedance in 50 years. This USGS (2008) seismic hazard map provides an a_{max} value of 0.4 g, with the major contributing source having M_w = 7.3.

As indicated by Eq. (1), accurate estimates of CRR may require correction for age, cementation, and/or stress history. Hayati and Andrus (2009) recommended the following equation to estimate K_{DR} based on age:

$$K_{DR} = 0.13 \log_{10}(t) + 0.83 \tag{14}$$

where t is the time since initial soil deposition or last critical disturbance (e.g., liquefaction) in years. Because it is often difficult to determine t, Hayati and Andrus (2009) also proposed the following equation to estimate K_{DR} based on MEVR:

$$K_{DR} = 1.08\ \mathrm{MEVR} - 0.08 \tag{15}$$

Assuming t = 200,000 years and average MEVR = 1.34 for Layer A, Eqs. (14) and (15) provide K_{DR} values of 1.51 and 1.37, respectively. The lower value of 1.37 is assumed in this analysis for this study.

The results of the liquefaction potential analysis are presented next by the field test method, beginning with the V_s-based approach.

V_s-based Analysis

Presented in Fig. 10 is the V_S-based CRR curve by Andrus and Stokoe (2000) adjusted for age (or MEVR) using Eq. (1) and adjusted for probability of liquefaction. The CRR curve by Andrus and Stokoe (2000) for M_w = 7.5 and fines content (FC) $\leq$ 5% is defined as:

$$\mathrm{CRR} = 0.022[V_{s1}/(100\mathrm{MEVR})]^2 + 2.8[1/(215-V_{s1}/\mathrm{MEVR})-1/215] \tag{16}$$

where V_{s1} is in m/s. Because Eq. (16) has been characterized as a 26% probability of liquefaction (P_L) CRR curve, it has been slightly adjusted in Fig. 10 to correspond to P_L = 30% using the following relationship (Juang et al. 2002):

$$P_L = 1/[1 + (FS/0.73)^{3.4}] \tag{17}$$

A P_L = 30% is consistent with the most common SPT-based deterministic CRR curves (Juang et al. 2002).

Also presented in Fig. 10 are the CSR data points from the Hobcaw borrow pit site. It can be seen that the data points based on the 1886 earthquake correctly plot in the region of predicted non-liquefaction; and the data points based on the 2008 USGS seismic hazard map plot in the region of predicted liquefaction.

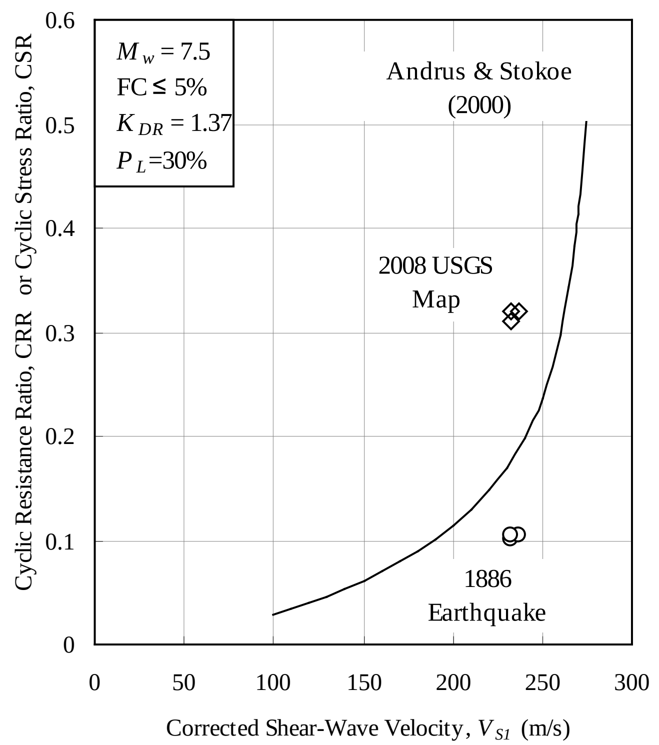

Fig. 10. Shear-wave-based CRR for clean sands corrected for age (or MEVR) with data from Hobcaw.

CPT- and SPT-based Analysis

Presented in Fig. 11 are various CPT-based CRR curves for M_w = 7.5, FC $\leq$ 5%, K_{DR} = 1.37 and P_L = 30%. The CRR curve by Andrus et al. (2009) is obtained by substituting Eq. (12) into Eq. (16) and adjusting to P_L = 30% using Eq. (17). The CRR curve by Robertson and Wride (1998) is obtained by adjusting to P_L = 30% using a relationship by Juang et al. (2002). The CRR curve by Idriss and Boulanger (2004) is plotted without any adjustment, except the K_{DR} correction. The CRR curve by Moss et al. (2006) is based on their relationship with P_L = 30%. It can be seen in Fig. 11 that three of the CRR curves plot fairly close together, whereas the curve by Moss et al. (2006) predicts significantly higher cyclic resistances.

