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MAGNITUDES OF PREHISTORIC EARTHQUAKES IN THE SOUTH CAROLINA COASTAL PLAIN FROM GEOTECHNICAL DATA

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ABSTRACT

In-situ geotechnical data were collected near paleoliquefaction features in the South Carolina Coastal Plain (SCCP) to assess the magnitudes and peak ground accelerations of prehistoric earthquakes. Currently used paleoseismic analysis methods were used to back-calculate the magnitudes of prehistoric earthquakes and peak ground accelerations that can cause paleoliquefaction features in SCCP. The results from multiple methods yielded similar results and indicated that the prehistoric earthquakes in SCCP that occurred during the past 6000 years and caused paleoliquefaction features have magnitudes ranging from 5.3 to 7.8. The peak ground acceleration needed to cause liquefaction are also consistent among the different methods and ranged between about 0.1 and 0.3g for M7.5 and between about 0.1 and 0.5g for M6.0 earthquakes. These results were used together with those of earlier paleoseimological investigations to estimate the magnitudes and peak ground accelerations associated with prehistoric earthquake episodes. For earthquake episodes centered at Charleston, the estimated magnitudes and peak ground accelerations range from 6.8 to 7.8 and 0.16 to 0.24g, respectively. For episodes centered near Georgetown, the estimated magnitudes and peak ground accelerations range from 5.5 to 7.0 and 0.21 to 0.42g, respectively.

INTRODUCTION

Talwani and Schaeffer (2001) reanalyzed the results of 15 years of paleoliquefaction investigations in the South Carolina Coastal Plain (SCCP) and identified the occurrence of seven prehistoric earthquakes based on radiocarbon ages of the paleoliquefaction features. Table 1 shows the suggested two scenarios for paleoearthquake ages and source zones. In the first scenario, there are three possible source zones: Charleston (central source), Georgetown (northern source) and Bluffton (southern source). In the second scenario, all earthquakes occurred in the Charleston seismic zone.

Although the locations and dates of these paleoearthquakes have been verified with some degree of confidence, the estimated magnitudes are perhaps good to ± 0.5 units. The magnitudes were evaluated by a qualitative comparison of the spatial extent of liquefaction features observed in 1886 earthquake with those associated with prehistoric earthquakes. Talwani and Schaeffer (2001) also estimated magnitudes based on the epicentral distance to the farthest location of liquefaction based on the method developed by Youd and Perkins (1978) and Ambraseys (1988). In-situ soil properties at the locations of liquefaction features were not directly considered with these methods.

The work presented herein considers the in-situ soil properties at the locations of paleoliquefaction features to estimate magnitudes and accelerations of paleoearthquakes in the SCCP. In our paper 1, we have described the results of SPT, CPT, and shear wave velocity investigations at four locations of paleoliquefaction features in the SCCP. In this paper, we will use the engineering properties for the source sands, described in paper 1, to back-calculate paleoearthquake magnitudes and accelerations at the studied sites.

DATA

Over 50 prehistoric sandblows with datable material have been discovered at 17 sites along the South Carolina Coastal Plain (Figure 1). They are concentrated within three zones of paleoliquefaction: in the northeast, Georgetown and Myrtle Beach area; in the middle, Charleston area; and in the southwest, Bluffton and Hilton Head area. Four sites were investigated for this study: Ten Mile Hill sites A and B in the central paleoliquefaction zone, and Sampit and Gapway in the northeast paleoliquefaction zone. For details of the soil properties at each site, the reader can refer to our paper 1.

At each site, CPT, SPT and shear wave velocity tests were performed. The soil profiles were determined by analyzing the CPT and SPT data and the source sands were identified (paper 1). The data for the source sands are summarized in Table 2.

METHODS

Much progress has been made in the past few years to develop of techniques for back-calculating the acceleration and magnitude of paleo-earthquakes using paleoliquefaction features. These techniques are especially applicable in regions of the world that experience infrequent, but damaging, earthquakes such as southeastern and central U.S. Recently Obermeier and Pond (1999) reviewed the issues in using liquefaction features for paleoseismic analysis and summarized the methods currently in use for estimating the magnitudes and peak accelerations of prehistoric earthquakes that produced paleoliquefaction features. Selection of the appropriate method depends on the quality and type of data available at the paleoliquefaction sites. There are uncertainties associated with each method, such as the location of epicenter, the determination of soil profile, the knowledge of crustal structure, and hence the estimated values can have large margins of error.

MAGNITUDE EVALUATION

To determine the magnitude of each inferred paleoearthquake episode, Talwani and Schaeffer (2001) compared the areal extent of liquefaction features associated with a particular prehistoric episode with the areal distribution of sandblows associated with the 1886 earthquake. Assuming similar soil and groundwater conditions, if the sandblows associated with a specific paleoearthquake episode had comparable spatial distribution with the 1886 earthquake, the paleoearthquake magnitude was estimated to be of the same order of magnitude as 1886 earthquake and was assigned a magnitude of 7+. If the sandblows associated with a specific paleoearthquake episode had smaller spatial distribution than the 1886 earthquake, the paleoearthquake was believed to have a smaller magnitude than the 1886 earthquake and was arbitrarily assigned a magnitude of 6.0 (Table 1).

