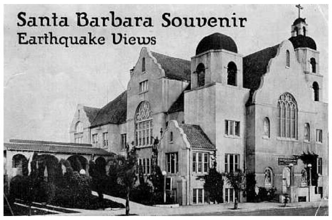

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SITE FACTORS FOR ESTIMATING PEAK GROUND ACCELERATION IN CHARLESTON, SOUTH CAROLINA, BASED ON V_{S30}

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ABSTRACT

Site factors for estimating peak ground surface acceleration in Charleston, South Carolina, based on average shear wave velocity in the top 30 meters (V_{S30}) are derived in this paper from one-dimensional, total stress equivalent linear and nonlinear dynamic response simulations with representative soil/rock profiles and ground motions. The site factors are determined by dividing the peak ground acceleration (PGA) obtained from dynamic site response analysis by the PGA for the soft rock outcrop. From the results of over 9000 simulations, plots of site factor versus V_{S30} are prepared for soft rock outcrop motions with PGAs of 0.05, 0.1, 0.2, 0.3, 0.4 and 0.5 g. Site factors generally decrease with increasing PGA for a given V_{S30}. The plots exhibit peak site factors between V_{S30} values of 165 and 420 m/s, depending on soft rock outcrop PGA. The computed site factors are significantly less than the NEHRP site factors for all values of outcrop PGA when $V_{S30} \leq 180$ m/s. When $V_{S30} \geq 180$ m/s, the computed site factors are similar to the NEHRP site factors.

INTRODUCTION

Surface geology can significantly influence the amplitude, frequency content and duration of seismic motions felt at the ground surface. Kramer (1996) reviewed several notable cases where distinct differences in seismic motions occurred at neighboring sites with different geologic and soil conditions. Near-surface conditions of importance include the thickness of soil layers, the small-strain stiffness and material damping of soil layers, the variation of stiffness and material damping with shear strain amplitude, and the site topography.

Small-strain stiffness of soil and rock is often represented by shear wave velocity (V_s). The significant contribution of V_s to seismic motion has been noted by many investigators (e.g., Seed et al., 1976; Idriss, 1990; Borcherdt, 1994; Boore et al., 1994; Joyner et al., 1994; Midroikawa et al., 1994) and has led to a seismic site classification system used in building codes and guidelines (BSSC, 1995, 2010; Dobry et al., 1999; ICC, 2000; SCDOT, 2008).

The seismic site classification system used in many building codes and guidelines is based on the average shear wave velocity in the top 30 m (V_{S30}) defined as (Borcherdt, 1994):

$$V_{s30} = \frac{30}{\sum_{i=1}^{n} \frac{H_i}{V_{si}}} \tag{1}$$

where H_i is the thickness in meters of layer i; V_{si} is the shear wave velocity in m/s of layer i; and n is the number of layers in the top 30 m. Values of $V_{S30} > 1500$ m/s, $760 < V_{S30} \leq 1500$ m/s, $360 < V_{S30} \leq 760$ m/s, $180 < V_{S30} \leq 360$ m/s, and $V_{S30} \leq 180$ m/s correspond to site classes designated as A, B, C, D and E, respectively. These site classes are often referred to as the National Hazard Reduction

Program (NEHRP) site classes after the recommended provisions published in BSSC (1995) where they were first introduced.

The NEHRP site classes are used in simplified design procedures to select factors for adjusting available estimates of peak ground surface acceleration and other spectral values available for soft rock sites with V_{S30} = 760 m/s. This paper deals with the site factor for peak ground acceleration (F_{PGA}) which is defined as:

$$F_{PGA} = \frac{PGA}{PGA_{outcrop}} \tag{2}$$

where PGA is the peak ground surface acceleration at the site; and $PGA_{outcrop}$ is the peak ground surface acceleration at the soft rock outcrop.

Recent studies sponsored by the South Carolina Department of Transportation (SCDOT) have shown that F_{PGA}, and other spectral site factors, computed from results of site-specific ground response analysis can be significantly different from values recommended in the NEHRP provisions (BSSC, 1995, 2010) and adopted in building codes (ICC, 2000). Sometimes the difference is on the unconservative side. There is a need to better understand and to quantify the uncertainty associated with the NEHRP site factors used in simplified seismic design procedures.

The objective of this paper is to compute values of F_{PGA} for conditions typical of Charleston, South Carolina, and compare the computed F_{PGA} values with values recommended in the building codes. Computed F_{PGA} values are plotted with respect to V_{S30}. Median, 10% lower bound, and 90% upper bound curves are established for the plotted V_{S30}-F_{PGA} data pairs.

GEOLOGY AND SEISMOLOGY

Presented in Fig. 1 is a map of the Charleston area. Gridlines in the figure represent 7.5-minute quadrangle boundaries. The Charleston area is located in the Coastal Plain physiographic province where near-surface sediments are typically unconsolidated Quaternary deposits ranging from beach/barrier island sand to estuarine sand and clay to fluvial sand and silt (McCartan et al., 1984; Weems et al., 2011). Underlying the Quaternary sediments are compacted and weakly litified Tertiary and Cretaceous sediments that extend to depths of 700 to 1000 m in the area shown in Fig. 1 (Chapman and Talwani, 2002). Beneath the Coastal Plain sediments are hard Mesozoic/Paleozoic basement rock. Both the younger and the older Coastal Plain sediments can significantly influence PGA.

