Characteristics of Nonlinear Response of Deep Saturated Soil Deposits

by Shean-Der Ni, Raj V. Siddharthan, and John G. Anderson

Abstract  Recent EPRI seismic design guidelines call for dynamic soil properties (shear modulus ratio and damping) and liquefaction strength curves to be characterized as a function of the effective vertical stress (or depth). A modified version of the DESRA2 constitutive model for saturated soil has been applied to study the nonlinear seismic response including liquefaction of medium dense soil deposits of various thicknesses. The results of the stress-dependent soil properties model show lower deamplification and higher first-mode (resonant) frequency than that of the stress-independent soil properties model. By using the stress-dependent model with impulse base excitation, the nonlinear behavior of various soil deposits has been investigated under a variety of conditions. The results show that (1) the saturated soil deposit has a smaller surface amplitude and significantly lower resonant frequency than the unsaturated soil deposit of the same thickness; (2) for the saturated soil conditions, the larger the base excitation, the lower the surface amplification and the resonant frequency; (3) the deep soil deposits show lower surface amplification and resonant frequency compared to the response of shallow deposits; (4) when shallow and deep deposits are compared, the shallow deposits develop much higher residual pore-water pressure; and (5) the amplification and residual pore-water-pressure response of deposits deeper than 100 m or so are very similar. The application of the method has also been illustrated using a strong synthetic base excitation applied to the base at a site near Reno. The results in general are consistent with those computed using the impulse loading. The study reveals that the response predicted from the conventionally used stress-independent soil properties model is unconservative for deep deposit.

Introduction

The nonlinear behavior of soil plays an important role in altering the characteristics of ground motions when subjected to strong shaking. Numerical approaches to predict the nonlinear response of soil can be classified as either an equivalent secant approach [e.g., the SHAKE program by Schnabel et al. (1972)] or a direct nonlinear approach [e.g., the DESRA2 program by Lee and Finn (1978) and the CHARSOIL program by Streeter et al. (1974)]. Many studies have compared the response computed by both of these methods. Such studies have concluded that the secant approach cannot reproduce some of the important characteristics of the ground motion, especially for the case of strong ground shaking (e.g., Constantopoulos et al., 1973; Streeter et al., 1974; Finn et al., 1978). Yu et al. (1993) used DESRA2 to examine the differences between linear and nonlinear soil response with various levels of base excitations. By using this direct nonlinear approach, they showed that in strong excitations soil nonlinearity causes deamplification and also a shift in peak frequencies to lower values for an unsaturated shallow soil deposit of 20 m thickness.

Response evaluation of deep saturated deposits, which is the topic of this article, differs in many ways from that of shallow unsaturated deposit. This article uses the term saturated to refer to a soil column with a shallow (usually 3 m) water table, and unsaturated to mean that the water table is below the soil column and the pore-water pressure is assumed to be zero. The soil properties and characteristic behavior of deep deposits are quite different from that of shallow deposits, as noted by Electric Power Research Institute (EPRI, 1993). The material properties are a function of effective stress and, consequently, a function of depth. Thus, hysteretic nonlinear soil behavior of a deep strata should be characterized using many sublayers, with each sublayer assigned a set of uniform properties. The sublayer properties, as pointed out later, should also account for a stress- (or depth-) dependent damping ratio and shear modulus ratio variations with shear strain (EPRI, 1993). In addition, the coupling of pore-water pressure and ground shaking play an important role in soil response. The influence of pore-water pressure becomes significant when the strength of excitation is large. The generation of residual pore-water pressure reduces the effective stress during excitation and, as the input...
excitation becomes larger, can cause liquefaction. These factors are vital to the dynamic response of deep soil columns subject to base excitation. This study extends the work of Yu et al. (1993) to deep soil deposits and accounts for the aforementioned factors.

Proposed Model

This study uses a numerical code to evaluate the nonlinear soil response, including pore-water-pressure generation and dissipation. The numerical model used in this study is a modified version of the DESRA2 program developed by Lee and Finn (1978). The model is based on the finite-element method and is capable of evaluating motions within a soil column for a vertically propagating shear-wave input. The dynamic equations of motions are numerically integrated in the time domain (direct method) using the tangent approach. It is an effective stress-based approach in which the soil shear stiffness is modified as the residual pore-water-pressure generation takes place. Each finite element in the physical model follows a nonlinear shear-strain relationship, and the hysteretic behavior of soil is modeled using Masing criteria as described by Finn et al. (1977).

