Evaluation of Liquefaction Potential at a Silt Site in Providence, Rhode Island

Given the fact that liquefaction potential depends on specific site conditions, it is necessary to perform such evaluations for any given project subject to risk of liquefaction.

A.S. BRADSHAW, R.A. GREEN &
C.D.P. BAXTER

Assessing liquefaction potential in the Providence, Rhode Island, area is complicated by two factors:

- there are limited available ground motion records from large earthquakes in the region; and,
- much of the city is underlain by deep deposits of non-plastic silt, for which little is known about its cyclic resistance.

To provide insight on this issue, a deterministic evaluation of liquefaction potential was conducted at a site in Providence using a cyclic stress-based approach. The cyclic demand of the earthquake was evaluated by performing site response analyses using scaled intra-plate bedrock motions. The cyclic resistance of the silt was estimated using a soil-specific shear wave velocity-based correlation developed by the authors from an extensive laboratory testing program, as well as using existing standard penetration test (SPT) and cone penetration test (CPT) correlations.

Background

"Liquefaction" refers to the phenomenon that occurs when the soil skeleton of loose, saturated sand and/or non-plastic silt collapses and

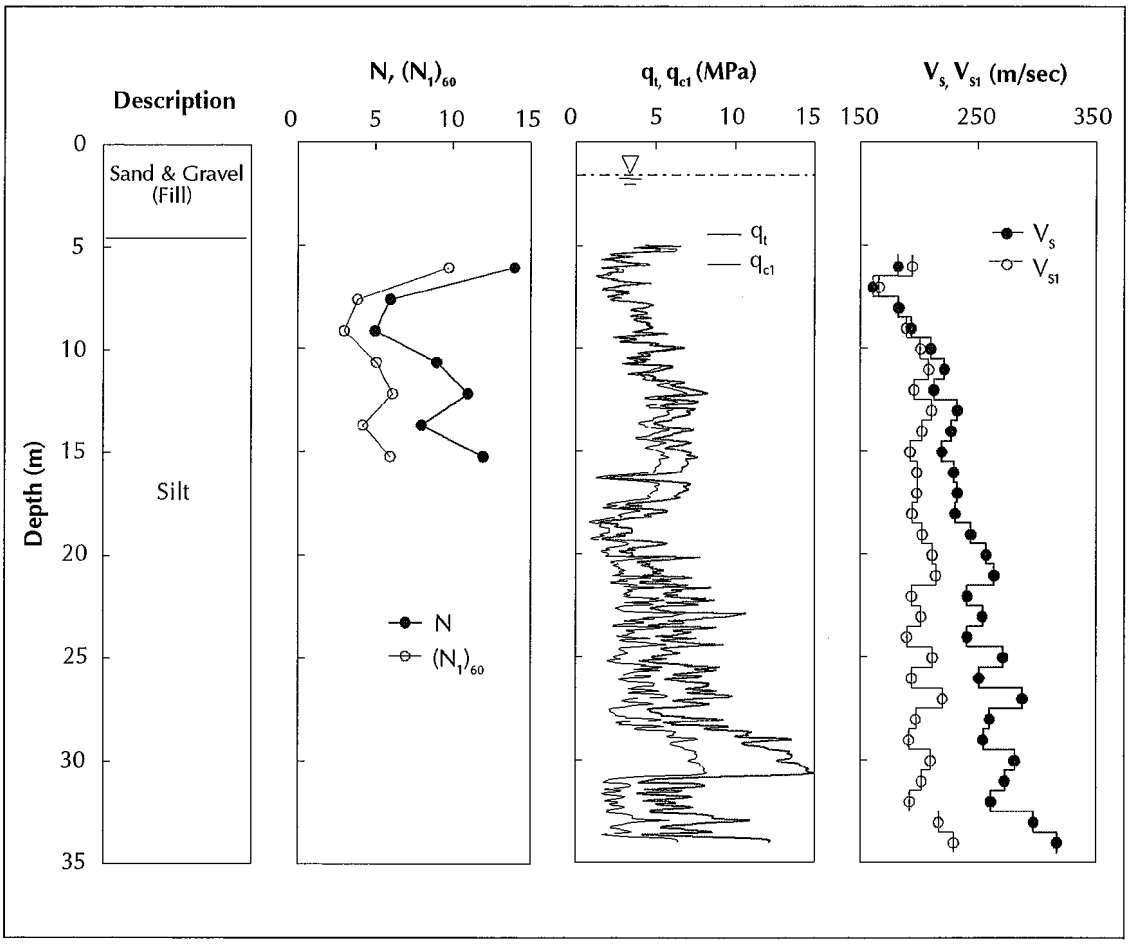

FIGURE 1. A representative geotechnical profile resulting from the site investigation.

there is a temporary transfer of overburden stress from the soil skeleton to the pore fluid. The collapse of the soil skeleton can be initiated in a variety of ways, one of which is earthquake shaking. There are numerous documented cases where earthquake-induced liquefaction led to the partial or complete loss of soil shear strength, resulting in building settlements, damage to utilities, and slope instabilities.[1]

The precise cause of earthquakes in the northeastern United States is uncertain. However, for earthquakes that are large enough to be of engineering interest, a generally accepted hypothesis is that they result from movements on ancient failed rift zones or passive margins, where the movements are driven by stresses transferred from the plate boundaries.[2,3] Because most seismic sources in stable continental regions (i.e., central and eastern United States) are not well defined, it is difficult to quantify the potential of such sources to generate future earthquakes.[4] However, some geologic studies have shown that prehistoric earthquakes in the Northeast have been large enough to cause liquefaction.[5-7] One study in Newburyport, Massachusetts, in particular uncovered two generations of buried liquefaction features, including sand dikes and sills, with the causative earthquake estimated to have a return period of approximately 4,000 years.[6]

In Rhode Island, the evaluation of liquefaction hazard is further complicated by the fact that most of the coastal areas, including Providence, are underlain by silt that is composed of approximately 95 percent fines. Assessing the cyclic resistance of the Providence silt is uncertain because the semi-empir-

ical liquefaction evaluation procedures commonly used in practice were primarily developed using data from deposits containing less than 35 percent fines.[8] It is the combined difficulty of defining the design earthquake motions and the uncertainty in the cyclic resistance of the Providence silt that makes the assessment of liquefaction potential a challenging problem in this area.