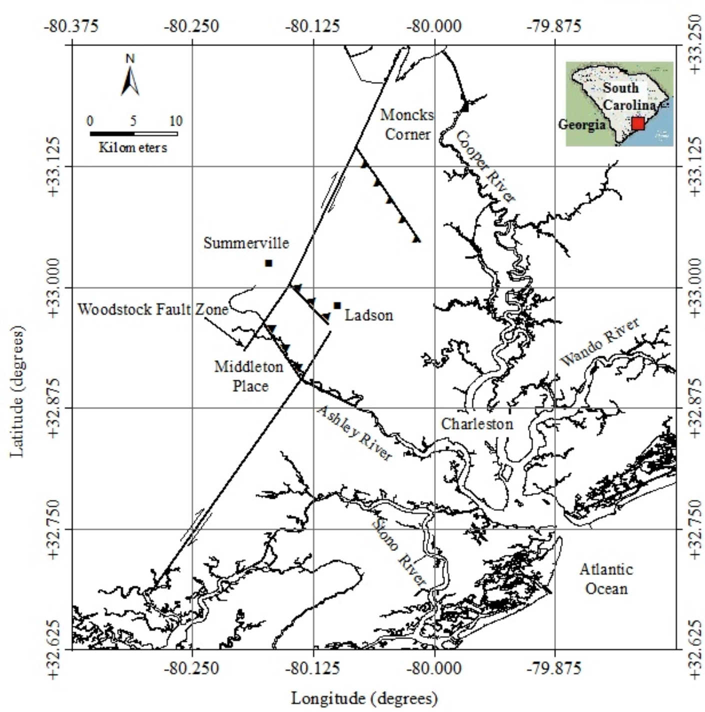

Fig. 1. Map of the Charleston area showing the Woodstock fault zone as delineated in Durá�Gómez and Talwani (2009).

The Charleston earthquake of August 31, 1886 with moment magnitude of 6.9 ± 0.3 (Bakun and Hopper, 2004) to 7.3 ± 0.3 (Johnston, 1996) is the largest historic earthquake to have occurred in the eastern United States (Bollinger, 1977). The epicentral area was located in the general area of Summerville, Ladson and Middleton Place (Dutton, 1889). Plotted in Fig. 1 is the Woodstock fault zone as delineated in Durá-Gómez and Talwani (2009), which is the likely source of the 1886 earthquake.

Based on paleoliquefaction investigations, several prehistoric liquefaction-inducing earthquakes have occurred in the Coastal Plain of South Carolina during the last 6000 years (Talwani and Schaeffer, 2001). The recurrence rate for 1886-like earthquake is estimated to be between 500 and 600 years.

DYNAMIC SOIL/ROCK MODEL

The dynamic soil/rock model used in the ground response analysis consists of small-to-large-strain shear wave velocity (or shear modulus) and material damping ratio relationships for each layer down to a soft rock half-space with V_s = 760 m/s, which corresponds to the NEHRP B-C boundary. Small-strain shear wave velocity and shear modulus are directly related by

$$G_{\max} = \rho V_s^2 \tag{3}$$

where G_{max} is the small-strain shear modulus; and ρ is the mass density of soil. Presented in Fig. 2 are thirty-two V_s profiles that were used to represent the soil/rock conditions in the Charleston area.

The reference V_s profile shown in Fig. 2 above the depth of 80 m is based on results of the study by Andrus et al. (2006). Andrus et al. (2006) compiled V_s data from in situ measurements performed at ninety-one sites by different investigators (i.e., Applied Research Associates, Inc.; ConeTec, Inc.; Georgia Institute of Technology; Gregg In Situ, Inc.; RedPath Geophysics; S&ME, Inc.; WPC, Inc.; and U.S. Geological Survey) during the years of 1998 to 2004. Between the depths of 0 to 10 m, V_s is 190 m/s in the reference profile and corresponds to the average value for the 100,000-year-old Wando Formation. Between the depths of 10 and 80 m, V_s increases from 400 to 530 m/s which are average values for Tertiary-age sediments at the respective depths.

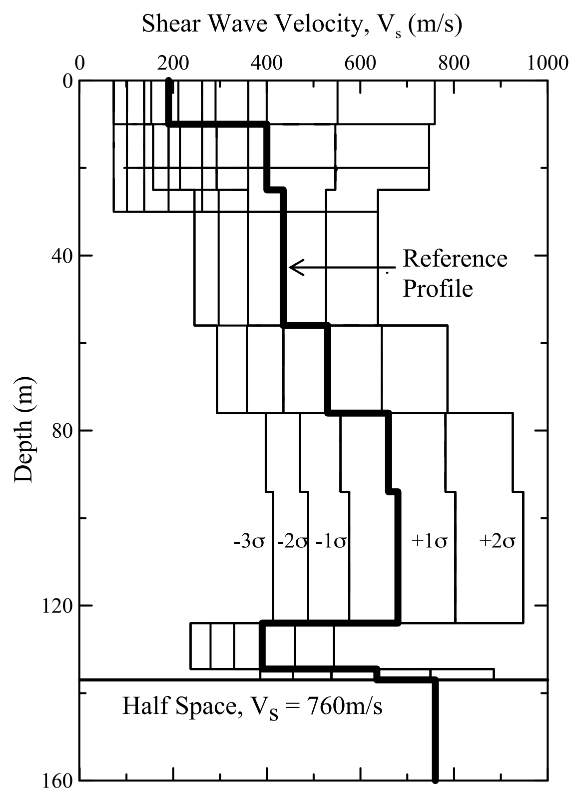

Fig. 2. Shear wave velocity profiles considered in the analysis.

