Non-Linear Site Response Analysis for Deep Deposits in the New Madrid Seismic Zone

Youssef M. A. Hashash
University of Illinois at Urbana-Champaign, Urbana, IL

Duhee Park
University of Illinois at Urbana-Champaign, Urbana, IL

Jungsuk Lee
University of Illinois at Urbana-Champaign, Urbana, IL

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Youssef M.A. Hashash
University of Illinois at Urbana-Champaign
Urbana, IL-USA-61801

Duhee Park
University of Illinois
Urbana, IL-USA-61801

Jungsuk Lee
University of Illinois
Urbana, IL 61801

ABSTRACT

The New Madrid Seismic Zone, the most seismically active zone in the Eastern US, is overlain by deep unconsolidated deposits of the Mississippi Embayment. The deposits range in thickness from about 20 m in the St. Louis area to about 1 km in the Memphis Area and consist of silts, clays and sands. The influence of these deposits on the propagation of seismic waves to the ground surface remains a major source of uncertainty. A new non-linear one-dimensional site response analysis model is introduced for the vertical propagation of horizontal shear waves in deep soil deposits. The model accounts for the effect of large confining pressures on the strain dependent modulus degradation and damping of the soil. The capability of the new model is illustrated using soil columns at three typical locations within the Mississippi Embayment including a 1000 m column representative of conditions in Memphis. The analyses show that high frequency components usually filtered using conventional wave propagation methods, are preserved. The analyses show that spectral amplification factors for the deep deposits in the period range of 0.6-5sec range between 2 and 6, and at longer long periods (up to 10 sec) can be as high as 8.

INTRODUCTION

Earthquakes in the New Madrid Seismic Zone (NMSZ) are characterized as low probability high consequence events. The estimate of ground motion characteristics in the NMSZ is required for assessing seismic vulnerability of structures and susceptibility of soils to liquefaction. The presence of very deep (up to 1000 m) unconsolidated deposits in the Mississippi Embayment has an important, though poorly understood effect on the propagation of seismic waves.

Earthquake activity elsewhere has shown the importance of local site conditions on propagated ground motions. Strong motion records from recent earthquakes including Loma Prieta, 1989, Northridge, 1994, Hyogoken-Nanbu, 1995, and Chi-Chi, 1999, in Taiwan show significant differences between soil sites and nearby rock sites. Such records are not available for the NMSZ and the Mississippi Embayment.

In the absence of strong motion records, numerical models can be used to develop an understanding of the wave propagation characteristics of the Mississippi Embayment. This paper proposes an enhancement of an existing one–dimensional wave propagation non-linear model to account for the effect of very high confining pressures encountered in the Mississippi Embayment.

GEOLOGY OF THE MISSISSIPPI EMBAYMENT

The Mississippi Embayment is a syncline or a trough-like depression that plunges southward along an axis, which approximates the course of the Mississippi River. The Embayment, beginning near the Gulf of Mexico and extending north to the confluence of the Ohio and Mississippi Rivers, is surrounded by the Illinois Basin to the north, the Nashville dome and southern Appalachian Plateau to the east, and the Ouachita and Ozark uplifts to the west as shown in Fig. 1. The Paleozoic rock that forms the bedrock floor of the Mississippi Embayment is located about 1000 m below Memphis and Shelby County, which is near the central part of the Mississippi Embayment, as shown in Fig. 1 (Ng et. al., 1989). The Embayment is filled with sediments of clay, silts, sand, gravel, chalk and lignite ranging in age from Cretaceous to Recent. There is no well-consolidated rock above the Paleozoic rock, except some local beds of sandstone and limestone.

The axis of the Embayment is nearly coincident with the underlying Reelfoot rift, which is the most prominent buried structure in the northern Embayment. The New Madrid seismic zone (NMSZ), a clustered pattern of earthquake epicenters between 5 and 15km deep, lies mostly within the Reelfoot rift. The NMSZ is also shown in Fig. 1a.

