Effect of Magnitude and Type of Damping on Soil Amplification

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SYNOPSIS: Soil Amplification studies conducted to obtain site specific seismic motions at the free surface of a soil deposit or at any other elevation (convolution process), or to determine compatible base motions at a given depth for soil structure interaction analyses (deconvolution) assume, when performed in the frequency domain simulating nonlinear soil behavior through an iterative linear analysis, that the internal soil damping is of a linear hysteretic nature. This tends to filter out excessively the high frequency components of motion for convolution studies and leads to eventual instability of the solution at a given depth (function of the soil properties) when performing deconvolution. In this paper, the results obtained using constant frequency independent, linear proportional and inverse proportional damping in the iterative solution are compared to those provided by true nonlinear analyses using consistent soil models.

INTRODUCTION

It has long been recognized that, in order to study the propagation of seismic waves through a soil deposit for moderate or large earthquakes, it is necessary to account, at least approximately, for nonlinear soil behavior. Two procedures can be used for this purpose:

a) Iterative linear analyses making use of equivalent linearization techniques. After each analysis, maximum strains are computed at representative points for each soil sublayer, finite element or discrete spring. If the solution is carried out in the frequency domain, as is normally the case, this implies conversion to the time domain to obtain time histories of strains and scanning for the maximum value in each time record. From experimental curves relating shear modulus and damping to shear strain, like those suggested by Seed and Idriss (1970) or later improved versions, values of these two parameter can be obtained corresponding to a characteristic shear strain. For steady state harmonic excitation and response at a single frequency the characteristic strain is the maximum computed strain (the strain amplitude). For transient responses, as in the case of earthquake motions, the characteristic strain is typically taken as two thirds of the maximum. A new analysis is then performed using the soil properties so determined. The process is continued until the values of the strains, or the soil properties, computed in two consecutive cycles differ by less than a specified tolerance (typically 5 or 10%). This procedure is implemented in computer programs such as SHAKE (Schnabel et al, 1972), LUSH (Lysmer et al, 1974), or FLUSH (Lysmer et al, 1975), which have been extensively used in practice. SHAKE uses a continuum solution, like that presented by Roesset and Whitman (1969), while LUSH and FLUSH use a discrete finite element model.

b) Nonlinear analyses in the time domain, using an appropriate set of constitutive equations for the soil. This alternative is implemented in the programs CHARSOIL modelling the soil with a Ramberg Osgood model and carrying out the solution with the method of characteristics (Richart and Wylie, 1977, Streeter et al 1974), and STEALTH (Hofman, 1976) using an explicit finite difference code. A similar solution can be obtained modelling the soil with finite elements or for the simple case of vertically opagating waves a system of lumped masses and interconnecting nonlinear springs (close coupled shear beam model).

A question has been repeatedly raised, and conveniently ignored, as to the validity of the iterative procedure and the accuracy of its results. The first comprehensive evaluation of the equivalent linear solution was performed by Constantopoulos (1973) using a Ramberg Osgood model for the soil and curves of modulus and damping versus level of strain corresponding to the same model. Constantopoulos compared the results obtained using true nonlinear analyses with those of the iterative approach assuming frequency independent (linear hysteretic) and stiffness proportional (linear viscous) damping, and concluded that out of these two models the linear hysteretic one was by far the best. His studies indicated that with this approach the iterative scheme tended to overestimate the peak ground acceleration at the free surface by 20% or so while displacements and strains could be underestimated by 50% and were much less reliable. They were limited, however, to two relatively shallow soil deposits (100 ft deep) with maximum base acceleration of 0.35 g. Richart (1977) compared results obtained with SHAKE with those provided by CHARSOIL and found that the response spectra of the surface motions computed with the former were much lower than those derived with the latter in the high frequency range. A report by D'Appolonia (1979) showed similar results and concluded that the iterative procedure as implemented in SHAKE is applicable for relatively shallow soil deposits and small earthquake excitations but it will yield unreliable results for deep profiles and high intensity motions. A similar conclusion was reached in a report by Dames and Moore (1978), comparing SHAKE and STEALTH, but on the basis of apparently opposite findings. The results of this study indicated that for deep profiles and high levels of shaking the iterative solution overestimated the maximum surface acceleration by a factor of 2 or more.

A second point of concern is the application of the iterative procedure to the deconvolution process. Figure 1 shows typical amplification curves representing the amplitude of the transfer functions from the bottom to the surface of a homogeneous soil layer with linear hysteretic damping.

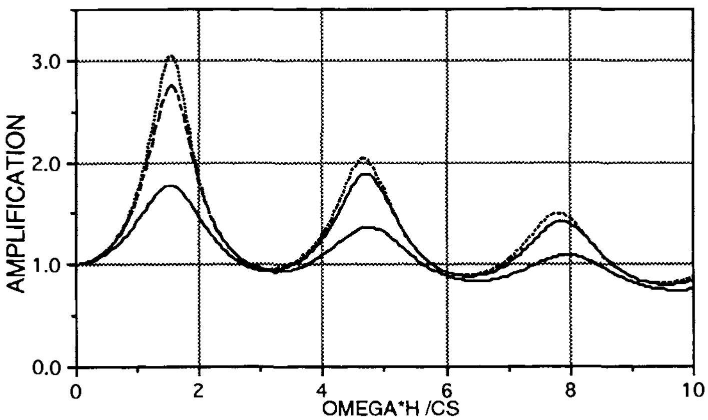

Figure 1. Amplification functions for SH waves at various angles rock outcrop to soil surface.

Figures 2 and 3 show the amplitudes of the transfer functions from the surface to the bottom for the same profile. They are simply the inverse. The amplitude of the latter increase without bound for increasing values of the parameter fh/c_s where f is the frequency in Hz, h the depth and c_s the shear wave velocity of the soil. For a system with frequency independent damping as the deconvolution process proceeds down the soil profile, layer by layer, the amplitudes of the high frequencies increase continuously and will eventually cause numerical problems. When using this procedure in practice it is necessary at times to suppress from the surface motions any components with frequencies above 8 or 10 Hz, an adjustment which is illogical and inconsistent with other requirements in seismic regulations. These problems with high frequencies do not occur when performing actual nonlinear analyses.

