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ABSTRACT

Seismic site response analysis is of paramount importance for many problems in earthquake engineering and has been studied extensively over the last 50 years. More recently, the observed response of deep stiff soil profiles during seismic events has indicated the possibility of significant ground amplification. In this study, a new enhanced hysteretic constitutive model is used for the evaluation of dynamic site response of deep granular soil deposits. The constitutive laws are implemented in a finite element computer code, AMPLE2000. The response of two soil profiles to different earthquake records was calculated using the newly developed model implemented in AMPLE and the computer program, SHAKE, which employs the equivalent linear procedure. The importance of soil nonlinearity with increasing levels of shaking and deposit depth on the acceleration at the ground surface is examined.

INTRODUCTION

Site response analysis is an integral component in any earthquake engineering analyses. Typically, it provides the input necessary for the evaluation of structural performance during a seismic event. Most site response analyses seldom include depths exceeding 50-100 m, with the notable exception of analyses for offshore soil deposits (e.g., Biscontin et al., 2000). For instance, the 1997 Uniform Building Code only considers the top 30 m (~100 ft) of the soil profile when developing characteristics for the soil profile following the recommendations by Borcherdt (1994). More recently, there has been significant interest in the evaluation of seismic response of deep stiff soil deposits (e.g., Chang et al., 1997).

Similarly to their mostly cohesive counterpart, deep deposits of primarily dense granular material is believed to significantly amplify ground motions. The 1967 Caracas earthquake, with magnitude 6.4, originally focused attention on this effect and provided an undisputed evidence of the effect of "local soil conditions" on structural performance. The city of Caracas is located in an alluvium filled valley consisting (primarily) of sand and gravel which in the Palos Grandes area is up to 230m deep. The epicenter for this earthquake was 56.3 km (35 miles) from Caracas, and although the peak acceleration in the rock was estimated to be only in the order of 0.03g, significant ground amplification was observed resulting in significant damage to buildings 10 stories and higher in the Palos Grandes area (e.g., Seed et al., 1972).

Although the amplification of higher period motion by deep deposits is a well known phenomenon, the effects of confining stress and soil non-linearity on the behavior of deep granular deposits has not been a subject of much study. An enhanced hysteretic constitutive law is used here to describe the non-linear behavior of the granular deposits. The proposed model is able to simulate the observed soil nonlinearity and the effect of confining pressure on the shear modulus degradation and damping coefficients, while at the same time achieving a robust and computationally efficient model formulation. A series of numerical simulations have been performed using the newly developed model implemented in the finite element computer code AMPLE2000 (Pestana and Nadim 2000) and with the computer code, SHAKE (Schnabel et al. 1972), which uses the equivalent linear procedure. The results from the enhanced hysteretic model in AMPLE2000 are compared to the results from SHAKE to evaluate the importance of including the effects of soil nonlinearity in the site response analysis.

MODEL FORMULATION

Small Strain Shear Modulus, G_{max}

A new formulation for G_{max} is included in the implementation of the enhanced hysteretic model in AMPLE2000. The formulation for G_{max} is based on extensive work done by many researchers (e.g., Jamiolkowski et al., 1994). A review of available data and bender element tests performed at UC Berkeley is available in the literature (Salvati and Pestana, 2000). For granular materials, the most important factors controlling G_{max} are the void ratio and the confining pressure. The formulation is based on the concept of the generalized stiffness with separable functions for the volumetric state (i.e., void ratio) and confining stress as proposed by Pestana (1994):

$$G_{max} / p_{at} = f(e).f_p(p / p_{at}) \tag{1}$$

where $f(e)$ and $f_p(p/p_{at})$ are functions of the void ratio and mean effective stress, respectively and p_{at} is the atmospheric pressure. The function f_p is typically described by a power law formulation. For sands, an average exponent of 0.5 has been reported by many researchers and common values range between 0.40 to 0.60 (e.g., Salvati and Pestana, 2000). A series of bender element tests were performed at UC Berkeley on dry pluviated Sacramento River Sand with relative densities ranging from 50% to 83% to evaluate the dependence of G_{max} on the confining stress. The results from the lowest and highest density tests are shown in Fig. 1. The confining pressures used for the tests ranged from 10 kPa to 588 kPa (0.1 to 5.8 atmospheres), which is larger than the typical range of measurements. All of the individual values of G_{max} measured are shown with the open symbols, and the average G_{max} at each confining pressure is shown with the solid symbols. The results show that exponent of 0.5 in the power law relation describes the effect of confining pressure over the entire range of pressures tested as shown by the dashed lines.