To increase the confidence in the estimated magnitude, Talwani and Schaeffer (2001) used the magnitude-bound method to further evaluate the paleoearthquake magnitudes. The magnitude-bound method is based on the field observations. For a given earthquake magnitude, M, the liquefaction-induced ground failures that occur during the earthquake are confined within a particular epicentral distance, R_e, beyond which liquefaction is not usually observed. Ambraseys (1988) investigated the correlation between maximum epicentral distance, R_e, at which liquefaction had been observed and associated earthquake magnitude, M, using liquefaction-producing earthquakes worldwide to develop the following magnitude-bound equation:

$M= -0.31+2.65*10^{-8}R_e+0.99(\log R_e)$ (1)

where R_e (in cm) is measured from the inferred epicenter to the most distant site where clear evidence of liquefaction-induced ground failure is present.

Use of this technique requires systematically studying the regional distribution of liquefaction effects to estimate the epicenter and extensive searching for liquefaction features over large geographic areas to locate the most distant effects of liquefaction. Obermeier and Pond (1999) suggested that when data from historical earthquakes in the study area are available to calibrate for the influence of local factors such as stress drop, focal depth, liquefaction susceptibility on the extent of liquefaction and attenuation of bedrock shaking, the confidence in interpretation of paleoearthquake magnitude is high. (In our study we did not attempt to calibrate the results with data from 1886.)

Neither of these methods explicitly considers the in-situ soil properties at the locations of the liquefaction features. Therefore, Pond (1996) developed a correlation between the seismic energy arriving at a site and the liquefaction susceptibility index of the site. The empirical representation of the seismic energy arriving at a site can be expressed as the seismic energy intensity T, which is a function of magnitude M and hypocentral distance R in kilometers:

$T = 10^{1.5M}/R^2$ (2)

This equation is valid only beyond meizoseismal zones and assumes that geometric spreading comprises most of the attenuation within hypocentral distances where liquefaction occurs. The average corrected SPT blow counts, $(N_1)_{60}$, value through the liquefiable layer is used as a liquefaction susceptibility index. The correlation between the seismic energy intensity and corrected SPT blow counts can be described as follows:

$(N_1)_{60} = (T/1.445)^{0.165}$ (3)

Obermeier and Pond (1999) indicated that this relationship is applicable for any tectonic setting worldwide. The energy-stress method was developed by combining equations (2) and (3) to obtain the following equation:

$M = 2/3*\log[1.445R^2*(N_1)_{60}{}^{6.06}]$ (4)

This equation can be used to estimate the minimum paleoearthquake magnitude that can cause liquefaction at a site based on the average corrected SPT blow count numbers, $(N_1)_{60}$, at the paleoliquefaction site and assumed hypocentral distance, R. This method assumes geometric spreading of energy and ignores other factors such as focusing of seismic energy due to shallow crustal structures. It can lead to unrealistically low estimates of magnitudes if geometric spreading of energy is not the mechanism of seismic energy dissipation.

PEAK GROUND ACCELERATION EVALUATION

For peak ground acceleration evaluation, the Ishihara method (1985) relates peak ground acceleration to the formation of sandblows based on the relative thicknesses of the liquefied and nonliquefied portions of the soil profile (Figure 2). This method is for sandblows caused by hydraulic fracturing. The maximum height of fracture is controlled by the thickness of liquefied sediment and the peak ground acceleration. Obermeier and Pond (1999) suggested that peak ground acceleration is important in sandblow formation because high peak ground acceleration is (1) more likely to cause breakage of the cap, irrespective of hydraulic fracturing; (2) more likely to be associated with a longer duration of strong shaking; and (3) likely to induce a greater thickness of completely liquefied sediment, providing more water for hydraulic fracturing.

The Ishihara curves were originally developed using data from only a few earthquakes whose magnitudes were about 7.5 and higher. When developing the curves, Ishihara did not consider the influence of earthquake magnitude, however, earthquake magnitude has a very important influence on liquefaction because larger earthquake magnitudes are more likely associated with longer durations of strong shaking. For the same soil profile, the effects of liquefaction are very different for different earthquake magnitudes. Youd and Garris (1995) discussed a case that demonstrated the importance of magnitude. They noticed that in the Marina District of San Francisco no liquefaction was observed by the M 5.3 1957 Daly City earthquake but the same area had extensive liquefaction following the M7.1 1989 Loma Prieta earthquake.

Youd and Garris (1995) further evaluated and verified Ishihara’s criteria for a wide range of earthquake and site conditions. Their investigations covered 15 different earthquakes, ranging in magnitude from 5.3 to 8.0. They found that the thickness bounds proposed by Ishihara (1985) appear valid only for the prediction of ground surface disruption at sites that were not susceptible to ground oscillation or lateral spread. They also examined the magnitude dependence of thickness relationships controlling the development of surface liquefaction effects. However, the data set were inadequate, particularly for smaller earthquakes, to clearly discern any influence of magnitude.

Two important factors that influence surface liquefaction manifestation but were not factored into Ishihara’s chart are the relative density and fines content of the liquefied source sands. For a given layer thickness of liquefied source sand, very loose source sands should cut through thicker layers than sands with higher relative densities. According to Ishihara (1985), the method appears to be valid for sand with SPT N-value as high as 20. Obermeier and Pond (1999) state that the Ishihara (1985) method is applicable when "the cap thickness is reasonably uniform (or at least does not slope much along the base) and the source sands range from very loose to moderately compact, at least for M>7.5 earthquakes." The Ishihara method provides a reasonable estimate of peak ground accelerations in the SCCP because the liquefaction features at the four SCCP sites studied herein were caused by hydraulic fracturing and, as shown in Table 1 for scenario 2, are associated with earthquakes of M 7+.

