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ABSTRACT

The shear wave velocity of soil and rock is one of the key components in establishing the design response spectra, and therefore the seismic design forces, for a building, bridge, or other structure. The shear wave velocity can be measured from in-situ field tests, such as cross-hole or downhole testing. The shear wave velocity can also be estimated based on empirical correlations with other field collected information. This paper presents case histories from 8 bridge projects performed in the northeastern United States where in-situ measurements of shear wave velocities were performed for site-specific ground motion studies. A comparison of these measurements with several empirical correlations indicates that the empirical correlations do not approximate the shear wave velocity very well. Therefore, the use of the empirically derived shear wave velocities may result in an inaccurate determination of the seismic forces imparted to the soils and the structure. Therefore, based on these results, it is concluded that the use of empirically derived shear wave velocities should be used as a preliminary assessment for development of response spectra and liquefaction susceptibility parameters.

INTRODUCTION

It is well known that the soil conditions beneath a structure have an impact on the propagation of ground motions from the bedrock to the ground surface. The soil conditions may amplify certain spectral accelerations and attenuate spectral accelerations at other periods. Utimately, the spectral accelerations are used to estimate the earthquake induced forces that are imparted to a structure. In addition, the shear stresses imparted to the soil by the earthquake are affected by the soil properties and have a direct effect on the liquefaction potential of the soils. The shear wave velocity (or shear modulus) of the soils is the soil property that has the greatest effect on the determination of an appropriate response spectra and estimation of shear stresses.

This paper presents the results of in-situ shear wave velocity measurements from eight bridge projects located throughout the northeastern United States. Comparisons between the measured values and those derived based on empirical equations related to the N-values of test borings are presented. Site specific response analyses were performed using idealized shear wave velocities based on the measured and empirical values. Comparisons between the response spectra and shear stresses from these site response analyses are made and conclusions are presented based on the results of these analyses.

PROJECT DESCRIPTIONS

A summary of the location and size of the bridges, the depth to bedrock, and the method used to measure the in-situ shear wave velocities is included in Table 1. Additional information regarding the subsurface conditions at specific bridge locations are presented elsewhere in this paper.

Table 1 – Project Descriptions

Site ID Number	Location	Bridge Size	Depth to Bedrock	Method of In-situ Vs Measurements
1	Jersey City, NJ	5000 ft long steel girder	30 – 125 ft	Cross Hole
2	Ridgefield Park, NJ	500 ft long steel girder	110 – 150	Down Hole
3	Harrison, NJ	7000 ft long steel girder	80	Cross Hole
4	Brooklyn, NY	300 ft long steel girder	500	Down Hole
5	Chelmsford, MA	50 ft long steel girder	50	Cross Hole

6	Bridesburg, PA	8500 ft long steel truss	80	Cross Hole
7	Perth Amboy, NJ	7000 ft long steel girder	120	Cross Hole
8	Brooklyn, NY	150 ft long bascule	>300	Cross Hole

SHEAR WAVE VELOCITY COMPARISONS

A comparison between the measured shear wave velocities of the eight projects listed in Table 1 and those obtained from the use of an empirical equation using the N-value (N) from test borings is shown in Figure 1. The N-values are from test borings performed at the location of the in-situ shear wave velocity measurements. The shear wave velocity from the empirical equation (shown in Figure 1 as a line) is an average of the following four empirical equations:

$Vs = 280 * N^{0.348}$ (Ohta and Goto [1978]) (1)

$Vs = 185 * N^{0.5}$ (Seed, et. al. [1983]) (2)

$Vs = 318 * N^{0.314}$ (Imai and Tonouchi [1982]) (3)

$Vs = 350 * N^{0.27}$ (Sykora and Stokoe [1983]) (4)