The objective of this paper is to illustrate some of the limitations involved in dynamic analyses with linear hysteretic damping, particularly in the context of soil amplification studies. To illustrate the nature of the problem and the approximations introduced by the iterative procedure with frequency independent damping a soil deposit subjected to vertically propagating shear (SH) waves will be considered and modelled as a close coupled multidegree of freedom system. Analysis will be carried out both in the time domain with nonlinear springs and in the frequency domain using the iterative linear approach. In the second case results will be obtained assuming that the damping is inversely proportional to frequency, frequency independent (linear hysteretic) or

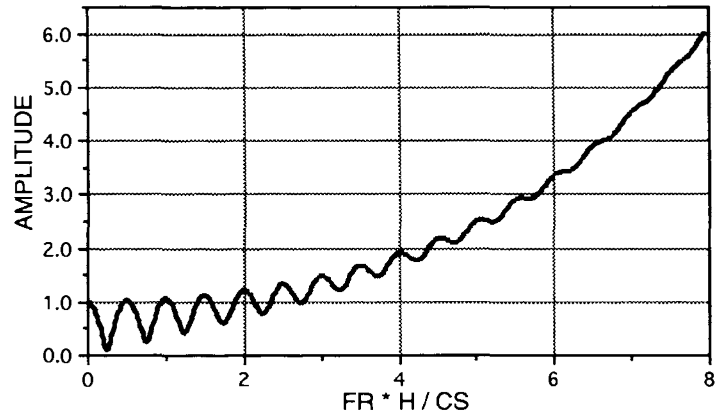

Figure 2. Variation of amplitude with depth. Decomvolution. Damping = 0.05

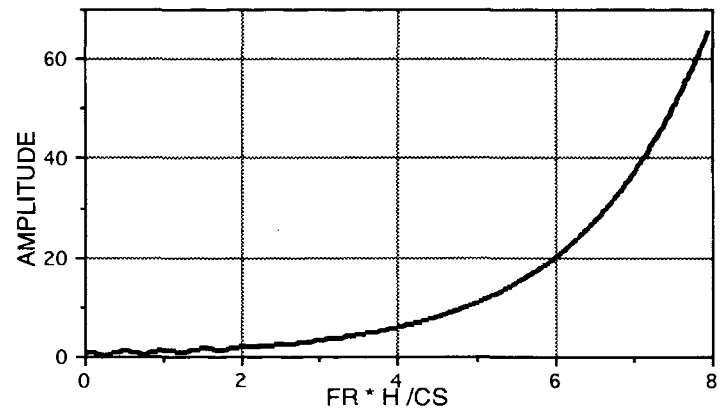

Figure 3. Variation of amplitude with depth. Decomvolution. Damping = 0.10.

stiffness proportional (linear viscous damping proportional to frequency). The same discrete model will be used for the four sets of analyses and the curves relating the variation of the stiffness (or shear modulus) and the damping to the level of shear strain for the last three sets (iterative analyses) will be those corresponding to the nonlinear springs used in the time domain solution.

FORMULATION

A uniform soil profile with a depth of 100 ft, a shear wave velocity of 800 ft/sec and a unit weight of 199 lbs/cu.ft was used for the first series of studies. The soil layer was subdivided into 10 sublayers and each one of these was represented by a nonlinear shear spring. The corresponding multidegree of freedom system had thus 10 masses and 10 springs. The top mass was equal to 18.5 lbs x sec^2/ft and the others were 37. Each spring had an initial stiffness for very low levels of strain of 7.616 x 10^6lbs.

The variation of the shear modulus of the soil with shear strain was given by the values of G/G_{max} versus γ listed in table 1, where G_{max} is the initial shear modulus of 76.16 x 10^6 lbs/sq.ft. These values correspond to a real soil tested at the Geotechnical Center of the University of Texas at Austin. There are also, of course, the ratios of the spring stiffnesses to their initial values.

Table 1. Variation of shear modulus and damping with level of strain.

$\gamma \times 10^6$	G/G_{max}	D
5.35	1.000	0.0
10.35	0.991	0.003
19.42	0.973	0.007
40.88	0.925	0.017
99.33	0.824	0.035
170.70	0.727	0.063
304.20	0.623	0.083
569.20	0.505	0.109
1201.00	0.371	0.142

For the analyses in the time domain each nonlinear spring was modelled by a set of 9 elastic-perfectly plastic springs in parallel, selected so as to provide the same variation of the stiffness (shear modulus) with level of strain. Figure 4 shows typical hysteresis loops for a resulting nonlinear spring under harmonic excitation. The value of damping associated to these hysteresis loops are also listed in table 1. These are slightly different from those that had been obtained experimentally: when fitting the variation of the shear modulus obtained in laboratory tests with a multilinear spring which satisfies Masing's law it is often found that the resulting value of damping will not match exactly the measured data.

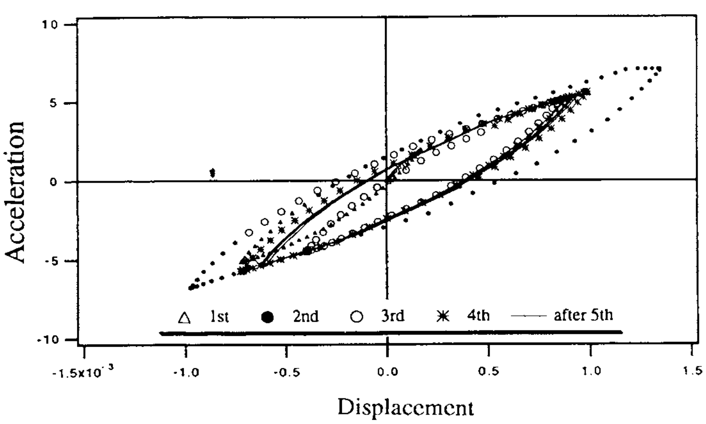

Figure 4: Hysteretic loops for material model.