Figure 2 shows a comparison of two formulations describing the effect of void ratio, e, on G_{max}. To isolate the effect of void ratio on G_{max}, only values measured at confining pressures within 10% of p_{at} ~ 101.3 kPa are shown. The first formulation, given by:

$$f(e) = 900 \frac{(2.17 - e)^2}{1 + e} \tag{2a}$$

is commonly used for granular materials and was first suggested by Iwasaki and Tatsuoka (1977). The second formulation was proposed by Jamiolkowski et al. (1994) and is given by:

$$f(e) = G_b / e^{1.3} \tag{2b}$$

where the material constant, G_b, is best determined by a regression analysis on available G_{max} values for a particular sand. As seen in Fig. 2, the second formulation fits better the suite of data with a range of G_b of 400 to 800, to account for the different materials and is used for the enhanced hysteretic model. An advantage of this formulation is the ability to describe sands over a large range of void ratios, such as those typical of calcareous sands.

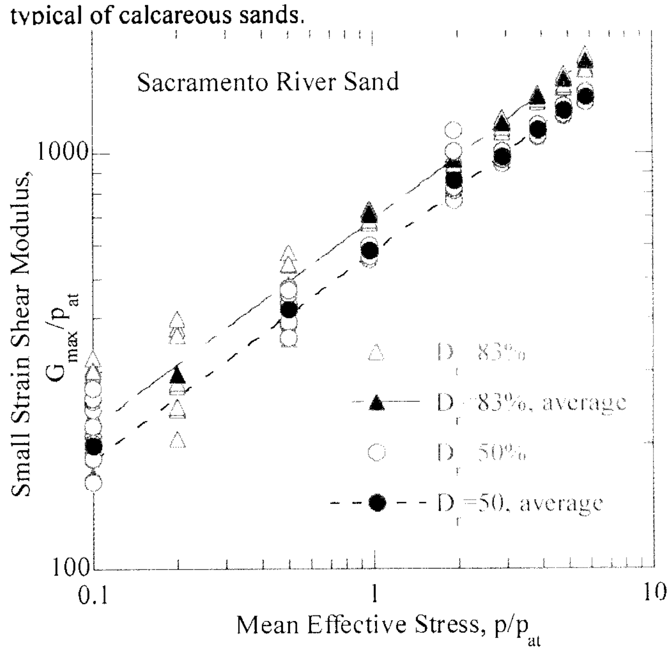

Fig 1. G_{max} from bender element tests for Sacramento River Sand (after Salvati and Pestana, 2000)