Below the depth of 80 m, values of V_s in the reference profile are based on average suspension logger measurements made by GEOVison, Inc. in 2006 for SCDOT. The suspension logger measurements were extended to a depth of 240 m. Although measured V_s values below 137 m were fairly constant with an average of 640 m/s, a soft rock half-space with V_s = 760 m/s was assumed in the ground response analysis of this study. Results of ground response analysis performed with half-space V_s values of 700 and 760 m/s indicated that the variations caused by the half-space V_s assumption are small compared to other factors. For this reason, a half space of 760 m/s is considered an adequate assumption.

The other thirty-one V_s profiles plotted in Fig. 2 were created to represent the range of likely variations in thickness of the Quaternary and V_s of the Quaternary and Tertiary within the study area. Quaternary thicknesses were assumed to be 0, 10, 20, and 30 m. Variations in V_s were included by applying ±1, ±2 and -3 standard deviations (σ) of $\ln(V_s)$ to the reference profile above the half space. The mean values of σ reported by Andrus et al. (2006) were 0.32 for the Wando Formation, and 0.14 to 0.31 for the Tertiary-age sediments.

Each layer in the V_s profiles was divided into sublayers such that the computed fundamental frequency (= $V_s/4H$, where H is the thickness of the sublayer) was at least 25 Hz. The 25 Hz frequency requirement was used because frequencies above 10-25 Hz contain a relatively small amount of energy of the earthquake loading and the amplitude of these frequencies can often be set to be equal to 0 without causing any significant change in the responses within the soil/rock system (Schnabel et al., 1972). Several simulations were conducted to show the criterion that layer frequency ≥ 25 Hz was sufficient to ensure ''layer-independent'' results.

Normalized shear modulus (G/G_{max}) and material damping ratio (D) relationships developed by Zhang et al. (2005) for Quaternary and Tertiary and older sediments were used to model variations with shear strain (γ). Selection of these relationships was based on reasonable values of plasticity index (PI) and mean effective confining pressure (σ'_m). Relationships for ± 1σ based on Zhang et al. (2008) were also assumed to capture the uncertainty associated with the Zhang et al. (2005) relationships. Presented in Fig. 3 are sample G/G_{max}-γ and D-γ relationships for depths of 0.5 m (or mean effective confining stress of 15 kPa) and 137 m (or mean effective confining stress of 1400 kPa).

For the soft rock half-space, purely linear relationships of G/G_{max}-γ and D-γ were assumed. This was done by entering G/G_{max} = 1 and D = 0.5% for all γ values. A damping ratio of 0.5% was taken to be representative for B-C material in the South Carolina Coastal Plain (SCDOT, 2008).

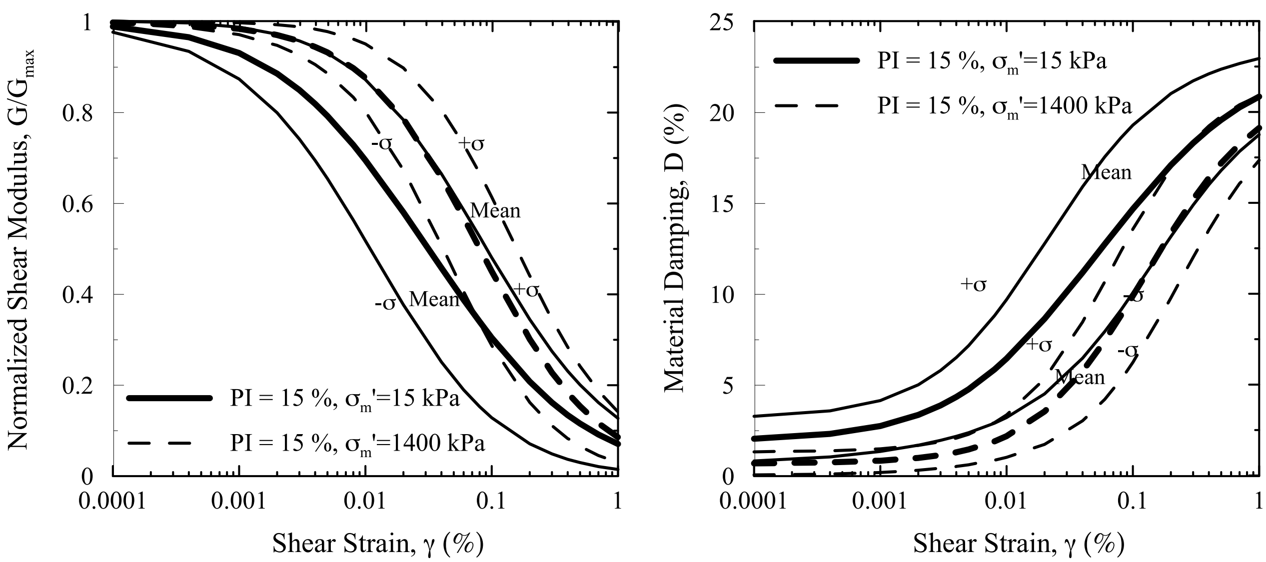

Fig. 3. Sample G/G_{max}-γ and D-γ relationships for depths of 0.5 and 137 m (Zhang et al., 2005, 2008).