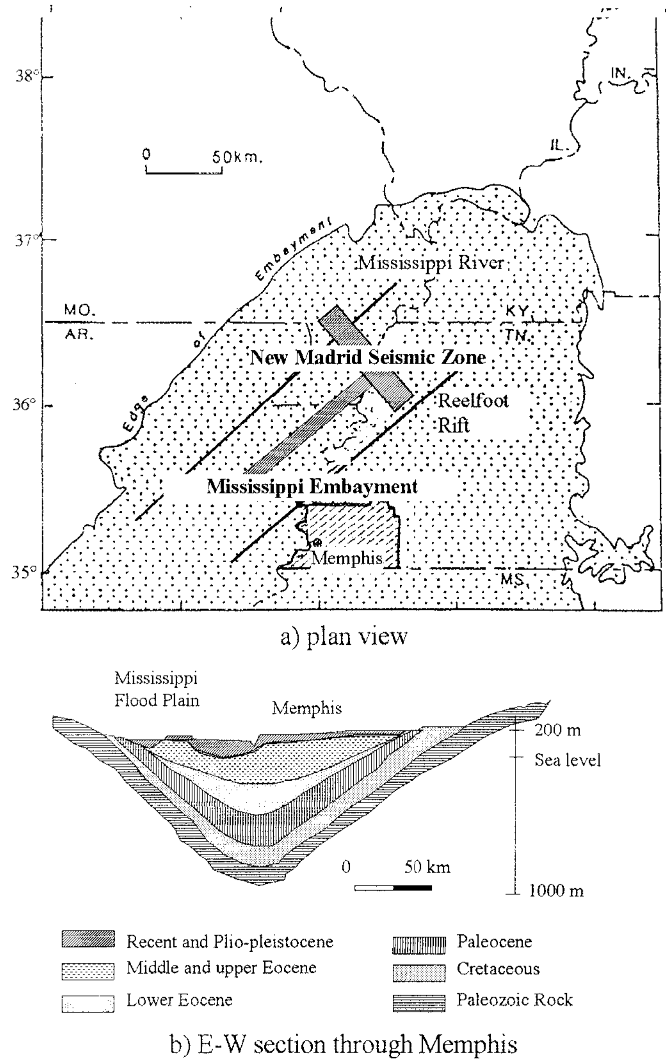

Fig. 1. The Mississippi Embayment (after Ng. et al., 1989)

The presence of thick unconsolidated deposits adds significant uncertainty regarding the nature of seismic ground motion propagation and attenuation in the Embayment. The effect of soil deposits on propagated ground motion is well documented in other parts of the world (e.g. Mexico City). However limited information is available regarding wave propagation through very thick deposits (up to 1000 m) such as those found in the Mississippi Embayment.

In Fig. 1b, where the vertical dimension is highly exaggerated, the trough representing the Embayment has a shallow slope of less than 1/150. The geologic layers can be considered nearly horizontal. Analysis of wave propagation through these deposits is approximated as a one-dimensional vertical propagation of horizontal shear waves.

Three profiles, 1000m, 500 m, and 100 m deep, shown Fig. 2, are selected to represent the range of soil depths encountered in the Embayment (Ng. et. al, 1989). The 1000 m profile is representative of conditions in the Memphis, Shelby County area while the 100 m profile represents conditions south of the St. Louis Area. The selected shear wave velocity profile is based on a combination of surface information and few deep wells as compiled by Rix et. al. (2000). The wave propagation analyses presented in this paper use the profiles shown in Fig. 2.

CYCLIC SOIL BEHAVIOR

The behavior of soil under cyclic loading is highly non-linear and dependent on a range of factors including amplitude of loading, number of loading cycles, soil type and in situ confining pressure. In its simplest form, the nonlinear behavior is commonly characterized by a secant shear modulus and viscous damping to represent the hysteretic soil response (Seed and Idriss, 1970; Hardin and Drnevich, 1972). The secant shear modulus, normalized by the maximum shear modulus, is shown to decrease with increasing magnitude of cyclic shear strain. Damping, which is a measure of energy dissipation in a loading cycle, increases with increasing magnitude of cyclic shear strain. Modulus degradation and damping curves for a wide range of soils have been developed by several researchers (e.g. Vucetic and Dobry, 1991). These curves have been extensively used in estimating the seismic site response in relatively shallow deposits (<30 m). Soil parameters such as plasticity index, void ratio and relative density influence dynamic properties. For cohesionless soil, the variation of the dynamic curves with change in soil properties is small and therefore it is assumed that the modulus degradation and damping curves fall within a narrow range (Seed & Idriss, 1970).