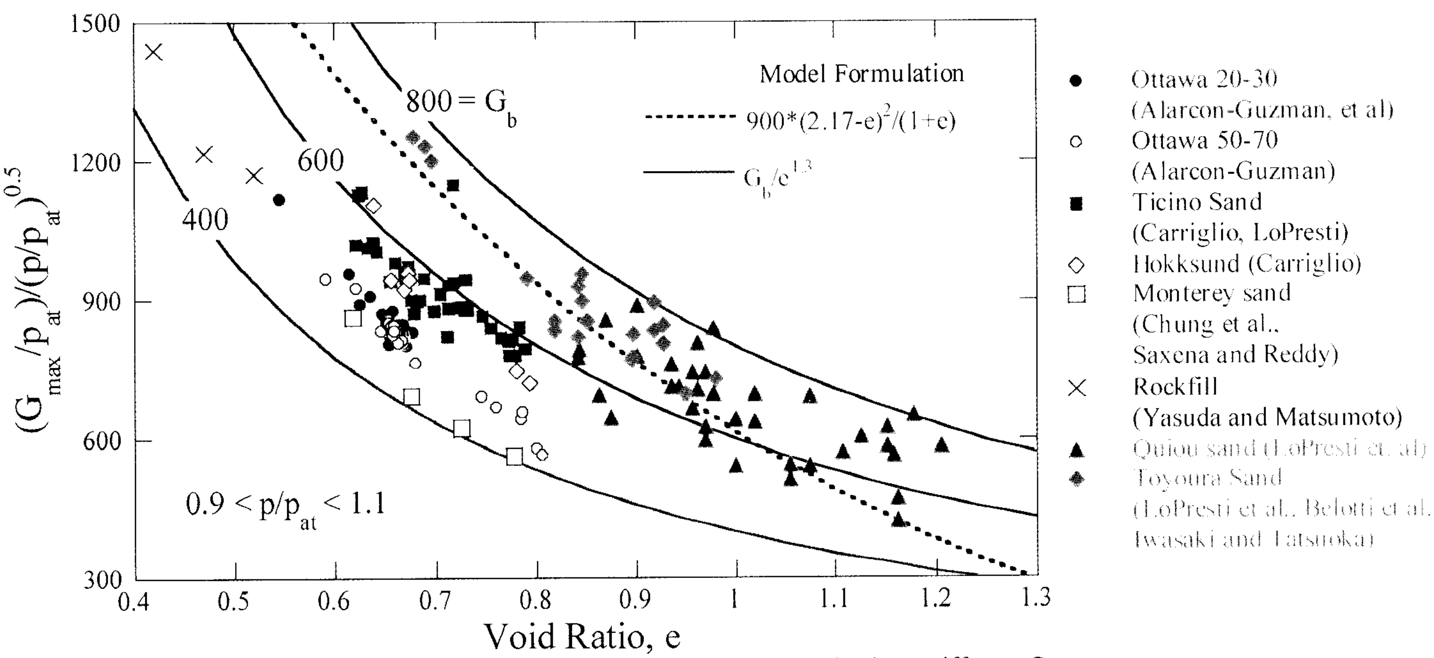

Fig. 2 Effect of void ratio on the small strain shear stiffness, G_{max}

Based on the previous arguments, the following equation was selected to calculate G_{max} in the implementation of the enhanced hysteretic model in AMPLE2000.

$$G_{\max} / p_{at} = G_b (p / p_{at})^{0.5} / e^{1.3} \qquad (3)$$

HYSTERETIC COMPONENT

An enhanced, hysteretic model was used to evaluate the dynamic site response of granular soil deposits. The model is based on the perfectly hysteretic component of a generalized model for clays and sands (Pestana, 1994), but includes several refinements to better describe the shear modulus degradation and damping ratios for soils (Lok, 1999; Lok and Pestana, 2000). The key elements of this model are its ability to a) accurately describe the soil non-linearity and damping over a large range of strains, b) independently match the modulus degradation and damping curves, and c) correctly track stress reversals under a general loading condition. The model only uses four parameters, three to describe the hysteretic behavior and the fourth to determine the maximum shear modulus, G_{max}, of the material. The main assumption establishes that the tangential shear stiffness, G, is the harmonic mean of the stiffness at small strains, G_h, and the stiffness at larger strains, G_p, :

$$\frac{1}{G} = \frac{1}{G_h} + \frac{1}{G_p} \qquad (4)$$

In the "Perfect Hysteretic" response, the tangent stiffness is related to the most recent stress reversal state as originally proposed by Huekel and Nova [1979]. The stress reversal point is determined by the direction of strain rates, which is based on the observation that the non-linearity of soil is most appropriately described by its past strain history. (Hight et al., 1983). Using the perfectly hysteretic formulation mentioned above, the small strain shear stiffness is described as.

$$\frac{G_h}{G_{\max}} = \frac{1}{(1+\omega_1 \xi_s)(1+c\|\eta_{\max}\|)}, \quad \text{where } \xi_s = \left\| \eta - \eta_{rev} \right\| \qquad (5)$$

where ξ_s describes a dimensionless measure in stress space, η (= s/p where s is the deviatoric stress tensor and p is the mean effective stress) is the shear stress ratio tensor, and η_{rev} is the shear stress ratio at the stress reversal point. The parameter, ω_1, describes the small strain non-linearity while parameter c controls the shear modulus immediately after each load reversal. As ω_1 increases, the secant shear modulus decreases, and the damping level increases, for all strain levels.. As c increases, the stiffness reached at each load reversal decreases, and the secant shear modulus and the damping level also decrease for all strain levels (Lok and Pestana, 2000).