Martin and Clough (1994) found that the Ishihara relations are not exact, and some subjectivity is required in their interpretation and use. They suggested that confidence is low when the Ishihara boundary curve is the only method used to back-calculate peak accelerations at sites of liquefaction and, therefore, this method should be combined with other methods to improve confidence when employed for paleoliquefaction studies.

The cyclic stress method is another common method used to back-calculate peak ground acceleration. It is based on the representative penetration resistance obtained at a paleoliquefaction site in conjunction with a correlation for liquefaction resistance of sandy soils (Seed et al. 1983, 1985; Youd and Idriss, 1997). Olson et al. (2001) found that interpretation of the representative penetration resistance for back-analysis of peak ground acceleration is affected by the following factors: (1) destruction of pre-earthquake soil structure and aging effects during liquefaction; (2) postliquefaction consolidation and densification; and (3) postliquefaction aging. They suggested the need to evaluate each of these factors on a site-specific basis and to estimate a range of peak accelerations rather than a single value.

In the cyclic stress method, the liquefaction resistance of sandy soils is expressed using cyclic resistance ratio (CRR), which is a function of peak acceleration:

$$CRR = (t_h)_{ave}/s_o' = 0.65(a_{max}/g)(s_o/s_o')r_d \quad (5)$$

where $(t_h)_{ave}$ is the average earthquake-induced horizontal cyclic shear stress; a_{max} is the peak acceleration; s_o is the total overburden stress at the depth of interest and s_o' is the initial effective overburden stress at the same depth; and r_d is a stress reduction factor decreasing from 1 at the ground surface to 0.9 at depth of 10 m.

Figure 3 shows the correlation between CRR and SPT and CPT penetration resistance. Youd and Idriss (1997) recommended the following equations to approximate the clean sand CRR curve:

(6)

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes; x is the corrected SPT blow count for clean sand, $(N_1)_{60cs}$; 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. This equation is only valid for $(N_1)_{60}$ less than 30.

Or

$CRR_{7.5} = 0.833[(q_{cl})_{cs}/1000]+0.05$ if $(q_{cl})_{cs} < 50$ (7a)

$CRR_{7.5} = 93[(q_{cl})_{cs}/1000]^3+0.08$ if 50 £ $(q_{cl})_{cs} < 160$ (7b)

where $(q_{cl})_{cs}$ is the clean sand cone penetration resistance normalized to 100 KPa (approximately 1 tsf).

For sands with fines, Youd and Idriss (1997) suggested the following fines content correction formulas:

$(N_1)_{60cs} = a + b\,(N_1)_{60}$ (8a)

a = 0; b = 1.0 for FC £ 5% (8b)

$a = \exp[1.76-(190/FC^2)]$; $b = 0.99+FC^{1.5}/1000$ for 5% < FC < 35% (8c)

a = 5.0; b = 1.2 for FC ³ 35% (8d)

where FC is the fines content in percent.

For earthquake magnitude other than 7.5, the CRR should be corrected using the following equation:

$CRR_M=CRR_{7.5}*MSF$ (9)

where MSF is the "magnitude scaling factor" and can be obtained by the following equation (Youd and Idriss, 1997):

$MSF=10^{2.24}/M^{2.56}$ (10)

where M is the earthquake magnitude.

Andrus and Stokoe (2000) also developed a correlation between CRR and normalized shear wave velocity. As described in our paper 1, there is significant discrepancy between this correlation and our data for the SCCP.

Martin and Clough (1994) introduced a method which combined both the cyclic stress and Ishihara methods to give a more reliable estimate of minimum peak ground accelerations at which surface liquefaction evidence would begin to form. This method is mainly for cases where the threshold acceleration level is thought to lie between those estimated from the cyclic stress procedure and those estimated from Ishihara's guideline. That is, in some cases, it is believed that liquefaction over the entire lateral and vertical liquefiable layer is not necessary to produce liquefaction features and therefore, the liquefied thickness is some percentage of the source bed thickness.

The first step of Martin and Clough (1994) method is to use the cyclic stress method with SPT or CPT penetration data to determine which layers within the soil profile would liquefy at various levels of peak acceleration. Then for each acceleration level, the Ishihara curves are used to determine whether the liquefied layers were sufficiently thick to allow sandblows to be formed at the ground surface. The lowest value of peak ground acceleration at which both methods agreed that sandblows would be formed was considered the threshold acceleration.

RESULTS

For the evaluation of prehistoric earthquake magnitude, one example is given herein to illustrate the analysis procedure. Sandblows, which were associated with earthquake Episode A that occurred about 500 years ago (Talwani and Schaeffer, 2001), were identified at Sampit in the northeast, Hollywood near Charleston and Bluffton in the southwest (Figure 1). The spatial distribution of these sandblows was on the same order as the spatial distribution of sandblows associated with the 1886 earthquake. Talwani and Schaeffer (2001) ascribed the seismic source of this episode to Charleston. So the epicentral distance (or the hypocentral distance) to the most distant sandblow (BLUF-C or SAM-02) was about 100 to 140 km. SAM-02 is one of the sandblows encountered at Sampit (see paper 1 for details). The magnitude was estimated to be 7.0 to 7.2 using Ambraseys' magnitude-bound method (equation (1)). From the in-situ geotechnical data (Table 2), the source sands at SAM-02 had an average corrected SPT blow count number, $(N_1)_{60}$, of 14 based on representative data from SAM-04 (paper 1). Using the energy stress method (equation (4)), the estimated threshold magnitude was 7.4 to 7.6. Similar procedures were used to analyze other paleoearthquake episodes. The results are given in Table 3.