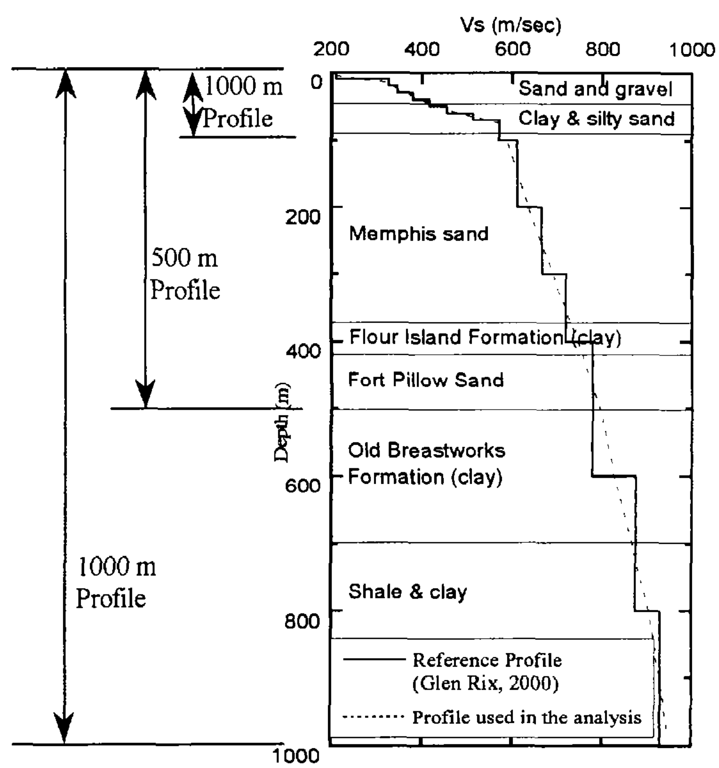

Fig. 2. Soil and Shear Wave Velocity Profiles used in the Analysis

The effect of confining pressure on dynamic properties, which is significant compared to other soil properties, has been widely recognized (e.g. Hardin et. al. 1994). Hardin et. al. (1994) present high pressure (up to 3.5 MPa) test data on sand and conclude that "damping ratios at conventional pressures are approximately equivalent to those at pressures up to 3.5 MPa." They report a damping ratio of 0.5% at strains less than 10^{-3} %. Hardin et. al. (1994) suggest that additional research is necessary to further understand cyclic soil response at very high pressures. Ishibashi and Zhang (1993) published relations relating modulus reduction to confining pressure and soil plasticity index.

Laird and Stokoe (1993) performed resonant column and torsional shear tests at strain levels up to 10^{-1}% and confining pressures up to 3.5 Mpa using remolded sand specimens, as well as undisturbed specimens such as sands, silty sands, silts, lean clays, and fat clays. Low and high amplitude cyclic torsional shear and resonant column tests were used to determine the effect of strain amplitude and confinement on shear modulus and damping curves. In this paper, only the results of the remolded sand specimens (washed motar sand) are used. The testing was part of the investigation for the ROSRINE project (http://rccg03.usc.edu/rosrine/) examining the local site response in Los Angeles Basin. The data shows that as the confining pressure increases, less degradation in the shear modulus is measured. The confining pressure has significant influence on the damping as well. The data of Laird and Stokoe (1993) show significantly less degradation of the shear modulus ratio compared to degradation curves by Seed and Idriss (1970).

ONE-DIMENSIONAL WAVE PROPAGATION IN SOILS

One-dimensional site response analysis is used to solve the problem of vertical propagation of horizontal shear waves (SH waves) through a horizontally layered soil deposit. In its simplest form, the analysis assumes the horizontal soil layers to behave as a Kelvin-Voigt solid where by soil properties are characterized by a constant elastic shear modulus and viscous damping. Solution of the wave propagation equations is performed in the frequency domain. In order to better capture the non-linear cyclic response of soil, Seed, Idriss and co-workers introduced the equivalent linear approximation. The strain dependent modulus degradation and damping curves are used to obtain revised values of shear modulus and damping. An iterative scheme is required to arrive at a converged solution. This approach has provided good results compared with field measurements and is widely used in practice (e.g. SHAKE, Schnabel et. al., 1972).