The program DESRA2 has been widely used by many researchers and practicing engineers to compute soil response (e.g., Hushmand et al., 1987; Finn, 1981; Finn, 1988). Modifications to the original program were such that some of the recently available nonlinear soil behavior data can readily be incorporated. The main changes to the original DESRA2 are to include new characterization for stress-strain and volumetric-strain relationships since better characterizations have been proposed (EPRI, 1993; Byrne, 1991). The modifications have been chosen such that only a minimum number of soil parameters are used in the model, while retaining many of the convenient features of the original model.

The new shear stress-strain equation for initial loading is defined as

\[ \tau = \frac{G_{\text{max}} y}{1 + \frac{\gamma}{\gamma_y} \frac{\sigma'}{\sigma_y}} \]  

where \( \tau \) and \( \gamma \) are shear stress and strain; \( G_{\text{max}} \) is the shear modulus at very low strain level; \( \gamma_y \) is the reference strain, defined as \( \gamma_y = \frac{\tau_{\text{max}}}{G_{\text{max}}} \); and \( a \) is a constant. It may be noted that in the original model, \( a \) is set to equal unity. A recent study by Nakagawa and Soga (1995) also proposed this type of variation based on a large database of laboratory soil behavior. The subsequent unloading and reloading are given by Masing stress-strain curves (Masing, 1926). During the strong shaking, the effective vertical stress decreases because of residual pore-water-pressure generation. The increment in residual (excess) pore-water pressure, \( A_u \), is evaluated using the pore-water-pressure model of Martin et al. (1975), given by

\[ A_u = \tilde{E}_r \Delta \varepsilon_{vd} \]  

in which \( \Delta \varepsilon_{vd} \) is the increment in volumetric compaction strain, and \( \tilde{E}_r \) is the one-dimensional rebound modulus. The \( \Delta \varepsilon_{vd} \) is a function of accumulated volumetric strain \( \varepsilon_{vd} \) and shear strain \( \gamma \). Martin et al. (1975) used four coefficients to evaluate \( \Delta \varepsilon_{vd} \). Byrne (1991) provided a new, much simpler relation,

\[ \frac{\Delta \varepsilon_{vd}}{\gamma} = c_1 \exp \left( - c_2 \frac{\varepsilon_{vd}}{\gamma} \right) \]  

in which \( c_1, c_2 \) are constants that depend on the relative density of sand.

The rebound modulus \( \tilde{E}_r \) is a function of effective stress level \( \sigma'_{\varepsilon} \), and the relation is given by Martin et al. (1975) as

\[ \tilde{E}_r = \frac{(\sigma'_{\varepsilon})^{1-m}}{mK_r(\sigma'_{\varepsilon})^{n-m}} \]  

in which \( \sigma'_{\varepsilon} \) is the initial value of the effective stress and \( K_r, m, n \) are experimental constants for a given sand. Since soil behavior is governed by effective stress, the soil stiffness decreases as the pore-water pressure increases. The modified values of \( G_{\text{max}} \) and \( \tau_{\text{max}} \) are

\[ G_{\text{max}} = (G_{\text{max}}) (\sigma'_{\varepsilon} - u)^{1/2} \]  

and

\[ \tau_{\text{max}} = (\tau_{\text{max}}) (\sigma'_{\varepsilon} - u) \]  

in which \( (G_{\text{max}}) \) and \( (\tau_{\text{max}}) \) are the values that correspond to zero residual pore-water pressure (Finn et al., 1977).

If the pore water is allowed to drain during shaking, the dissipation condition must be considered. The dissipation effect is evaluated by solving

\[ \frac{\partial u}{\partial t} = \tilde{E}_r \frac{\partial}{\partial z} \left( k \frac{\partial u}{\partial z} \right) + \tilde{E}_r \frac{\partial \varepsilon_{vd}}{\partial t} \]  

in which \( k \) is the permeability and \( \gamma_u \) is the unit weight of water. This equation resembles the heat conduction in a bar.