The solution in the time domain was carried out using a step by step numerical integration of the equations of motion with the central difference formula. The iterative analyses were carried out in the frequency domain. Starting in each case with the initial material properties corresponding to very low levels of strain a complete solution was obtained. The time histories of the deformations of each spring were computed using the Fast Fourier transform. For harmonic steady state excitation the amplitude of the deformation, once a steady state response had been reached, was used as characteristic strain. For transient analyses using an earthquake record the characteristic strain was selected as two thirds of the maximum. The values of the secant modulus and damping corresponding to the characteristic strain were then computed for each nonlinear spring from the set of 9 elasto-plastic springs in parallel. The damping was introduced through the use of a complex modulus of the form G(1+2id) or a complex stiffness

$$k = G(1+2iD) / h \qquad (1)$$

where h is the thickness of the soil sublayer. New analyses were then conducted and the iterations were continued until the maximum strains in all the springs differed by less than 5% in two consecutive cycles. To simulate damping inversely proportional to frequency the stiffnesses were computed as

$$k= G(1+2iD\omega / \Omega) / h \qquad (2)$$

and for a stiffness proportional damping, increasing linearly with frequency

$$k= G(1 + 2id\Omega / \omega) / h \qquad (3)$$

In these two expressions, Ω would be the frequency of vibration whereas ω is a reference frequency. Initially ω was selected as the fundamental natural frequency of the soil deposit for very low levels of strain. In a second series of studies ω was selected as the fundamental natural frequency of the soil deposit corresponding to the levels of strain (and associated values of stiffnesses) obtained at the end of the previous cycle. In all cases at the value of the frequency Ω equal to the reference frequency all three models would produce the same damping.

RESULTS

Figure 5 shows the acceleration time histories of the input base motion with a peak ground acceleration of 1 ft/sec^2, and the corresponding response accelerations of the top mass (motion at the free surface of the soil deposit) obtained with the nonlinear soil model and the three different versions of the iterative linear analyses (with damping inversely proportional to frequency, with constant damping independent of frequency and with damping increasing linearly with frequency). In all cases, the damping is defined at the original natural frequency of the soil deposit (which was 2 Hz in this case). The results obtained defining the damping at the effective natural frequency of the soil deposit accounting for the variation in shear moduli due nonlinear behavior did not present any significant differences and are therefore not shown. The maximum surface acceleration obtained with the nonlinear model is of the order of 1.8 ft/sec^2 indicating an amplification of the peak ground acceleration by a factor of 1.8. The iterative solution with inversely proportional damping yields a peak surface acceleration of 2.4 ft/sec^2 which is roughly 33% higher than the nonlinear solution. The linear hysteretic, frequency independent, damping results in a peak surface acceleration of 1.6 ft/sec^2 only 11% smaller than the prediction of the nonlinear analysis while the linear proportional damping yields much smaller accelerations with a maximum value of about 1.1 ft/sec^2. It is also clearly noticeable that the linear viscous damping model, increasing with frequency, shows a time history of accelerations with a much lower high frequency content than any of the other solutions. From this point of view, the response provided by the inversely proportional damping is the one most similar to the nonlinear solution. This point is further illustrated in Figure 6, which shows the amplitude Fourier spectra of the five time histories of Figure 5 (the input earthquake at the base and the surface accelerations predicted by the four models considered). It is clearly seen that the linear iterative solution with inversely proportional damping overestimates the amplitudes of the response over most of the frequency range. The constant, hysteretic, damping overestimates the amplitude of the first peak (at around 2 Hz), closely predicts the 2nd peak (at about 6 Hz) but underpredicts all the following ones, filtering out excessively the high frequency components. The linear viscous damping overestimates the amplitude of the first peak, severely underestimates that of the second and filters out entirely all the following ones.

The corresponding results for the same soil layer (100 ft depth, 2 Hz initial fundamental frequency) and the same earthquake scales up to a peak acceleration of 3 ft/sec^2 are shown in Figures 7 and 8. Figure 7 shows the time histories of the accelerations. The peak surface acceleration from the nonlinear analysis is approximately 4ft/sec^2 indicating an amplification of this parameter of 1.33. The corresponding values using inversely proportional damping, constant hysteretic damping, and linear proportional, viscous damping are 4.9 ft/sec^2, 3.2 ft/sec^2 and 2.2 ft/sec^2. Again, the first model overestimates the peak acceleration but only by about 20% now. The linear hysteretic damping model underestimates it by 20%, while the viscous damping model underestimates it very badly. Again the differences in the frequency contents of the four motions are clearly apparent with the inversely proportional damping yielding more similar results to those of the nonlinear analysis as far as high frequency content is involved. Figure 8 shows the amplitude Fourier spectra of the motions. The first iterative model overpredicts the amplitudes of most of the peaks; the constant damping model (frequency independent damping) overpredicts the amplitude of the first peak, underpredicts substantially the second (at around 5.5 Hz) and has almost no other peaks. The last model has essentially one peak, the one corresponding to the fundamental frequency of the soil and filters out almost entirely all the other frequencies.

Figures 9 and 10 show the corresponding results when the base motion is scaled up to a peak acceleration of 9 ft/sec^2. The peak acceleration at the free surface of the soil deposit using the nonlinear model is now about 7 ft/sec^2, smaller than the base acceleration. The results for the three linearized analyses are 10 ft/sec^2, 7 ft/sec^2 and 4 ft/sec^2 respectively. The first model overestimates again the response (by 45%) and the third one underestimates it badly. The constant (frequency independent) damping model predicts almost exactly the peak acceleration. The frequency content of the resulting motion is, however, smaller than the true one for frequencies above 5 Hz.

