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NUMERICAL METHODS IN SITE RESPONSE ANALYSIS FOR NUCLEAR APPLICATIONS

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ABSTRACT

Site response analysis is a precursor to soil-structure interaction analysis, which is an essential component in the seismic analysis of safety-related nuclear structures. Current practice in calculating site response mainly involves a one-dimensional analysis using an equivalent linear method in the frequency-domain. Nonlinear time-domain methods using hysteretic material models have been employed for the assessment of buildings, bridges and petrochemical facilities, but are not widely used at this time in the nuclear industry. None of the site response analysis methods in use have been thoroughly validated for large strains and high frequencies, which is essential for nuclear applications. Although these methods are used with confidence for traditional civil engineering applications, their utility for nuclear applications has not been established. The purpose of this study is to shed light on the applicability of equivalent linear and nonlinear programs across a broad range of frequencies and earthquake shaking intensity.

Four hypothetical sites are chosen for this study. Two sites represent the Central and Eastern United States (CEUS) and two sites represent the Western United States (WUS). The CEUS sites are subjected to three ground motions, and the WUS sites are subjected to six ground motions (three ordinary and three pulse motions). Site response analyses for these cases are performed using the frequency-domain equivalent linear program, SHAKE, and the time-domain nonlinear programs, DEEPSOIL and LS-DYNA. Comparison of the responses calculated from these programs showed large differences between equivalent linear and nonlinear programs, especially for large strains and higher frequencies. Significant differences are also observed in the nonlinear responses for some cases, underlining the need for thorough validation studies.

INTRODUCTION

Site response analysis is the process of calculating the response of a soil deposit to an earthquake ground motion in the absence of structures; i.e., calculating the free-field response. It is a nonlinear three-dimensional wave-propagation problem involving the excitation of a soil deposit by a wave-field comprising of body and surface waves. However, in practice, site response analysis is most often performed using one-dimensional methods, which are based on the following assumptions: a) soil profiles comprise of horizontal layers stacked on top of each other, b) the soil layers are homogeneous along the horizontal plane, and c) the ground motion incident on the soil deposit is comprised of vertically propagating shear waves. These assumptions simplify the site response phenomenon to a one-dimensional problem and have enabled the use of simplified numerical methods for analysis. One-dimensional site response analysis involves the excitation of a soil profile using the horizontal component of a ground

motion, and calculating the response of the individual soil layers. An upward wave propagation analysis (or a convolution analysis) typically involves the input of a rock outcrop motion (ground motion recorded at a nearby, outcropping bedrock) at the soil-bedrock interface, and the bedrock is replaced by a transmitting boundary to enable the radiation of outgoing waves. In a downward wave propagation analysis, the surface free-field motion is input at the topmost layer of the soil, and the response of underlying layers is calculated. This process is termed deconvolution.

STATE-OF-THE-ART IN SITE RESPONSE ANALYSIS METHODS

One-dimensional site response analysis is typically performed using a) the equivalent linear method in the frequency-domain or b) the nonlinear method in the time-domain. Site response analysis for nuclear applications is performed traditionally using an equivalent linear method, especially the numerical program, SHAKE (Schnabel *et al.*, 2012). Time-domain programs that use the nonlinear method are presently used for certain building and non-nuclear infrastructure design. Some of the well-known nonlinear time-domain programs are DEEPSOIL (Hashash *et al.*, 2011), D-MOD_2 (Matasovic, 2006), DESRA-2 (Lee and Finn, 1978) and TESS (Pyke, 2000). Finite element programs such as OpenSees (Mazzoni *et al.*, 2009) and LS-DYNA (LSTC, 2009) are also used for nonlinear site response analysis.

Equivalent linear methods

The equivalent linear method involves the calculation of an approximate nonlinear response using a linear analysis with the soil properties adjusted to account for softening during earthquake shaking. These layer properties are calculated by an iterative procedure involving a series of linear analyses that can be performed either in the frequency or time domains. A linear analysis is first performed using the initial values of shear modulus and damping ratio, and the peak strains in the soil layers are computed. An effective shear strain is calculated for each layer by multiplying the peak shear strain by an effective shear strain ratio. This strain value, along with the modulus reduction and damping curves, is used to update the shear modulus and damping of each layer. The new values are used to perform another iteration of linear analysis, and the iterations are continued until the shear strains from consecutive analyses match within a pre-defined tolerance. The effective shear strain ratio is a key parameter that relates the loading conditions occurring during the earthquake to those in the laboratory tests that are used to calculate the modulus reduction and damping curves (Kramer, 1996). The SHAKE user's manual (Schnabel *et al.*, 2012) recommends a value of 0.65, which has traditionally been used in practice. However, this recommendation is only experiential and has not been rigorously validated by experimental results, to the knowledge of the authors.

