An applied method for analysis of nonlinear seismic response of layered soil deposit

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Abstract. The available methods of Seismic ground response analysis including equivalent linear and nonlinear methods perform the analysis in the time or frequency domain. Nonlinear methods usually provide accurate results however, these methods are usually time-consuming because of their step by step integration procedure in time domain. On the other hand, simplicity, flexibility and less computational effort are the advantages of the frequency domain methods. Therefore, in this paper seismic response of layered soil deposits is performed by hybrid frequency-time domain procedure. The soil deposit is modeled as a discrete system composed of a finite number of lumped masses connected by springs and dashpots. A seismic motion is applied to the system at the base level. Pseudo-forces are applied to the system in a rational iterative procedure in the time domain to evaluate the nonlinear soil behavior. This paper presents an applied method of the nonlinear seismic ground response analysis without the need for more input data than general data provided in the most of the geotechnical investigations. Verification of the accuracy of the proposed method is made by comparing its results including acceleration and displacement time histories and acceleration response spectrum to the recorded data of different earthquakes. Further investigation is conducted by comparing the results of the proposed method with the results of equivalent linear (i.e. SHAKE computer program) and nonlinear (i.e. DEEPSOIL computer program) methods.

Keywords: Soil dynamics; Seismic analysis; Nonlinear analysis; Damping; Hybrid method; Earthquake response analysis

Introduction

Seismic response analysis of soil deposits is a part of geotechnical earthquake engineering which deal with the site effects on the propagated seismic wave in the ground. Among the most important factors

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involved in the site effects, local geologic conditions, topography, soil layering and geotechnical properties of soil to the depth of 50 m could be noted (Aki, 1988, Faccioli, 1991). Seismic ground response analyses provide researchers with important results including design response spectrum, liquefaction susceptibility estimation and seismic slope stability assessment (Kramer, 1996). Generally, methods of predicting the ground seismic response could be divided into empirical and analytical categories. Among the empirical methods, empirical equations (Day, 2002) , microtremors (Diagourtas, et al., 2001) and spectral ratio analysis (Heisey, 1982) could be mentioned. On the other hand, some researchers demonstrated that differences between characteristics of bedrock excitation and soil deposit movement during an earthquake could be analyzed and predicted (Wiggins, 1964). Analytical methods are mainly categorized into linear, equivalent linear and nonlinear methods. These methods differ in their modeling of soil behavior under earthquake loading. Available methods of the seismic ground response analysis can also be categorized in terms of their calculation domain. Based on this aspect, these methods are categorized into time and frequency domain methods. In this classification, the equivalent linear procedure is among the frequency domain methods and most of nonlinear methods are among the time domain methods.

Available methods of seismic ground response analysis

Nowadays, the non-linear behavior of soils under cyclic loading is well known. The equivalent linear method is one of the available ones to estimate the nonlinear behavior of soil. Seed and Idriss (1970) used this method for the first time in geotechnical earthquake engineering. Afterwards, other researchers tried to complete their computational procedure (Assimaki and Kausel, 2002, Yoshida, et al., 2002). Based on dynamic characteristics of the soil, including shear modulus and damping ratio, this method calculates transfer functions to perform seismic ground response analysis. Then, the mentioned soil dynamic characteristics are re-estimated according to the shear modulus degradation and damping ratio curves. These curves are provided by laboratory tests. Many researchers have offered these curves for various types of soil (Seed, et al., 1984, Seed and Idriss, 1970, Sun, et al., 1988). Based on the re-estimated dynamic characteristics of the soil, calculations are repeated. This process is continued until the error becomes less than a previously considered value. SHAKE (Schnabel, et al., 1972) is among the first computer programs presented to analyze the seismic ground response. This program implements the equivalent linear procedure to perform one-dimensional ground response analysis for soil deposits that idealized as horizontally stratified, viscoelastic medium (Idriss and Sun, 2001, Idriss and Sun, 1992). EERA (Bardet, et al., 2000) is another program which also using the equivalent linear procedure to perform seismic ground response analysis.

Nonlinear seismic ground response analysis methods are composed of a solution scheme for the wave equation and the constitutive model as their two main components. Finite element and finite difference methods could be mentioned among the solution schemes (Wood, 2004). These methods are used in forming required equations for integration in the small time steps. It is noteworthy, these two methods have developed for their part which have led to completion of their computational procedure (Bagheripour and Zhao, 1992, Chuhan and Zhao, 1987, Von Estorff and Firuziaan, 2000). The constitutive models are applied to estimate the nonlinear soil behavior under cyclic loadings. These constitutive models are based on laboratory test results. Hyperbolic (Hardin and Drnevich, 1972), Ramberg-Osgood (Ramberg and Osgood, 1943), Davidenkov (Pyke, 1979), Prevost (Prevost, 1977) and Iwan (Iwan, 1967) models could be mentioned among these models. Hyperbolic model has been used more than others. This model, is often used with Masing (Masing, 1926) and Modified Masing (Pyke, 1979) criteria to model the soil hysteresis behavior. Lee and Finn (1978) and Matasovic (1993) used this technique to analyze the nonlinear soil response under cyclic loadings. These researchers used Wilson θ (Wilson, 1968) and Newmark β (Chopra, 1995) numerical methods, respectively in their computational process. Hashash and Park (2001, 2002) and Phillips and Hashash (2009) also used the hyperbolic model and Newmark β method to solve the equation of motion in the time domain. Based on their proposed method, latter researchers presented DEEPSOIL computer program (Hashash, et al., 2011). Streeter et al. (1974) implemented Ramberg-Osgood model alongside Masing criteria to analyze the stratified soil deposits. Martin and Seed (1978), used Davidenkov model and finite element method to perform one-dimensional seismic soil response analysis. Some researchers, including Elgamal et al. (1998) and Prevost (1989) implemented more sophisticated constitutive models of porous media to predict soil behavior under the earthquake loading. Using more sophisticated constitutive models offers more accurate results and provides more realistic predictions of soil behavior under earthquake loading such as liquefaction, dilation and excess pore water pressure. However, due to the complexity of implementation of these models and calibration of their required parameters, these constitutive models are always faced difficulties for practical considerations (Lo Presti, et al., 2006).

Few downhole vertical array data has led to uncertainty about the appropriate method of analysis of the vertical seismic response of the ground. Although the common assumptions of one-dimensional ground response analysis are very common, however in some cases two or three dimensional seismic analysis is necessary. Slope ground, heavy and rigid buried structures are among these issues (Kramer, 1996). Many researchers have also analyzed the three-dimensional response of the ground under dynamic loads while most of them have used the finite element method (Kim and Roesset, 2004, Maheshwari, et al., 2005, Zamani and Shamy, 2012). Detailed explanation of multidimensional analyses is beyond the scope of this paper however, Ferrini (2001), Lo Presti and Ferrini (2002) are compared these methods in detail.

Some researchers tried to combine frequency and time domain methods to benefit from the advantages of both methods. Kawamoto (1983) proposed Hybrid frequency-time domain (HFTD) procedure for the first time to solve structural dynamics problems. In his proposed method, he solved the equation of motion in the frequency domain while nonlinear effects due to cyclic loading are applied to the system by pseudo-forces in the time domain. Thenceforth, other researchers began to complete and implement his proposed method (Darbre and Wolf, 1988). Wolf (1986) implemented the HFTD procedure to analyze nonlinear soil-structure interaction problems. After him, other researchers considered the various aspects of his work and improved it for use in soil-structure interaction problems (Bernal and Youssef, 1998, Ding and Liao, 2001). Mansur et al. (2000) used the HFTD procedure to solve multi-degree of freedom nonlinear structural problems under time-dependent loadings. They segmented the applied load into smaller time intervals to increase the accuracy of their calculations. In recent years, efforts have been made to implement the hybrid frequency-time domain procedure in the soil dynamics problems, including nonlinear analysis of seismic ground response. One-dimensional nonlinear seismic response analysis of a single layer soil deposit is among these efforts (Asgari and Bagheripour, 2010). In another study, the nonlinear response of the soil deposits analyzed using dynamic stiffness matrix to prevent recursive calculation (Bazrafshan moghaddam and Bagheripour, 2011). In the mentioned study, nonlinear soil behavior is modeled by implementation of the HFTD procedure.

In this paper, a hybrid frequency-time domain (HFTD) approach has been implemented to analyze the nonlinear seismic response of a multi layered soil deposit. Calculations are performed in a tangible and comprehensible way without the use of matrix operation techniques. In order to verify the accuracy of the results of the proposed method, the obtained results including time histories of acceleration, displacement and acceleration response spectra are compared to the recorded data of real earthquakes. The results of the proposed method are also compared to similar results obtained from the equivalent linear (i.e. SHAKE) and nonlinear time domain (i.e. DEEPSOIL) methods.

The proposed method

Analyzing the dynamic equation of motion by the numerical integration methods in the time domain have been used by many researchers. On the other hand, in some problems the frequency domain is preferred to the time domain for analyzing the problems. The main advantages of the frequency domain analyses to the time domain ones are their significant reduction in the calculation efforts, simplicity and flexibility. Providing the frequency content of the loadings and calculation of response spectra of the earthquake characteristics such as acceleration are the other advantages of the frequency domain methods (Clough and Penzien 1993, Paz, 1997). The basic shortcoming of the frequency domain methods is their inherent linear aspect of their calculations. These methods are not able to calculate the nonlinear behavior of the systems and just estimate it (Lo Presti, et al., 2006). Therefore, many researchers choose to analyze the nonlinear systems in the time domain, however in geotechnical systems, it involves difficulties because of practical considerations. In many cases the required parameters of these procedures are not recorded accurately in the reports and the obtaining process is usually expensive and time-consuming (Bazrafshan moghaddam and Bagheripour, 2011).

Another important advantage of the frequency domain methods is implementation of superposition principle in their calculation procedure. Various frequency components of the loading could be decomposed by using this principle. This is used in the calculation and investigation of response spectra. In the frequency domain analyses, earthquake loading could be assigned at any arbitrary level of the system while the response could be analyzed for any level. This could be mentioned as another advantage of the frequency domain methods since in the time domain methods, seismic loading should be assigned at the base level. As an important result, deconvolution process could be named which helps to estimate a unique input motion at the base level based on the observed seismic motion at the surface. It is important to remember, different input motion can be perceived for an observed motion at the surface in time domain methods (Kramer, 1996). Regarding these advantages, in the current study equation of motion is solved in the frequency domain so above advantages can also be considered as the advantages of the proposed method. Besides, in the current study the needed parameters to evaluate the nonlinear behavior of the soil under cyclic loadings are obtained from the shear modulus reduction and damping ratio curves provided by laboratory tests. In this paper, nonlinear behavior of the soil is considered by applying pseudo-forces calculated in time domain. Thus, in addition to benefiting from the frequency domain advantages, nonlinear soil behavior is considered in the time domain.