Site Investigation & Soil Conditions

A geotechnical site investigation was performed at the site of a historic farmers' market in Providence, which included rotary wash borings with SPTs performed adjacent to seismic CPTs. The SPTs were accomplished using a standard split-spoon sampler with the inside liner removed. The efficiency of the donut hammer system was measured using a pile driving analyzer and ranged from 30 to 40 percent, with an average of 37 percent. Hammer efficiency measurements are critical for correcting the SPT blow counts to a standard efficiency of 60 percent for evaluating liquefaction resistance. Representative samples of silt were recovered between SPT intervals using a 7.5-centimeter diameter split-spoon sampler with a core catcher. These samples were reconstituted and used in a laboratory cyclic strength testing study.[9] Seismic CPTs included near-continuous measurements of tip resistance, sleeve resistance and pore water pressure. Shear wave travel time measurements were made "down-hole" and a shear wave velocity model was used to interpret the shear wave velocity profile at 1-meter depth intervals.

A representative profile of the soil conditions at the farmers' market including SPT blow counts (N), cone tip resistance corrected for pore pressure effects (q_t), and shear wave velocity (V_S) is shown in Figure 1. Values corrected to an overburden stress of 1 atmosphere (about 100 kPa) — *i.e.*, $(N_1)_{60}$, q_{c1N}, and V_{S1}) are also shown in this figure. The site consisted of approximately 4 to 10 meters of sand and gravel fill underlain by a thick layer of non-plastic silt. The water table was encountered at a depth of about 1.6 meters below the ground surface. The silt was deposited as glacial lake sediments during the last glacial retreat and is characterized by seasonal varves. Grain size analyses of bulk samples indicated that the silt was composed of about 95 percent fines (less than 0.075 millimeters), and Atterberg Limits could not be determined. The silt was therefore classified as a non-plastic silt (ML) according to the Unified Soil Classification System. Previous geotechnical explorations indicated that the silt was underlain by a very thin layer of till overlying bedrock.[10] The depth to bedrock ranged from about 36 to 61 meters below the ground surface and the rock consisted of a sandstone conglomerate.

Assessment of the Seismic Hazard

The first step in evaluating the liquefaction potential at a particular site is to assess the seismic hazard at the site. This study could be a regional hazard assessment based on building code provisions,[11,12] or a site-specific hazard analysis that is typically performed for important and/or hazardous facilities.[13] Ultimately, the results of a seismic hazard analysis are the design ground motions that are required for an analysis of liquefaction potential.

In lieu of performing a site-specific seismic hazard analysis, the design ground motions used herein are in general accord with current seismic design recommendations and standards for new buildings. The 2003 edition of the National Earthquake Hazard Reduction Program (NEHRP) provisions recommends seismic design motions having a 2 percent probability of exceedance in 50 years (*i.e.*, motions having an approximately 2,500-year return period).[14] This criterion was also adopted by the 2006 International Building Code (IBC).[15] The seismic hazard maps that accompany these building codes and additional information available from the United States Geological Survey (USGS) were used to determine the design ground motions at the study site. This approach is particularly attractive because the USGS provides online access to the seismic hazard data that were used to develop the seismic hazard maps that accompany the NEHRP and the ICC provisions.[16] A

description of the procedures that were used to generate these map is given in Driscoll and Baise.[17] The hazard data can be accessed by entering the zip code or the latitude and longitude of the site of interest.

Methods Used to Evaluate Liquefaction Potential

Liquefaction potential was evaluated using a cyclic stress-based approach, which is commonly used in engineering practice.[8] The factor of safety against liquefaction is estimated at any depth in the soil profile from the following equation:

$$FS = CRR/CSR \quad (1)$$

Where:
CRR = cyclic resistance ratio of the soil; and
CSR = cyclic stress ratio induced in the soil by the earthquake.

Note that in using Equation 1 both CRR and CSR need to be evaluated for the same earthquake magnitude and initial stress conditions.

Cyclic Stress Ratio (CSR). The cyclic resistance correlations correspond to a magnitude 7.5 earthquake. Therefore, to be consistent with this magnitude, the CSR was calculated from the following expression:

$$CSR = 0.65 \cdot \frac{\tau_{max}}{\sigma'_{v0}} \cdot \frac{1}{MSF} \quad (2)$$

Where:
τ_{max} = maximum cyclic shear stress estimated at a given depth;
σ'_{v0} = initial effective overburden stress at a given depth; and
MSF = magnitude scaling factor.

The range for MSFs recommended by Youd *et al.* for M5.5 to M8.5 is shown in Figure 2.[8] For this study, an average of the recommended range was used and was extrapolated for M less than 5.5.

Site response analyses were performed using SHAKE91 to determine the maximum cyclic shear stress induced in the soil column.[18] The analysis required time histories of bedrock motion as inputs, along with a profile of soil stiffness and damping properties. The time histories were applied as outcrop motions at bedrock, which on average is at a depth of 49 meters below the ground surface. The small strain shear modulus (G_{max}) was computed from the shear wave velocity data obtained using the seismic cone (for example, see Figure 1) and estimated bulk density values. A bedrock shear wave velocity of 1,982 meters/second was used based on cross-hole seismic measurements made at a nearby site.[13] Shear modulus and damping properties as a function of strain were estimated at various depths using empirical relationships developed by Ishibashi and Zhang.[19]

One of the most important and challenging aspects of performing a site response analysis is selecting the input rock outcrop motions. It is common to generate synthetic motions that match a building code design spectrum or a uniform hazard spectrum (UHS) for the site of interest.[13,20] However, neither of these spectra is associated with a single earthquake scenario (i.e., a single magnitude, *M*, and site-to-source distance, *R*). Therefore, a limitation of this approach for evaluating liquefaction potential is the difficulty in determining an appropriate value of MSF to compute the CSR (see Equation 2).