To evaluate the minimum peak ground accelerations of prehistoric earthquakes in SCCP that caused paleoliquefaction features, the Ishihara, cyclic stress, and Martin and Clough methods were used. The soil profiles at the four sites: Ten Mile Hill sites A and B, Sampit and Gapway have been obtained from the analysis of CPT and SPT data. The thickness of the liquefied sand layer H_2 and the thickness of the penetrated surface layer H_1 were obtained from the estimated soil stratigraphy and plotted into the Ishihara chart to assess the minimum peak ground acceleration that can cause sandblows at these sites. The data are plotted in Figure 4. For Ten Mile Hill site A (Figure 4a), two points were to the left of the 0.2g boundary curve and three points fell between the 0.2g and 0.3g boundary curves. Thus, a peak ground acceleration of about 0.3g is interpreted to cause extensive liquefaction at this site. For Ten Mile Hill site B (Figure 4b), three points were to the left of the 0.2g boundary curve and two points fell on the 0.2g boundary curve. Thus a peak ground acceleration of about 0.2g will cause extensive liquefaction at Ten Mile Hill Site B. For Sampit (Figure 4c) and Gapway (Figure 4d), all the data points are to the left of the peak ground acceleration 0.2g boundary curve, so a peak ground acceleration 0.2g will cause liquefaction-induced ground damage these sites. Recall, the Ishihara method assumes M>7.5 and provides only a rough and semi-quantitative evaluation of threshold peak ground acceleration that can cause surface liquefaction manifestation at a site.

Back-calculation of the peak ground acceleration using the cyclic stress method is illustrated using SAM-04 as an example. The source sands at SAM-04 have a corrected SPT blow count of 14 and fines content of 2% (Table 2). Using equation (8), we get the corrected SPT blow count for equivalent clean sand, $(N_1)_{60cs} = 14$. Then using equation (6) or Figure 3a, the cyclic resistance ratio for earthquake magnitude 7.5, $CRR_{7.5} = 0.15$. According to the magnitude evaluation of prehistoric earthquakes in the SCCP described before, the prehistoric liquefaction-inducing earthquakes may have magnitudes ranging from 5.3 to 7.8. Therefore, representative magnitudes 6.0 and 7.5 were selected to assess the threshold peak ground accelerations that can cause surface liquefaction evidence. For an earthquake with M6.0, equations (9) and (10) were used to calculate the equivalent cyclic resistance ratio, $CRR_6 = 0.27$. Then equation (5) is rearranged as follows:

$a_{max}/g = CRR/(0.65*(s_o/s_o')*r_d)$ (11)

and used to calculate the peak acceleration. For SAM-04, the total overburden stress and effective overburden stress for the sands are 0.93 tsf and 0.64 tsf respectively (Table 2). The stress reduction factor, r_d, was calculated using the following equation (Youd and Idriss, 1997):

$r_d = 1.0-0.00765z$ for z£ 9.15m (12)

where z is the depth in meters to the middle point of source sand layer. For SAM-04, z = 5 m (Table 2) and from equation (12), we get r_d. Then equation (11) was used to calculate the threshold peak ground accelerations for SAM-04.

The results are $a_{max}/g = 0.17$ for earthquake magnitude 7.5 and $a_{max}/g = 0.30$ for earthquake magnitude 6.0. The peak ground accelerations for other locations were calculated using the same procedure and the results are presented in Table 4.

At Ten Mile Hill site A, most of the boreholes have moderate blow count numbers of about 18. The back-calculated peak ground accelerations range from 0.21 to 0.27g for an earthquake with M7.5 and 0.37 to 0.47g for an earthquake with M6.0. Borehole TEN-02 has a blow count of 30 which is the upper limit value for liquefaction, so the back-calculation result for this borehole is high. For Ten Mile Hill site B, the blow count numbers are less than 10, which indicates that this site is highly liquefiable. Peak ground accelerations ranging from 0.08 to 0.12g for earthquake magnitude 7.5 or 0.14 to 0.22g for earthquake magnitude 6.0 may cause sandblows at this site. For the Sampit site, the blow counts range from low to moderate (9 to 16). This indicates that this site is liquefiable. The back-calculation indicates the peak ground accelerations at this site range from 0.11 to 0.20g for earthquake magnitude 7.5 and 0.19 to 0.35g for earthquake magnitude 6.0. At the Gapway site, the blow counts range from 8 to 16, thus the back-calculated peak ground accelerations range from 0.14 to 0.27g for earthquake magnitude 7.5 and 0.24 to 0.48g for earthquake magnitude 6.0.

When using normalized CPT tip resistance to evaluate the peak ground accelerations, the procedures are the same as using SPT blow counts. SAM-04 is used again as an example to illustrate the back-calculation procedure. The normalized CPT tip resistance of source sands at SAM-04 was 80 tsf. The fines content of source sand was 2%, From equation (8), the normalized CPT tip resistance for equivalent clean sand, $(q_{cl})_{cs}$ is 80 tsf. Then use equation (7) or Figure 3b to get the cyclic resistance ratio at earthquake magnitude 7.5, $CRR_{7.5} = 0.13$. Representative magnitudes 6.0 and 7.5 were selected to assess the threshold peak ground accelerations as was done when using SPT blow counts. For earthquake with M6.0, equations (9) and (10) were used to calculate the equivalent cyclic resistance ratio, $CRR_6 = 0.22$. Then using equation (12) to get r_d and equation (11), the threshold peak ground accelerations for SAM-04 were calculated. The results are $a_{max}/g = 0.14$ for earthquake with M7.5 and $a_{max}/g = 0.25$ for earthquake with M6.0. The peak ground accelerations for the other locations were calculated using the same procedure and the results are presented in Table 5.