Figures 11 and 12 show the corresponding results for a soil profile with the same properties but a depth of 200 ft and therefore an initial fundamental frequency of 1 Hz. The input

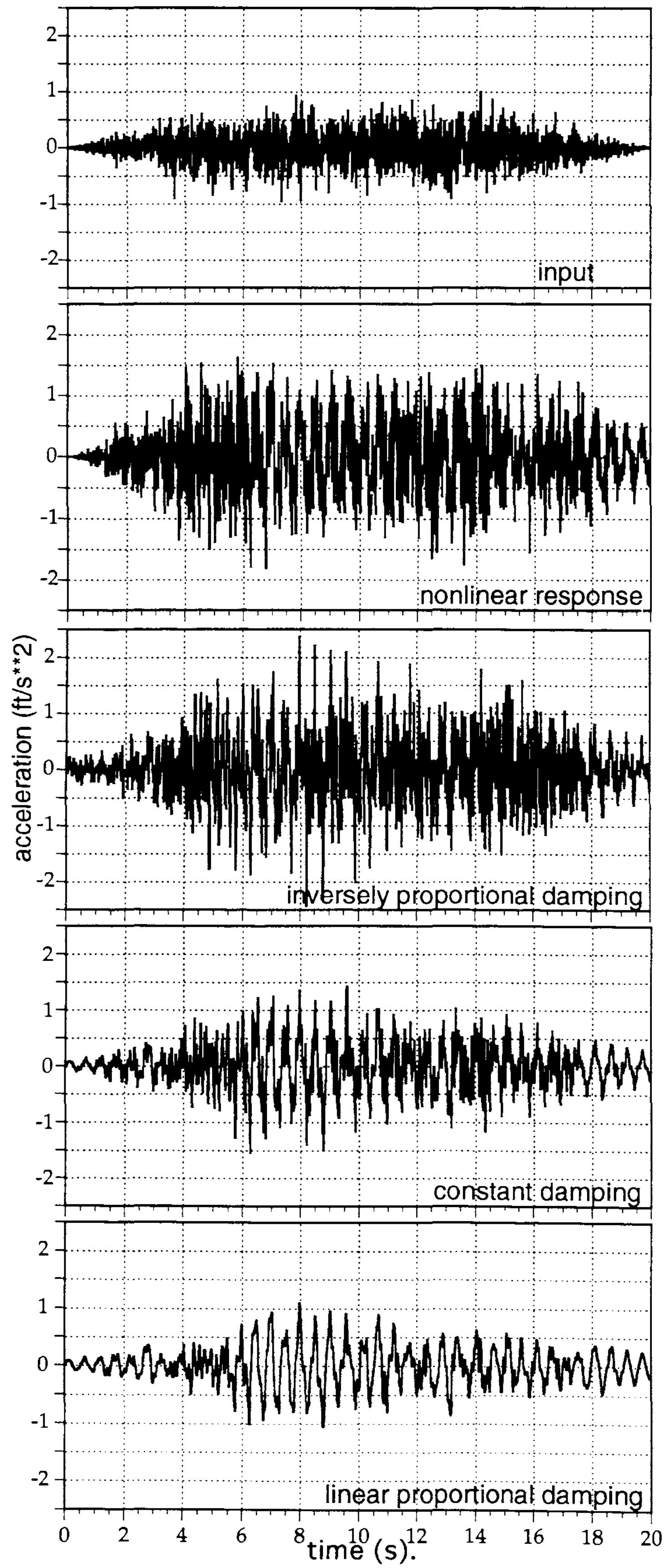

Figure 5 . Acceleration time histories.