As an alternative, the motions associated with one or more controlling earthquake scenarios can be used. These scenarios, called *modal events*, have the most significant contribution to the seismic hazard at the site. The *M* and *R* of the modal events can be determined from the deaggregation matrices for the 2,500-year UHS. The deaggregation matrices provide a breakdown of the percent contribution of a given earthquake scenario to each spectral acceleration defining the UHS. The deaggregation matrices can be obtained directly from the USGS website for spectral accelerations defining the 5 percent damped, 2,500-year UHS at periods of 0.0, 0.1, 0.2, 0.3, 0.5, 1.0 and 2.0 seconds.[16] As an example, the deaggregation matrices for 0.0- and 2.0-second spectral accelerations for Providence are plotted as histograms in Figures 3 and 4. As may be observed from these figures, the predominant modal event for the 0.0-second spectral accel-

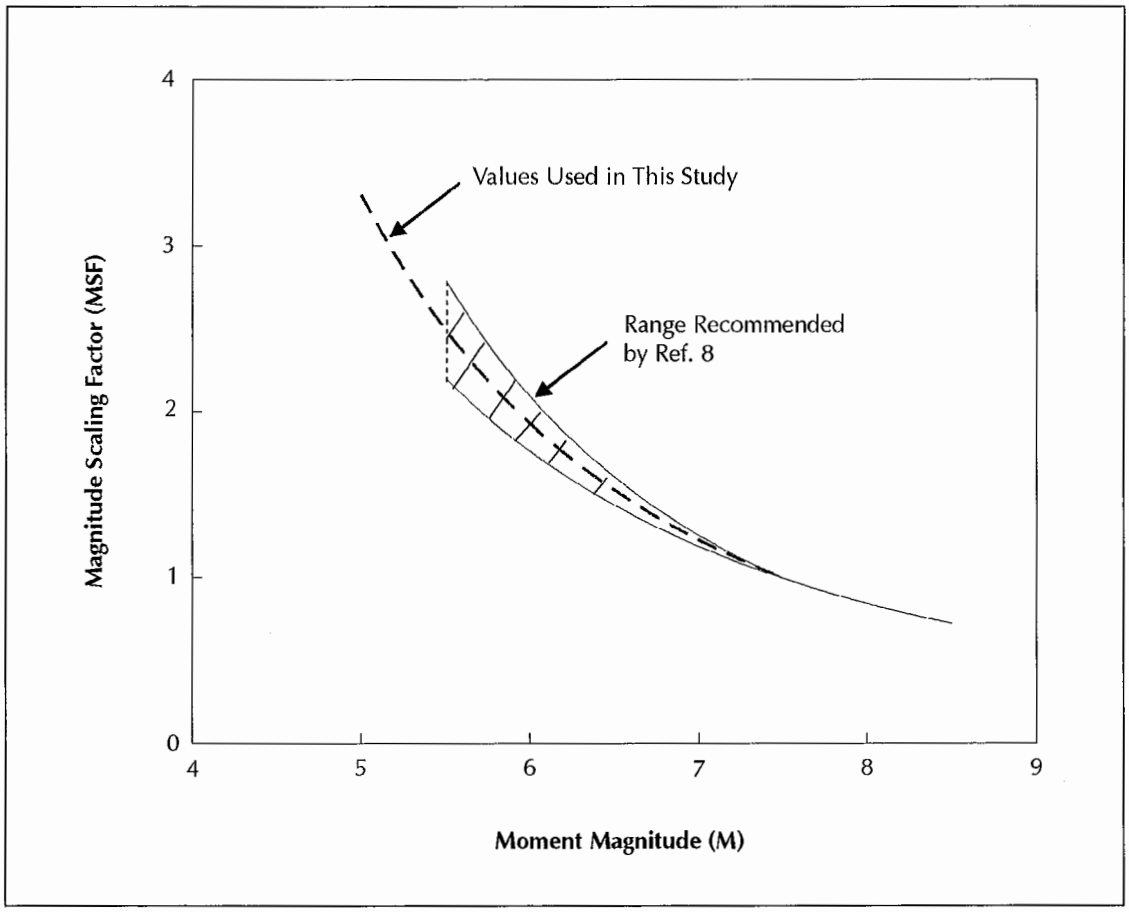

FIGURE 2. Magnitude scaling factors used in this study.

eration (*i.e.*, peak ground acceleration) is M of about 5 and R of about 15 kilometers. In contrast, the predominant modal event for the spectral acceleration corresponding to 2.0 seconds is M greater than 7 and R about 400 kilometers. Other modal events may be observed for both the peak ground acceleration and the 2.0-second spectral acceleration, but they are less pronounced than those identified.

For liquefaction evaluations, the modal events associated with the spectral accelerations at oscillator periods closest to the fundamental period of the site are of most interest. This is because motions corresponding to these events will induce the largest cyclic shear stresses in the soil column. A preliminary estimate of the fundamental period (T) can be made using the following equation:[21]

$$T = 4H/V_{S\,avg} \tag{3}$$

Where:

H = total thickness of the soil profile; and

$V_{S\,avg}$ = average shear wave velocity of the soil profile.

$V_{S\,avg}$ can be obtained by equating the travel time for a seismic wave to propagate from bedrock to the ground surface in the actual soil profile and an "equivalent" uniform profile:

$$V_{S\,avg} = \frac{H}{\sum_{i=1}^{m} \frac{h_i}{V_{Si}}} \tag{4}$$

Where:

h_i = thickness of sublayer i;

V_{Si} = shear wave velocity in sublayer i; and

m = total number of sublayers.