Redistribution of pore-water pressure can take place during the excitation, and the net effects of contemporary generation and dissipation is given by (6). Equations (1) through (6) define the stress-strain relation and the pore-water-pressure generation and dissipation model. These equations are solved numerically in conjunction with the dynamic equations of motion of the sand layer using a step-
by-step integration procedure in the time domain. More details on the procedure may be found in Finn et al. (1977) and Lee and Finn (1978).

Selection of Soil Parameters

Consider a deep stratum of medium dense sandy deposit (relative density, \( D_r = 60\% \)) with water table at 3 m from the surface (Fig. 1). The properties of the sand vary only in the vertical direction. The sand deposit is assumed to rest on an impermeable rock with a shear-wave velocity given by 2500 m/sec. The input motion is present at the interface between the soil and rock.

The deposit is divided into sublayers of 1 m in thickness, each with approximately uniform properties. The maximum shear modulus, \( G_{\text{max}} \), in any sublayer is computed as

\[
G_{\text{max}} = 218.8 \ (K_2)_{\text{max}} (\sigma_m')^{1/2}
\]

in which \((K_2)_{\text{max}}\) is the constant that depends on the relative density of soil and \(\sigma_m'\) is the effective mean normal stress of the layer (Seed and Idriss, 1970). Here \(G_{\text{max}}\) and \(\sigma_m'\) are given in kPa.

Traditionally, the nonlinear behavior of soil is defined by two strain-dependent soil parameters: normalized shear modulus ratio, \(G/G_{\text{max}}\), and damping ratio, \(\zeta\) (Seed and Idriss, 1970; Hardin and Dvnevich, 1972). There is a large database for these parameters for many types of soil. However, a vast majority of these laboratory tests that were used to estimate these parameters were obtained from tests conducted at a confining pressure range of 100 to 300 kPa. Therefore, the applicability of these parameters is limited to shallow deposits of depth up to 20 to 30 m or so.

The EPRI has recently recommended the use of depth-(or stress-level) dependent soil properties (\(G/G_{\text{max}}\), and \(\zeta\)) in the evaluation of deep soil response (EPRI, 1993). This recommendation was arrived at by using many laboratory tests under low and high confining pressures. Figure 2 shows the EPRI recommendation for normalized shear modulus ratios, \(G/G_{\text{max}}\), and damping ratios, \(\zeta\), for sand at different depths as a function of shear strain (EPRI, 1993). It is seen that as the depth of soil increases, the values of \(G/G_{\text{max}}\) and \(\zeta\) for a given strain level increases and decreases, respectively. In other words, the soil elements under large confining pressure have lower damping and do not exhibit a strong nonlinear behavior.

The stress-strain relationship (equation 1) can be rewritten as

\[
\frac{G}{G_{\text{max}}} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_y}\right)^n},
\]

The soil parameters \(\gamma_y\) and \(a\) in (8) have been evaluated such that equation (8) provides a best fit to the \(G/G_{\text{max}}\) and

The EPRI Guidelines

- 0 - 6 m
- 6 - 15 m
- 15 - 37 m
- 37 - 76 m
- 76 - 152 m
- 152 - 305 m

Figure 2. Characteristic behavior of sand (after EPRI, 1993).
The Masing criteria that defines the unloading and reloading gives the area enclosed by a strain cycle, which is proportional to $\zeta$. An optimization technique has been utilized to estimate different sets of $\gamma_s$ and $\alpha$ values for each depth (or stress level). It may be noted from Figure 2 that there is a certain damping $\zeta_0$ at the low ($10^{-4}$%) strain level. This damping amount is considered not to be due to hysteretic soil behavior and therefore was subtracted from EPRI data in the optimization. This damping, which is around 0.6% to 1.5%, is quite small and is incorporated through the damping matrix in the dynamic equilibrium equations (Finn et al., 1977).

Table 1 gives the optimum values of $\gamma_s$ and $\alpha$ for fitting all the relationships shown in Figure 2. The EPRI data and the soil properties, $G/G_\infty$, and $\zeta$, computed from the model used in this study are shown in Figure 3. The $\zeta_0$ values have been subsequently added to the computed damping values when plotted in the figure. Only two sets of data (depths 0 to 6 m and 76 to 152 m) are shown. The figure reveals that the predicted values provide a good fit to EPRI data.