Soil deposit modeling

In this study, horizontally stratified soil deposit is modeled as a one-dimensional discrete system consists of a finite number of lumped masses which are connected by springs and dashpots. As shown in figure 1, a multi-layer soil column is modeled as a multi degree of freedom lumped mass system. In order to calculate the corresponding lumped mass of each layer, the following equation can be used (Ohsaki, 1982):

$$\mathrm{m}_z = \rho_z h_z \qquad , \mathrm{z}=1, 2, 3, \ldots, \mathrm{Z} \tag{1}$$

where z is the layer number, Z is the number of layers in the soil deposit, ρ, h and m are density, thickness and lumped mass of each soil layer, respectively. Stiffness of the springs is computed as (Ohsaki, 1982):

$$k_z = G_z / h_z \qquad , z=1, 2, 3, \ldots, Z \tag{2}$$

where k is the spring stiffness and G is the shear modulus. After a soil column is modeled, the bedrock behavior should be considered. Bedrock can be assumed to be rigid or elastic. If the earthquake is recorded within the soil column (e.g. vertical array), rigid bedrock is a more appropriate option (Kwok, et al., 2007) because the rigid bedrock reflects all the downward seismic waves. In other words, the bedrock motion is not affected by the movement of the overlying soil deposit. On the other hand, if the earthquake is recorded on an outcrop, elastic bedrock provides a better model for the situation, because some downward seismic waves are dissipated by the bedrock. In this case, a damper is used to connect the bedrock to soil deposit (Hashash and Park, 2001):

$$c_r = \rho_r v_r \tag{3}$$

where c_r is damping, ρ_r and v_r are density and velocity of seismic wave in the bedrock.

Equation of motion in frequency domain

Equation of motion for multi layer soil deposit is written in time domain as:

$$m_z \ddot{U}_z(t) + c_z \dot{U}_z(t) + k_z U_z(t) = F(t) \tag{4}$$

where m is mass, c is damping, k is stiffness and $U_z(t)$, $\dot{U}_z(t)$, $\ddot{U}_z(t)$ are displacement, velocity and acceleration, respectively. The index z is soil layer number as aforementioned. In the above equation $F(t)$ is the harmonic load applied to the system and written as:

$$F(t) = f e^{i\omega t} \tag{5}$$

where f is the harmonic load amplitude, t and ω are time and angular frequency, respectively. Thus, a harmonic displacement occurs due to this harmonic load, expressed as (Kramer, 1996):

$$U_z(t) = u_z e^{i\omega t} \tag{6}$$

where u_z is the amplitude of the harmonic displacement. By derivation with respect to time of the equation (6), one can write:

$$\frac{dU_z(t)}{dt} = u_z i\omega e^{i\omega t} \quad (7)$$

$$\frac{d^2U_z(t)}{dt^2} = -u_z\omega^2 e^{i\omega t} \quad (8)$$

thus, equation (4) can be rewritten as:

$$\left(-m_z\omega^2 + ic_z\omega + k_z\right)u_z e^{i\omega t} = F(t) \quad (9)$$

the term $(-m\omega^2+ic\omega+k)$ is called the dynamic stiffness of the soil layer. In the above equations, damping ratio can be used instead of viscous damping. Therefore, the concept of complex stiffness is used by the following equations (Kramer, 1996):

$$K_z^* = i\omega c_z + k_z \quad (10)$$

where K^* is the complex stiffness. It can be expressed by the following equation as well (Kramer, 1996):

$$K_z^* = (1+\eta_z i)k_z \quad (11)$$

where η is the hysteresis damping. Based on the energy balances, the correlation between hysteresis and viscous damping has been presented as (Roesset and Whitman, 1973):

$$\eta_z = 2\xi_z \quad (12)$$

where ξ is the damping ratio. Comparing equation 10 to 12, one can write:

$$K_z^* = (1+2\xi_z i)k_z \quad (13)$$

thus, equation (9) can be rewritten as:

$$\left[-m_z\omega^2 + (1+2\xi_z i)k_z\right]u_z e^{i\omega t} = F(t) \quad (14)$$

Calculation of displacement

It is necessary to obtain the required motion parameters from propagating seismic wave in the ground to perform a seismic analysis. For this purpose, there are devices that record seismic data (e.g.

acceleration) at specific time intervals. Various techniques are available to relate the discrete recorded data to desired data of an analysis. Assuming the $\varphi(t)$ as a function which is defined by A discrete points in Δt time intervals, it can be written as follows:

$$\phi(t) = \phi(a\Delta t) \qquad , a=0, 1, \ldots, A-1 \tag{15}$$

Discrete Fourier Transform (DFT) of this function is presented as (Bazrafshan moghaddam and Bagheripour, 2011):

$$\Phi(b\Delta\omega) = \Delta t \sum_{a=0}^{A-1} \phi(a\Delta t) e^{-ib\Delta\omega a\Delta t} \qquad , b=0, 1, \ldots, A-1 \tag{16}$$

where $\Phi(\omega)$ is the Fourier transform of the $\varphi(t)$, $\Delta\omega$ is the angular frequency interval. Since the $\Phi(\omega)$ has the period of ($2\pi/\Delta t$) and it is defined by A discrete points:

$$\Delta\omega = \frac{2\pi}{A\Delta t} = \frac{2\pi}{T} \tag{17}$$

where T is the time duration of recorded data. In practice, the highest frequency that can be analyzed is the Nyquist frequency which is calculated as (Idriss and Sun, 2001):

$$\omega_{nyq} = \frac{\pi}{\Delta t} \tag{18}$$

In this study, the above equations are used to relate the time and frequency domains.

According to equation (14), displacement of each harmonic component of the loading is calculated as:

$$u_z(\omega) = f(\omega) \Big/ \left[-m_z\omega^2 + k_z(1 + 2\xi_z i) \right] \tag{19}$$

where $f(\omega)$ and $u_z(\omega)$ are the Fourier transforms of the exciting force and of the displacement of the z^{th} soil layer, respectively. The linear displacement in the frequency domain is calculated by the latter equation. Nonlinear behavior of the soil which is time-dependent is applied to the soil by the pseudo-forces in a rational iterative procedure. Therefore, the displacement in the time domain is obtained based on the calculated displacement in the frequency domain. As a result, the effective shear strain is computed as a percentage of peak shear strain. The computed response is not particularly sensitive to this percentage, it is often taken as 65% (Kramer, 1996). The new values of shear modulus and damping ratio of each soil layer are estimated by the shear modulus reduction and damping ratio curves. Figure 2 shows an example of these curves. Pseudo-forces are calculated in time domain as:

$$\Delta Q_{z,j}(t) = (c_{z,o} - c_{z,j})\dot{U}_{z,j-1}(t) + (k_{z,o} - k_{z,j})U_{z,j-1}(t) \tag{20}$$

where *ΔQ(t)* is the pseudo-force in time domain and *j* is the number of iteration. The total value of pseudo-forces in j^{th} iteration is computed as:

$$Q_{z,j}(t) = \sum_{k=1}^{j} \Delta Q_{z,k}(t) \tag{21}$$

using the calculated *Q(t)*, displacement of the z^{th} soil layer is recalculated as:

$$u_{z,j}(\omega) = f_{z,o}(\omega) + Q_{z,j}(\omega) \Big/ \left[-m_z\omega^2 + k_{z,j}(1+2\xi_z i)\right] \tag{22}$$

where *Q(ω)* is the Fourier transform of the pseudo-force, *Q(t)*. This iteration process is continued until the pseudo-forces become smaller than a pre-defined value. It is obvious that the values of the aforementioned forces depend on the estimated dynamic properties of the soil layer in each step of iteration which are obtained based on the effective shear strain values. Thus, the convergence is controlled by comparing shear strain values of two consecutive iterations (Idriss and Sun, 2001):

$$\left|\frac{\gamma_{z,j+1} - \gamma_{z,j}}{\gamma_{z,j}}\right| < \varepsilon \qquad , 10^{-5} < \varepsilon < 10^{-3} \tag{23}$$

Criteria of Convergence

In each step of iterations, the following conditions must be met to establish the convergence of the calculations:

- The value of the dynamic stiffness should not be zero because in this case the displacement of soil layer can not be calculated.
- The calculated values of the pseudo-forces in each step of iteration should be smaller than their corresponding values of the previous iteration.
- The differences between calculated values of shear strain in successive steps of iteration must be descending until it becomes smaller than a pre-defined negligible value. This causes the values of the pseudo-forces to become negligible.

Considering that dynamic and geometric characteristics of the soil deposit differ in each case of analysis, the convergence of the process in each step of iteration must be carefully controlled in accordance with the foregoing.

Verification of the proposed method

The accuracy of the proposed method is verified by comparing its results with the recorded data of the real earthquakes. A computer program is written in the MATLAB environment to perform the calculations of the proposed method. The recorded earthquake accelerograms at Gilroy 1 (rock) and recorded data at Gilroy 2 stations are used as input motions and controlling data, respectively. Gilroy 2 is located 2 km east of Gilroy 1 where its soil profile is composed of various layers of gravel, sand and clay and level of the water table is at a depth of 21 m (EPRI, 1993). Gilroy 2 soil profile properties are presented by Table 1. The shear modulus degradation and damping ratio curves which are provided from laboratory tests, depicted in figure 2 (EPRI, 1993). The necessary and sufficient information for the equivalent linear method are those that given in Table 1 and Figure 2. This is all the data SHAKE needed. It is noteworthy that DEEPSOIL uses the same input parameters to compute the backbone curve which is best fit the problem. Philips and Hashash (2009) has described this calculation procedure in detail. It is obvious that more realistic results need more in-situ and laboratory tests to obtain more detailed soil parameters.

Presenting an applied method which is using the general data provided in the most of the geotechnical investigations (without the need for more dynamic in-situ or laboratory test) to analyze the seismic ground response is one of the advantages of the proposed hybrid method. Most often, the equivalent linear method is used for this purpose which provides an approximation to the ground response. Therefore, in most cases a nonlinear analysis is also performed. However, due to its nonlinear calculation procedure the HFTD method can prevent this repetition.

The results of the proposed method, in addition to the recorded data are compared with the results of the equivalent linear and nonlinear methods. In this study, the computer programs SHAKE (Idriss and

Sun, 2001) and DEEPSOIL (Hashash, et al., 2011) are applied to perform the equivalent linear and nonlinear analyses, respectively.

Site response analysis at Gilroy 2 underwent Loma Prieta earthquake

Loma Prieta earthquake (M=6.9) is occurred in a close distance of the Gilroy array. Figure 3 shows this earthquake accelerogram and its Fourier spectrum while Table 2. presents the summary of parameters for this motion. Figure 4(a) shows the recorded acceleration time history at Gilroy 2 and the result of the current study at the surface level. As could be seen, the result of the proposed method is in good agreement with the recorded data. The result of the HFTD method is compared to the ones of the equivalent linear and nonlinear methods in Figure 4(b). According to this figure, the results of different methods show good agreement, however the predicted values of the equivalent linear method are higher than other methods as well as the recorded values. Figure 5 compares the acceleration Fourier amplitudes on the ground surface. It is observed that in the frequencies less than 1 Hz the provided results by SHAKE and DEEPSOIL have the largest and the smallest values, respectively. It is also observed that in the frequencies up to 2.5 Hz, the provided results of the HFTD method are larger than the recorded. It worth to note, SHAKE is using the conventional equivalent linear procedure. This method is inherently linear and estimates the seismic ground response after some iteration steps. It generally tends to overestimate results in strong earthquakes and vise versa. This issue has been examined several times by different researchers (Lo Presti, et al., 2006, Ohsaki, 1982, Yoshida, et al., 2002). Therefore, some studies have tried to improve this method (Assimaki and Kausel, 2002, Kausel and Assimaki, 2002). On the other hand, damping calculations play an important role in the accuracy of the nonlinear soil behavior models. In general, there are two major shortcomings while using the reloading-unloading criteria including those used in DEEPSOIL (i.e. Masing criteria) in the calculation of the hysteresis loop:

- Refraining from small strains, the behavior is assumed as linear whereas in the small strains, damping controls soil behavior (Bazrafshan moghaddam and Bagheripour, 2011).
- At the large strains, damping is overestimated (Hashash and Park, 2002).