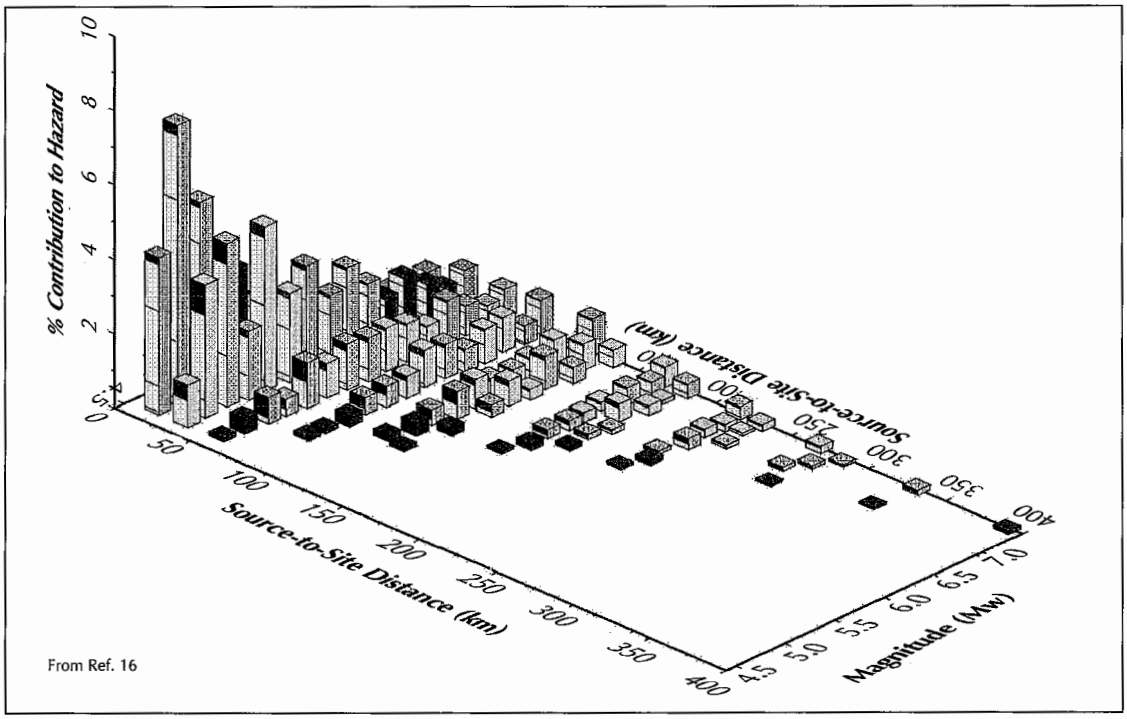

FIGURE 3. Deaggregation data for the study site for 2,500-year ground motions at a period of 0.0 seconds.

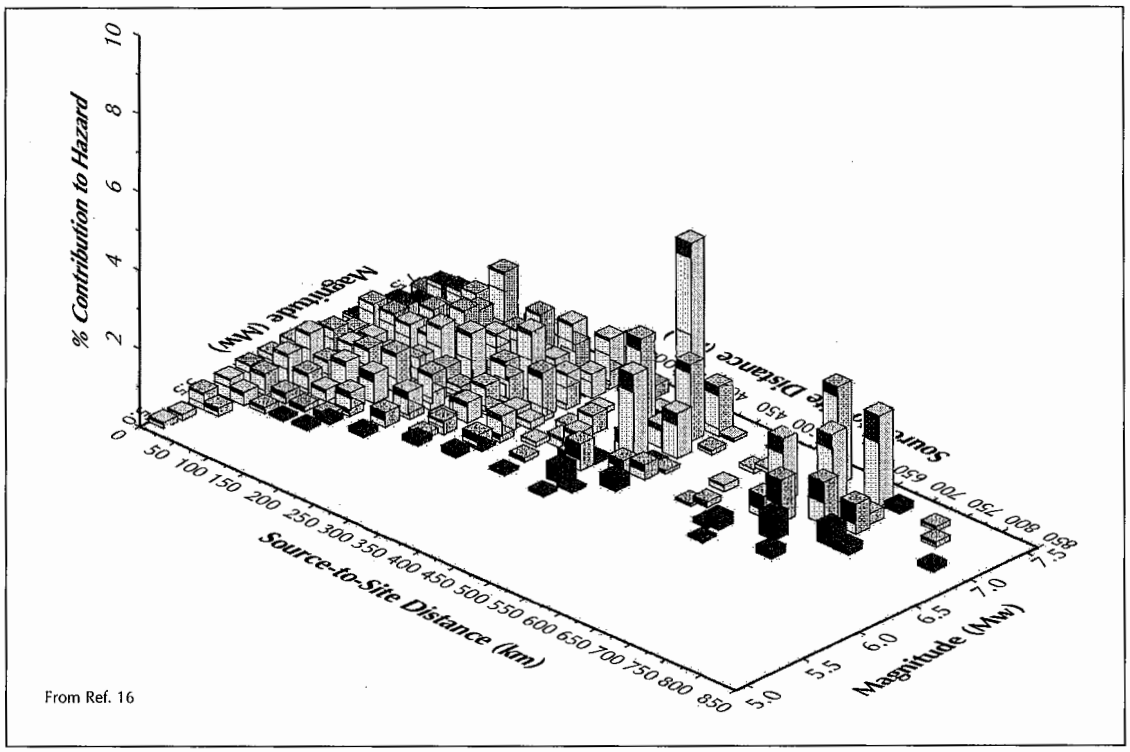

FIGURE 4. Deaggregation data for the study site for 2,500-year ground motions at a period of 2.0 seconds.

Using this equation, the fundamental period of the study site profile was determined to be approximately 0.85 seconds.

The closest oscillator periods for which the deaggregation matrices are provided by the USGS for the 5 percent damped, 2,500-year spectral accelerations are 0.5 and 1.0 seconds. Based on these matrices, two predominant modal events were identified:

- a small, local event with *M* approximately equal to 5.4 and *R* approximately equal to 15 kilometers; and,
- a large, distant event with *M* approximately equal to 7.4 and *R* approximately equal to 400 kilometers.