Nonlinear methods

Nonlinear methods simulate the hysteretic stress-strain response of the soil and are therefore potentially more realistic than equivalent linear methods. The soil profile in a nonlinear method can be modeled using two approaches: 1) the lumped-mass approach, and 2) the finite element approach. In the lumped-mass approach, the soil layers are lumped into adjacent nodal masses, which are connected by springs that model the soil stress-strain behavior in shear. The input ground motion is applied at the base of the soil column, and the dynamic equations of motion are integrated to calculate the response of the soil layers. In the finite element approach, the soil layers are modeled using solid elements stacked on top of each other, and constrained to only move in shear. One-dimensional nonlinear material models are typically characterized by 1) the backbone curve, and 2) a set of rules that govern the hysteresis path under an irregular cyclic loading. Various expressions have been proposed for the backbone curve [e.g. the Ramberg-Osgood model (Ramberg and Osgood, 1943), the hyperbolic or the Kodner-Zelasko model (Lee and Finn, 1978), the modified hyperbolic model or the modified Kodner-Zelasko (MKZ) model (Matasovic, 1993)], each developed to better reproduce the experimental stress-strain data. The most

well-known set of hysteresis rules are the extended Masing rules, which are used in DEEPSOIL. Because they define the shape of the loops, the hysteresis rules significantly affect the nonlinear response, and especially hysteretic damping.

Validation of one-dimensional site response analysis methods

One-dimensional site response analysis programs are typically validated by comparing the numerical response of a real soil profile to recorded field data. Validation studies have been performed for SHAKE using field data (Chang, 1996; Idriss, 1990) but these data either involved 1) weak ground motion shaking, or 2) approximation of a three-dimensional wave field using a one-dimensional wave propagation model. One study to verify one-dimensional wave propagation was performed by Chang *et al.* (1990). They used data gathered from the Lotung LSST site in Taiwan and concluded that the site response calculated using the nonlinear program, DESRA-2 resulted in a better agreement with the field data than the equivalent linear program, SHAKE. Kwok *et al.* (2008) examined the results of blind predictions of the site response of Turkey-Flat site near Parkfield, California. Site responses calculated using six nonlinear programs, including DEEPSOIL, were compared with recorded data, and none resulted in a close prediction. The differences could be attributed to the input motions, characterization of the soil profiles, characterization of the nonlinear properties of each layer, and the necessary assumption that the true three-dimensional wave field could be collapsed into a vertically propagating shear wave. Indeed, validation of a one-dimensional site response analysis program ideally requires data recorded from a true one-dimensional wave field that is not a product of real earthquakes. Validation can also be performed through laboratory experiments using a laminar box in a geotechnical centrifuge (Elgamal *et al.*, 2005). Validation studies at a larger scale using a laminar box installed on an earthquake simulator have not yet been undertaken.

NUMERICAL ANALYSES

Selection of soil profiles for site response analysis

Four hypothetical soil profiles, namely, Site E1, Site E2, Site W1 and Site W2, are considered in this study. Sites E1 and E2 are representative of soil conditions in the Central and Eastern United States (CEUS), and Sites W1 and W2 are representative of soil conditions in the Western United States (WUS). Each soil site is 100m deep and is underlain by linear elastic bedrock. Site E1 is a uniform site composed of hard rock with a small-strain shear wave velocity of 2500 m/s. Site E2 comprises a 28m layer of sand (shear wave velocity 300 m/s) lying atop a 72m layer of hard rock. Site W1 and Site W2 are composed of 100m of soft rock (shear wave velocity 1000 m/s) and 100m of sand (shear wave velocity of 300 m/s)respectively. The bedrock for the CEUS sites has a shear wave velocity of 2500 m/s, and the bedrock of the WUS sites has a shear wave velocity of 1000 m/s. A unit weight of 23.3 kN/m^3 is assigned to both soft rock and hard rock, and a unit weight of 20.1 kN/m^3 is assigned to the sand. Nonlinear properties (stress-strain curves and damping curves) are assigned to each of these materials and are presented in Figure 2 below.

Preliminary modal analyses are performed in LS-DYNA to calculate the four site periods. Small strain linear properties are used in these analyses, along with a damping ratio of 1.1% (Bolisetti, 2013). The first two modal periods for Site E1 are 0.16 sec and 0.05 sec. For Site E2 (W1, W2), these periods are 0.39 (0.40, 1.32) sec and 0.14 (0.13, 0.44) sec. Bolisetti (2013) shows the modal periods of the sites are sufficiently close to the results from analytical calculations.