Figure 6 shows the acceleration response spectra for the damping of 10%. As this figure shows, the HFTD method presents close values compared to the recorded data. The equivalent linear method tends to overestimate the results in the periods up to about 1 second and then presents close values compared to the HFTD and recorded data. On the other hand, the nonlinear method tends to underestimate the results, however its trend is in good agreement with the other spectra. Figure 7 shows the displacement time history at the ground surface of the site for Loma Prieta earthquake. Figure 7(a) compares the obtained results of the current study to the recorded data at the site, where close values are observed. Figure 7(b) shows results of different analysis methods. The results of all applied methods follow the same trend and predict close values, although overestimation is observed in the results of the equivalent linear method. Peak ground acceleration profile is shown in Figure 8. Based on this figure, similar trends are presented by all three methods, however nonlinear method predicts smaller values, in general. Maximum of the PGA values are observed between depths of about zero to 5 meters, besides PGA is increased at the depth of 40 m, compared to its upper and lower levels. Noted, at the depth of 0-5 and 40 m the soil profile is composed of sand layers.

Site response analysis at Gilroy 2 underwent Morgan Hill earthquake

Morgan Hill earthquake (M=6.2) has occurred in the vicinity of the Gilroy array and is studied in this paper. Figure 9 shows this earthquake acceleration time history and its Fourier spectrum. The summary of this earthquake parameters is provided in Table 3. Fig 10 shows the time histories of acceleration at the ground surface, obtained from analyses and recorded data. Fig 10(a) depicts a comparison between the results of the HFTD method and recorded data at the site. According to Figure 10(b), the results of the applied methods provide approximately the same values. Besides, these results are reasonably accurate compared to the recorded data. As can be seen in Figure 10, the peak acceleration value in all obtained results is increased compared to the input motion [Figure 9(a)] which shows the site effects on the applied motion. The acceleration response spectra for damping of 10% are shown in Figure 11. According to this figure, the nonlinear method provides close prediction to the recorded data. The HFTD procedure presents slightly higher values compared to the recorded ones in the periods up to 1 second. Due to the small differences in the values of these two spectra, these differences provide safety in the calculation. After this period, results of all methods show close values, however the HFTD presents the closest values to the recorded spectrum. As observed in Figure 10 and 11, the equivalent linear method predicts the smaller values compared to the other methods. Unlike Loma Prieta, Morgan Hill earthquake is classified among the weak earthquakes with regard to its acceleration and frequency content. As can be seen, the equivalent linear method tends to underestimate the results for this event, however in strong Loma Prieta motion the results are quite contrary. This is in agreement with the conclusions of the other researchers (Lo Presti, et al., 2006, Yoshida, et al., 2002). Figure 12 shows the acceleration Fourier amplitude on the ground surface. As could be seen in this figure, provided results of all methods are nearly the same. The reason is low intensity of this earthquake which causes the small values of shear strain. Figure 13 shows the comparison of time histories of displacement at the ground surface for this earthquake. According to Figure 13(a), the results of the current study and recorded data are nearly the same. As Figure 13(b) shows, the HFTD and nonlinear methods are almost provided the same values for all loading duration, while the result of the equivalent linear method is associated with some differences in values. The observed differences in the equivalent linear result compared to other data are because of its inherently linear nature. On the other hand, the HFTD and nonlinear methods are modeled nonlinear behavior of soil properly. It must be remembered, although DEEPSOIL is a nonlinear program, but it does not mean that it can predict the actual behavior and response of the soil deposits. This program provides acceptable results based on its simple input data. It uses the hyperbolic backbone curve which is the simplest soil constitutive model (Hashash and Park, 2002). This program also uses modified Rayleigh damping matrix (Hashash and Park, 2002) which withdraws some terms of the soil damping for the simplicity of the calculations and it has the inevitable problems of the unloading-reloading criteria (Hashash and Park, 2002). Figure 14 shows the peak ground acceleration profile in the soil column. Results of all applied methods are increased at depth of 0-8 m, however PGA changes are smooth in other depths.

Conclusion

Hybrid frequency-time domain procedure is used to analyze nonlinear one-dimensional seismic response of a layered soil deposit. Fourier Transform and its inverse is paved the way to solve the dynamic equation of motion in the frequency domain. Nonlinear behavior of soil under dynamic loadings is modeled in the time domain by applying pseudo-forces in a rational iteration procedure. This paper is presented a method that performs nonlinear seismic ground response analysis without need for calibration of the constitutive models parameters which are inevitable in the conventional nonlinear methods. In the proposed method, the computational efforts are significantly reduced by solving the equation of motion in the frequency domain. The results of the proposed method are verified by comparing them to the recorded data of two different earthquakes and results of the equivalent linear and nonlinear methods which are provided by SHAKE and DEEPSOIL computer programs, respectively.

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Table 1

No.	Thickness (m)	Soil type	Shear wave velocity (m/s)	γ (kN/m^3)
1	3	Sand	184	18.9
2	5.1	Sand	269	18.9
3	3.10	Gravel	341	18.9
4	3.8	Sand	341	18.9
5	4.5	Clay	430	18.9
6	2.3	Gravel	430	18.9
7	12.9	Clay	270	18.9
8	3.3	Sand	334	20.9
9	2	Sand	504	20.9
10	12	gravel	504	20.9
11	3	gravel	717	20.9
12	5.8	sand	717	20.9
13	2.2	clay	717	20.9
14	11	gravel	717	20.9
15	4.2	gravel	527	20.9
16	16.6	clay	527	20.9
17	4.2	gravel	527	20.9
18	26	gravel	704	20.9
19	bedrock	-	1190	22.6

Table 2

Parameters	Units	Values
Peak acceleration	(g)	0.436
Arias intensity	(m/sec)	1.324
Acceleration spectrum intensity	(g.sec)	0.492
Predominant period	(sec)	0.380

Table 3

Parameters	Units	Values
Peak acceleration	(g)	0.099
Arias intensity	(m/sec)	0.060
Acceleration spectrum intensity	(g.sec)	0.062
Predominant period	(sec)	0.140

Figure 1

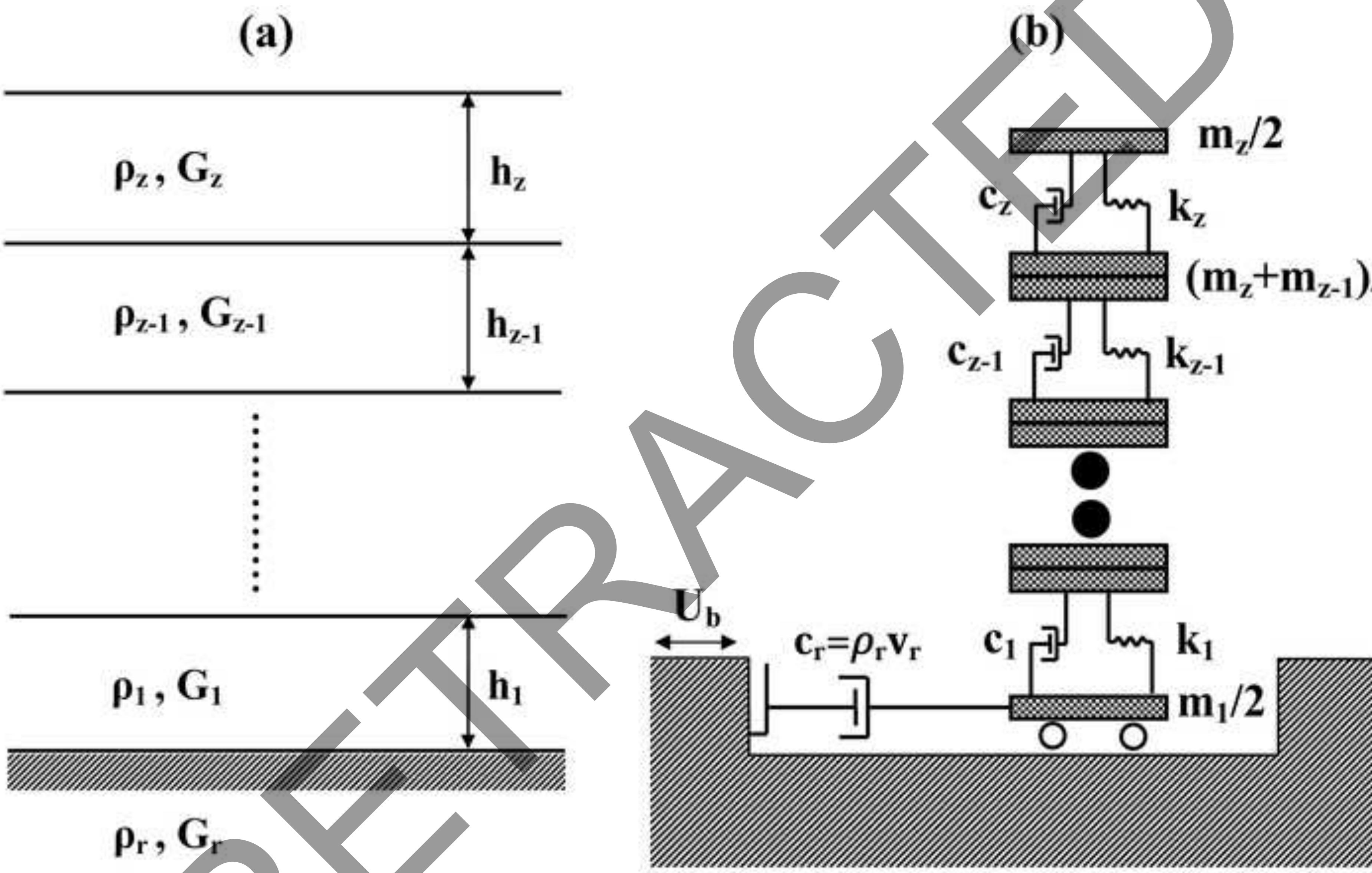

(a)
ρ_z, G_z
h_z
ρ_{z-1}, G_{z-1}
h_{z-1}
ρ_1, G_1
h_1
ρ_r, G_r
(b)
$m_z/2$
c_z
k_z
$(m_z+m_{z-1})/2$
c_{z-1}
k_{z-1}
U_b
$c_r=\rho_r v_r$
c_1
k_1
$m_1/2$