The equivalent linear approach does not capture the full range of cyclic behavior of soil, including modulus degradation due to number of loading cycles, permanent straining of soil, and excess pore pressure generation. A constitutive model represents the cyclic behavior of the soil. In non-linear analysis the dynamic equation of motion is solved in time domain:

$$[M]\ddot{u} + [C]\dot{u} + [K]u = -[M]\ddot{u}_g \qquad (1)$$

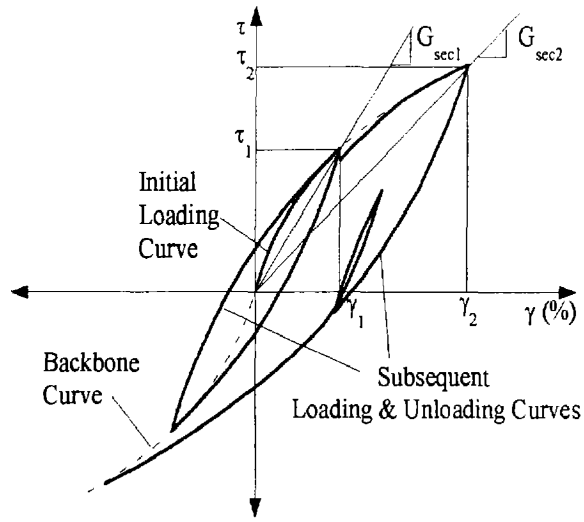

Fig. 3. Hyperbolic Non-linear Soil Model

where [M], [K] and [C] are the mass, damping and stiffness matrices respectively, $\ddot{u}, \dot{u}$, and u are the acceleration, velocity and displacement relative to the base, and $\ddot{u}_g$ is the base acceleration. The matrices are assembled using the constitutive properties of the soil layers. The earliest constitutive relations use a simple model relating the shear stress to the shear strain, whereby the backbone curve is represented by a hyperbolic function. Modulus and damping degradation curves are used to define of the backbone curve. The Masing criteria (Masing, 1962) and extended Masing criteria (Pyke, 1979) defines the unloading-reloading criteria and the behavior under irregular loading as illustrated in Fig. 3. . Lee and Finn (1978) developed the one-dimensional seismic response analysis program using the hyperbolic model. Matasovic (1993) further extended the model with modification of the hyperbolic equation and also included the constitutive model for clays. Soil damping is captured through the hysteretic behavior of the soil model. The damping matrix may be used as a mathematical convenience or to include damping at very small strains. Plasticity models have also been used to represent cyclic soil behavior.

PRESSURE DEPENDENT 1-D CYCLIC SOIL MODEL

The backbone curve for the model by Matasovic (1993) and implemented the code D_MOD, is described by:

$$\tau = \frac{G_{mo}\gamma}{1 + \beta\left(\frac{G_{mo}}{\tau_{mo}}\gamma\right)^s} = \frac{G_{mo}\gamma}{1 + \beta\left(\frac{\gamma}{\gamma_r}\right)^s} \qquad (2)$$

where τ = shear stress, γ = shear strain, G_{mo} = initial shear modulus and τ_{mo} = initial shear strength. τ_{mo} is selected as the shear stress at approximately 1% of strain. $\gamma_r = \tau_{mo} / G_{mo}$ is the reference shear strain (Hardin and Drnevich, 1972) and considered as a material constant. The model is a modification of the hyperbolic model by Kondner and Zelasko (1963), through the addition of the two parameters β and s which adjust the shape of backbone curve to better model soil behavior. The model is extended in this paper to capture the influence of confining pressure effects.

Strain Dependent Shear Modulus and Confining Pressure

Hardin and Drnevich (1972) show using laboratory test data on clean dry sand that the reference strain is dependent on the confining pressure and that it can be used as a normalizing strain to capture modulus degradation and damping variation with confining pressure. In D_MOD, γ_r is used as a constant material property. In the new model the reference strain, γ_r, is:

$$\gamma_r = a\left(\sigma' / \sigma_{ref}\right)^b \tag{3}$$

σ_{ref}, reference confining pressure, is 0.18 MPa. Fig. 4 shows that using the proposed equation, the model can capture the variation in shear modulus measured from lab experiments. The figure shows extrapolated modulus degradation curve at an effective stress of 10000 kPa (depth~1000m).