Other important model parameters are those that define the volume change and pore-water-pressure generation behavior. The volume change behavior (constants $c_1$ and $c_2$) has been extensively studied by Byrne (1991), who recommends values as a function of the relative density of soil. Recommended values for $c_1$ and $c_2$ are 0.24 and 1.66, respectively, for a sand with $D_r = 60\%$. A convenient way of obtaining pore-water-pressure model parameters is to match a specified liquefaction potential curve with the one predicted by the pore-water-pressure generation model used in the approach (Finn et al., 1982). In using the DESRA2 model, it is customary to select model constant $K_r$ (equation 4) such that there is a close match between the predicted and specified liquefaction potential curves. The liquefaction curve for given $D_r$ (or SPT value, $N_l$) can be obtained from an extensive field database compiled by Seed and Idriss (1982). Since this database is applicable for shallow deposits (stress level $\approx 100$ kPa), it should be modified when using it for a deep deposit. Guidelines reported by Marcuson et al. (1990) can be used to obtain the liquefaction potential curves that are relevant for higher confining pressures by scaling the curve obtained from Seed’s data. Figure 4 shows the three predicted and target liquefaction potential curves.
may be noted that liquefaction resistance when specified as a ratio \((z/\sigma'_{iq})\) is higher for low confining stress case. It is clearly evident from Figure 4 that by selecting an appropriate value for \(K_r\) it is possible to closely match a given liquefaction potential curve. Table 1 lists the optimum value of \(K_r\) for all the stress levels reported in the EPRI study.

Results and Discussions

In this study, we use the approach of applying an impulse base excitation that is the same as that of Yu et al. (1993) to examine the nonlinear behavior. We designate this impulse \(a(t)\) (Fig. 5). Its corresponding Fourier amplitude spectrum, \(A(f)\), gives the advantage of a flat frequency response below a corner frequency at 20 Hz (Fig. 5). Acceleration response at the surface will be designated as \(a(t)\), with Fourier spectrum \(A(f)\).

The shear-wave velocity can be estimated from the maximum shear modulus that is based on the mean effective stress at different depth (equation 7). Since the maximum shear modulus is proportional to the square of the shear-wave velocity and to the square root of depth (equation 7), the shear-wave velocity is proportional to \(1/4\) power of depth. Figure 6a illustrates the shear velocity model used in the study. One of the ways to understand the response of soil of different thickness is to pay attention to the first-mode (resonant) frequency. This is because the largest spectral amplifications typically occur at the first-mode frequency. Figure 6a illustrates the shear velocity model used in the study. One of the ways to understand the response of soil of different thickness is to pay attention to the first-mode (resonant) frequency. This is because the largest spectral amplifications typically occur at the first-mode frequency. Figure 6b shows the first-mode frequency, \(f_0\), computed using \(f_0 = (V_s)_{avg}/(4H)\), in which \((V_s)_{avg}\) is the arithmetic average of shear-wave velocity within the thickness of soil being considered. This is the simplest way to estimate the first-mode frequency, \(f_0\), and it assumes linear soil behavior. The program DESRA2 also has been used to obtain \(f_0\) for the linear soil model. This variation, shown in Figure 6b, matches well with that computed using the arithmetic average velocity.

Figure 6b shows that the first-mode frequency decreases as the thickness of soil increases. The decrease is most rapid when thickness is less than 40 m. The first-mode frequency falls below 1 Hz when the thickness is greater than 80 m. Two important interpretations can be made from Figure 6. Though this interpretation is strictly based on linear soil behavior. Since the variation in the first-mode frequency is small for deeper deposits (depth above 100 m), in general, we expect more similarities than differences between the soil models with thickness > 100 m. Secondly, small earthquakes, which typically contain more energy in high frequencies, can excite only higher modes of the deep soil deposits. It should be noted that these interpretations, which are based on linear soil model, may not be applicable for nonlinear soil. This aspect is revisited subsequently in this article.