For Ten Mile Hill site A, tip resistance at soundings TEN-01 and TEN-02 were larger than the upper limit value of 160 tsf for liquefaction, therefore, the back-calculation for these two boreholes could not be performed. Soundings TEN-03 and TEN-04 have tip resistances close to the upper limit value, resulting in back-calculated peak ground accelerations that are unreasonably high. For Ten Mile Hill site B, the tip resistance was relatively low, resulting in back-calculated peak ground accelerations ranging from 0.11 to 0.13g for an earthquake with M7.5 and 0.19 to 0.23g for an earthquake with M6.0.

The source sand at the Sampit site has moderate tip resistance (77 to 114 tsf), therefore the back-calculated peak ground accelerations range from 0.14 to 0.28g for earthquake with M7.5 and 0.25 to 0.50g for earthquake with M6.0. At the Gapway site, peak ground accelerations ranging from 0.13 to 0.23g for earthquake with M7.5 and peak ground accelerations ranging from 0.23 to 0.41g for earthquake with M6.0 may cause sandblows.

The peak ground acceleration evaluation procedures using the Martin and Clough (1994) method are illustrated using Ten Mile Hill site A and the results for all the investigated sites are presented in Figure 5. In the analysis of Ten Mile Hill site A, the sand stratum from 1.95 to 4.05 m is considered the potentially liquefiable layer. The first step is to use the cyclic stress method to calculate the percentage of CPT values of the liquefiable layer under different acceleration levels and plot the relationship between peak ground acceleration and percentage of CPT values liquefied (curves for magnitude M = 6.0 and 7.5 in Figure 5a). The Martin and Clough method assumes that the percentage of CPT values liquefied within a particular layer represents the percentage of that layer which is actually liquefied. So these two curves show the percentage of the 2.10 m sand layer that would be susceptible to liquefaction under various levels of peak acceleration.

The next step is to use the Ishihara guideline to evaluate the percentage of the source layer required to liquefy to cause surface evidence under different acceleration levels. The Martin and Clough method assumed that the liquefied portion of the potentially liquefiable layer lies immediately below the nonliquefiable layer. So when using the Ishihara curves (Figure 2) to analyze the percentage of liquefied layer, the thickness of penetrated surface layer, H_1 (1.95 m in this example), remains constant. Then several percentages (20, 40, 60, 80, 100%) of source layer liquefied were analyzed using Ishihara curves to determine the peak ground acceleration. For instance, when only 20% (0.42 m) of the 2.10 m sand layer is liquefied, the Ishihara curves predict that a peak ground acceleration of more than 0.6g would be required to produce sandblows; when the entire (100%) layer is liquefied, only 0.2g would be required. This result is shown in Figure 5a and labeled "layer effect curve."

The last step is to superimpose the cyclic stress acceleration curve and the Ishihara layer effect curve as shown in Figure 5a. The crossover point of these two curves is considered the threshold acceleration at which both methods predict that sandblows would initially develop. The threshold peak ground accelerations for M7.5 and 6.0 earthquakes for the various sites were as follows: Ten Mile Hill site A, 0.37g and 0.46g; Ten Mile Hill site B, 0.20g and 0.23g; Sampit, 0.19g and 0.25g; and Gapway, 0.19g and 0.25g.

Finally, the threshold peak ground accelerations evaluated from all the methods are summarized in Table 6. The Ishihara method uses the relative thickness of liquefied sand layer and unliquefied surface layer to estimate the peak ground accelerations. This method does not use the in-situ soil properties at the specific site, thus these estimated peak ground accelerations may only be reliable within ± 0.05g. The cyclic stress method uses the in-situ soil properties (SPT blow count or CPT tip resistance) to evaluate the required peak ground accelerations. The results from this method produce a wide range of peak ground accelerations rather than a single value. The Martin and Clough method combines use of both the Ishihara and cyclic stress methods and is believed to give a more reliable back-calculation result. This method also gives a single value of peak ground accelerations for a given magnitude. Except for Ten Mile Hill site A, which was located about 50 m from paleliquefaction features, the peak ground acceleration needed to cause liquefaction at the other sites are consistent among the different methods and ranged between about 0.1 and 0.3g for M7.5 and between about 0.1 and 0.5g for M6.0 earthquakes.

The results described above were used together with those of earlier paleoseimological investigations (Talwani and Schaeffer, 2001) to estimate the magnitudes and peak ground accelerations associated with prehistoric earthquake episodes. Using the estimated magnitudes from the energy-stress method for the prehistoric earthquake episodes and corrected SPT blow counts, $(N_1)_{60}$, of source sands at the location of sandblows due to the prehistoric earthquakes (Table 3), the minimum peak ground accelerations that can cause sandblows for these prehistoric earthquakes were back-calculated using the cyclic stress method. For example, Episode A (546±17 ybp) was associated with a sandblow at SAM-02. Using the energy stress method, its magnitude was estimated to lie between 7.4 and 7.6 (Table 3). For this magnitude range, the minimum peak ground acceleration needed to cause liquefaction based on SPT blow counts at SAM-02 was estimated to lie between 0.16 and 0.18 g (Table 7). The estimated magnitudes and peak ground accelerations for the prehistoric earthquake episodes in SCCP are summarized in Table 7. Since Ten Mile Hill sites A and B were not in the immediate vicinity of any dated sandblows, the earthquake episode(s) they were associated with could not be identified, however assuming them to be associated with a Charleston source, the estimated magnitude and acceleration were estimated and are given in Table 7.