Having two predominant modal events corresponding to a small, local event and a large, distant event is characteristic of the seismic hazard in the central and eastern United States. Specific to Providence, the small, local event is associated with a background areal source used in the probabilistic seismic hazard analysis, and the large, distant event is associated with the Charlevoix Seismic Zone, which is located on the St. Lawrence River about 100 kilometers downstream (northeast) from Quebec City.

Obtaining earthquake records representative of these modal events is particularly difficult in the northeastern United States because there are limited recorded ground motions for earthquakes having M greater than or equal to 4.5. The only sizable events for which motions were recorded in this region were the 1988 M5.9 Saguenay earthquake and aftershocks, which occurred just south of Chicoutimi, Quebec, Canada, in a relatively aseismic region 75 kilometers north of the Charlevoix Seismic Zone. As a result, it is common practice in the Northeast to generate and use synthetic ground motions having characteristics representative of the modal events.

As an alternative to using synthetic motions, ground motion records from a database compiled by McGuire *et al.* were used.[22] McGuire *et al.* "scaled" motions recorded in active seismic regions (*i.e.*, inter-plate motions) using response spectral transfer functions that relate active seismic region motions to stable continental region motions (*i.e.*, intra-plate motions).[22] The transfer functions were based on a single-corner, point source model.[22-24] The use of scaled motions is preferred over synthetic ones because the former preserve many of the characteristics of the original recorded motions, particularly the distribution of energy over the time of shaking. An example of a scaled time history and associated response spectrum is shown in Figure 5.

TABLE 1.
Binned Scaled Rock Motions

Bin	*M*	*R* (km)
1	5-6	0-50
2	5-6	50-100
3	6-7	0-10
4	6-7	10-50
5	6-7	50-100
6	6-7	100-200
7	7+	0-10
8	7+	10-50
9	7+	50-100
10	7+	100-200

Note: *M* is defined using the moment magnitude scale and *R* as the closest distance to the fault.

McGuire *et al.* binned the scaled rock motions in their database according to magnitude (*M*) and site-to-source distance (*R*).[22] In general, motions recorded on profiles having an average shear wave velocity of the upper 30 meters (V_{S30m}) greater than 360 meters per second were classified as "rock motions." This criterion includes NEHRP site classes A, B, and C.[14] The motions were further subdivided into 10 *M-R* bins, each containing at least 30 records. These bins were as noted in Table 1.

For each of the predominant modal events identified above, six representative time histories were selected from the McGuire *et al.* database.[22] The records for the large, distant event (*i.e.*, *M* approximately 7.4 and *R* approximately 400 kilometers) were taken from Bin 10 (see Table 1). Additional scaling was then applied