Figure 2

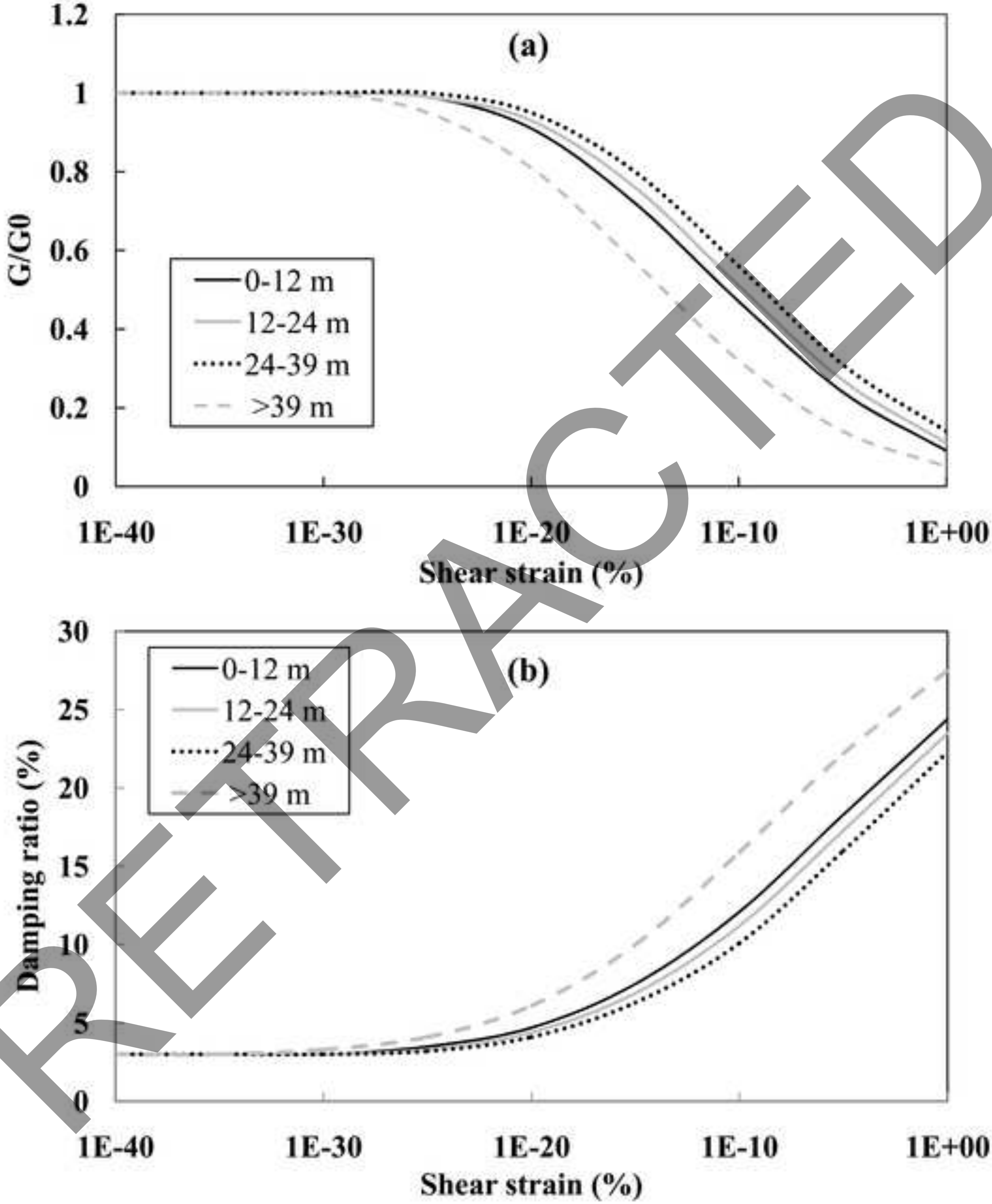
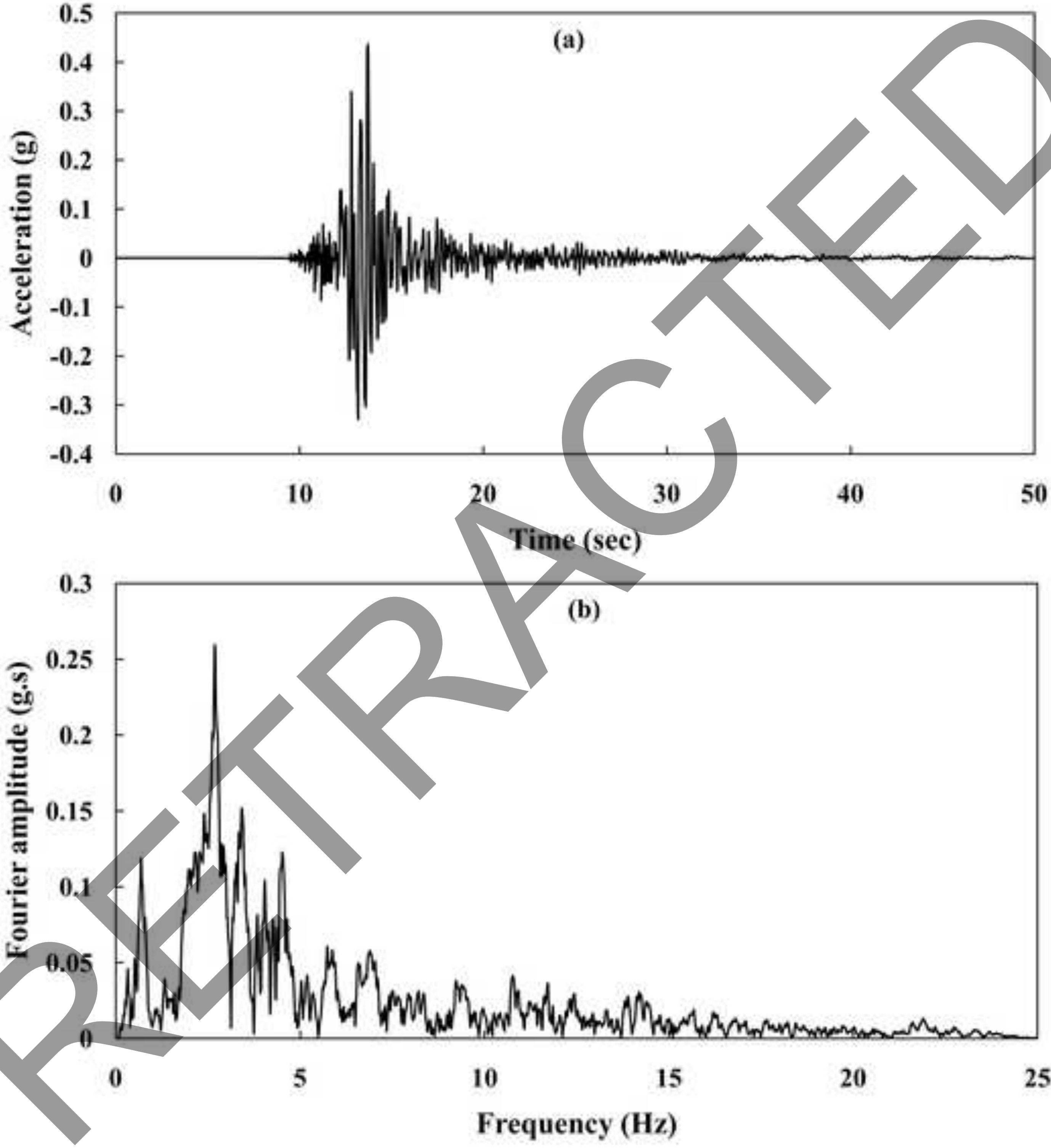
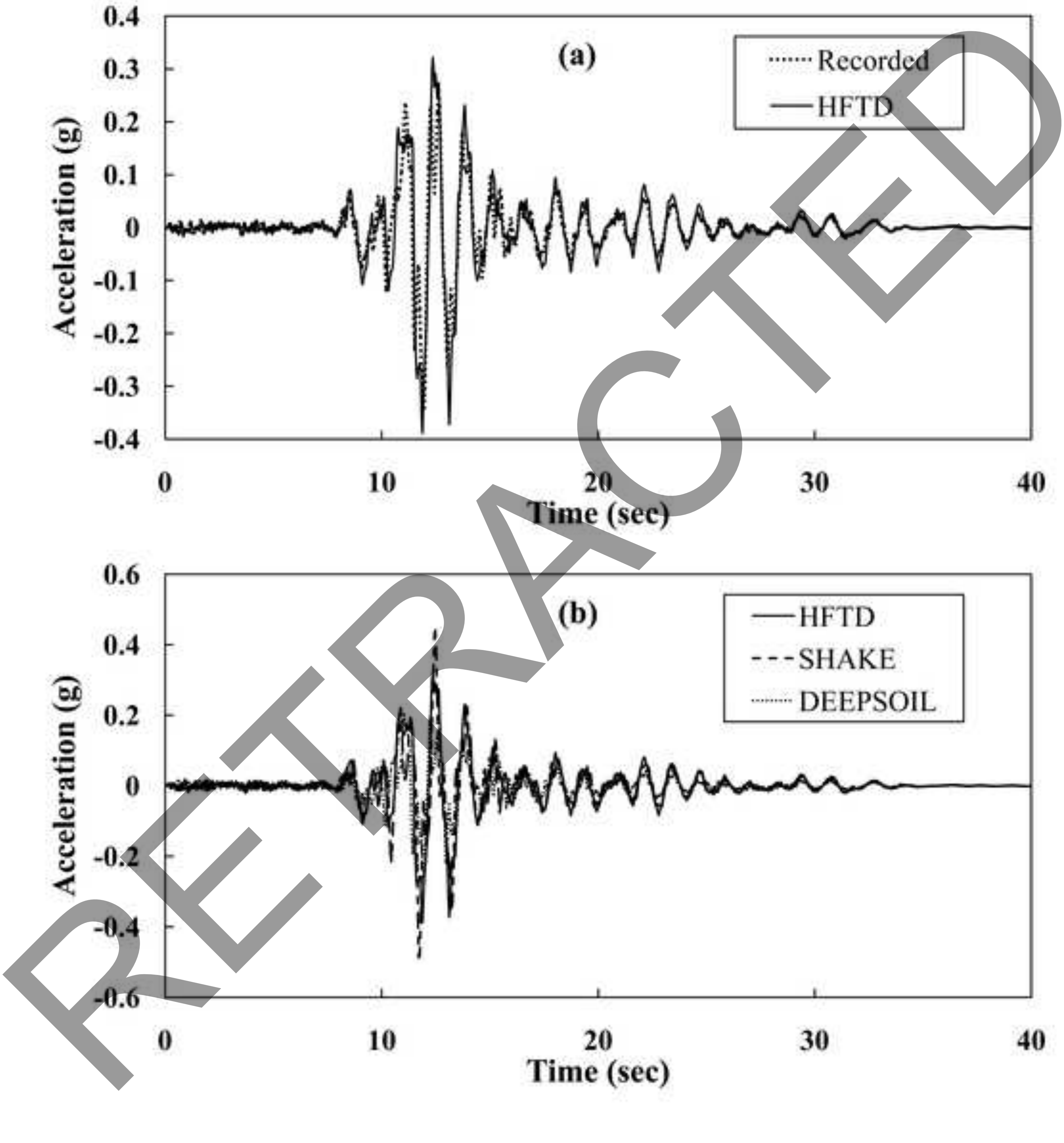