By applying a 0.5 g impulse loading shown in Figure 5 to the soil deposit shown in Figure 1, the influence of stress-dependent soil properties can be demonstrated (Fig. 7).
soil deposit is assumed to be 100 m thick. The depth- (or stress-) dependent properties listed in Table 1 are assigned for the stress-dependent soil behavior model. Results from a stress-independent soil behavior model also are presented in Figure 7, in which soil properties for \( \sigma' \approx 100 \text{ kPa} \) (second row in Table 1) were applied for the entire deposit. It should be noted that the stress-independent soil model has been used routinely in response analysis in the past. Time series (normalized by the peak value of the input impulse) and normalized Fourier spectrum (normalized by input Fourier spectrum) of the surface motion along with that of the input motion are provided in Figure 7. Though both soil models show deamplification, the stress-dependent soil behavior model predicts larger response, i.e., lower attenuation of the input excitation. The dominant frequency is also slightly higher. Many factors play roles in influencing the response differences between the soil models used. The stress-dependent model is initially stiffer and has lower damping (Fig. 2) but lower liquefaction strength (Fig. 4). Therefore, it is not straightforward to ascertain that the stress-dependent soil properties model will exhibit higher dominant frequency. For the deep soil deposits, the stress-independent soil properties model will underestimate the surface amplitudes. In other words, the routine analyses, which use a stress-independent soil model, can give rise to unconservative soil amplification response. In the subsequent investigations reported below, the stress-dependent soil model has been used.

Figure 8 compares the soil surface response calculated from linear and nonlinear representation for soil. When applicable, the influence of including pore water (or water table) also is presented. In all cases, the impulse excitation with \( a_{\text{max}} = 0.5 \text{ g} \) is used. Figure 8a shows the acceleration time history of the surface motion normalized with respect to the maximum input motion (i.e., 0.5 g), and Figure 8b shows the respective normalized Fourier spectrum for all cases. Since Yu et al. (1993) reported on the unsaturated soil case, this case is also considered by ignoring the presence of the water table in the analysis. When the linear case is considered, the soil shear modulus is kept at its initial value,
and residual pore-water-pressure generation effects are ignored. On the other hand, in the nonlinear case, the soil is modeled as nonlinear hysteretic and can generate residual pore-water pressure.

As seen by Yu et al. (1993), the nonlinear characterization of soil consistently gives a lower peak surface acceleration, and the spectral peaks shift to lower frequencies. This is expected since the nonlinear characterization gives rise to a softer soil deposit. When the influence of pore water (i.e., saturated) is included, the soil stiffness is further reduced, resulting in further reduction in surface acceleration. The largest deamplification is computed for the case of saturated soil deposit with nonlinear soil behavior. This means that since the pore water strongly affects the characteristics of ground motion, it must not be ignored in the assessment of seismic hazard analysis.

Yu et al. (1993) pointed out that spectral amplitudes are increased in the high-frequency \( f > 10 \text{ Hz} \) band when nonlinear soil response is compared with linear soil response. The revised model used here also reproduces that prediction for deep deposits, and for the case of saturated soil conditions.

Influence of excitation strength on the soil response is presented in Figure 9 for the case of saturated soil deposit. The format of the presentation is similar to that of Figure 8. There is surface amplification for the case of \( a_{\text{max}} = 0.001 \text{ g} \), while for higher base excitations, there is deamplification. The deamplification for the case of \( a_{\text{max}} = 0.5 \text{ g} \) is by as much as 60%. The spectral plot reveals that first-mode and higher-mode frequencies shift to lower frequencies, and the corresponding amplitudes also decrease as the excitation strength increases. But in the higher-frequency range \( f > 10 \text{ Hz} \), the near-linear case \( a_{\text{max}} = 0.001 \text{ g} \) shows somewhat lower Fourier amplitudes.

Figure 10 shows the effects of soil deposit thickness on ground response for the nonlinear saturated case. Three soil thicknesses \( D = 20, 100, \) and \( 200 \text{ m} \) were considered, and all deposits were subjected to the impulse loading with \( a_{\text{max}} = 0.5 \text{ g} \). The peak amplitudes are substantially decreased as the thickness of the soil increased from 20 to 100 m, beyond which the increasing of the depth of the soil has negligible influence. A similar observation has also been made earlier based on linear soil behavior. Though the peak values of the normalized accelerations are similar for \( D = 100 \text{ and } 200 \text{ m} \), the spectral characteristics of the surface motion are different, as seen at the bottom of Figure 10. The resonant frequency for \( D = 200 \text{ m} \) is smaller at 0.45 Hz as opposed to 0.7 Hz for \( D = 100 \text{ m} \). This may be an important aspect for structural response since structures are quite sensitive to the frequency (or period) of the input motion. The response of the structures for motions with similar amplitudes but with different frequency contents can be substantially different. Therefore, care should be taken when using a shallow soil model to represent a deeper strata.