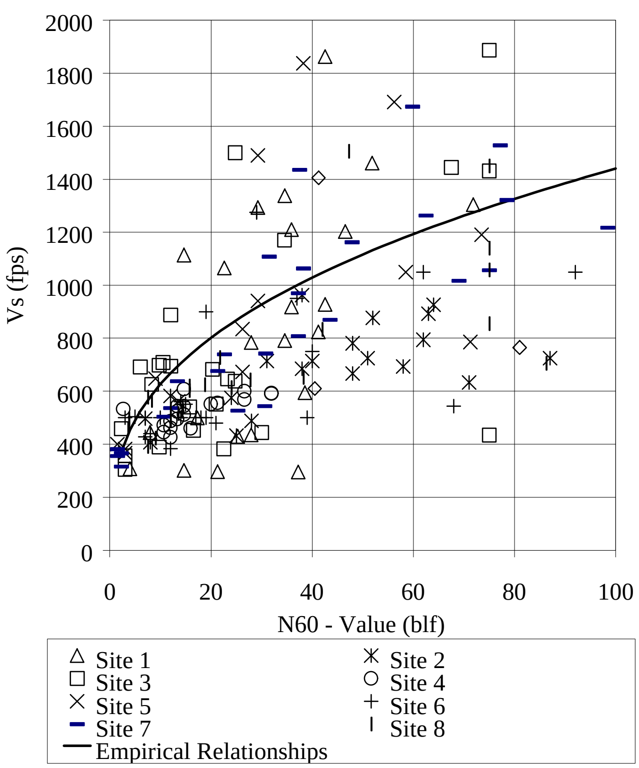

Figure 1 - Comparison of Measured and Empirically Derived Shear Wave Velocities

The above empirical relationships were developed for primarily sandy soils. The results indicate that the use of the N-value in determining the shear wave velocity of sandy soils is not very reliable. The results from specific projects indicates that the average of the empirical equations sometimes predicts the shear wave velocity well but at other projects, the predicted shear wave velocities may be significantly different than the measured values.

SITE RESPONSE ANALYSIS RESULTS

Site response analyses are performed for some of the project locations given in Table 1 for the purpose of evaluating the effect of using empirically derived shear wave velocities and measured shear wave velocities. Specifically, the response spectra at the foundation level is evaluated since this is used in determining the seismic forces imparted to a structure. In addition, the shear stress is evaluated since this is a critical component in evaluating liquefaction potential.

One dimensional site response analyses were performed using the ProShake computer program. A site response analysis requires an acceleration time history and a soil profile. The acceleration time history used for the analyses consists of an actual time history from the Magnitude 6.5 Saquenay earthquake in Quebec, Canada that occurred in October 1988. The time history has been matched to the New York City Department of Transportation (NYCDOT) recommended response spectrum for Soil Class B (Rock) with a 10% probability of exceedance in 50 years (NYCDOT, [1998]). The soil profile includes the total unit weight and the shear wave velocity of the soils. In addition, relationships between the shear wave velocity and the shear strain are needed.

Site response analyses were performed for Sites 3, 5, 7, and 8. These locations were selected so that a relatively wide range of subsurface conditions could be evaluated. In addition, with the exception of Site 8, in-situ shear wave velocity measurements were obtained for the entire soil column and the bedrock. At Site 8, the influence of the bedrock depth on the site response analyses is relatively minor; therefore, a depth of 140 ft is used as the top of the bedrock.

For a given site, response analyses were performed using the measured shear wave velocities and the empirically derived shear wave velocities and the results are compared. All other parameters (e.g., bedrock motion, modulus degradation curves) used in the site response analyses remain constant. The soil profiles for the four sites are shown in Figures 2 through 5.

The response spectra, located at a depth of 5 ft below the ground surface, for the various sites evaluated are included in Figures 6 through 9. A comparison of the response spectra for a given site indicate that in some cases, such as at Site 7, there is very little difference between the response spectra using measured and empirically derived shear wave velocities. As expected, the measured and empirically derived shear wave velocities match relatively well at this location. However, at Site 8, the response spectra using the measured shear wave velocities generally results in lower spectral accelerations. In general, the empirically derived shear wave velocities at this location are greater than the measured values.