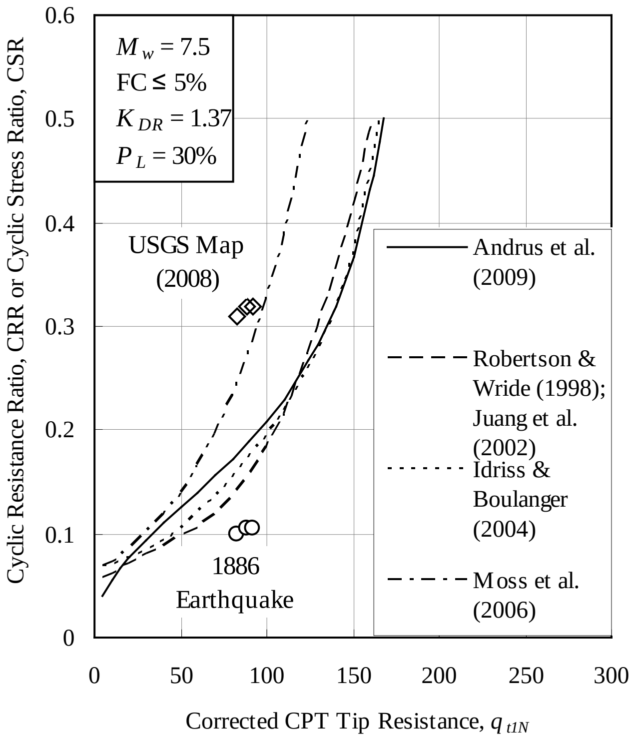

Fig. 11. CPT-based CRR curves for clean sands corrected for age (or MEVR) with data from Hobcaw.

Presented in Fig. 12 are various SPT-based CRR curves for M_w = 7.5, FC $\leq$ 5%, K_{DR} = 1.37 and P_L = 30%. The CRR curve by Andrus et al. (2009) is obtained by substituting

$$(V_{s1})_{cs} = 87.8[(N_1)_{60cs}]^{0.253} \tag{18}$$

from Andrus et al. (2004b) into Eq. (16), and adjusting to P_L = 30% using Eq. (17). The CRR curve by Youd et al. (2001) is obtained by adjusting to P_L = 30% using a relationship by Juang et al. (2002). The CRR curve by Idriss and Boulanger (2004) is plotted without any adjustment, except the K_{DR} correction. The CRR curve by Cetin et al. (2004) is based on their relationship for P_L = 30%. It can be seen in Fig. 12 that all four CRR curves are in general agreement.

Also presented in Figs. 11 and 12 are the CSR data points from the Hobcaw borrow pit site. It can be seen that the data points based on the 1886 earthquake correctly plot in the region of predicted no liquefaction in both figures. The data points based on the 2008 USGS seismic hazard map generally plot in the region of predicted liquefaction. These predictions agree well with the V_s-based predictions.

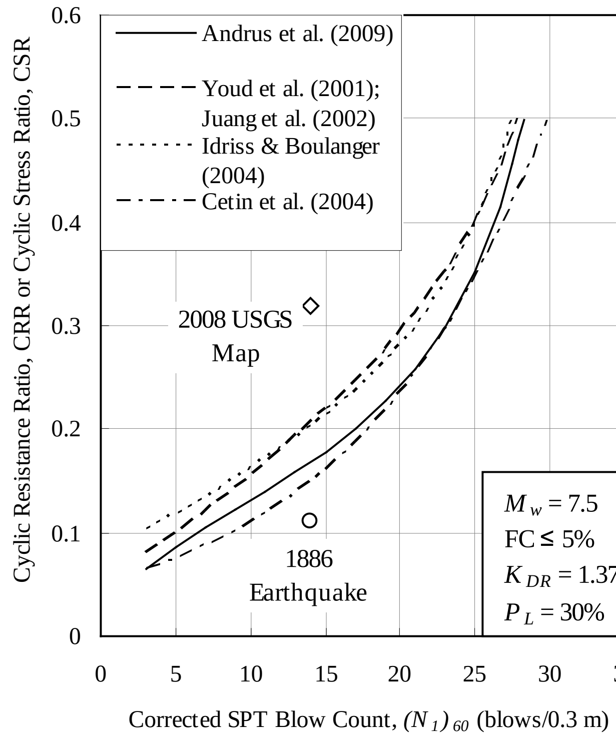

Fig. 12. SPT-based CRR curves for clean sands corrected for age (or MEVR) with data from Hobcaw

CONCLUSIONS

Three seismic cone soundings and one standard penetration boring were conducted at the Hobcaw Barony borrow pit geotechnical experimentation site to evaluate the liquefaction potential of near-surface sediments. Based on the results, the near-surface sediments were divided into three primary layers (A, B, C). Layer A is a poorly graded sand deposit that extends from the ground surface to approximately a depth of 8.8 m (28.9 ft). Layer B is a thin silty sand to clayey sand deposit ranging from depths of 8.8 to 9.6 m (29 to 32 ft). Layer C is a poorly graded sand deposit located below 9.6 m (32 ft). Both Layers A and C are susceptible to liquefaction.

The liquefaction potential of the 200,000-year-old beach sands of Layer A was evaluated using two earthquake scenarios. These earthquake scenarios were the 1886 Charleston earthquake, and the event predicted by the 2008 USGS seismic hazard map for 2% probability of exceedance in 50 years. CRR was estimated using the SCPTu, SPT, and shear-wave velocity results and various CRR curves. It is shown that the 1886 event did not cause liquefaction at the site, supporting the observation that no liquefaction occurred in the area. However, it is also shown that the Layer A sand is likely to liquefy during the 2,475-year return period earthquake.

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