The amplification in locations within the soil deposit is important for underground structures and for structures founded on piles. The peak values of acceleration response in the upper 50 m of the soil deposit are shown in Figure 11. These peak values are shown as an amplification ratio defined as the ratio of the peak value within the soil to the peak of the incoming pulse. There are significant variations in amplification ratio within the 20-m (shallow) deposit, while the changes within the 50 m of the deeper soil deposit are small. Here again, the differences in acceleration response between \( D = 100- \text{ and } 200- \text{ m soil deposit is minimum.} \)

The typical trend of amplification ratios shown in Figure 11 is that amplification ratio decreases as the distance from the surface increases, and then, subsequently it increases. The depth to the minimum amplification ratio may be referred to as the turning point for amplification (Fig. 11). The locations of these turning points vary with the thickness of deposit. This can be explained by studying the interference between the upgoing and downgoing waves and difference in their frequency contents as a result of prior propagation through different soil thicknesses. To illustrate this, Figure 12 shows the motions at different depths for the 100-m soil deposit. Because of the attenuation of high-frequency signals
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in the soil deposit, the period of the signal becomes larger as it approaches the surface. The minimum amplification ratio occurred (turning point) at the shallowest depth within the layer where the upgoing and downgoing waves do not constructively interfere. Above the turning point location, amplitudes increase to as much as twice the amplitude below it, because of constructive interference between the upgoing and downgoing waves. The depth of the turning point becomes greater, as shown in Figure 12, for thicker alluvial deposits in which the frequency content of the upgoing impulse is decreased. Figure 13 shows the Fourier amplitude spectra of the seismograms in Figure 12. The first-mode frequency does not change from the surface to the depth of approximately 70 m. Below 70 m, the downgoing waves are weak, so the spectra are dominated by input impulse, with spectra peaking at zero frequency (Fig. 5), and the first-mode frequencies become insignificant. The absence of the second-mode frequencies around the depth of 25 m shows evidence of destructive interference.

The amplification ratio within the soil deposit is also affected by the input strength, as shown in Figure 14. The characteristics of the surface motions for these cases have already been presented in Figure 9. The higher the strength of input, the lower the amplification ratio throughout the deposit. Another interesting observation is that even though amplification is indicated in the case of lower level of excitation ($a_{\text{max}} = 0.001 \text{ g}$), there is deamplification in deeper locations within the soil. This again illustrates the role of the trade-off between attenuation and the surface effect in controlling amplitude.

Figure 15 illustrates the maximum pore-water-pressure ratio that was computed within the duration of the shaking as a function of depth for the three thicknesses, $D = 20$, 100, and 200 m. Since this ratio is normalized by the initial vertical effective stress, they also reveal how close to liquefaction the soil deposit is. The liquefaction is indicated when pore-water-pressure ratio equals 1.0. The impulse loading with $a_{\text{max}} = 0.5 \text{ g}$ does not last long enough to cause liquefaction. The largest pore-water-pressure ratio is indicated for soil with $D = 20 \text{ m}$. Similar to an earlier observation relating to acceleration, the pore-water-pressure responses for $D = 100$ and 200 m are also essentially the same. The maximum pore-water-pressure ratio for three different input strengths is shown in Figure 16. The larger input

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**Figure 10.** Equivalent of Figure 7, illustrating the influence of soil deposit thickness on surface motion for $a_{\text{max}} = 0.5 \text{ g}$. This model has a water table at 3 m depth.**

**Figure 11.** Peak acceleration amplitude within the soil deposit for different soil thicknesses. These values have been normalized by the maximum acceleration of the upgoing pulse at the base of the soil, 0.5 g in this case.
Figure 12. Acceleration seismograms at various depths in a 100-m soil column, normalized by input $a_{max}$ (0.5 g), and corresponding peak values of the normalized seismograms.

Strength induces larger pore-water-pressure ratio in the deposit. There is no pore-water pressure computed for $a_{max} = 0.001$ g. A general feature for a 100-m soil column is that the pore-water-pressure ratio only varies slightly within the top 50 m.