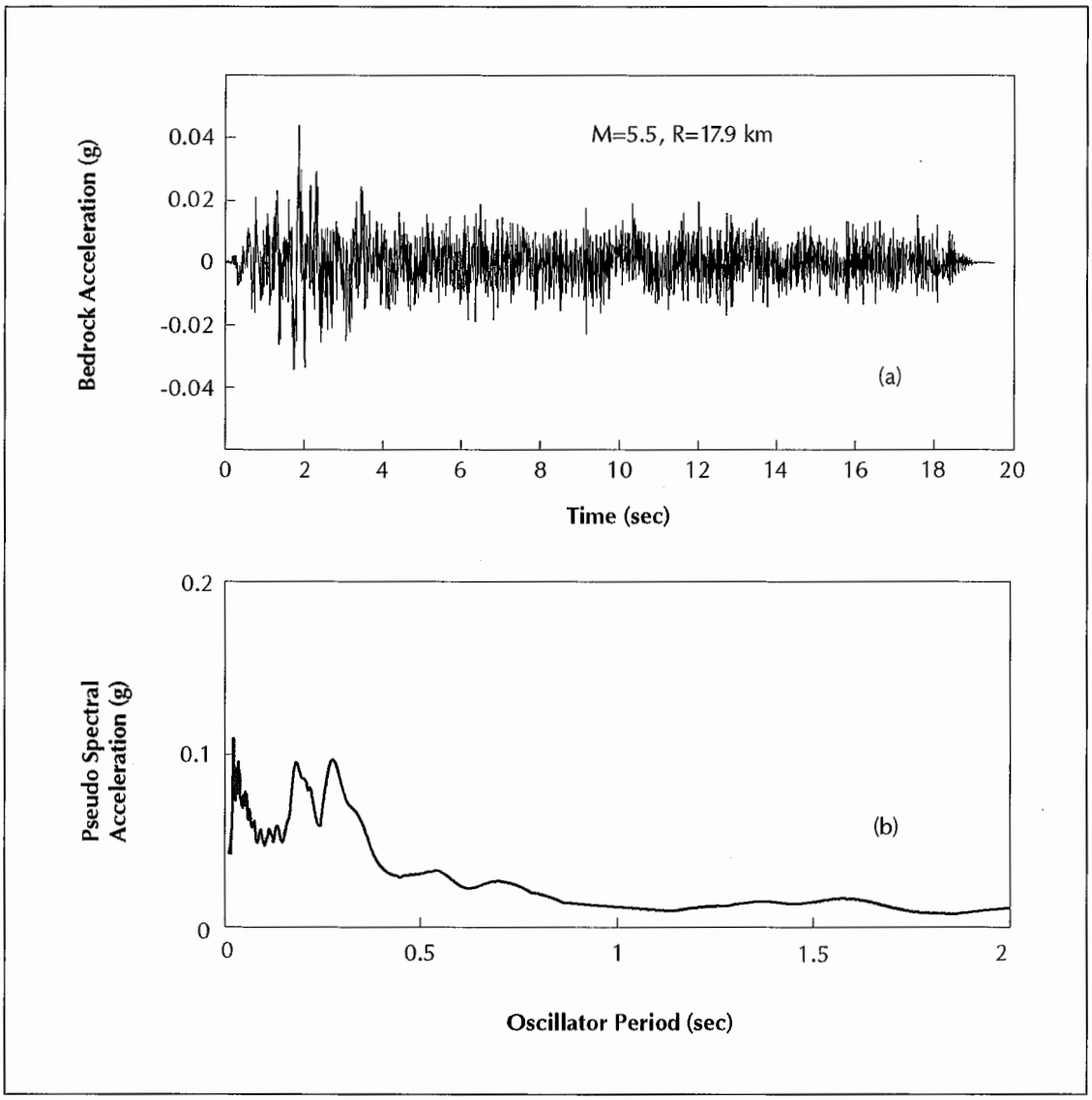

FIGURE 5. An example of a) scaled bedrock time history used in the site response analyses and b) the corresponding 5 percent damped response spectrum.

to these records to account for differences in the site-to-source distances. The scaling approach used was the same as that used by McGuire *et al.* to initially scale the motions. Although there are other studies in which the amplitude of recorded motions were scaled to match the 2,500-year spectral acceleration at a specific oscillator period, this scaling was not done since the basis for this additional amplitude scaling is questionable. The approach used herein is a straight deterministic analysis where the earthquake scenarios used in the analysis corresponded to the predominant modal events from the deaggregation of the probabilistic seismic hazard.

The twelve selected records were used in the site response analyses to compute the CSR induced in the soil profile as a function of depth. The median CSR profiles for each modal event are shown in Figure 6. As shown in this figure, the CSR profile for the smaller, local event is significantly higher than that for the larger, distant event, although both CSR profiles are relatively low.

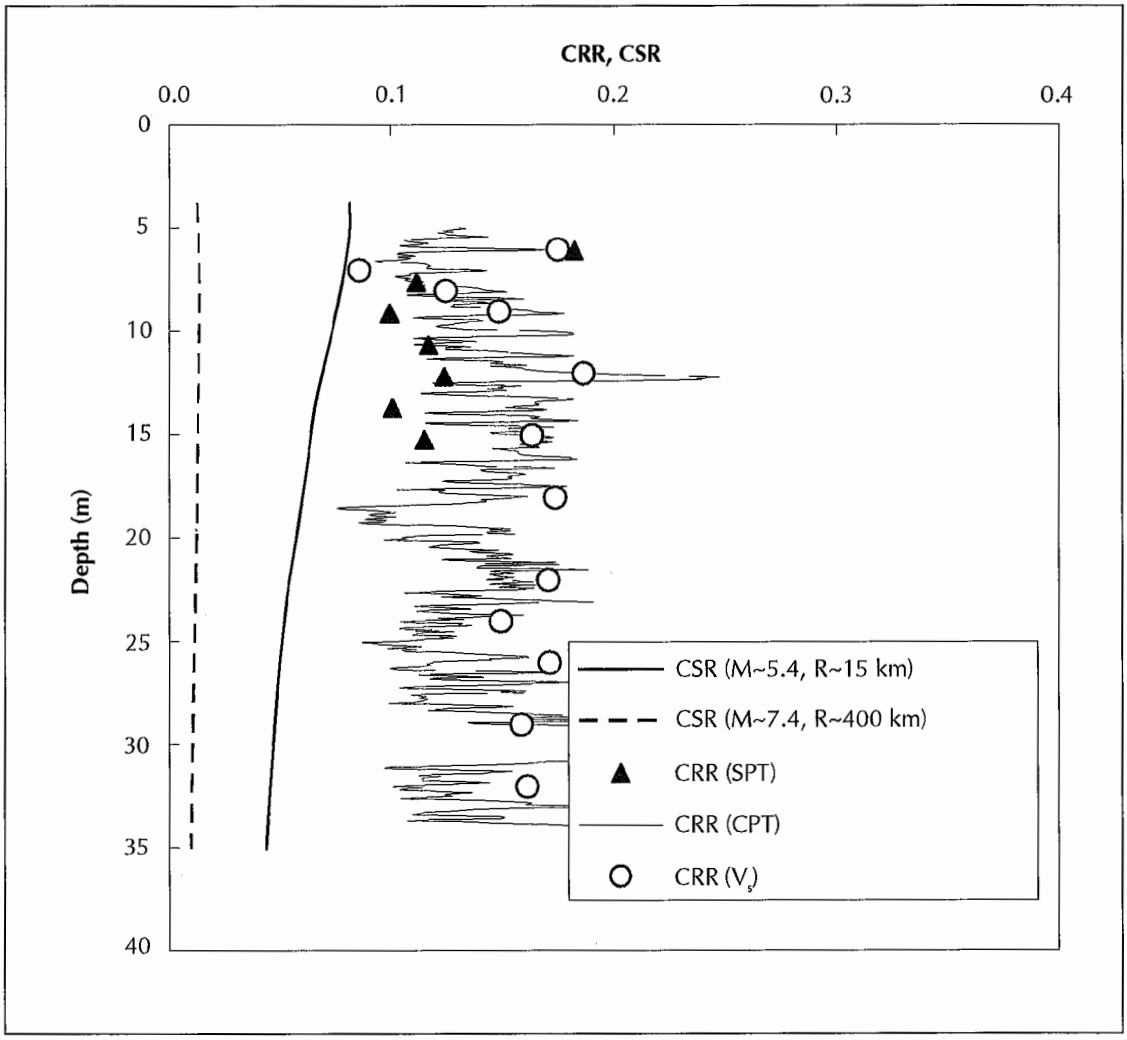

FIGURE 6. CSR profiles for a small magnitude, local event and a large magnitude, distant event and CRRs estimated using the SPT-, CPT- and VS-based methods.