Strain Dependent Damping Ratio and Confining Pressure

The hysteretic damping of the soil model defined by Matasovic (1993) can capture damping at strains larger than 10^{-4} to 10^{-2} %, depending on the value of the reference strain. The use of Eq. (2) captures the hysteretic damping dependency on confining pressure. However, the hyperbolic model is nearly linear at small strains (less than 10^{-4} to 10^{-2} %) with practically no damping which can cause unrealistic resonance during wave propagation. The model described by Matasovic (1993) incorporates additional damping to the dynamic equations in the form of [C] matrix. The damping matrix [C] is expressed as:

$$[C] = (2\xi / \omega)[K] \tag{4}$$

where ω is the circular frequency and ξ is the equivalent damping ratio. [C] is assumed to be independent of the strain level and therefore, the effect of hysteretic damping induced by the nonlinear soil behavior can be separated from, but is additive to the viscous damping. The damping value of ξ can be obtained from the damping ratio curves at small strains. A constant small strain viscous damping is used in D_MOD with a recommended range of 1.5 – 4 % for most soils.The dependency of small strain damping ratio on confining pressure is described as:

$$\xi = c / (\sigma')^d \tag{5}$$

σ' is the vertical effective stress. The recommended upper bound value of ξ is 1 – 1.6 %. Fig. 5. includes plots of total damping equal to hysteretic plus small strain damping.. The total damping curves fall within the range of measured data.

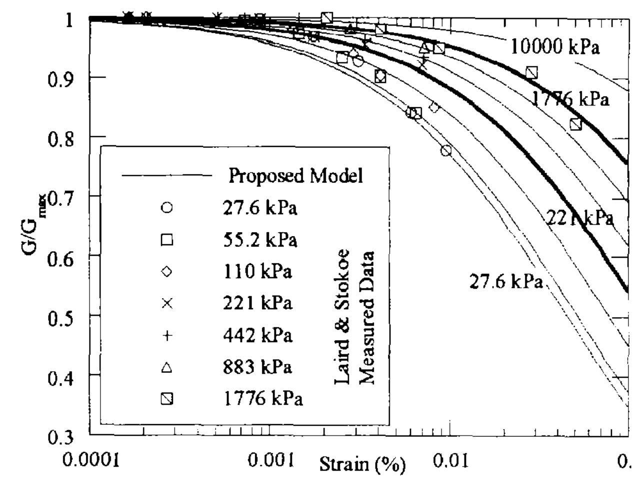

Fig. 4. Modulus Degradation Curves at different confining pressures for the extended Hyperbolic Model

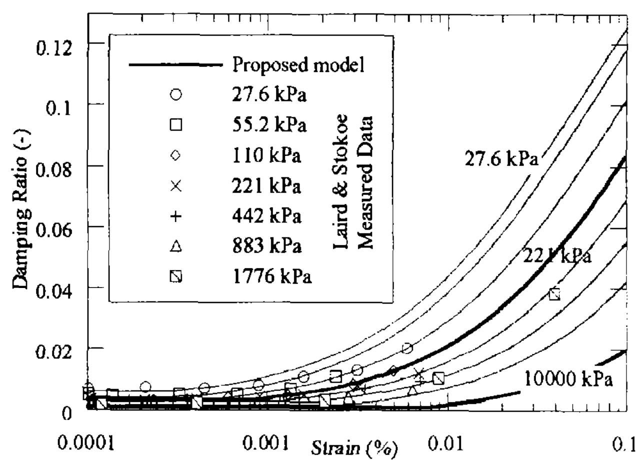

Fig. 5. Damping Curves at different confining pressures for the extended Hyperbolic Model

ANALYTICAL STUDY

The proposed model is implemented in a 1-D wave propagation code called DEEPSOIL. A series of analyses are presented using the proposed, confining pressure dependent to illustrate the capability of the new model. The analyses use recordings from a range of earthquake events as input ground motion time series. The recordings represent time series at an equivalent rock outcrop. The peak accelerations, a_{max}, for the recordings range from 0.0073 g to a high of 0.83 g. The three soil columns shown in Fig. 2 are used in the analyses. A rigid base is assumed in these analyses given the sharp contrast in shear wave velocity between the Embayment deposits and the underlying rock.

INFLUENCE OF CONFINING PRESSURE ON 1-D SITE RESPONSE ANALYSIS

The effect of confining pressure on 1-D site response analysis is apparent through comparisons of the new pressure dependent model (NLPD) with the original pressure independent model (NLPI). The influence of confining pressure is shown in Fig. 6. a,b for a soil thickness of 1000m. The influence of the confining pressure is very pronounced. Short period spectral accelerations are much larger for the NLPD compared to NLPI. There is a significant development of long period oscillations as can be seen in the acceleration TS and the presence of significant long period spectral accelerations. Similar observations can be made for a soil column thickness of 500 m. For a column thickness of 100 m, Fig. 6. c response spectra are also similar for a T>0.8 sec. For shorter period the NLPD spectral acceleration is larger than that of NLPI.