**Application to Field Problems**

The impulse loading study in a previous section provides for a fundamental understanding of the nonlinear response of deep deposits. An application to a field situation is expected to provide additional insights. The results presented below employ a synthetic accelerogram as input to nonlinear calculation, to predict the soil response for a large earthquake that could affect the Reno area. The synthetic accelerogram that represents “earthquakelike” excitation provides a flexible way for seismic hazard assessment because they can be generated from a specific fault plane and propagated to assumed site location with different earth models.

**Reno Scenario Earthquake and Base Input**

Reno is the largest city with a sizable concentration of population in northwest Nevada, located at the western margin of the Basin and Range Province. A nearby large earthquake is possible. One fault system that could generate destructive motion in Reno is the Carson range fault system; the segments of this system that are close to Reno city are the Washoe valley segment, the Mt. Rose segment, and the Reno segment (dePolo et al., 1996). These three fault segments are assumed to rupture together to generate a Reno scenario earthquake (dePolo et al., 1996). The postulated scenario earthquake has magnitude of 7.1. The locations of these three fault segments, the epicenter, and the site of interest are shown in Figure 17.

A synthetic accelerogram for a Reno scenario earthquake is generated from the composite source model (Zeng et al., 1994; Yu, 1994; Anderson and Yu, 1996). Several studies have shown that synthetic seismograms from a composite source model can provide realistic estimations on ground motions of future large earthquakes (Zeng et al., 1994; Su et al., 1994a, 1994b; Anderson and Yu, 1996). Fault parameters shown in Table 2, which were obtained from geological descriptions of the faults or from prior experience, were used with the composite model to generate the base excitation. The velocity, $Q_s$, and density model used in the composite model to generate ground motion on rock are listed in Table 3. The N-S component of the corresponding motion is then applied at the base of the deep soil deposit.

Beeston et al. (1992) provided a soil profile and the
field-measured shear-wave velocity for a site near Reno (Fig.

Figure 14. Equivalent of Figure 11, showing the influence of different excitation strengths for a 100-
m soil deposit with 3-m water table.

Figure 15. Pore-water-pressure response within the soil deposits for different soil thickness. The input pulse has $\sigma_{\text{max}} = 0.5 \text{ g}$ for all soil thicknesses.

a conservative estimation of soil response, although the frequency peaks are potentially different.

Figure 19 shows the input seismogram and the nonlinear soil response. The maximum acceleration of the input is 0.41 g. The computed surface motion (Fig. 19b) indicates deamplification and has a much lower-frequency component than the input motion. The computed deamplification values are consistent with those obtained for impulse loading with $\sigma_{\text{max}} = 0.5 \text{ g}$ (see Fig. 11). This means that the amplification results described for the impulse loading can be used as a guide to estimate soil behavior when subjected to earthquakelike motions.

For comparison, a stress-independent soil properties model (using only the second row of soil properties in Table 1) for the selected Reno site is also presented in Figure 19c. The surface response in Figure 19c is smaller than that in Figure 19b. This observation is also consistent with the result reported with the impulse loading (Fig. 7), indicating that the results from stress-dependent soil properties model predictions are unconservative for deep deposits. The stress-independent model assigns softer soil properties to soils located in deeper depths.

The difference between these two models is significant for pore-water generation. Figure 20 shows the maximum pore-water-pressure ratios for stress-dependent and stress-
The study presented here used a direct nonlinear approach to investigate the soil response of deep deposits. The program DESRA2 (Lee and Finn, 1978) has been modified so that it can account for recently developed guidelines for nonlinear soil properties provided by EPRI. These recent guidelines propose to use depth- (or stress-) dependent shear modulus ratio \((G/G_0)\) and damping ratio \((\zeta)\). The stress-dependent behavior of the liquefaction potential curves (Marcuson and Hynes, 1990) also have been incorporated in the DESRA2 computations. This was achieved by calibrating the pore-water-pressure generation model of DESRA2 against the stress-dependent liquefaction potential curves.