The large strain shear stiffness is defined as:

$$G_p = G_h / (\omega_2 \xi_s^2) \qquad (6)$$

The parameter ω_2 has a similar effect on the shear modulus reduction and damping curves as parameter ω_1 but it only takes effect at strains larger than 0.01%. By calibrating parameters ω_1, ω_2, and c, the modulus and reduction curves can be matched nearly independently and have been selected using numerical optimization subroutines to match observed soil response (e.g., Lok, 1999; Lok and Pestana, 2000).

SELECTION OF MODEL PARAMETERS

A single set of parameters can be selected that will simulate the behavior of a sand over a wide range of confining pressures and is described in detail by Salvati and Pestana (2000). Using the parameters listed in Table 1, the proposed model is able to predict modulus degradation and damping curves which are in excellent agreement with the modulus reduction curves reported by Iwasaki et al. (1978) for confining pressures of approximately 0.25 to 2.0 kg/cm^2 and the damping curves described by Seed and Idriss (1970) as shown in Figure 3.

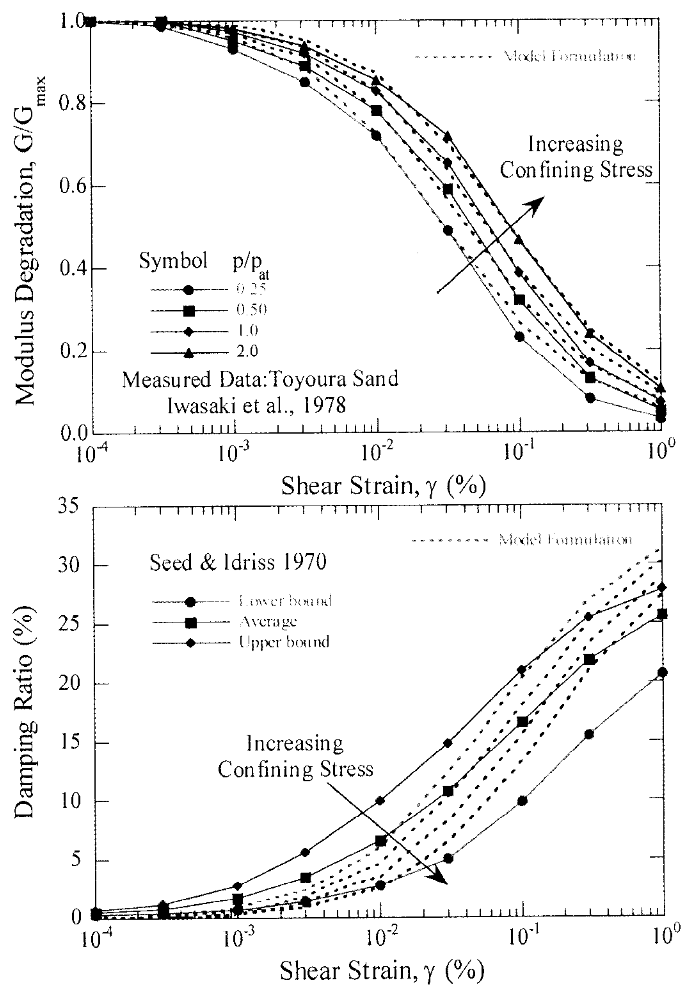

Table 1. Model Parameters for Toyoura sand

Variable	G_b	ω_1	ω_2	c
Value	700	1.3	0.5	0.1

The analyses presented in the following section use these parameters to predict modulus reduction and damping curves at higher confining pressures, which is necessary for deep soil deposits. SHAKE is unable to do without inputting a set of curves for each confining pressure. One of the shortcomings of any model based on the "perfectly hysteretic formulation" is that they are unable to describe the small but non-zero damping which is seen at very small strains (~0.5-1.0%). The program AMPLE2000 uses a Rayleigh damping formulation to describe this small amount of viscous damping (Pestana and Nadim, 2000).