SUMMARY AND CONCLUSIONS

Different methods were used in this study to estimate the magnitudes of prehistoric earthquake episodes and threshold peak ground accelerations that can cause surface liquefaction features at the investigated sites in SCCP. The comparison of the spatial extent of paleoliquefaction features due to a prehistoric earthquake with those due to a historic earthquake whose magnitude was known by other means provided a rough, qualitative estimation for the prehistoric earthquake magnitudes.

The magnitude-bound method is based on field observations of a large number of liquefaction-producing earthquakes worldwide. It can provide a quantitative estimate of prehistoric earthquake magnitudes, but may not match site-specific conditions because it does not consider the soil properties and geological settings of different sites. This method is also dependent upon an extensive field investigation to locate the most distant liquefaction features.

The energy-stress method relates the magnitudes estimation of prehistoric earthquake to the in-situ soil properties at the locations of liquefaction features due to the prehistoric earthquake. It is sensitive to an accurate assessment of the blowcounts of the source sands and other sources of error. For example, the determination of the seismic energy source may be very uncertain; the energy attenuation within hypocentral distances may not be controlled by geometic spreading; and the current in-situ soil properties (penetration resistance) may not be representative of the in-situ pre-earthquake soil properties. Therefore it is important to evaluate magnitudes of prehistoric using multiple methods. If the results from multiple methods yield similar results, the confidence in the results is higher.

Thus, we used the Ishihara, cyclic stress (SPT and CPT based) and Martin and Clough methods to back-calculate the threshold peak ground accelerations that can cause surface liquefaction damage for a given magnitude earthquake at four sites investigated in the SCCP. The Ishihara method provided the threshold peak ground acceleration for earthquake magnitude 7.5 or above. The estimation of peak ground acceleration from the Ishihara method is only a semi-quantitative assessment. The cyclic stress method relates the peak ground acceleration to the in-situ corrected SPT blow counts, $(N_1)_{60}$, and normalized CPT tip resistance, q_{c1}. This method is the most widely used to back-calculate peak ground accelerations because of the wide usage of penetration test and availability of abundant field performance data. The Martin and Clough method combined both the Ishihara and cyclic stress methods to provide a more reliable estimate of threshold peak ground acceleration.

The results of these investigations suggest that the magnitudes of prehistoric earthquakes that caused liquefaction in the SCCP ranged between 5.3 and 7.8 and were associated with peak ground motions between 0.16 and 0.42g. For earthquake Episodes A, B, C', E and F' centered at Charleston the estimated magnitudes and peak ground accelerations range from 6.8 to 7.8 and 0.16 to 0.24g, respectively. For Episodes C and F centered near Georgetown the estimated magnitudes and peak ground accelerations range from 5.5 to 7.0 and 0.21 to 0.42g, respectively.

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Table 1 Two Scenarios for Paleoearthquake Ages and Source Zones in South Carolina Coastal Plain (after Talwani and Schaeffer, 2001)

Liquefaction Age, years Episode B.P.	Scenario 1	Scenario 2
	Source M	Source M
1886 AD 115	Charleston 7.3	Charleston 7.3
A 546± 17	Charleston 7+	Charleston 7+
B 1001± 33	Charleston 7+	Charleston 7+
C 1648± 74	North 6.0	… -
D 1966± 212	South 6.0	… -
C' 1683± 70	… -	Charleston 7+
E 3548± 66	Charleston 7+	Charleston 7+
F 5038± 166	North 6+	Charleston 7+
G 5800± 500	Charleston 7+	Charleston 7+

Table 2 In-situ Geotechnical Data for the Source Sands

Site	Location	z (m)	h (m)	s o (tsf)	s o' (tsf)	$(N_1)_{60}$	q_{cl} (tsf)	V_{sl} (m/s)	Fines (%)
Ten Mile Hill A	TEN-01	2	1.5	0.38	0.34	18	163	235	7
	TEN-02	3	1.5	0.56	0.42	30	204	400	3
	TEN-03	3	2.4	0.56	0.42	17	159	163	3
	TEN-04	3	2.7	0.56	0.42	18	83	214	3
	TEN-05	4	2.4	0.74	0.59	N/A	153	239	N/A
Ten Mile Hill B	TEN-06	4	3.8	0.74	0.59	9	46	170	4
	TEN-07	5	4.1	0.93	0.68	5	57	187	5
	TEN-08	5	4.2	0.93	0.68	8	57	177	4
	TEN-09	5	4.3	0.93	0.76	5	60	158	5
	TEN-10	6	5.3	1.13	0.85	6	66	165	5
Sampit	SAM-01	4	5.7	0.74	0.57	14*	114	277	3
	SAM-02	6	4.3	1.13	0.76	14*	108	250	1
	SAM-03	5	5.2	0.93	0.67	14*	77	288	0
	SAM-04	5	5.4	0.93	0.64	14	80	291	2
	SAM-05	5	5.8	0.93	0.6	16	95	334	4
	SAM-06	5	5.6	0.93	0.64	9	80	321	4
Gapway	GAP-01	2	0.7	0.38	0.38	10	33	181	N/A
	GAP-02	2	0.9	0.38	0.38	11*	58	220	9
	GAP-03	2	1.0	0.38	0.38	11	87	177	6
	GAP-04	2	1.1	0.38	0.38	8	83	240	N/A
	GAP-05	2	1.3	0.38	0.38	16	90	154	5