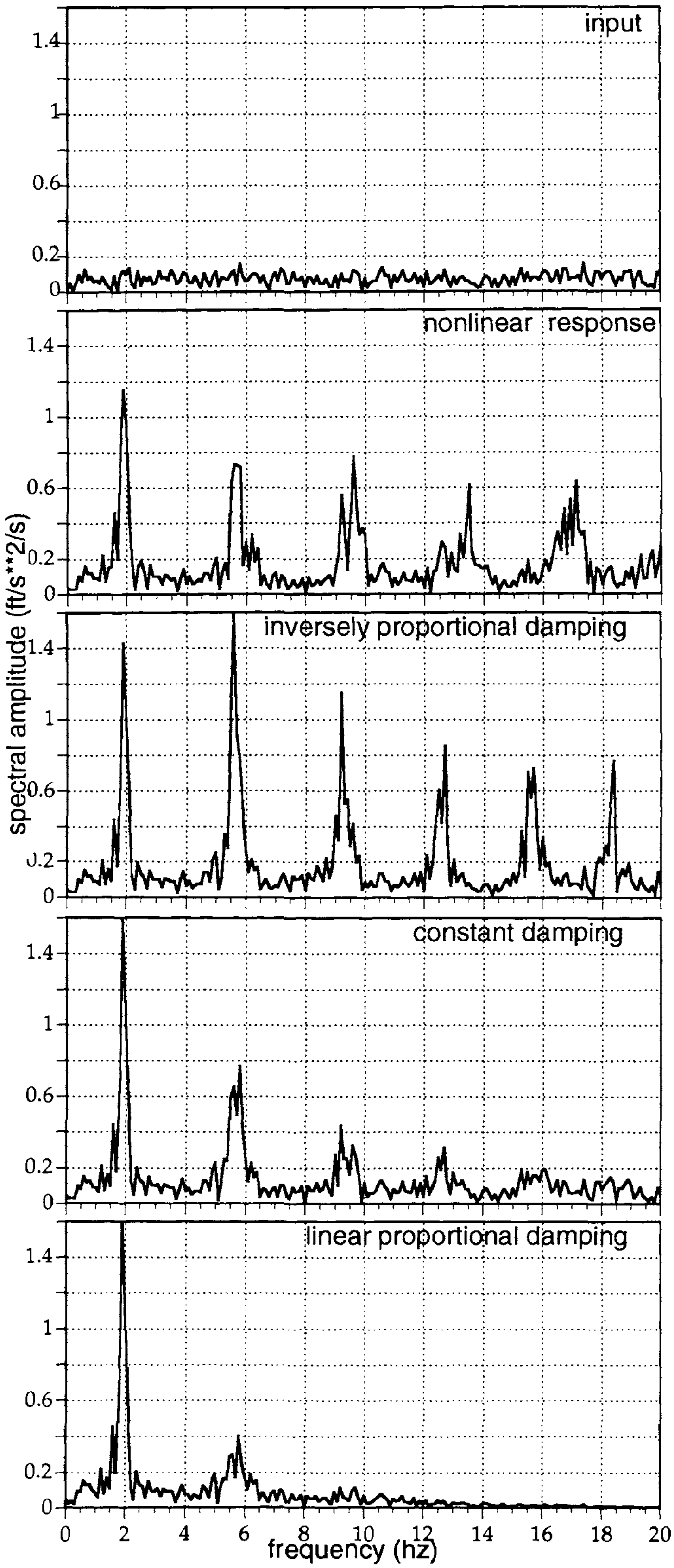

Figure 6 . Amplitude Fourier spectras.

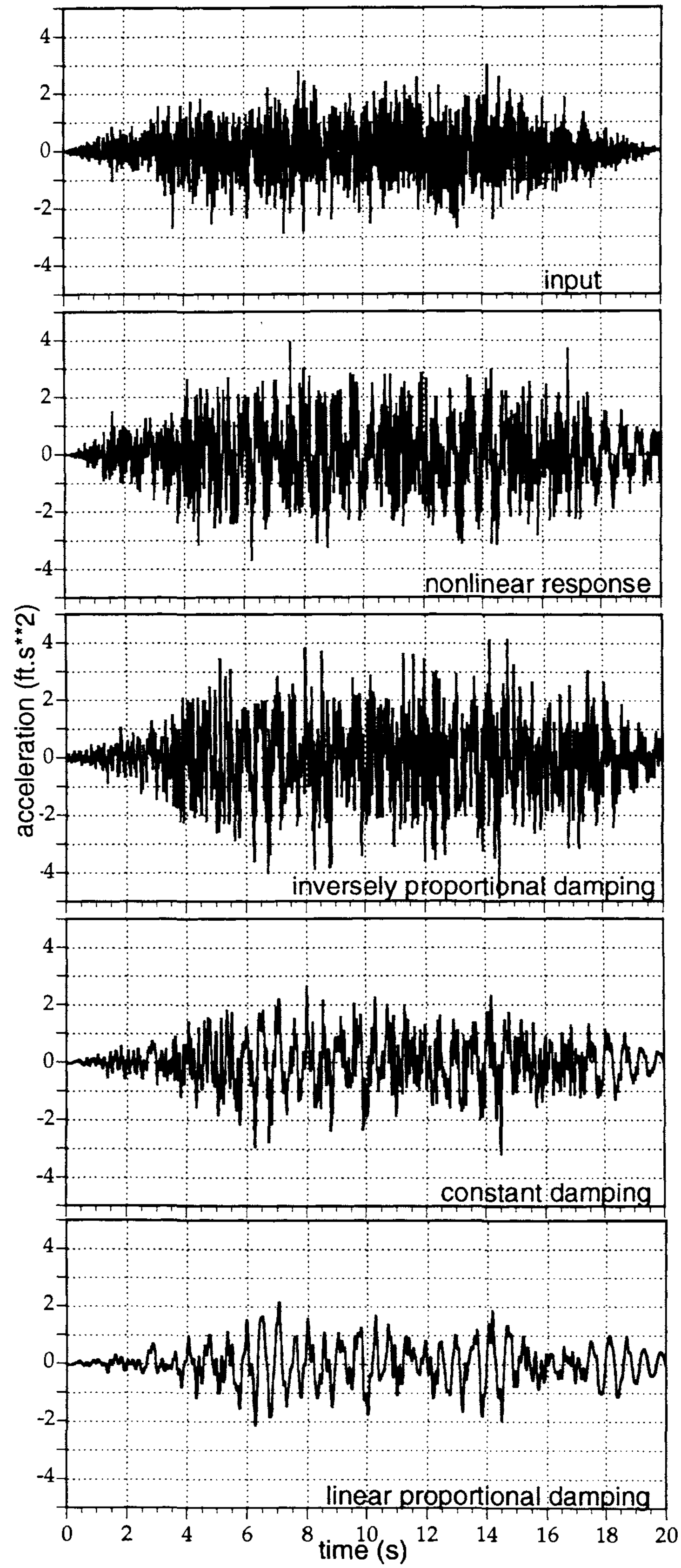

Figure 7 . Acceleration time histories.