SITE RESPONSE ANALYSIS

Using the Caracas Valley as a model, two idealized soil profiles consisting of a uniform, medium-dense (e = 0.7) sand to depths of 30.5 m (100 ft) and 183 m (600 ft), respectively, were examined here. In both profiles the water table was at a depth of 20 m (65.6 ft) (Seed et al., 1972). Motions recorded during the Loma Prieta Earthquake (1989), with magnitude M_w = 6.9, at rock outcrops in Gilroy (11.2 km to the closet fault rupture, PGA=0.473) and Rincon Hill (79.7 km to the closest fault rupture, PGA = 0.092) were used for the site response analyses.

Shallow Profile

Acceleration response spectra for the 30.5 m (100ft) profile were generated using the programs SHAKE and AMPLE2000 with the Rincon Hill and Gilroy records scaled to PGAs of 0.1g and 0.3g and the results are shown in Figure 4. The records were used as an outcropping motion.

For the Rincon Hill motion, the PGA predicted by SHAKE and AMPLE2000 are in good agreement, with SHAKE resulting in a slightly higher PGA. AMPLE2000 predicts a higher amplification than SHAKE at periods up to 0.4 to 0.6 sec, while the opposite is true for higher periods.

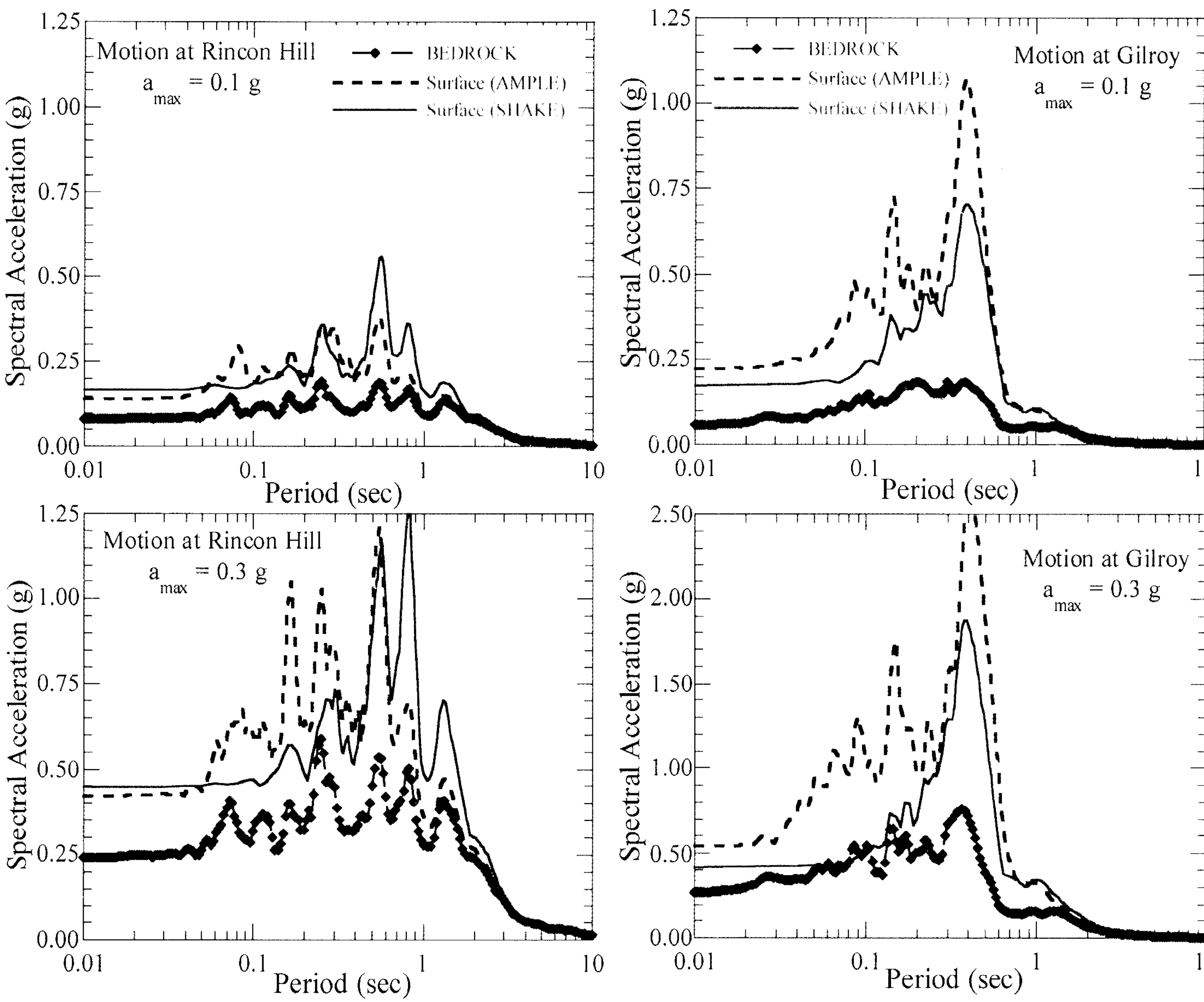

Fig. 4. *Acceleration response spectra for the shallow soil profile.*

The results from the AMPLE2000 analysis for the Gilroy motion show a significant greater amplification of the input motion than SHAKE, except at high periods (T>1 sec). One reason that SHAKE predicts higher amplifications than AMPLE2000 for the Rincon Hill motion compared to the Gilroy motion, is the difference in predominant periods for the soil