INPUT GROUND MOTIONS

Because actual strong motion records from the Charleston area were not available, synthetic soft rock outcrop motions were generated using the computer program called Scenario_PC developed by Chapman (2006) for seismic hazard analysis in South Carolina. Scenario_PC is based on a point-source stochastic model (Boore, 1983; Boore and Atkinson, 1987; Atkinson and Boore, 1995) to simulate outcrop motions. Input variables needed for Scenario_PC include: (1) rock model; (2) earthquake moment magnitude; (3) source-to-site distance; and (4) return period.

Two rock models are available in Scenario_PC which are based on the study by Chapman and Talwani (2002). The first rock model is referred to as the ‘‘geologic realistic condition’’ and involves a very thick outcropping layer of soft rock with V_s = 760 m/s. The second rock model is referred to as the ‘‘hard-rock outcropping condition’’ which consists of a weathered rock layer with V_s gradient from 760 to 2500 m/s on top of unweathered hard rock (V_s=3500 m/s). For this study, the ‘‘geologic realistic condition’’ was assumed. The advantage of assuming the ‘‘geologic realistic condition’’ is that the input V_s profile only needs to extend to a depth of 137 m, the assumed top of V_s = 760 m/s material, versus a depth of 700-1000 m which is the top of hard rock in the Charleston area.

Input earthquake moment magnitudes and source-to-site distances were obtained for the centers of the 7.5-minute quadrangles shown in Fig. 1 and the return periods corresponding to 10% and 2% probabilities of exceedance in 50 years (or 15% and 3% probability of exceedance in 75 years) using the 2002 USGS deaggregation program (https://geohazards.usgs.gov/deaggint/2002/index.php; accessed March 26, 2010). Earthquakes with these return periods are referred to as the Functional Evaluation Earthquake (FEE) and the Safety Evaluation Earthquake (SEE), respectively, in the SCDOT Geotechnical Design Manual (SCDOT, 2008). The 2002 USGS interactive deaggregated program provided results indicating moment magnitudes of 7.3-7.4 in the vicinity of the Woodstock fault zone as the modal contributors to the seismic hazard in the Charleston area for both return periods. Moment magnitudes greater than 7.2 contributed to 15% and 12% of the seismic hazard for the SEE and FEE conditions, respectively. Based on these results a magnitude of 7.3 was assumed for both the SEE and FEE conditions.

Presented in Fig. 4 are the synthetic motions generated for the FEE and SEE conditions, respectively, at the center of the Charleston quadrangle. These motions along with seven other sets (for centers of the seven neighboring quadrangles) with source-to-site distances of 15 to 36 km were scaled to have PGA values of 0.05, 0.1, 0.2, 0.3, 0.4 and 0.5 g for use in the ground response analysis. Eight sites (center of the Charleston quadrangle and centers of seven neighboring quadrangles), two earthquake evaluation conditions (FEE, SEE), and six PGA scaling values leads to a total of 96 acceleration time histories that were used as input soft rock outcrop motions.

The synthetic acceleration time histories were transformed into the frequency domain to evaluate how amplitudes are distributed among different frequencies. For both time histories plotted in Fig. 4, the predominant frequency (or period) of the ground motion is around 2.4 ± 0.2 Hz (0.42 ± 0.02 s). The bandwidth, defined in Kramer (1996) as the range of frequencies (or periods) from the first exceedance of the maximum amplitude divided by √2 to the last exceedance of the maximum amplitude divided by √2 is 1.51 to 5.55 Hz (0.18 to 0.66 s) for both time histories.

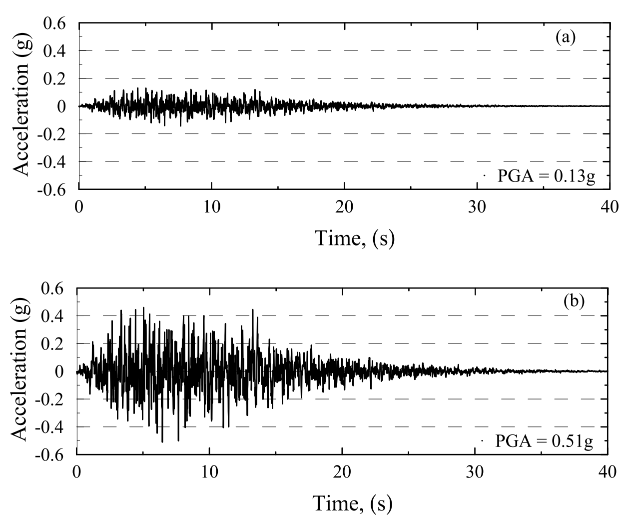

Fig. 4. Synthetic outcrop motions generated by Scenario_PC for the Charleston quadrangle for (a) 10% probability of exceedance in 50 years, and (b) 2% probability of exceedance in 50 years.

GROUND RESPONSE ANALYSIS

Total stress ground response analysis was performed using the computer programs SHAKE2000 (Ordóñez, 2011) and D-MOD2000 (Matasović and Ordóñez, 2011). Both programs assume one-dimensional ground response analysis with vertically propagating shear waves, which is considered appropriate because much of the Charleston area is flat with ground surface elevations less than about 15 m above mean sea level.