(a)
G/G0
1.2
1
0.8
0.6
0.4
0.2
0
0-12 m
12-24 m
24-39 m
>39 m
1E-40
1E-30
1E-20
1E-10
1E+00
Shear strain (%)
(b)
Damping ratio (%)
30
25
20
15
10
5
0
0-12 m
12-24 m
24-39 m
>39 m
1E-40
1E-30
1E-20
1E-10
1E+00
Shear strain (%)

Figure 3



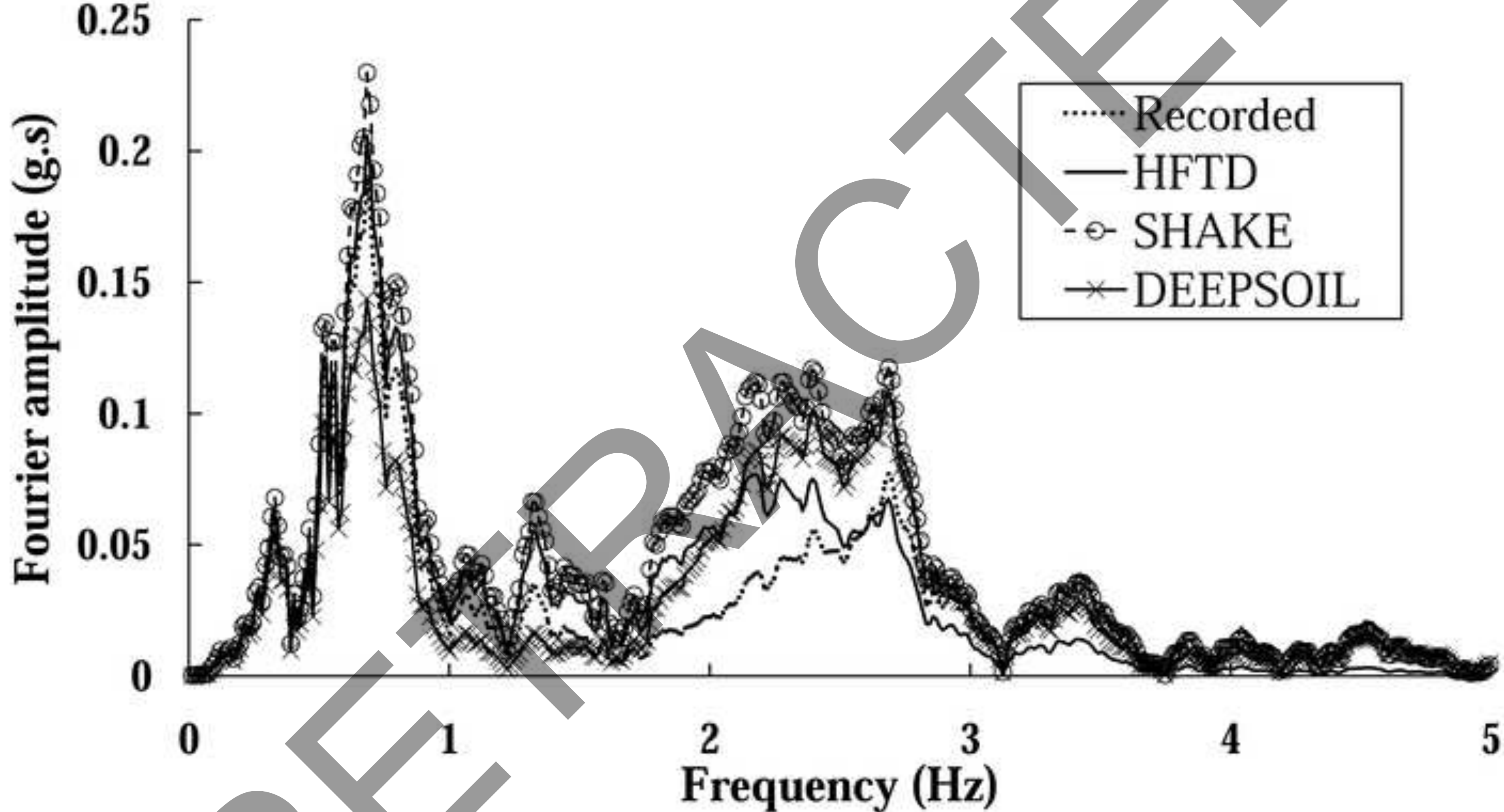
(a)
Acceleration (g)
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40
50
Time (sec)
(b)
Fourier amplitude (g.s)
0.3
0.25
0.2
0.15
0.1
0.05
0
0
5
10
15
20
25
Frequency (Hz)
RETRACTED

Figure 4



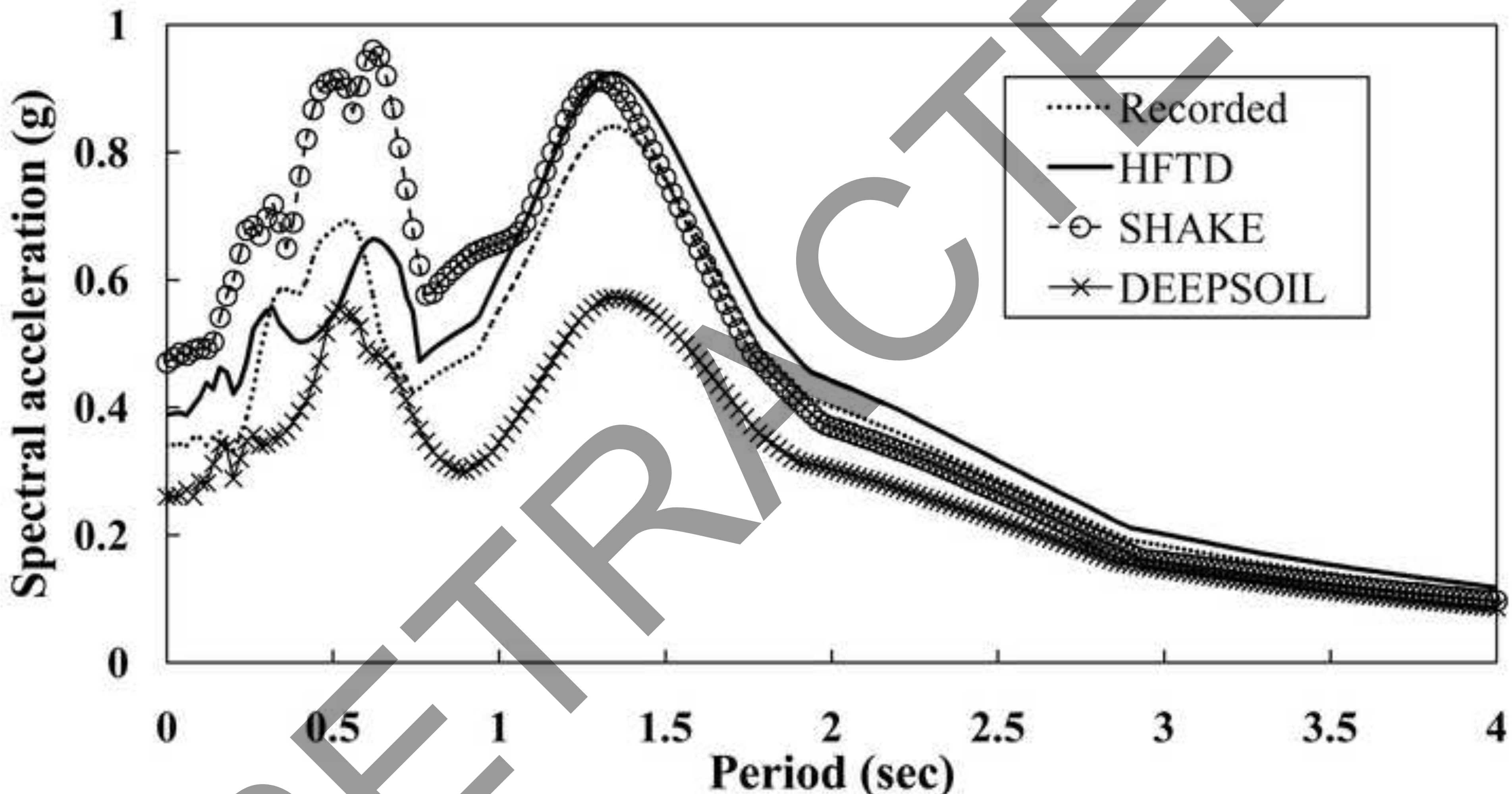
(a)
Recorded
HFTD
Acceleration (g)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
10
20
30
40
Time (sec)
(b)
HFTD
SHAKE
DEEPSOIL
Acceleration (g)
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
0
10
20
30
40
Time (sec)
RETRACTED