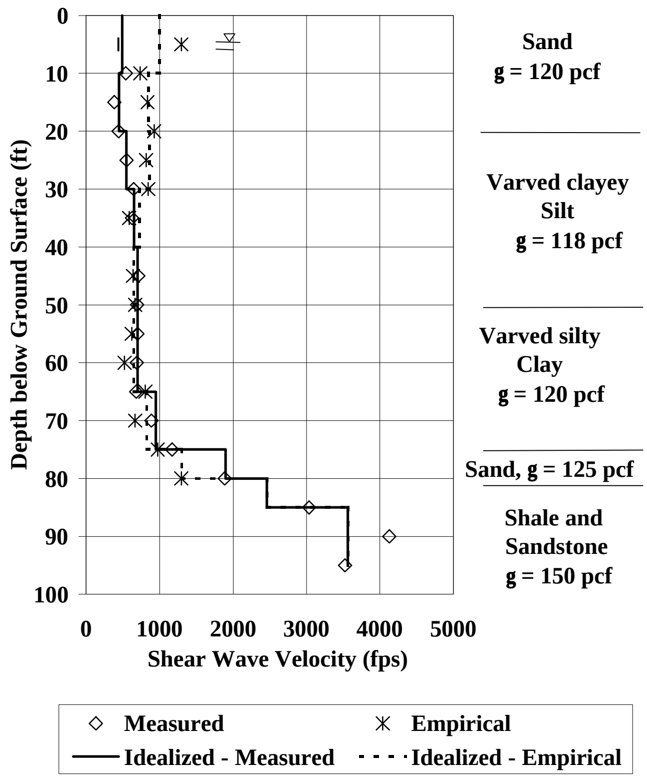

Figure 2 - Soil Profile - Site 3

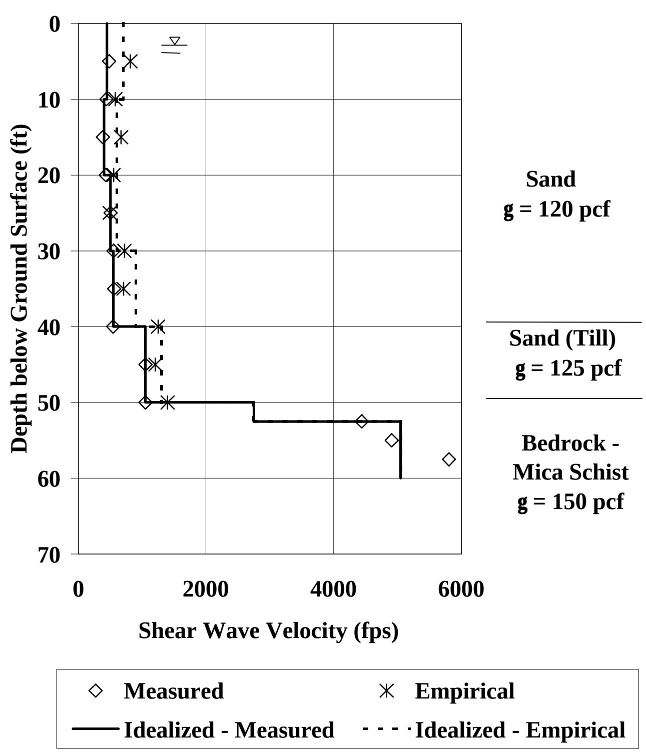

Figure 3 - Soil Profile - Site 5

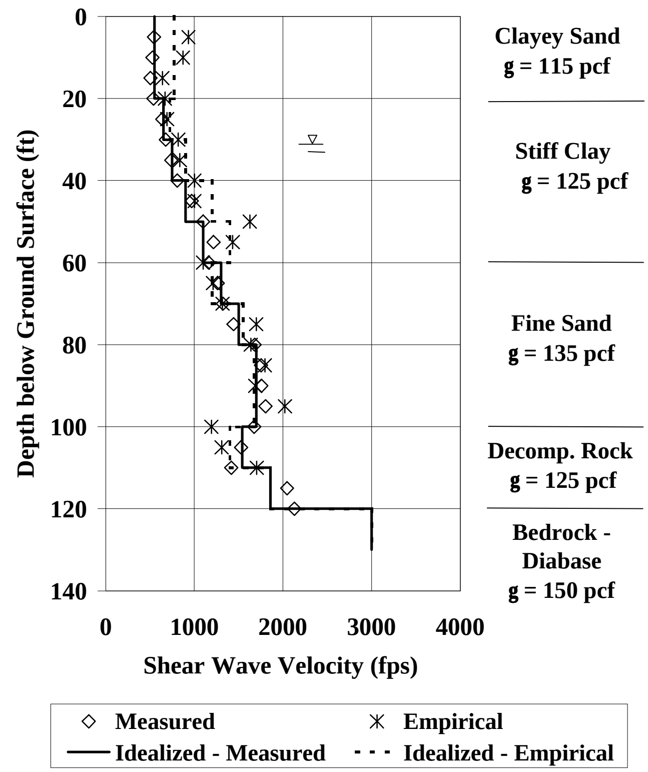

Figure 4: Soil Profile - Site 7

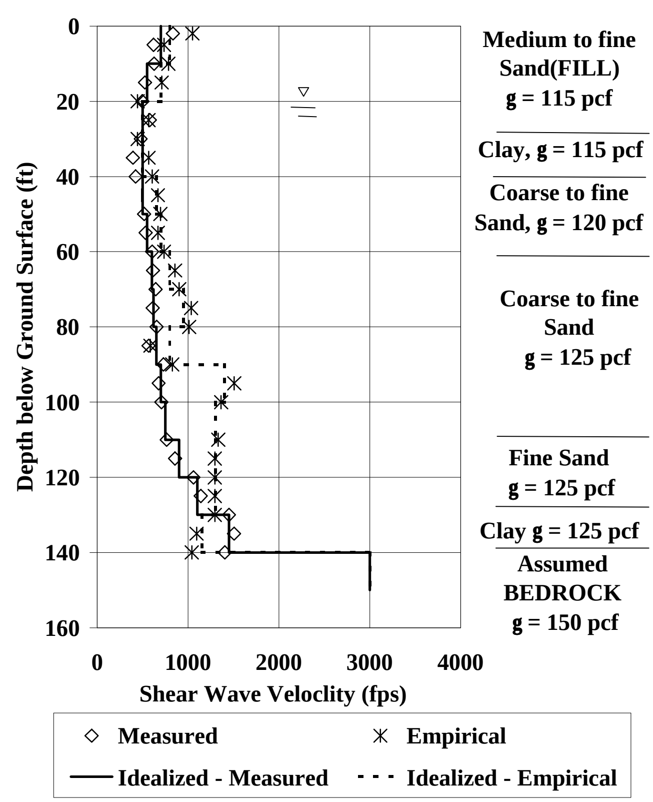