Initially, impulse loading and, subsequently, earthquake-like excitation were used to study the characteristic of the deep soil responses. Since the stress-independent soil properties model has been used routinely (conventionally) in the past, comparisons between the stress-dependent and independent soil properties models. The stress-independent model underestimates substantially the pore-water pressure. The stress-dependent model predicts liquefaction, and the stress-independent model does not. The liquefaction is indicated at depths below 107 m. This deep liquefaction is a consequence of the lower normalized liquefaction resistance for high confining stress (Fig. 4). It causes additional filtering of high frequencies later in the seismogram, as seen in Figure 19b. Although consistent with the reported shear velocity profile (Fig. 18), the assumption that the relative density is constant (60%) is significant for this result. It is an open research question whether this actually occurs. From the engineering perspective, liquefaction at such depths may not be detrimental to structures founded at the surface.

Conclusions

Figure 16. Influence of excitation strength on pore-water-pressure response for a 100-m soil deposit.

Figure 17. Surface projection of the fault plane, epicenter, and the site used for the Reno scenario seismograms in Figure 19. Heavy lines identify surface expression of individual segments identified by de-Polo et al. (1996).

Table 2

<table>
<thead>
<tr>
<th>Fault Parameter for Comprising the Reno Scenario Earthquake</th>
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<tr>
<td>Fault length (km)</td>
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<td>Fault width (km)</td>
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<tr>
<td>Seismic moment (dyne-cm)</td>
</tr>
<tr>
<td>Moment magnitude</td>
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<tr>
<td>Maximum depth (km)</td>
</tr>
<tr>
<td>Strike (degree)</td>
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<tr>
<td>Dip (degree)</td>
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<td>Rake (degree)</td>
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<tr>
<td>Rupture velocity (km/sec)</td>
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<td>Stress drop of subevents (bars)</td>
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<td>Maximum radius of subevents (km)</td>
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Table 3
The Earth Model for Reno Basin

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<tr>
<th>Thickness (km)</th>
<th>( V_p ) (km/sec)</th>
<th>( Q_p )</th>
<th>( V_s ) (km/sec)</th>
<th>( Q_s )</th>
<th>Density (g/cm³)</th>
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</table>

![Soil profile](image)

**Figure 18.** The soil profile at the selected Reno site.

Using the response of a 100-m soil deposit subjected to the impulse loading with \( a_{\text{max}} = 0.5 \text{ g} \). The saturated soil deposit with the ability to generate residual pore-water pressure shows larger deamplification and lower first-mode (resonant) frequency. The surface amplification ratio dropped from 0.70 to 0.45 when the saturated soil deposit was compared with the unsaturated soil deposit (Fig. 8). Since the pore water strongly affects the important characteristics of ground motion, it must not be ignored in the assessment of seismic hazard analysis.

The strength of base excitation and thickness of soil are the two important factors that influence soil response. Higher base excitation leads to a larger deamplification and a lower resonant frequency for the computed ground motions. This is consistent with the past results on unsaturated soil (Yu et al., 1993). High excitation strength also generates larger residual pore-water pressure.

There are major differences in responses between shallow and deep deposits. The shallow soil deposit has larger surface amplification and higher resonant frequency than that of a deep soil deposit. The residual pore-water pressures are also substantially higher in the case of shallow deposit. Among deeper soil deposits (thickness > 100 m), the difference in amplification ratios within and at the surface of the deposit is small, whereas the resonant frequency depends on the thickness of soil deposit. Similar to the acceleration response, the deep deposits (thickness > 100 m) also indi-
cate similar residual pore-water response. Since the acceleration and the residual pore-water pressure response are similar for deep deposits, the location of the “rock” where the base excitation is to be specified does not need to be known precisely in response studies. Only the resonant frequency is expected to be affected.

In the second phase of the study, a synthetic strong motion from a magnitude 7.1 earthquake was applied at the base of a deep soil deposit in Reno for which field-measured shear-wave velocity profile is available. The maximum input base acceleration from a composite model was 0.41 g. The acceleration response and pore-water pressure response were evaluated. Though the stress-dependent and stress-independent soil properties models predict deamplification, the acceleration response computed with the stress-dependent model is much larger. Similarly, the residual pore-water pressures generated in the stress-dependent model was higher. This study leads to an important conclusion that the soil response computed using a conventional stress-independent soil properties model is unconservative for deep deposits.

Acknowledgments

We would like to thank the reviewers for an excellent job. This research was supported by the Southern California Earthquake Center (SCEC). The SCEC publication number is SCEC 318.

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Program for Earthquake Response Analysis of Horizontally Layered Sites, Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, California.


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Manuscript received 1 August 1995.