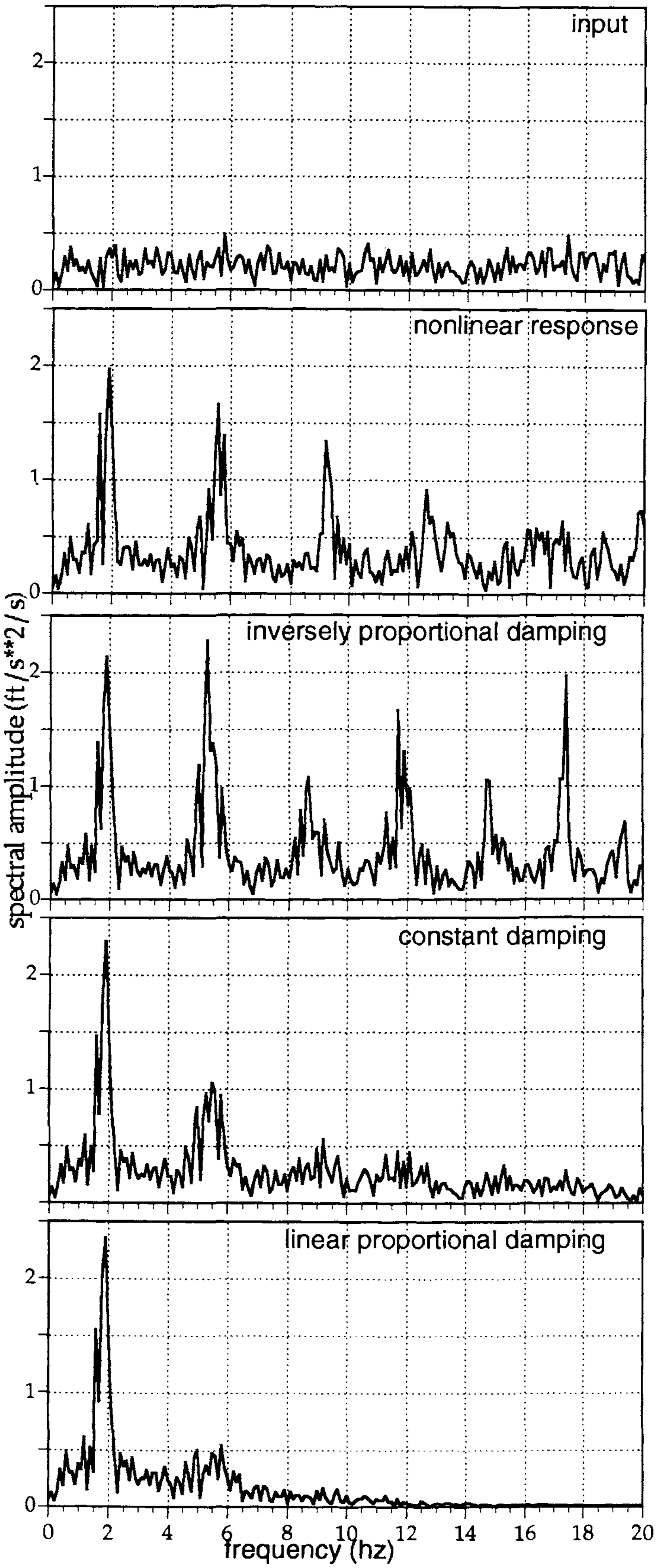

Figure 8. Amplitude Fourier spectras.

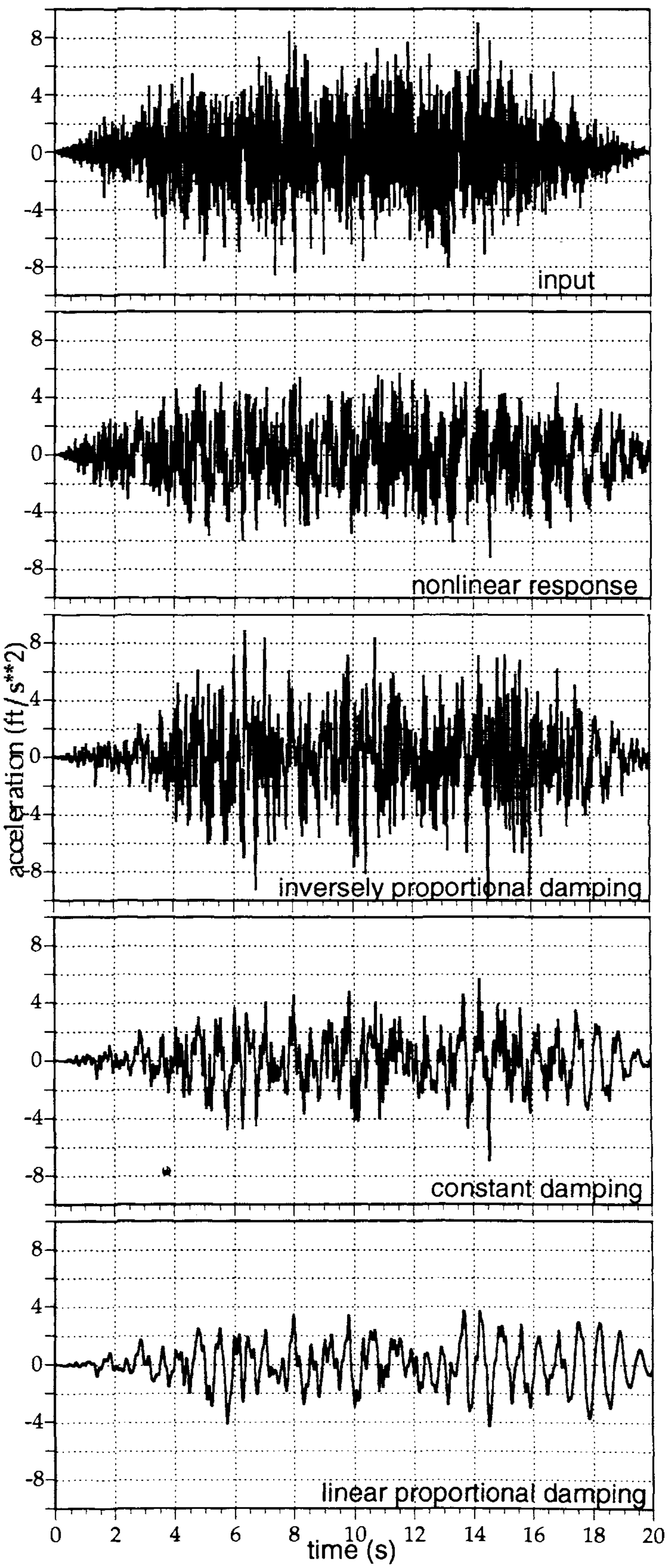

Figure 9. Acceleration time histories.