SHAKE2000 is based on the original SHAKE program by Schnabel et al. (1972) and uses an equivalent linear formulation. Although a nonlinear formulation is preferred for modeling nonlinear systems, SHAKE2000 is considered adequate when computed PGA is less than 0.4 g and computed values of γ are less than 2% (Kramer and Paulsen, 2004). The advantage of SHAKE2000 is that it takes less time to run than a computer program based on a nonlinear formulation.

D-MOD2000 is an enhanced version of D-MOD (Matasović, 1993) and uses a nonlinear formulation where the response of soil is modeled by a degraded backbone curve generated by unloading-reloading rules developed by Masing and extended by Pyke (1979). Because this type of formulation considers only hysteretic damping, an initial calibration step is required for D-MOD2000 to obtain the viscous damping and an odd integer related to the mode number. The calibration involves running D-MOD2000 at a low value of $PGA_{outcrop}$ and adjusting the viscous damping and the odd integer until the response spectrum from D-MOD2000 matches the response spectrum from SHAKE2000. The viscous damping and the odd integer that provide the best match are then used in running D-MOD2000 at the desired high $PGA_{outcrop}$ level.

For this study, SHAKE2000 was used when $PGA_{outcrop} \leq 0.3$ g and D-MOD2000 was used when $PGA_{outcrop} > 0.3$ g. Values of the viscous damping and the odd integer for use in D-MOD2000 were determined by running both programs mostly with $PGA_{outcrop} = 0.1$ g until the resulting response spectra matched.

RESULTS

Displayed in Figs. 5a-5f are calculated F_{PGA} plotted versus V_{S30} for $PGA_{outcrop}$ = 0.05, 0.1, 0.2, 0.3, 0.4 and 0.5 g, respectively, based on over 6000 SHAKE and 3000 D-MOD simulations. Each data point represents an average of the results from eight simulations with different synthetic soft rock outcrop motions. The plotted data in each figure exhibit three distinct features—(1) an increasing trend in F_{PGA} with increasing V_{S30}; (2) a zone of peak F_{PGA} values between V_{S30} of 165 and 420 m/s, depending on $PGA_{outcrop}$; and (3) a decreasing trend in F_{PGA} with increasing V_{S30} beyond the peak F_{PGA} zone. Based on these trends, a linear regression model is assumed when $V_{S30} <$ 165 to 420 m/s, and an exponential regression model is assumed when $V_{S30} \geq$ 165 to 420 m/s.

Median, 10% lower bound, and 90% upper bound regression curves for the plotted V_{S30}-F_{PGA} data pairs are also displayed in Figs. 5a-5f. The curves are defined assuming the following general relationships:

$$F_{PGA} = aV_{s30} \qquad \text{for } V_{S30} < V_{S30P} \tag{4a}$$

$$F_{PGA} = b\exp(cV_{s30}) \qquad \text{for } V_{S30} \geq V_{S30P} \tag{4b}$$

where a, b and c are the regression coefficients specific to $PGA_{outcrop}$; and V_{S30P} is the V_{S30} value corresponding to the estimated peak F_{PGA} value. The regression coefficients are derived so that F_{PGA} values computed from Equations 4a and 4b are the same at V_{S30P}. Presented in Table 1 are the regression values of a, b and c for the median curve, the 10% lower bound curve, and the 90% upper bound curve.

Table 1: Regression coefficients for determining F_{PGA} using Equations 4a and 4b

	Median curve				90% upper bound				10% lower bound			
$PGA_{outcrop}$	V_{S30P}	a	b	c	V_{S30P}	a	b	c	V_{S30P}	a	b	c
(g)	(m/s)	(x10^{-3})		(x10^{-3})	(m/s)	(x10^{-3})		(x10^{-3})	(m/s)	(x10^{-3})		(x10^{-3})
0.05	165	11.5	2.27	-1.08	165	13.7	2.70	-1.08	165	9.70	1.92	-1.08
0.1	190	8.42	1.87	-0.83	190	10.35	2.30	-0.83	190	6.92	1.54	-0.83
0.2	230	5.69	1.47	-0.51	230	7.28	1.88	-0.51	230	4.32	1.12	-0.51
0.3	270	4.44	1.33	-0.37	270	5.78	1.73	-0.37	270	3.40	1.02	-0.37
0.4	380	2.63	1.00	0.00	380	4.25	1.62	0.00	380	1.31	0.50	0.00
0.5	420	2.38	1.00	0.00	420	3.72	1.56	0.00	420	1.10	0.46	0.00