Figure 5: Soil Profile - Site 8

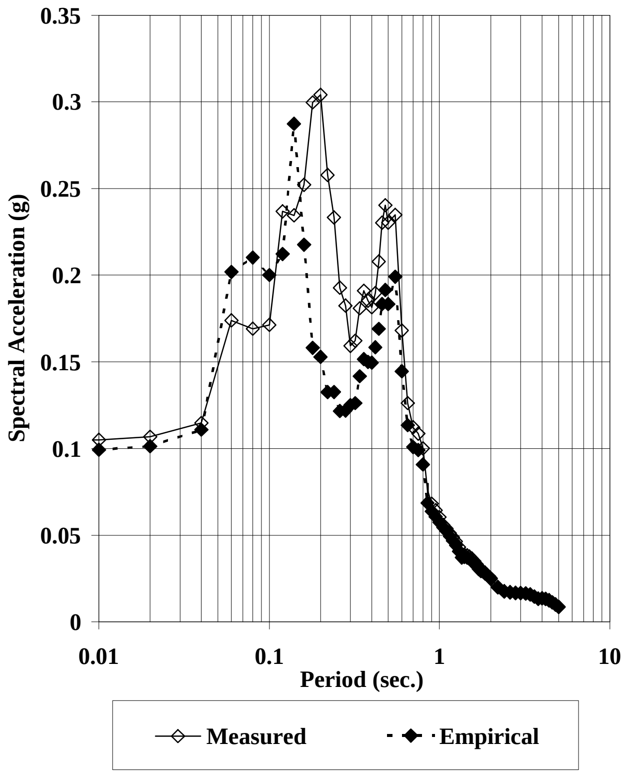

Figure 6 - Response Spectra at 5 ft. BGS - Site 3

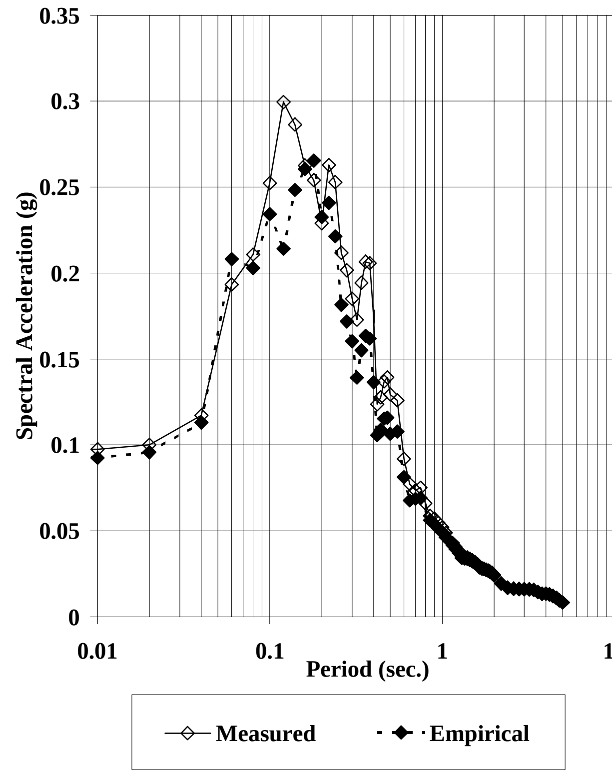

Figure 8 - Response Spectra at 5 ft. BGS - Site 7

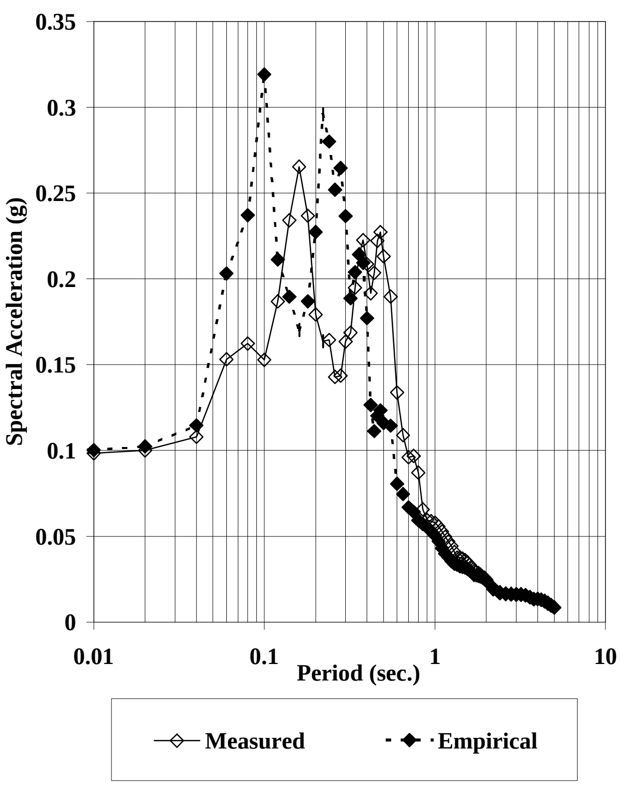

Figure 7 - Response Spectra at 5 ft. BGS - Site 5