Cyclic Resistance Ratio (CRR). To avoid problems associated with sample disturbance, the trend since the early 1980s has been to use in-situ test-based correlations to estimate cyclic resistance of soils.[25] Correlations with three different types of in-situ tests were used to estimate cyclic resistance at the site including the SPT, CPT and shear wave velocity (V_S). The former two were existing correlations, while the latter was developed specifically for the silt encountered at the study site.

The SPT-based approach was initially proposed independently by Seed and Idriss,[26] and independently by Whitman.[27] The current standard-of-practice is described in Youd *et al.*[8] and utilizes the data and correlations of Seed *et al.* with some modifications.[28] Cyclic resistance is correlated to $(N_1)_{60}$, defined as the blow count corrected to an effective overburden stress of 1 atmosphere (approximately 100 kPa) and a hammer efficiency of 60 percent. Since the majority of the soils used to develop the CRR correlation contained less than 35 percent fines, there is some uncertainty in using the existing correlations with the pure non-plastic silts commonly encountered in the Providence area.[29] However, the CRR profile for the site was computed using the $(N_1)_{60}$ values and the recommended fines corrections down to a depth of about 15 meters and is plotted in Figures 6. As

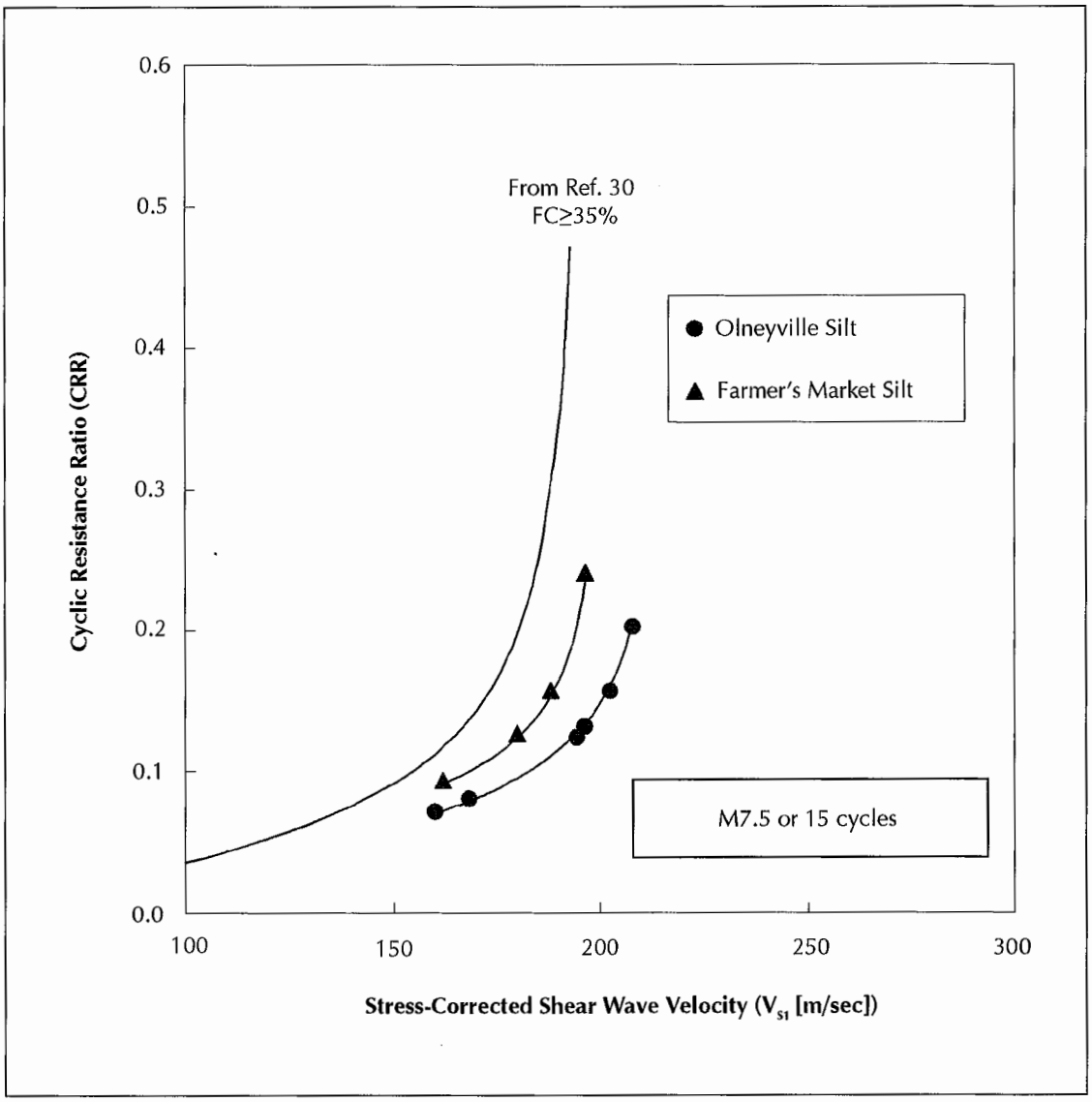

FIGURE 7. Shear wave velocity-based cyclic resistance correlations developed for two Providence silts, including silt recovered from the Farmers' Market site (after Ref. 9). The field based correlation developed by Andrus and Stokoe (Ref. 31) is also shown.

shown in Figure 6, the CRR at a depth of 6 meters is approximately 0.18 and then drops to approximately 0.1 at greater depths.

The procedure developed by Robertson and Wride forms the current standard of practice for evaluating liquefaction potential using CPT.[8,30] This approach correlates CRR with cone tip resistance normalized (q_{c1N}) to an effective overburden stress of 1 atmosphere (about 100 kPa). Like the SPT-based procedure, the cyclic resistance correlation was developed from case histories in relatively clean sands containing less than about 35 percent fines, and its applicability for the Providence silt is uncertain. Using q_{c1N} values for the study site (see Figure 1) and the recommended fines corrections, the CRR profile for the site was computed down to a depth of about 34 meters and is plotted in Figure 6. As shown in that figure, the CRR for the profile varies between about 0.1 to about 0.19 for the depths analyzed.