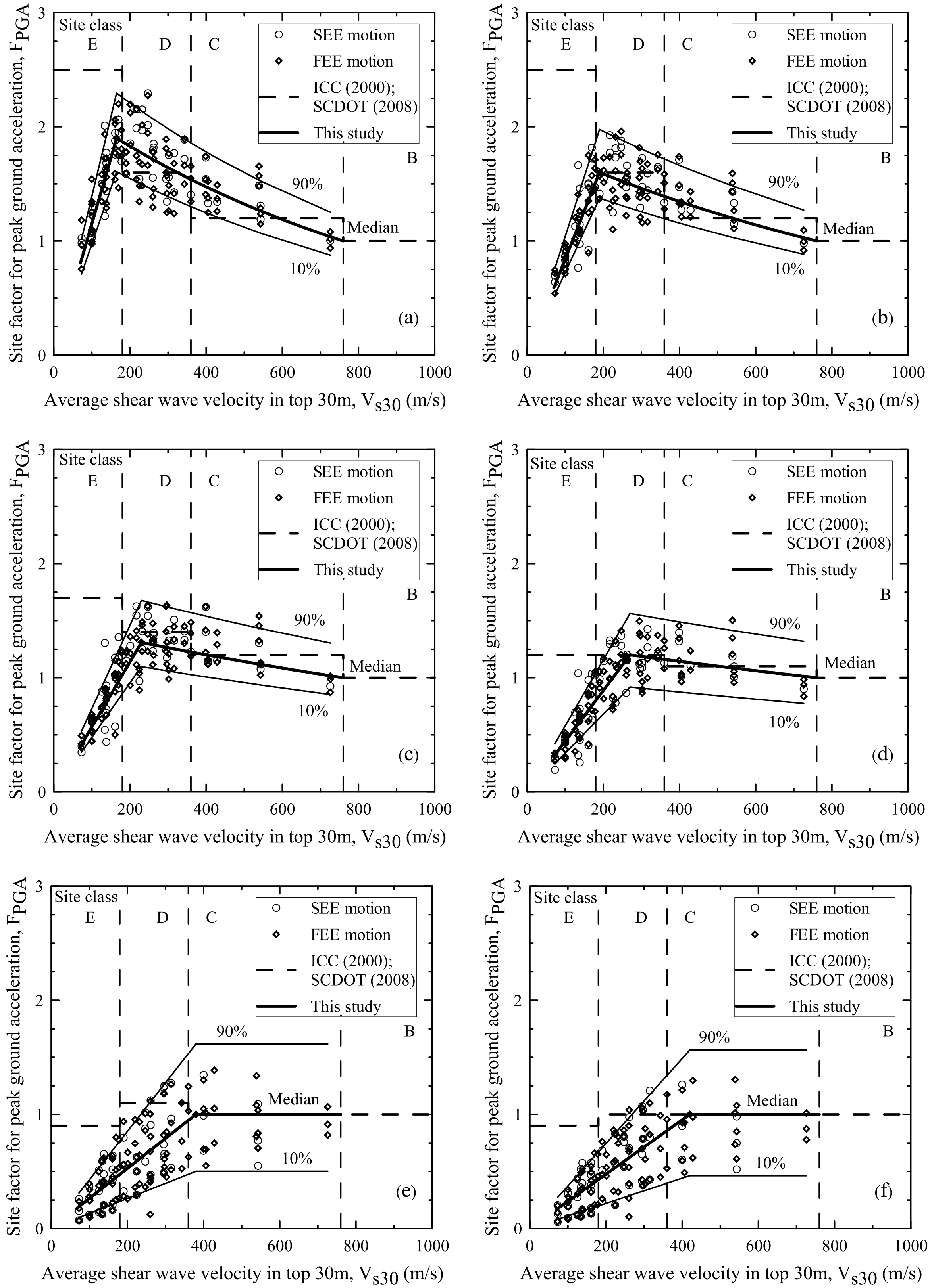

Fig. 5. F_{PGA} site factor for $PGA_{outcrop}$ equal to *(a) 0.05 g, (b) 0.1 g, (c) 0.2 g, (d) 0.3 g, (e) 0.4 g, and (f) 0.5 g.*

Equation 4a implicitly assumes $F_{PGA} = 0.0$ when $V_{S30} = 0$. This assumption agrees with the fact that zero stiffness (or strength) material cannot support shear waves and, for this reason, PGA should be zero regardless of $PGA_{outcrop}$. For the median curve, Equation 4b satisfies the condition that $F_{PGA} = 1.0$ when $V_{S30} = 760$m/s, which is the assumed B-C boundary condition for the South Carolina Coastal Plain. Both assumptions are supported by the data plotted in Figs. 5a-5f.

As expected, the larger values of F_{PGA} plotted in Figs. 5a-5f are associated with the lower values of $PGA_{outcrop}$. For $PGA_{outcrop} = 0.05$ g, the maximum value of F_{PGA} is about 2.5. For $PGA_{outcrop} = 0.1$ g, 0.2 g, 0.3, 0.4, and 0.5 g, the maximum values of F_{PGA} are 2.1, 1.64, 1.5, 1.4, and 1.35 respectively. It is also noted that V_{S30P} increases with increasing $PGA_{outcrop}$.

A study of residuals can show how well the median relationships fit the data plotted in Figs. 5a-5f. The residual, ε, is obtained by dividing the F_{PGA} of the data by the F_{PGA} of the median curve. Because F_{PGA} is greater than 0, the values of ε are expected to follow lognormal distributions. Displayed in Fig. 6 are probability density functions of ε assuming lognormal distribution of F_{PGA}. The plots show that the median value of ε is approximately equal to 1.0, which indicates the median relationship is unbiased in predicting F_{PGA}. In other words, the median relationships underestimate just as often as they overestimate. The increase in the spreadness of the probability density functions, for increasing $PGA_{outcrop}$, indicates the increase in scatter of ground response results.