Figure 5



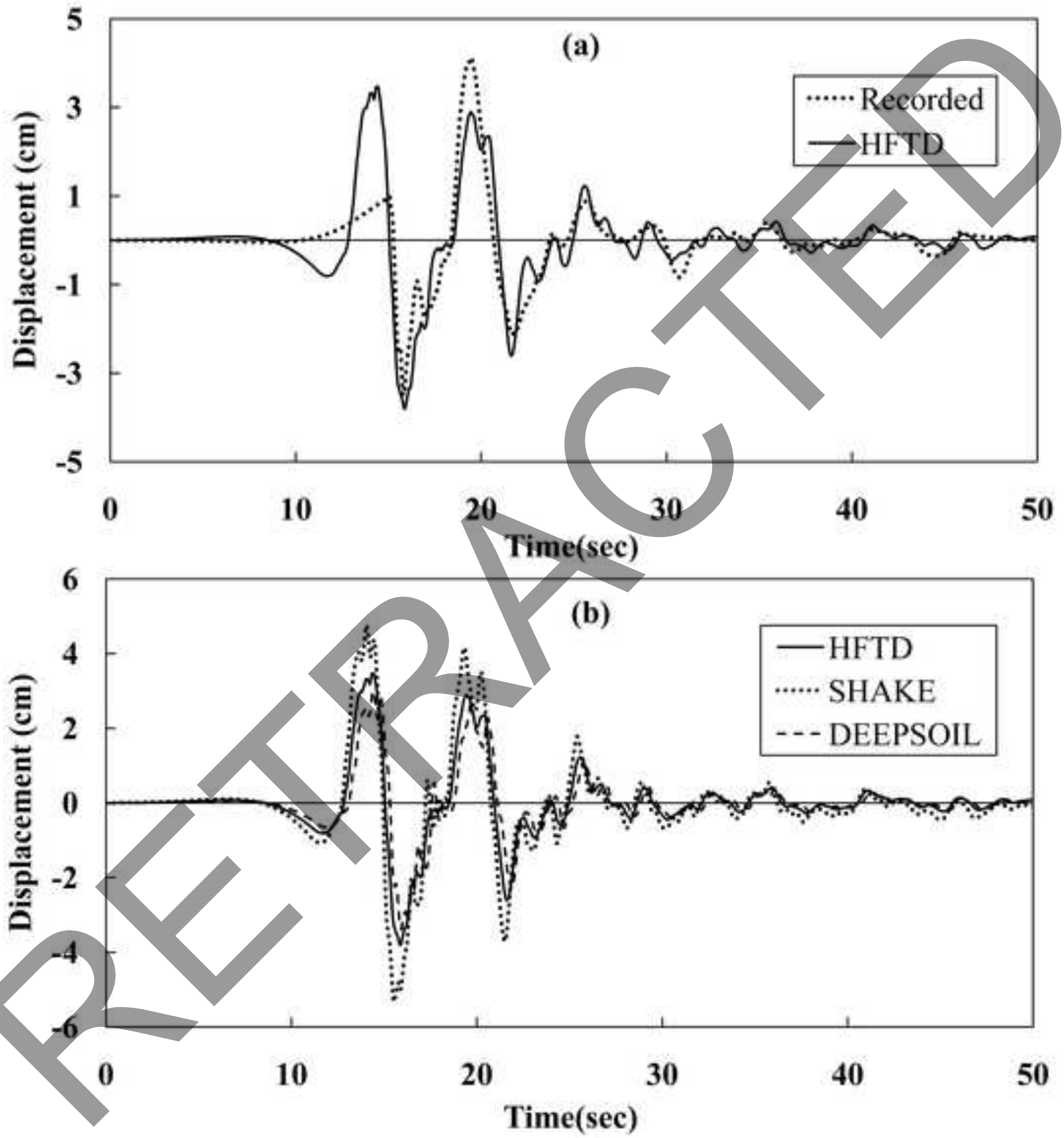
Fourier amplitude (g.s)
0.25
0.2
0.15
0.1
0.05
0
Recorded
HFTD
SHAKE
DEEPSOIL
0
1
2
3
4
5
Frequency (Hz)

Figure 6



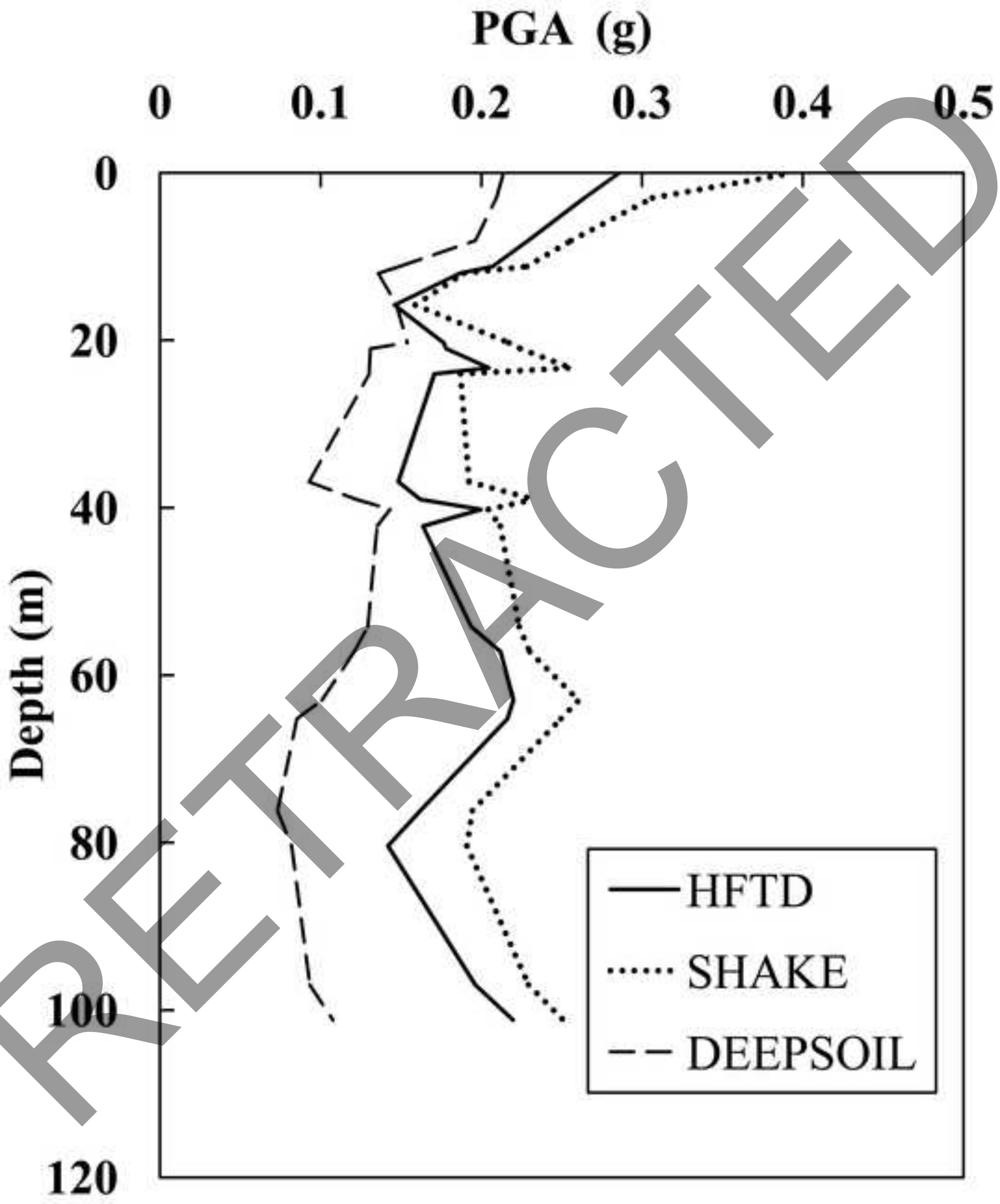
1
0.8
0.6
0.4
0.2
0
Spectral acceleration (g)
0
0.5
1
1.5
2
2.5
3
3.5
4
Period (sec)
Recorded
HFTD
SHAKE
DEEPSOIL

Figure 7



(a)
Recorded
HFTD
Displacement (cm)
Time(sec)
5
3
1
-1
-3
-5
0
10
20
30
40
50
(b)
HFTD
SHAKE
DEEPSOIL
Displacement (cm)
Time(sec)
6
4
2
0
-2
-4
-6
0
10
20
30
40
50
RETRACTED

Figure 8



PGA (g)
0
0.1
0.2
0.3
0.4
0.5
0
20
40
60
80
100
120
Depth (m)
HFTD
SHAKE
DEEPSOIL

Figure 9

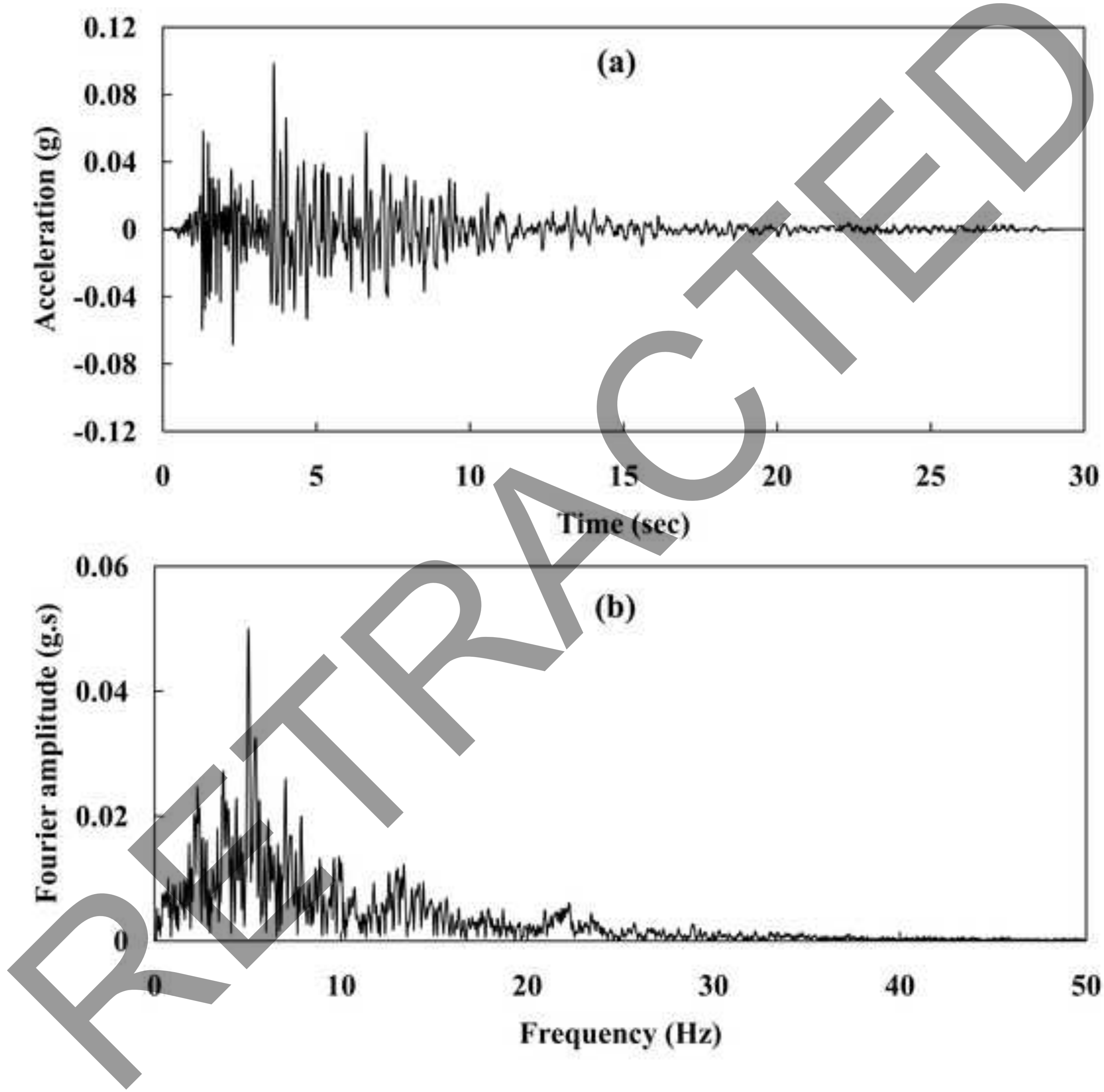

RETRACTED
(a)
Acceleration (g)
0.12
0.08
0.04
0
-0.04
-0.08
-0.12
0
5
10
15
20
25
30
Time (sec)
(b)
Fourier amplitude (g.s)
0.06
0.04
0.02
0
0
10
20
30
40
50
Frequency (Hz)