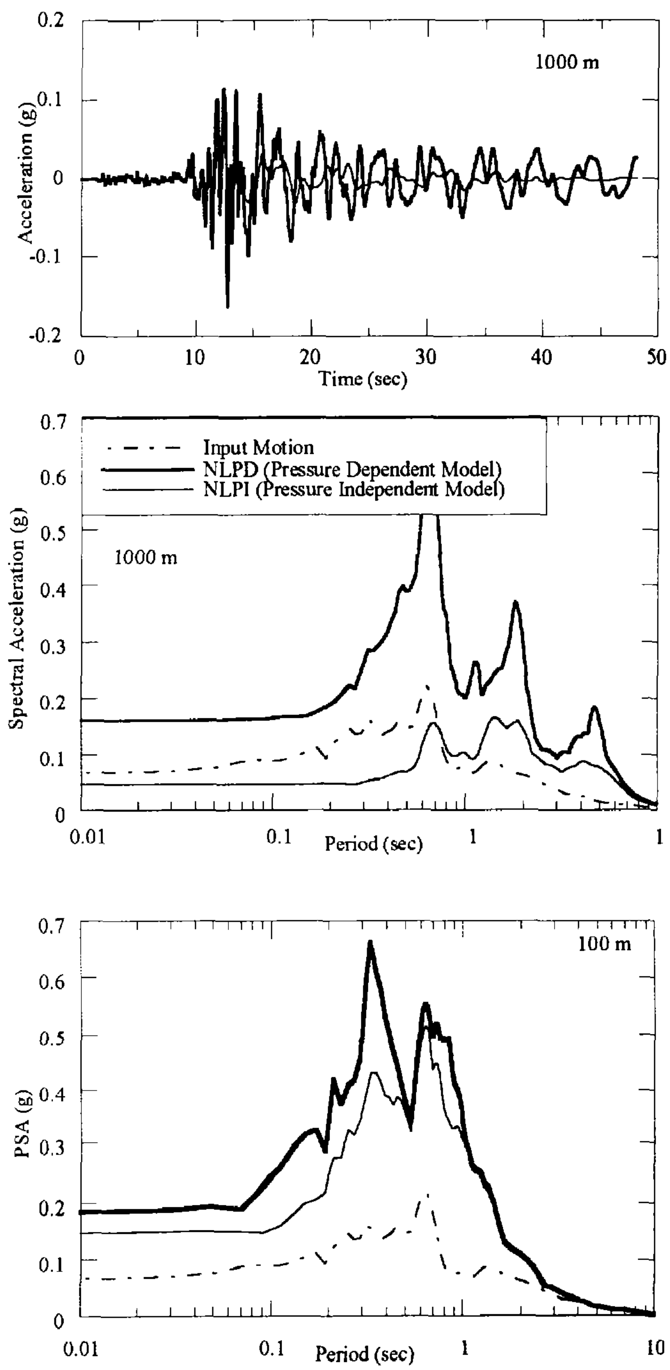

Fig. 6. Effect of Pressure Dependency on Surface Response Spectra (5% Damping), Input Motion; Yerba Buena, Loma Prieta, 1989, PGA=0.067g.

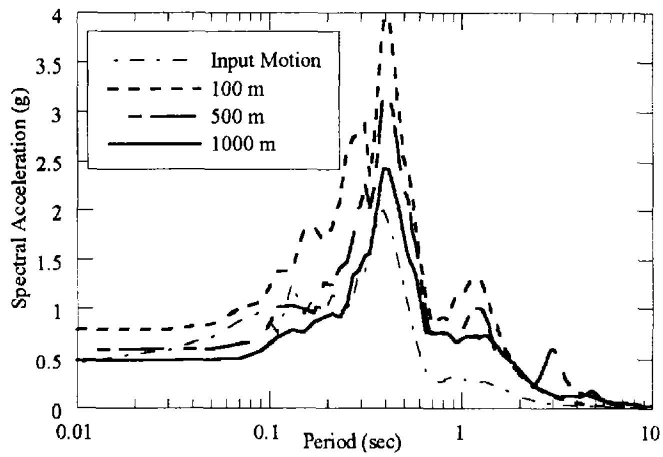

Fig. 7. Effect of Soil Column Thickness on Surface Response Spectra (5% Damping), Input Motion: Gilroy #1, Loma Prieta, 1989, PGA=0.43g..