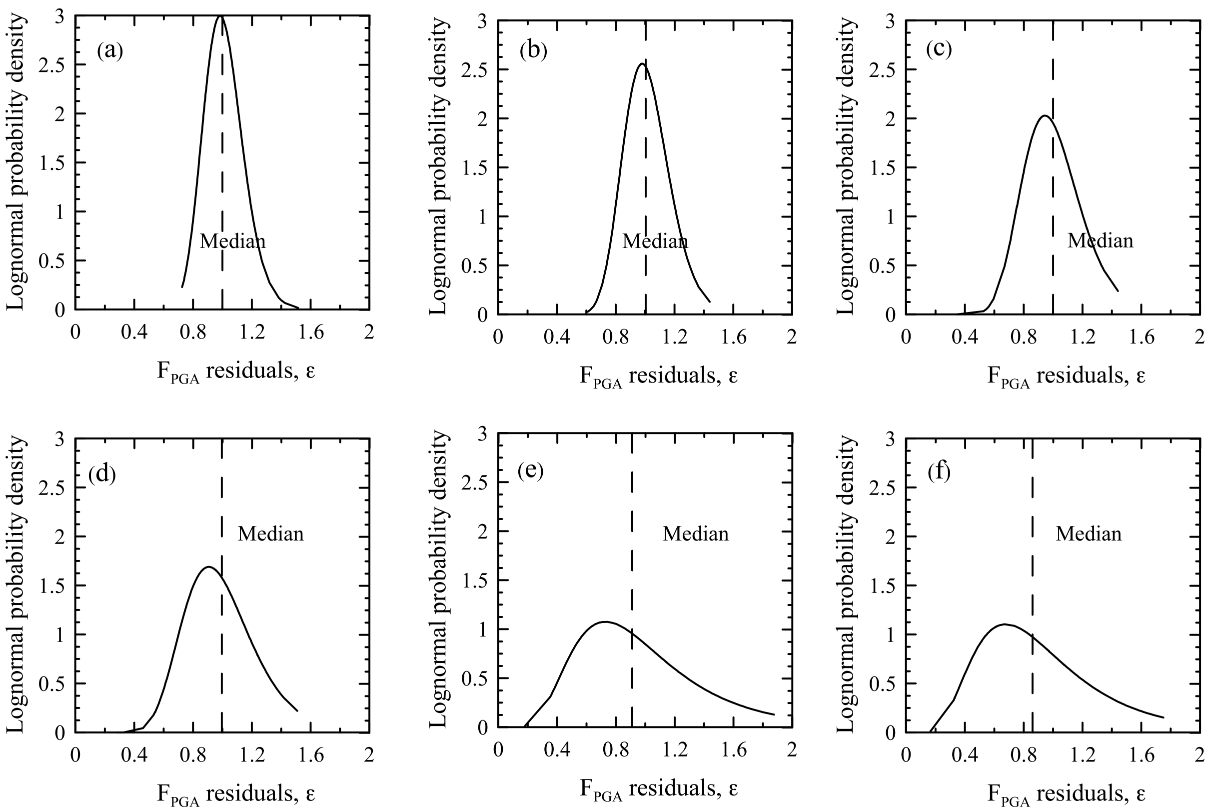

Fig. 6. Lognormal probability density functions of F_{PGA} residuals for $PGA_{outcrop}$ equal to *(a) 0.05 g, (b) 0.1 g, (c) 0.2 g, (d) 0.3 g, (e) 0.4 g, and (f) 0.5 g.*

Displayed in Fig. 7 are the values of ε plotted with respect to the predictor variable V_{S30}. It can be observed that the values of ε do not exhibit any systematic structure, indicating little or no bias in the median relationships expressed by Equations 4a and 4b. The median line at ε = 1 emphasizes the central tendency of the model, which indicates a reasonable regression fit to the data. With an increase in $PGA_{outcrop}$, the system nonlinearity increases and the data dispersion increases with increasing $PGA_{outcrop}$. Based on Fig. 7, the regression curves provide reasonable predictions of F_{PGA} given V_{S30}.