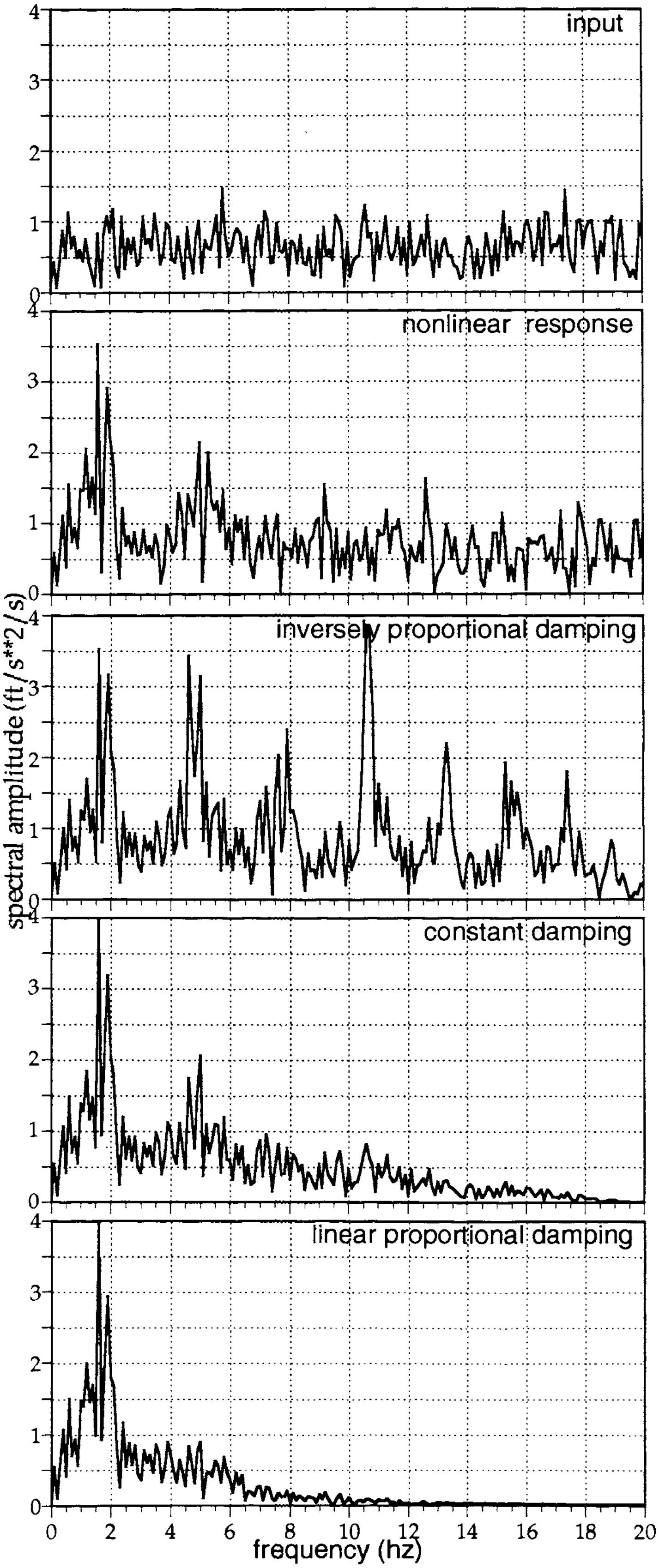

Figure 10 Amplitude Fourier spaectras.

motion at the base is the same earthquake with a peak acceleration of 9 ft/sec^2. The peak accelerations at the free surface are 3.4, 4.6, 3.2, and 3 ft/sec^2 respectively. The differences in the frequency content of the motions are even more pronounced in this case.

To further illustrate the practical significance of the frequency content, Figure 13 shows the 5% response spectra for the motions corresponding to the four models, the soil profile 200 ft deep, and the input motion with a peak acceleration of 9 ft/sec^2. It can be clearly seen that for systems with natural frequencies above 2 or 3 Hz the response to the motions computed with the nonlinear model are larger than those obtained with the iterative analysis and constant or linear proportional damping.