LAYER THICKNESS AND EXTENT OF NONLINEAR RESPONSE

This and the following sections present a synthesis of the analysis results using the new DEEPSOIL program only. Fig. 7 presents surface response spectra for the three soil columns using the Gilroy # 1, 1989 Loma Prieta Recording (PGA=0.43 g). The figure shows that generally the spectral response is largest for the 100m column and decreases with increasing soil depth. This is indicative of filtering of some motion components through the deep deposits. However, at periods greater than about 0.7 sec the spectral response is overall similar for all three columns.

AMPLIFICATION FACTORS

The spectral amplification factor, defined as the ratio of the surface spectral acceleration to the input motion spectral is plotted in Fig. 8 for a range of input motions with a peak ground acceleration ranging between 0.0073 g to 0.83 g using the 1000m soil column. The input motions were rock site recorded motions from selected earthquakes in California, Mexico and Japan. In the short period (high frequency) range up to T=0.03 sec, the amplification factor is nearly one for most ground motions. For 0.03 sec <T< 0.25 sec, there is significant attenuation of the spectral acceleration. The main exception is for T.H. No's. 2 & 3 which have an amplification factor of about 2-2.5 in T<0.25 sec. For T>0.25 sec, significant spectral amplification is computed with a value of up to 3 around T=1 sec and as high as 8 in the long period range (T>4 sec). Other analyses for the three soil columns show that the amplification factor at short periods, high frequency, generally increases with decreasing soil deposit thickness. The amplification factor at long periods decreases with decreasing column thickness.

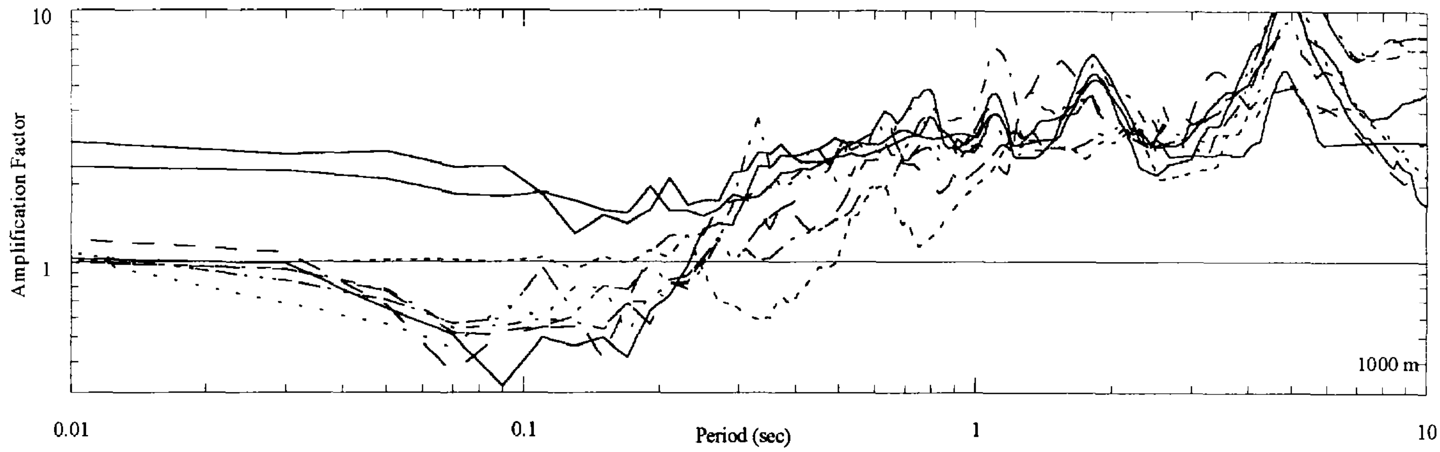

Fig. 8. Spectral Amplification Factors of Analyses with Input Motions PGA=0.0073g-0.83g

SUMMARY

The analyses presented in this paper show the importance of incorporating the effect of confining pressure on seismic site response analysis. The analyses show that:

1. Significant portions of high frequency components of the ground motion are propagated through deep soil deposits.
2. The propagation of seismic wave through very deep deposit results in the development of long period ground motion.
3. The spectral amplitude of the propagated ground motion is higher than what would be obtained using conventional wave propagation analyses.

These preliminary results are part of an ongoing study of site response issues in the Mississippi Embayment.

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