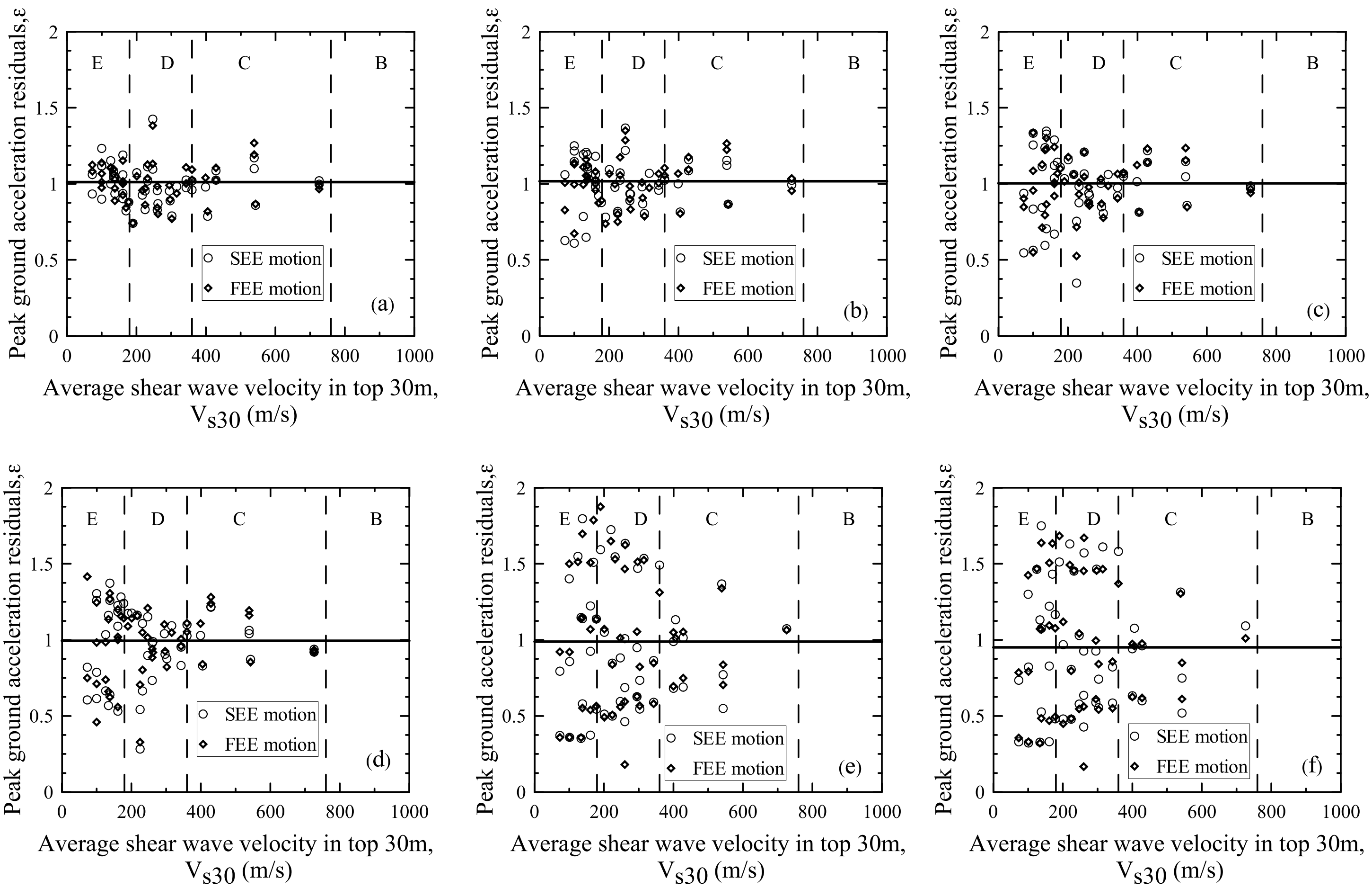

Fig. 7. Variation of F_{PGA} residuals with V_{S30} *for $PGA_{outcrop}$* equal to *(a) 0.05 g, (b) 0.1 g, (c) 0.2 g, (d) 0.3 g, (e) 0.4 g, and (f) 0.5 g.*

DISCUSSION

The results plotted in Fig. 5d agree well with the previous results reported in Andrus et al. (2006) and Fairbanks et al. (2008), who considered a soil/rock profile of the Charleston Peninsula that extended to a depth of 808 m. For moment magnitude of 7.3 and PGA of 0.3 g, their results indicated F_{PGA} value of 0.5 when V_{S30} = 110m/s; and F_{PGA} value of 0.75 when V_{S30} = 190m/s. These values plot close to the median curve in Fig. 5d.

The scatter in V_{S30}-F_{PGA} values plotted in Figs. 5a-5f is the result of several factors. The most significant factor causing the scatter is the different G/G_{max}-γ and D-γ relationships used for the same V_s profile. The influence of this factor seems to increase with increasing $PGA_{outcrop}$. Other factors include the different combinations of Quaternary and Tertiary layering producing similar V_{S30} values, the different V_s values below 30 m, and the frequency content differences between input ground motions. Nevertheless, the data plotted in Figs. 5a-5f support the use of V_{S30} as a useful predictor of site response.

NEHRP F_{PGA} site factors adopted by ICC (2000) and SCDOT (2008) are plotted in Fig 5a-5f for comparison with F_{PGA} site factors computed in this study. It can be seen that there is a good agreement between F_{PGA} values for $V_{S30} \geq 180$ m/s, with the NEHRP F_{PGA} values generally falling within the 10% and 90% bounds. For $V_{S30} \leq 180$ m/s, the NEHRP F_{PGA} values are much greater than the F_{PGA} values computed in this study. The F_{PGA} relationships derived in this study are recommended for the Charleston area because they are based on the regional conditions and provide a continuous function with V_{S30}.

The peak F_{PGA} zone between V_{S30} of 165 to 420 m/s may be explained by the predominant period of the synthetic input motions (0.42 ± 0.2 s) being close to the fundamental periods of the V_s profiles in the top 30 m. It is anticipated that the V_{S30P} values will vary with variation in the frequency content and the duration of the input ground motion.

CONCLUSION

In this paper, we developed relationships for estimating median, 10% lower bound, 90% upper bound values of F_{PGA} in the Charleston area as a function of V_{s30} and $PGA_{outcrop}$. The relationships were based on over 9000 simulations, and were expressed by a linear model for V_{s30} below the zone of peak F_{PGA} values and an exponential model for V_{s30} above the zone of peak F_{PGA}. NEHRP F_{PGA} site factors were shown to plot within the 10 - 90% range for V_{s30} > 180 m/s. For V_{s30} < 180 m/s, the NEHRP F_{PGA} site factors plot well above the relationships of this study. F_{PGA} is observed to decrease with increasing $PGA_{outcrop}$ for all site classes. The median relationships derived in this study are recommended for use in the Charleston area because they are based on regional conditions and are continuous with V_{s30}. The 10% lower bound and 90% upper bound F_{PGA} values can be used to quantify uncertainty in simplified code-based seismic design.

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