Note: z is the depth of the middle point of source sand layer; h is the thickness of source sand layer; s o, s o' are total overburden stress and effective overburden stress at the middle point of source sand layer; $(N_1)_{60}$ is the corrected SPT blow count number; q_{cl} is the corrected CPT tip resistance; V_{sl} is the normalized shear wave velocity; Fines is the percentage by weight passing through US #200 sieve. *The blow count values at SAM-01 to SAM-03 are based on data from SAM-04 and at GAP-02 on the data from GAP-03.

Table 3 Estimated Magnitudes of Prehistoric Earthquake Episodes in SCCP

Episode (Age)	Source	Found In	$(N_1)_{60}$ for Source Sands	R_e or R (km)	Estimated Magnitudes		
					Energy-Stress Method (Eq. 4)	Magnitude-Bound Method (Eq. 1)	Empirical Method (Talwani and Schaeffer, 2001)
A (546± 17)	Charleston	SAM-02	14	100-140	7.4 to 7.6	7.0 to 7.2	7+
B (1021± 30)	Charleston	SAM-04	14	100-140	7.4 to 7.6	7.0 to 7.2	7+
C (1648± 74)	Northeast	SAM-05	16	10-35	6.3 to 7.0	5.7 to 6.3	6
Or C’ (1683± 70)	Charleston	SAM-05	16	100-140	7.6 to 7.8	7.0 to 7.2	7+
E (3548± 66)	Charleston	GAP-02	11	100-140	6.8 to 7.0	7.0 to 7.2	7+
F (5038± 166)	Northeast	GAP-03	11	10-35	5.5 to 6.2	5.7 to 6.3	6
Or F’ (5038± 166)	Charleston	GAP-03	11	100-140	6.8 to 7.0	7.0 to 7.2	7+
?	Charleston	TMHA	18	10-35	6.5 to 7.2	5.7 to 6.3	-

?	Charleston	TMHB	9	10-35	5.3 to 6.0	5.7 to 6.3	-

Table 4 Peak Ground Acceleration Evaluation Based on SPT Blow Counts

Site	Location	a_{max} (g) (M = 7.5)	a_{max} (g) (M = 6.0)
Ten Mile Hill A	TEN-01	0.27	0.47
	TEN-02	0.59	1.03
	TEN-03	0.21	0.37
	TEN-04	0.22	0.4
	TEN-05	N/A	N/A
Ten Mile Hill B	TEN-06	0.12	0.22
	TEN-07	0.08	0.14
	TEN-08	0.11	0.19
	TEN-09	0.08	0.15
	TEN-10	0.09	0.16
Sampit	SAM-01	0.20	0.35
	SAM-02	0.17	0.29
	SAM-03	0.18	0.31
	SAM-04	0.17	0.30
	SAM-05	0.18	0.32
	SAM-06	0.11	0.19
Gapway	GAP-01	0.17	0.30
	GAP-02	0.19	0.33
	GAP-03	0.19	0.33
	GAP-04	0.14	0.24
	GAP-05	0.27	0.48

Table 5 Peak Ground Acceleration Evaluation Based on CPT Tip Resistances

Site	Location	a_{max} (g)	a_{max} (g)

		(M = 7.5)	(M = 6.0)
Ten Mile Hill A	TEN-01	N/A	N/A
	TEN-02	N/A	N/A
	TEN-03	0.53	0.93
	TEN-04	0.15	0.27
	TEN-05	0.54	0.95
Ten Mile Hill B	TEN-06	0.11	0.19
	TEN-07	0.12	0.21
	TEN-08	0.12	0.21
	TEN-09	0.13	0.23
	TEN-10	0.13	0.23
Sampit	SAM-01	0.28	0.50
	SAM-02	0.22	0.39
	SAM-03	0.14	0.25
	SAM-04	0.14	0.25
	SAM-05	0.17	0.30
	SAM-06	0.14	0.25
Gapway	GAP-01	0.13	0.23
	GAP-02	0.15	0.27
	GAP-03	0.22	0.39
	GAP-04	0.21	0.37
	GAP-05	0.23	0.41

Table 6 Estimated Peak Ground Accelerations of Prehistoric Earthquakes in SCCP

Earthquake	Paleoliquefaction	Estimated Peak Ground Accelerations (g)			
		Ishihara	Cyclic Stress Method		Martin &
			SPT	CPT	

					Clough Method	Method	Site	Magn
M = 7.5	Ten Mile Hill A	0.3	0.21 to 0.59	0.15 to 0.54	0.37			
	Ten Mile Hill B	0.2	0.08 to 0.12	0.11 to 0.13	0.20			
	Sampit	0.2	0.11 to 0.20	0.14 to 0.28	0.19			
	Gapway	0.2	0.14 to 0.27	0.13 to 0.23	0.19			
M = 6.0	Ten Mile Hill site A	0.3	0.37 to 1.03	0.27 to 0.95	0.46			
	Ten Mile Hill site B	0.2	0.14 to 0.22	0.19 to 0.23	0.23			
	Sampit	0.2	0.19 to 0.35	0.25 to 0.50	0.25			
	Gapway	0.2	0.24 to 0.48	0.23 to 0.41	0.25			