Figure 10

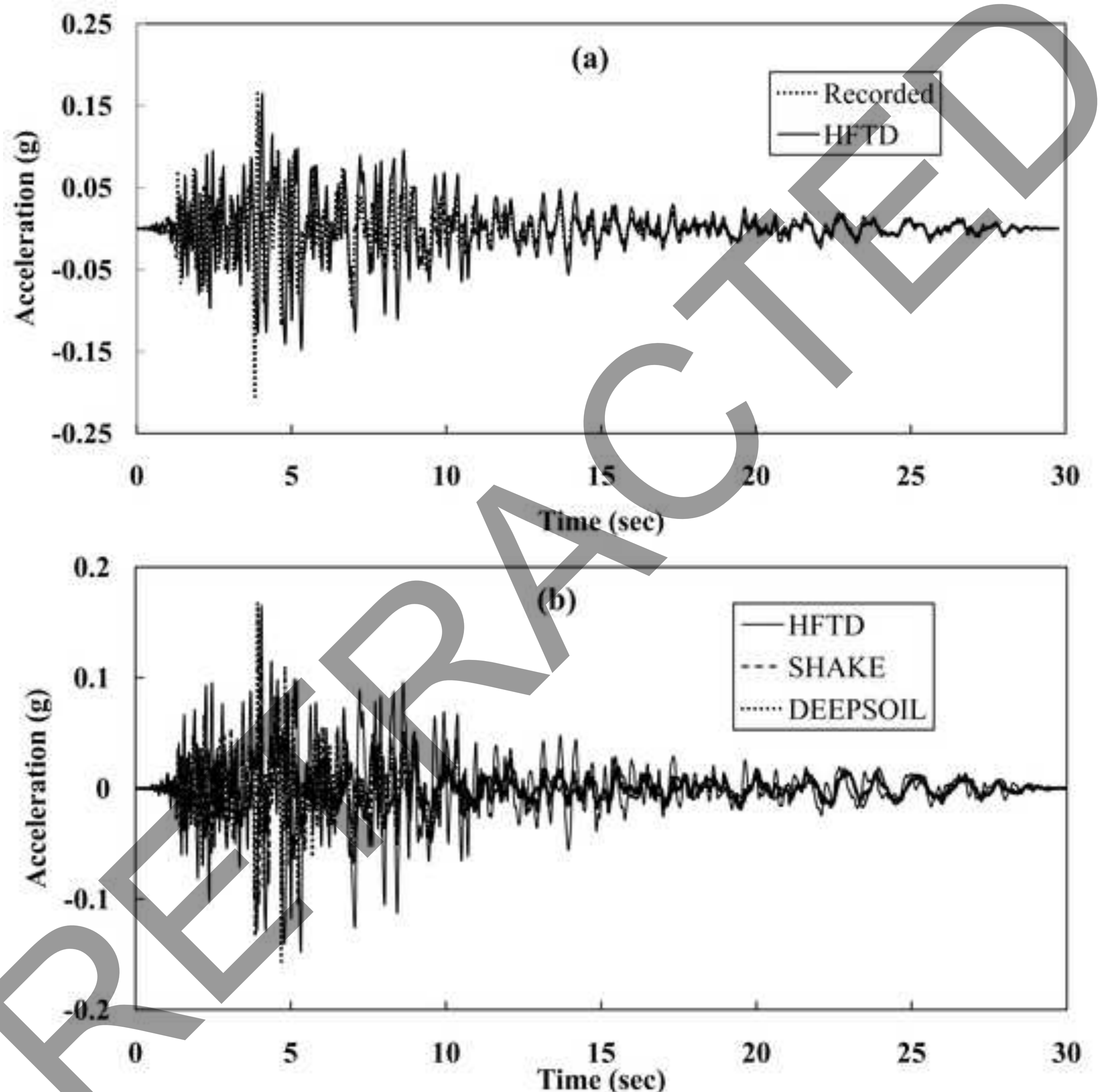
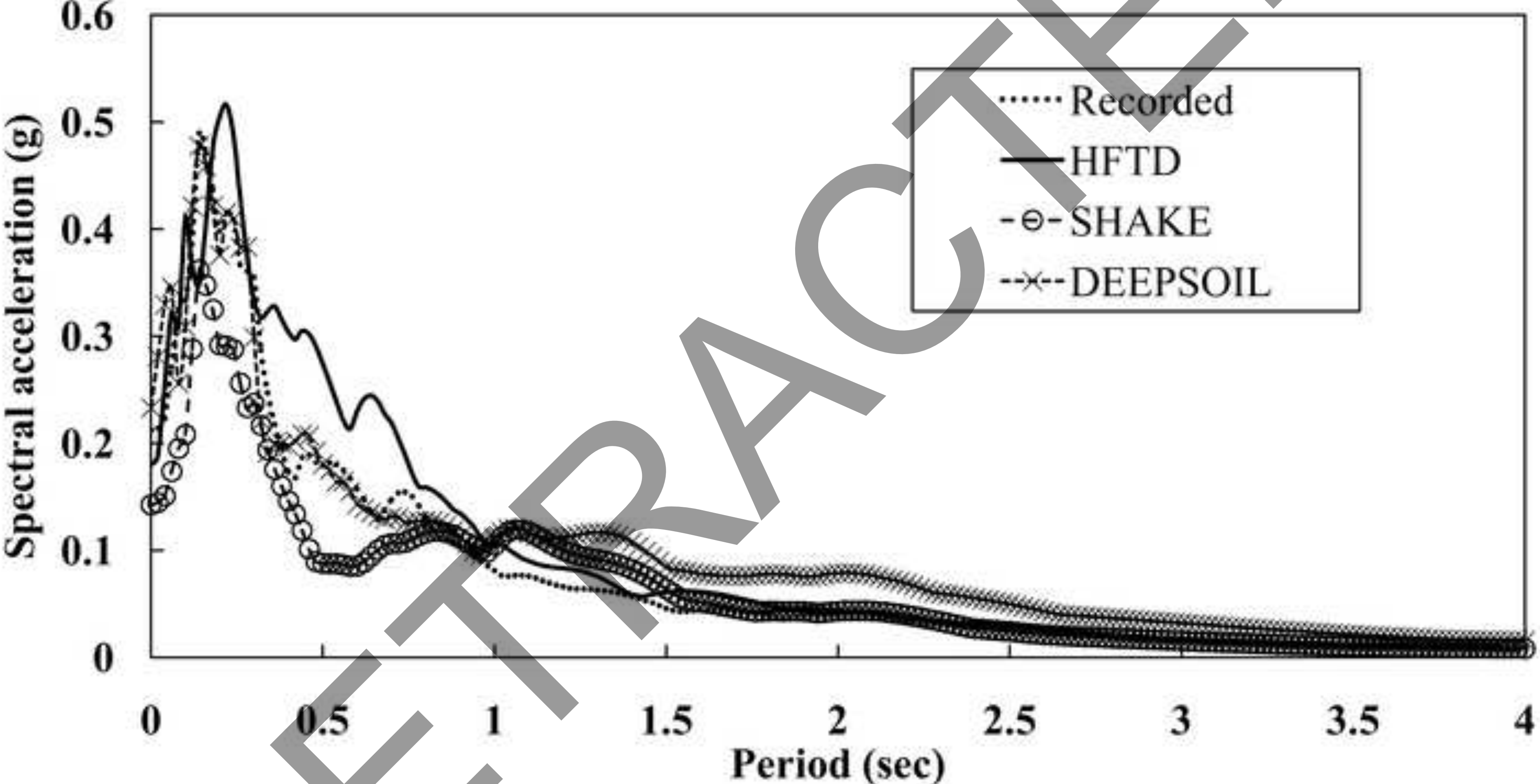
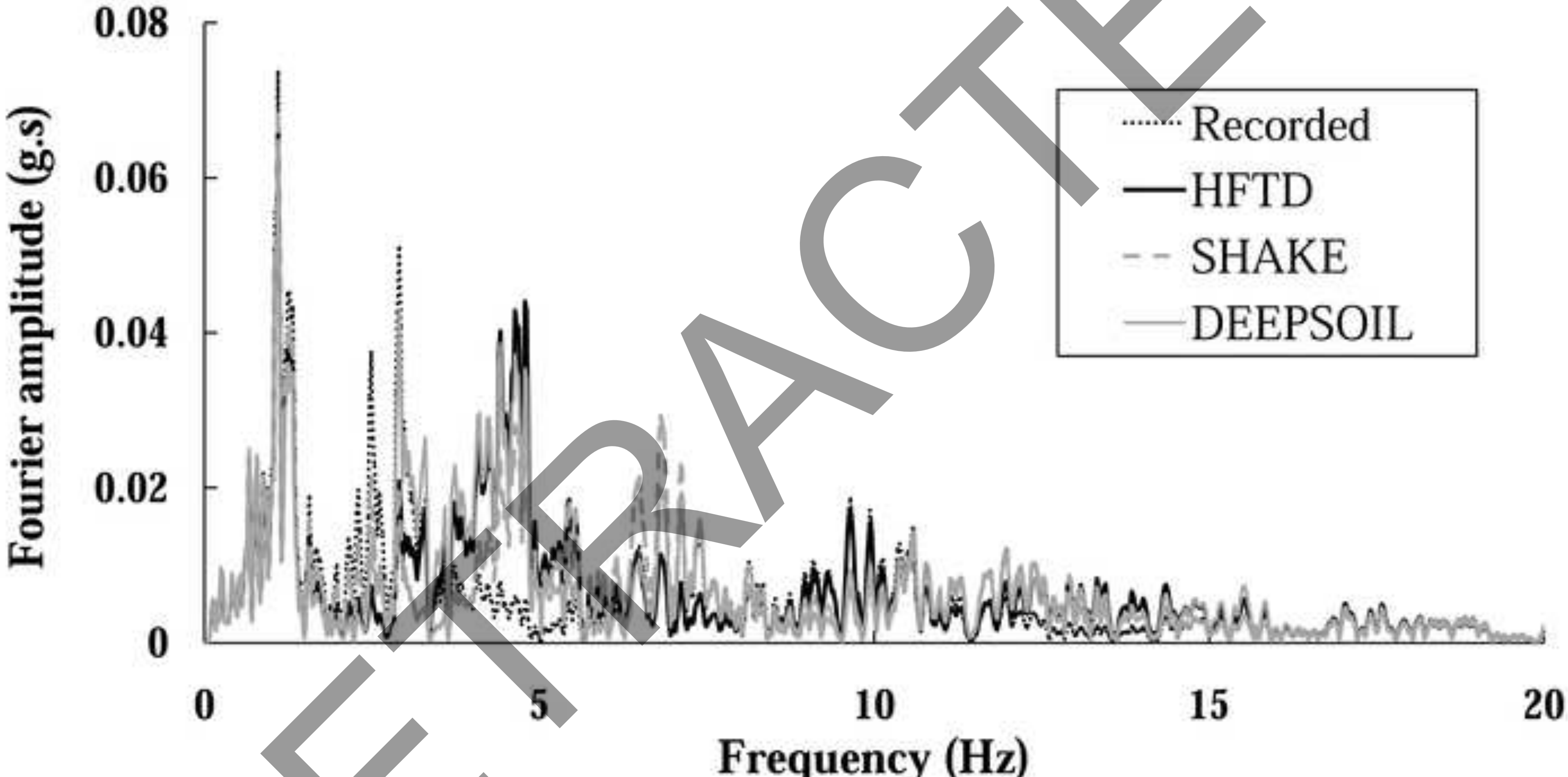