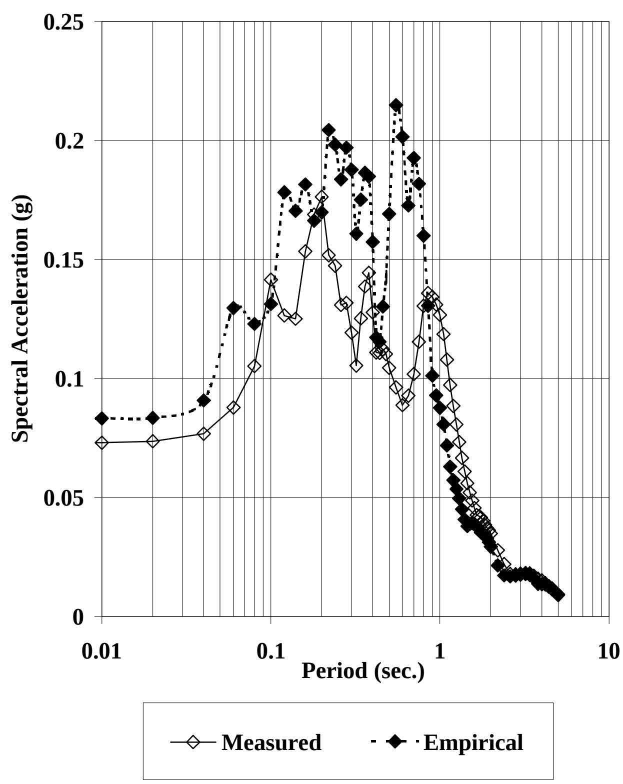

Figure 9 - Response Spectra at 5 ft. BGS - Site 8

Similar trends are observed when comparing the peak shear stresses using the measured and empirically derived shear wave velocities, as shown in Figures 10 through 13.