Table 7 Estimated Magnitudes and Peak Ground Accelerations for Prehistoric Earthquake Episodes in SCCP

Episode (Age)	Source	Found In	$(N_1)_{60}$ for source sands	R_e or R (km)	Estimated Magnitude	Estimated Peak Ground Acceleration (g)
A (546± 17)	Charleston	SAM-02	14	100-140	7.4 to 7.6	0.16 to 0.18
B (1021± 30)	Charleston	SAM-04	14	100-140	7.4 to 7.6	0.16 to 0.18
C (1648± 74)	Northeast	SAM-05	16	10-35	6.3 to 7.0	0.21 to 0.28
Or C' (1683± 70)	Charleston	SAM-05	16	100-140	7.6 to 7.8	0.16 to 0.17

E (3548± 66)	Charleston	GAP-02	11	100-140	6.8 to 7.0	0.23 to 0.24
F (5038± 166)	Northeast	GAP-03	11	10-35	5.5 to 6.2	0.31 to 0.42
Or F' (5038± 166)	Charleston	GAP-03	11	100-140	6.8 to 7.0	0.23 to 0.24
?	Charleston	TMHA	18	10-35	6.5 to 7.2	0.24 to 0.32
?	Charleston	TMHB	9	10-35	5.3 to 6.0	0.22 to 0.30

LIST OF FIGURE CAPTIONS

Figure 1 Dashed lines enclose three zones of paleoliquefaction along the South Carolina Coastal Plain. The explosion symbols represent three possible inferred epicentral locations. Triangles show the locations of sandblows in South Carolina Coastal Plain. Reports of liquefaction features extend to Columbia and Georgetown and to Sand Hills near Liberty Hill. Hollow triangles indicate the locations of in-situ engineering tests for this study. Abbreviations are as follows: Bluffton, BLUF; Colony Gardens, COLGAR; Conway, CON; Four Hole Swamp, FHS; Gapway, GAP; Georgetown, GEO; Hollywood, HOL; Malpherous, MAL; Martin Marietta, MM; Myrtle Beach, MYR; Sampit, SAM; and Ten Mile Hill, TMH (modified from Talwani and Schaeffer, 2001).

Figure 2 Liquefaction Boundary Curve (after Ishihara, 1985)

Figure 3 Liquefaction resistance relationships for sandy soils at earthquake magnitude M = 7.5 (a) CRR versus corrected SPT blow count, $(N_1)_{60}$ (From Youd and Idriss, 1997); and (b) CRR versus CPT normalized cone resistance, q_{c1} (after Olsen, 1996).

Figure 4 Peak Ground Acceleration Assessment Using Ishihara Chart: (a) Ten Mile Hill Site A; (b) Ten Mile Hill Site B; (c) Sampit; and d) Gapway.

Figure 5 Peak Ground Acceleration Assessment Using Martin and Clough (1994) method: (a) Ten Mile Hill site A; (b) Ten Mile Hill site B; (c) Sampit; and (d) Gapway.

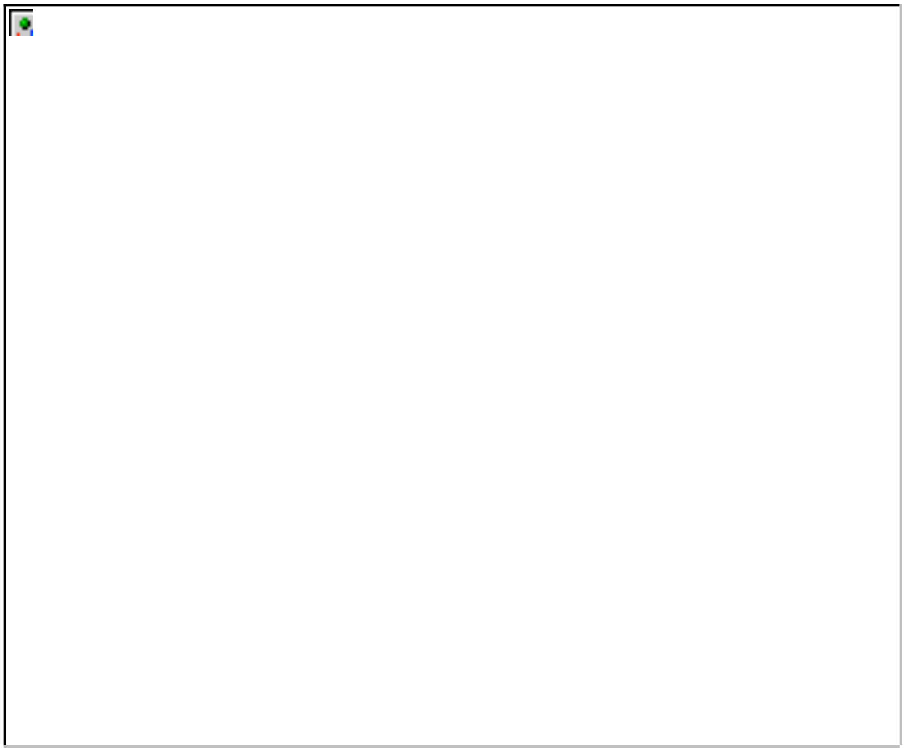

Figure
1

Figure 2

(a)

(b)

Figure 3

(a) (b)

(c) (d)

Figure 4

(a)

(b)

(c)

(d)

Figure 5