(a)
Recorded
HFTD
Acceleration (g)
0.25
0.15
0.05
-0.05
-0.15
-0.25
0
5
10
15
20
25
30
Time (sec)
(b)
HFTD
SHAKE
DEEPSOIL
Acceleration (g)
0.2
0.1
0
-0.1
-0.2
0
5
10
15
20
25
30
Time (sec)

Figure 11



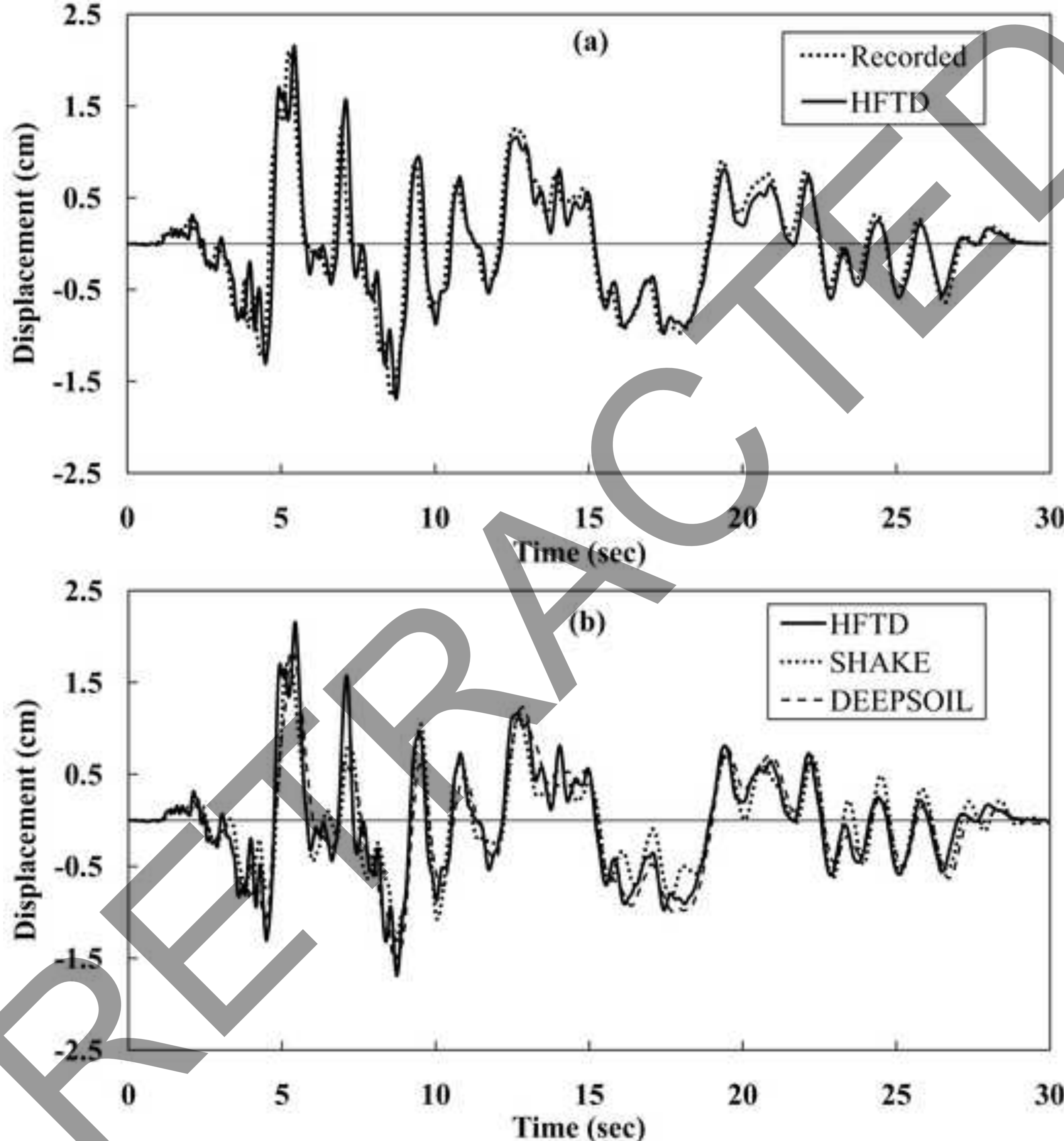
0.6
0.5
0.4
0.3
0.2
0.1
0
Spectral acceleration (g)
0
0.5
1
1.5
2
2.5
3
3.5
4
Period (sec)
Recorded
HFTD
SHAKE
DEEPSOIL

Figure 12



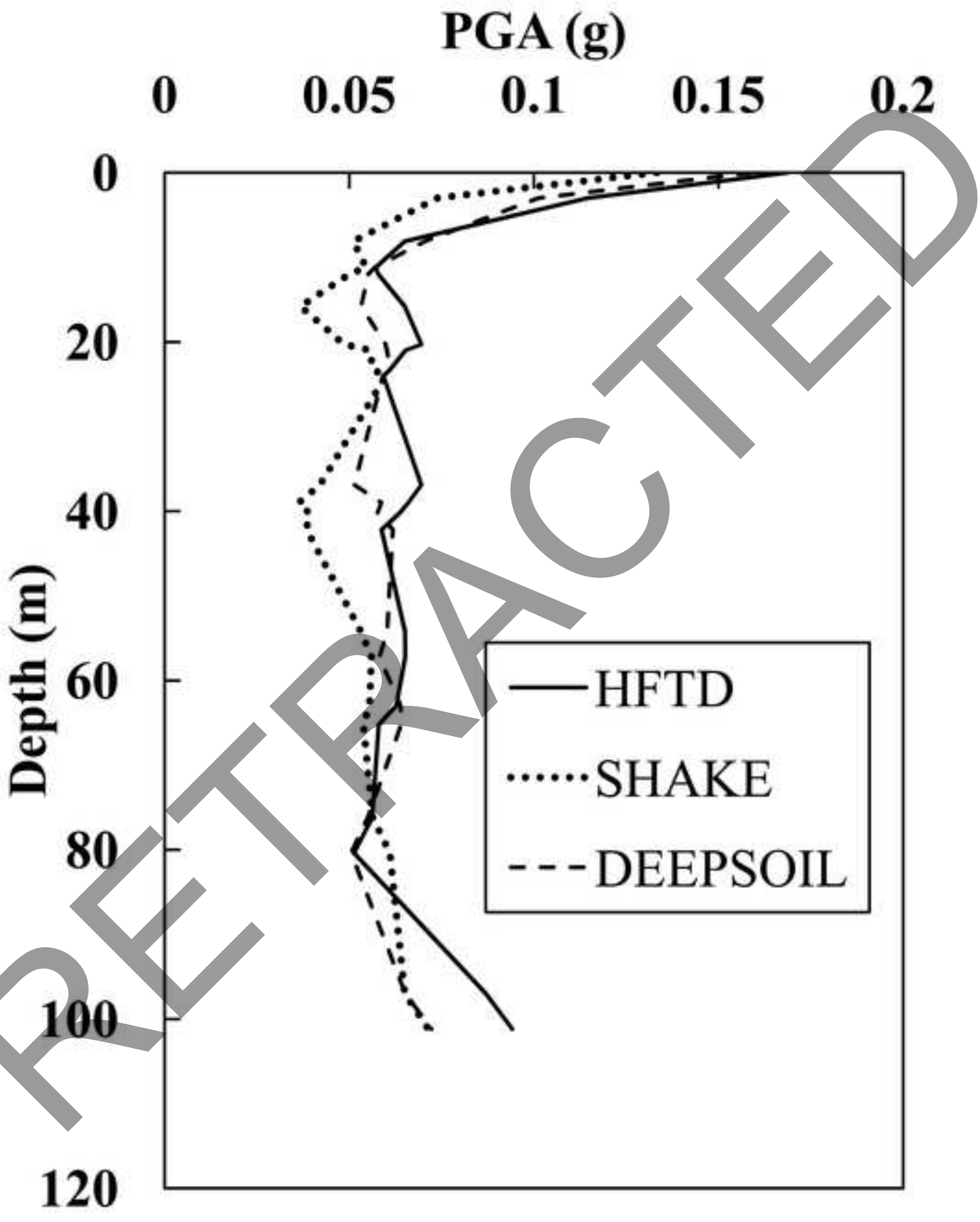
0.08
0.06
0.04
0.02
0
Fourier amplitude (g.s)
0
5
10
15
20
Frequency (Hz)
Recorded
HFTD
SHAKE
DEEPSOIL

Figure 13



(a)
Recorded
HFTD
Displacement (cm)
Time (sec)
(b)
HFTD
SHAKE
DEEPSOIL
Displacement (cm)
Time (sec)

Figure 14



PGA (g)
0
0.05
0.1
0.15
0.2
0
20
40
60
80
100
120
Depth (m)
HFTD
SHAKE
DEEPSOIL
RETRACTED

Artwork captions

Figures

Figure 1. (a) Horizontally stratified soil deposit; (b) lumped parameter equivalent model

Figure 2. (a) Shear modulus degradation; (b) damping ratio curves at Gilroy 2

Figure 3. (a) Acceleration time history; (b) acceleration Fourier spectrum of Loma Prieta earthquake (18/10/1989)

Figure 4. Comparison of acceleration time histories at the ground surface for the Loma Prieta earthquake obtained from HFTD method with (a) recorded data; and (b) with the results of SHAKE and DEEPSOIL computer programs

Figure 5. Acceleration Fourier amplitude of Loma Prieta earthquake on the ground surface

Figure 6. Comparison of the acceleration response spectra for the Loma Prieta earthquake (ξ=10%)

Figure 7. Comparison of time histories of displacement at the ground surface for the Loma Prieta earthquake obtained from HFTD method with (a) recorded data; and (b) with the results of SHAKE and DEEPSOIL computer programs

Figure 8. PGA profile for Loma Prieta earthquake

Figure 9. (a) acceleration time history; (b) acceleration Fourier spectrum of Morgan Hill earthquake (24/4/1984)

Figure 10. Comparison of acceleration time histories at the ground surface for the Morgan Hill earthquake obtained from HFTD method with (a) recorded data; and (b) with the results of SHAKE and DEEPSOIL computer programs

Figure 11. Comparison of the acceleration response spectra for the Morgan Hill earthquake (ξ=10%)

Figure 12. Acceleration Fourier amplitude of Morgan Hill earthquake on the ground surface

Figure 13. Comparison of time histories of displacement at the ground surface for the Morgan Hill earthquake obtained from HFTD method with (a) recorded data; and (b) with the results of SHAKE and DEEPSOIL computer programs

Figure 14. PGA profile for Morgan Hill earthquake

Tables

Table 1. Summary of soil properties at Gilroy 2

Table 2. Summary of parameters for Loma Prieta earthquake

Table 3. Summary of parameters for Morgan Hill earthquake