columns calculated by both codes and the effect of soil non-linearity. The period of the soil column in SHAKE is calculated as 0.5 seconds, which corresponds to a peak in the acceleration spectra of the Rincon Hill record. The predominant period of the soil column (based on the small strain stiffness) in AMPLE2000 is 0.36sec, which corresponds more closely to the predominant period of the Gilroy motion.

Deep Profile

Figure 5 shows the acceleration response spectra for the 183 m (600 ft) profile to the motions recorded at Rincon Hill and Gilroy scaled to 0.1g and 0.3g, respectively. When the input motion was scaled to 0.1g, AMPLE2000 predicts higher amplification at the surface than SHAKE at all periods less than 2 sec. In the case where the input motion is scaled to 0.3g, SHAKE not only predicts lower accelerations than AMPLE2000 for the surface at periods less than 3 seconds, but also predicts deamplification of the input motion for periods less than 0.9 sec. AMPLE2000 predicts significant amplification for all frequencies at both 0.1g and 0.3g input PGAs. SHAKE however predicts deamplification of the input motion up to 0.5 sec for the case where the input motion is scaled to 0.3g. This main difference between the two frameworks is primarily due to the effect of small strain nonlinearity. The enhanced model predicts smaller amount of shear modulus degradation and damping ratios for the same level of strains as the confining stress increases, whereas SHAKE constrains the shear modulus degradation to essentially three levels: < 0.5 p_{at}, 0.5-2 p_{at} and 2 p_{at} (Seed and Idriss, 1970).