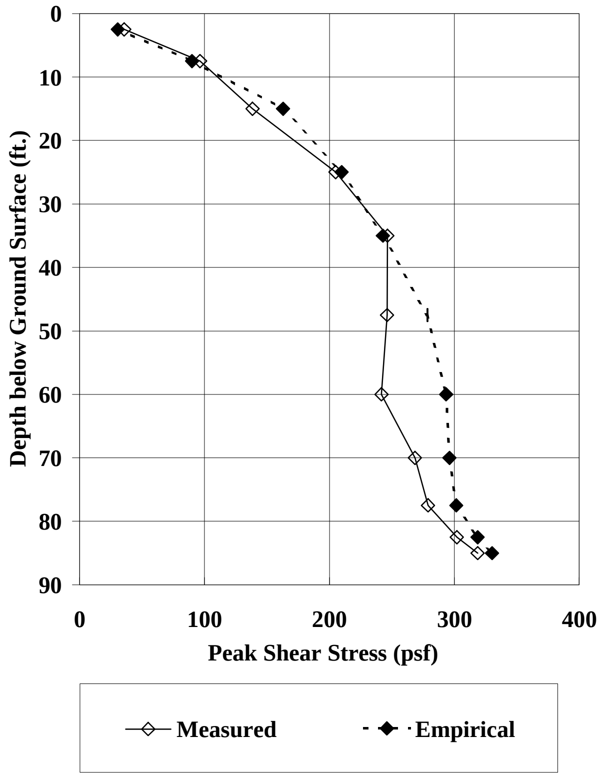

Figure 10 - Shear Stress Curve - Site 3

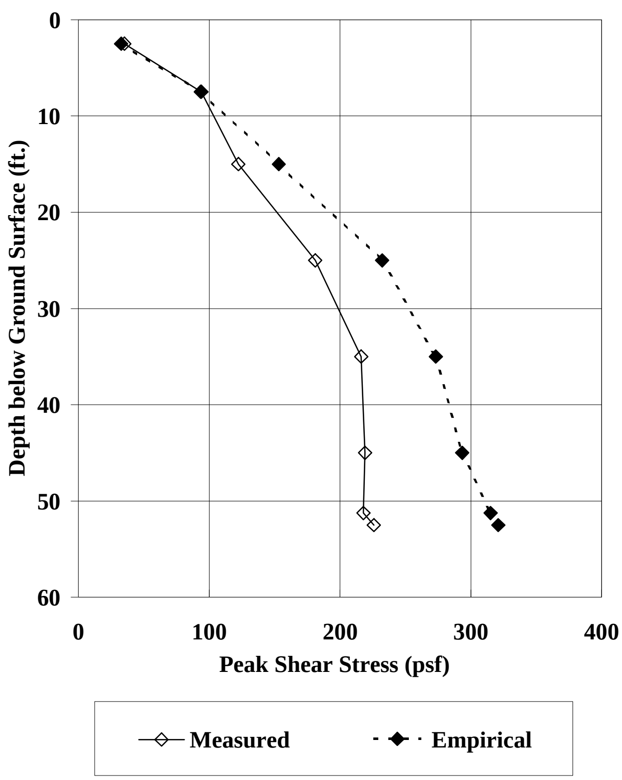

Figure 11 - Shear Stress Curve - Site 5

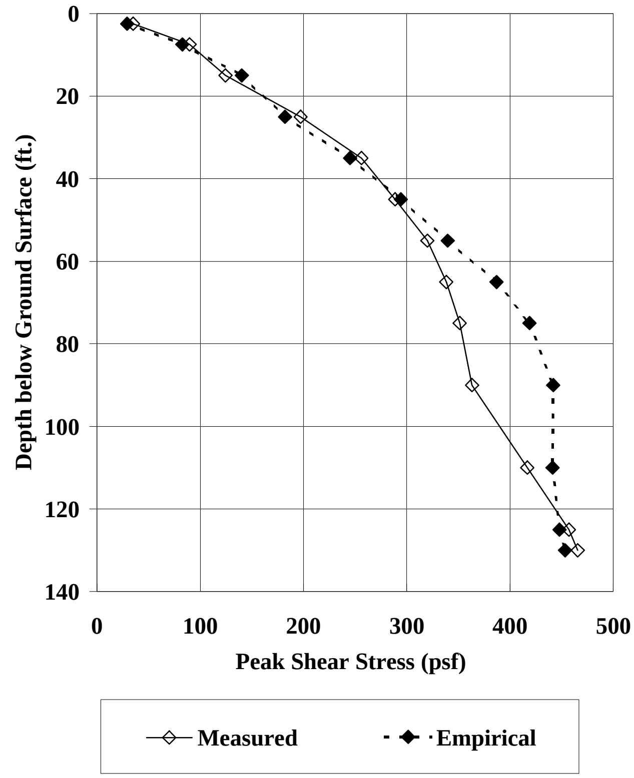

Figure 12 - Shear Stress Curve - Site 7

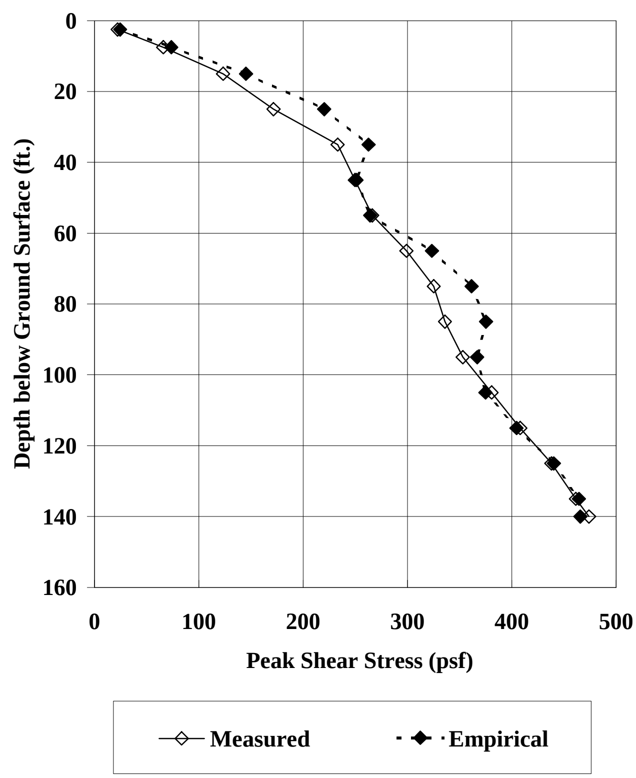

Figure 13 - Shear Stress Curve - Site 8

CONCLUSIONS

The shear wave velocity is an important parameter for the development of site specific response spectra and the evaluation of liquefaction potential at a site. The shear wave velocity can be estimated based on empirically derived relationships. However, these relationships can result in values that are significantly different than the measured values. As was shown herein, this may result in the improper estimation of the seismic response accelerations, and ultimately, the seismic forces imparted to the structure. In addition, the shear stresses from site response analyses based on empirically derived shear wave velocities may result in an incorrect assessment of liquefaction potential. Therefore, it is concluded that the use of empirically derived shear wave velocities should be used for a preliminary development of response spectra and liquefaction susceptibility parameters. Analyses should be performed where the shear wave velocity values are varied in order to evaluate the sensitivity of the results to changes in the shear wave velocity. If the results of these analyses indicate that the variation in the seismic forces or liquefaction potential are significant or are a concern, it may be prudent to perform more detailed analyses, including the measurement of in-situ shear wave velocities.

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