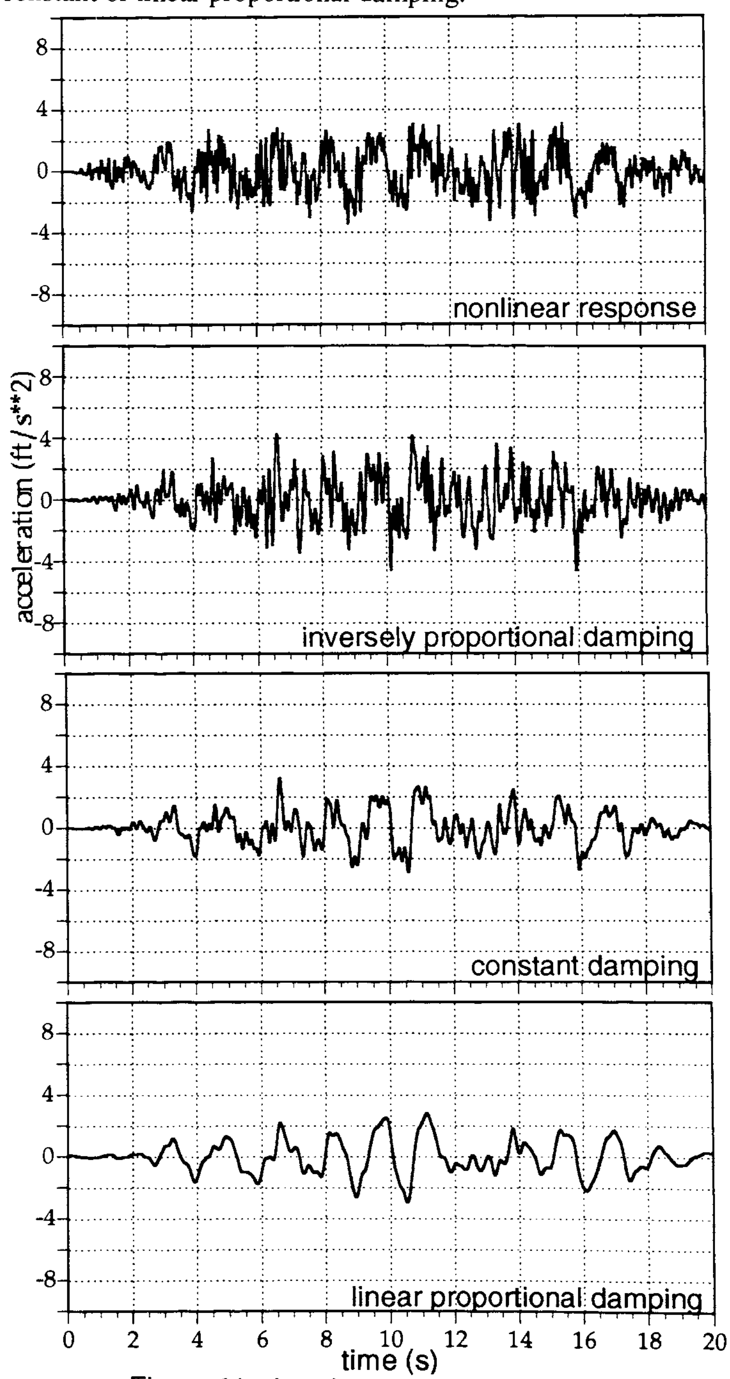

Figure 11 Acceleration time histories

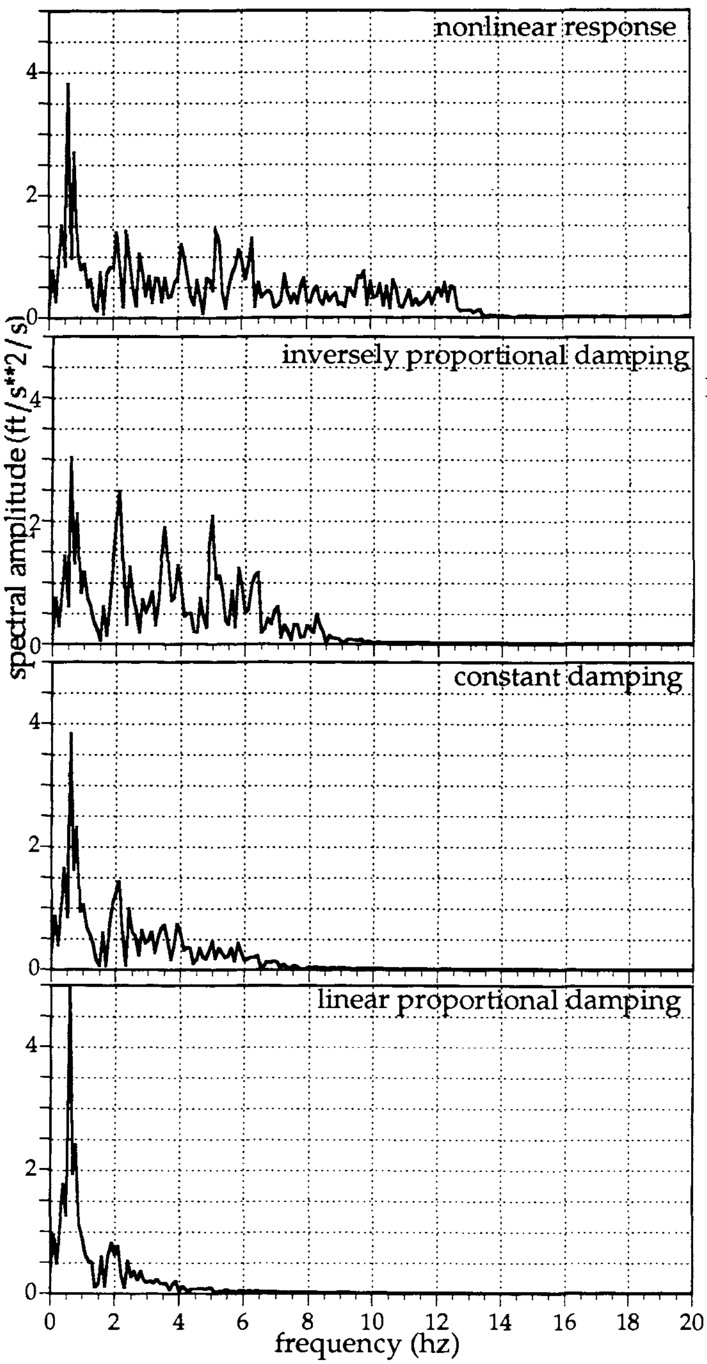

Figure 12 Amplitude Fourier spectras

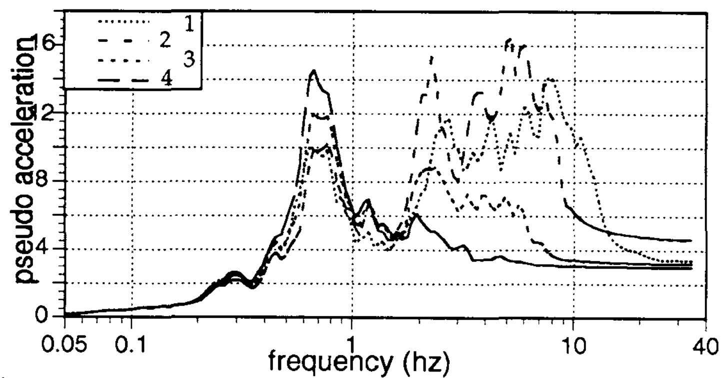

Figure 13. Response spectra for the soil profile 200 ft deep.
1: nonlinear response. 2: inversely proportional damping.
3: constant damping. 3: linear proportional damping.

SUMMARY AND CONCLUSIONS

The results of these studies show that although the model with linear hysteretic damping may be the one that predicts best the peak ground acceleration at the free surface, at least for the cases considered, the frequency content of the predicted motions does not agree well with the results of true nonlinear analyses. The model excessively filters the high frequencies. This explains the problems encountered in practice when performing deconvolution analyses for deep soil deposits starting with realistic motions at the free surface.

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