The current state-of-practice for evaluating liquefaction resistance using shear wave velocity field data was developed by Andrus and Stokoe,[31] and is also described in Youd *et al.*[8] The correlation relates CRR to shear wave velocity (V_{S1}) normalized to an effective overburden stress of 1 atmosphere. Recently, Baxter *et al.* developed similar correlations from an extensive laboratory study of two Providence silts, including silt obtained from the Farmers' Market site.[9] These two new correlations are shown in Figure 7. Cyclic triaxial tests were performed on both reconstituted[32] and block samples of the silt, where the V_S of the samples was measured using bender elements that were incorporated into the triaxial apparatus.[9] The difference between the two laboratory curves shown in Figure 7 further supports previous research in sands that has suggested that CRR-V_{S1} correlations are soil specific. Also note that the use of Andrus and Stokoe's field-based correlation for fines content (FC) greater than or equal to 35 percent would overestimate the CRR of the Providence silts.

The CRR-V_{S1} correlation developed for the Farmers' Market silt, shown in Figure 7, was used to calculate the CRR profile down to a depth of about 33 meters and is plotted in Figure 6. Note that the majority of the V_{S1} values for the profile (see Figure 1) fell within the range of the CRR correlation shown in Figure 7. As shown in Figure 6, the CRR for the profile reaches a low of approximately 0.09 at a depth of about 7 meters and then progressively increases over the next 3 meters, reaching a relatively constant value of about 0.16 for greater depths.

Discussion of Results

There is some uncertainty when using the SPT- and CPT-based CRR correlations for silt because these correlations were developed primarily from soils having fines contents less than 35 percent. The soil-specific V_S correlation is most representative of the in-situ strength of the silt at the study site, and was used as a baseline for a comparison of SPT- and CPT-based methods.[33] As shown in Figure 6, there is good agreement between the CRR values predicted by the CPT and soil-specific V_S correlations. In contrast, the SPT-based correlation consistently predicted the lowest CRRs. This occurrence is most likely due to the higher uncertainty of the SPT method itself,[34,35] with the increased scatter in the SPT data resulting in a more conservative placement of the cyclic resistance curve.

Figure 6 also illustrates some of the benefits of using CPT to evaluate liquefaction resistance. First, an almost continuous profile of CRR is obtained, which allows improved characterization of thin strata. Also, unlike SPT, CPT is entirely automated and, thus, less subject to human error that inherently leads to lower uncertainty in the results. However, from experience at other sites, both SPT and CPT may give erroneously high resistances in soil containing gravel; in which case the V_S-based procedure is a viable alternative for estimating cyclic resistance.

Liquefaction is predicted at depths where the CSR is greater than or equal to the CRR resulting in a factor of safety less than or equal to unity (see Equation 1). As shown in Figure 6, the factor of safety against liquefaction is significantly greater than one for the CSR profile computed for the large, distant earthquake. For the small local event, the factor of safety is also greater than one at all depths evaluated, although it is close to one at a depth of approximately 7 meters when the CRR is estimated using the soil-specific V_S correlation. Therefore, the analysis suggests that for the earthquake scenarios that contribute the most to the 2,500-year earthquake ground motions, there is a low potential for liquefaction at the site.

Summary & Conclusions

Recent geological, seismological and paleoseismological studies in the northeastern United States provide important clues regarding past occurrences of soil liquefaction in this region and, hence, highlight the future potential for liquefaction. Assessing liquefaction potential in the Providence, Rhode Island, area is complicated because of the uncertainty in the seismic ground motions as well as the uncertainty of the cyclic resistance of the silts

that underlie the city. To provide insight on this issue, a deterministic evaluation of liquefaction potential at a study site in Providence using a cyclic stress-based approach was presented.

The CSR was estimated for two earthquake scenarios. These scenarios corresponded to the two predominant modal events identified in the deaggragation matrices obtained from the USGS for the 2,500-year spectral accelerations for oscillator periods closest to the fundamental period of the site. The two scenarios considered were a small event (M5.4) occurring locally, and a large event (M7.4) originating from the Charlevoix Seismic Zone, Canada. The CRR of the silt at the study site was evaluated using empirical correlations based on SPT blow count, CPT tip resistance and in-situ shear wave velocity. The shear wave velocity correlation used in the analysis was developed from an extensive laboratory study[9] using the silt recovered from the study site. A comparison of the CRRs resulting from the three correlations suggests that the SPT-based method gave the lowest predictions, while the CPT-based method provided the highest resolution with depth.

Comparing the CSR and CRR predictions indicate that there is a low potential for liquefaction at the study site. However, given that the results depend to a large extent on the site characteristics, this finding is only applicable to the study site. This narrow finding further reinforces the need to perform site-specific evaluations of liquefaction potential for a given project.

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A.S. BRADSHAW *is an Assistant Professor in the Department of Civil Engineering at Merrimack College. He holds degrees from Tufts University (B.S.C.E.) and the University of Rhode Island (M.S.O.E. and Ph.D.C.E.). His research has focused primarily on marine geotechnical engineering and geohazards issues.*

R.A. GREEN *is an Associate Professor in the Department of Civil and Environmental Engineering at the University of Michigan. He holds degrees from Rensselaer Polytechnic Institute (B.S.C.E.), University of Illinois at Urbana-Champaign (M.S.C.E.) and Virginia Tech (Ph.D.). His research primarily focuses on earthquake engineering issues in the central and eastern United States.*

C.D.P. BAXTER *is an Associate Professor in the Departments of Ocean/Civil and Environmental Engineering at the University of Rhode Island. He holds degrees from Tufts University (B.S.C.E.), Purdue University (M.S.C.E.) and Virginia Tech (Ph.D.), and has been a post-doctoral fellow at the University of Rhode Island and the Norwegian Geotechnical Institute. His research interests include submarine slope stability and dynamic properties of soils.*

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