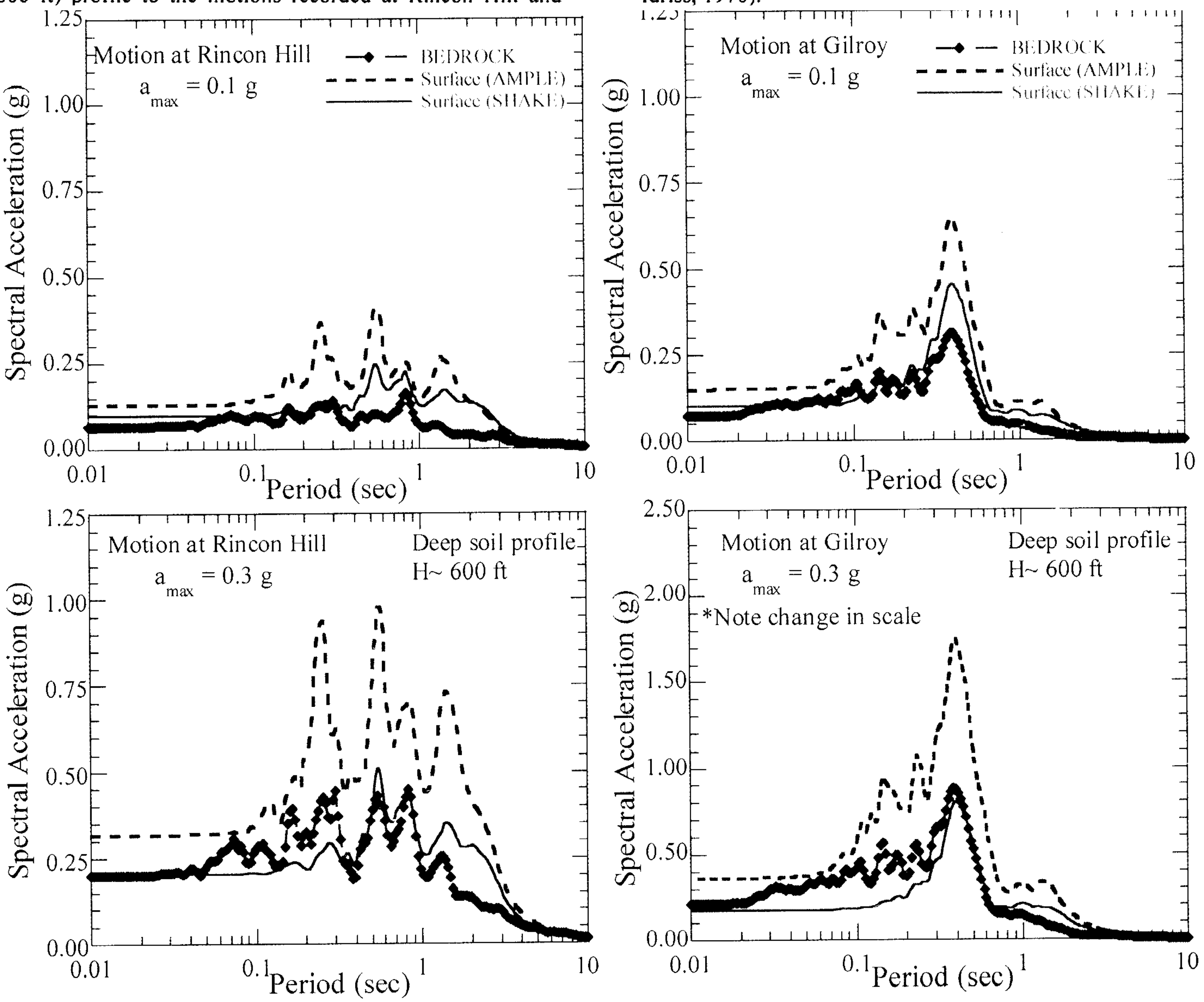

Fig. 5. *Acceleration response spectra for the deep soil profile.*

For both records and both shallow and deep soil profiles, the response spectra computed from SHAKE tend to be smaller over the higher frequency range (T<0.1 sec), and this is expected and attributed to the overdamping by the equivalent linear procedure in the frequency domain as generally known. In the high frequency regime, the nonlinear procedure using the hysteretic model may introduce a small amount of high frequency noise since the damping can be rather low for small strains. However, as the level of excitation increases the influence of the viscous damping at small strain becomes less significant to the overall computed response.

CONCLUSIONS

An enhanced hysteretic model was implemented into the site response analysis code, AMPLE2000, and a new formulation for the maximum shear modulus was included. With only a few parameters, the model is able to match the modulus reduction curves and damping independently, and describe the behavior of a material over a wide range of confining pressures. The new model gave similar results to SHAKE for the shallower profile and for the lower lever of excitation. In fact the new model suggests that SHAKE may overpredict amplifications in these cases. However, when the deeper deposits are excited, and especially with higher levels of shaking, the enhanced hysteretic model gives significantly higher acceleration response spectra than the equivalent linear procedures. This indicates a potential shortcoming in the analysis of deep cohesionless soil deposits such as those present in the alluvial valley of Caracas. A significant amount of amplification over the entire range of periods relevant for the analysis of structural response can be expected. This may explain in part the large amount of structural distress observed in the Palos Grandes area during the Caracas earthquake of 1967.

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