Selection of input ground motions for site response analysis

Single component bedrock outcrop ground motions are input at the base of each soil profile, through a viscous bedrock boundary. A total of nine ground motions are used: six motions for the WUS

soil profiles (three ordinary motions, WO1, WO2 and WO3, and three near-fault pulse motions with forward directivity, WP1, WP2 and WP3), and three motions for the CEUS soil profiles (EO1, EO2 and EO3). Details of these ground motions are presented in Table 1. The ordinary WUS motions and the CEUS motions are chosen to cover a range of frequency contents and peak accelerations. The pulse motions for the WUS soil profiles are chosen to cover a range of pulse periods. Figure 1 presents the acceleration response spectra of the nine ground motions. Panel a presents the spectra for the CEUS ground motions, while panels b and c present the spectra for the ordinary motions and pulse motions used for the WUS soil profiles.

Table 1. Input ground motions for site response analysis

GM ID	Earthquake Station	PGA (g)[a]	T_p (sec)[b]	dt (sec)[c]
EO1	Mineral, VA (2011) Charlottesville	0.10	-	0.005
EO2	New Hampshire (1982) Franklin Falls Dam	0.31	-	0.005
EO3	Saguenay, Canada (1988) Dickey	0.09	-	0.005
WO1	Northridge (1994) Vasquez Rock Park	0.15	-	0.02
WO2	Northridge (1994) Wonderland Ave	0.17	-	0.01
WO3	San Fernando (1971) Lake Hughes	0.19	-	0.01
WP1	Landers (1992) Lucerne	0.73	5.1	0.005
WP2	Northridge (1994) Rinaldi	0.83	1.5	0.005
WP3	Chi Chi (1999) TCU128	0.19	9.0	0.005

[a] Peak ground acceleration of the recorded ground motion

[b] Pulse period as specified in the PEER NGA ground motion database (PEER, 2011) for WP1 and WP2, and as calculated by Baker (2007) for WP3

[c] Time-step used for the discretization of the ground motion records

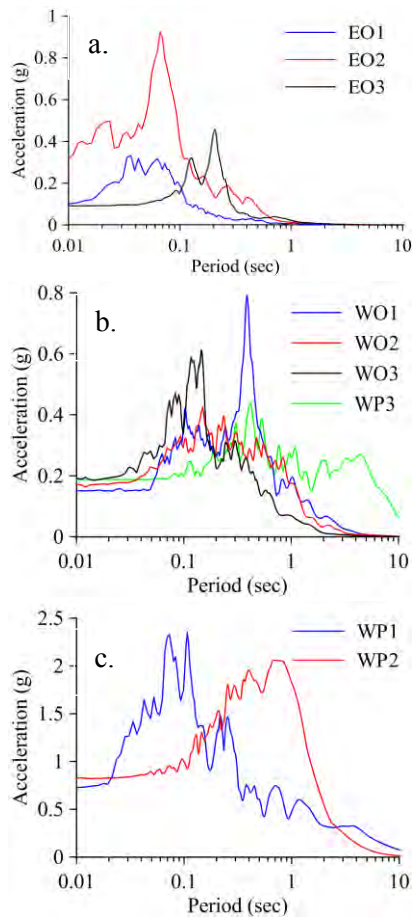

Figure 1. Acceleration response spectra of the input ground motions

Material modeling

The three numerical programs SHAKE, DEEPSOIL and LS-DYNA employ different soil material models to simulate the stress-strain response of the soil. SHAKE uses the equivalent linear model, which is defined by a modulus reduction curve (or a backbone curve), and a damping curve. DEEPSOIL and LS-DYNA use nonlinear hysteretic models to simulate cyclic behavior in shear. Although the material models in the different programs vary, the corresponding model parameters are chosen such that the resultant dynamic soil properties (modulus reduction and damping) are similar. Figure 2 shows that the backbone stress-strain curves input into the different programs for sand, hard rock and soft rock are almost identical.

The modulus reduction and damping curves are directly input into SHAKE as discrete points. An effective shear strain ratio of 0.65 is used for all analyses except for Site W2 subjected to pulse motions, where a ratio is 0.22 is used (refer to Bolisetti (2013) for details). DEEPSOIL provides the choice of two nonlinear material models: the 'MRDF Pressure-Dependent Hyperbolic Model', which uses modified-extended Masing criteria, and the 'Pressure-Dependent Hyperbolic model', which uses the extended Masing criteria (refer to Hashash *et al.* (2011) and Bolisetti (2013) for details). Both materials employ a hyperbolic model to define the backbone curve, and incorporate additional parameters to make it pressure-dependent (pressure-dependency is ignored for this study). LS-DYNA offers an extensive database of material models, of which several can be used for soil modeling. The MAT_HYSTERETIC model (with the LS-DYNA keyword *MAT_079, and referred to as the "Arup model") using the Masing rules was employed in this study. The Arup model requires the backbone curve to be defined using a maximum of 10 discrete stress-strain points. Preliminary analyses using this material model revealed an unrealistically large contribution of higher frequencies to the acceleration response, especially in cases involving large strains. To address this numerical problem, an alternative approach was adopted that involved superimposing 12 solid elements that act in parallel, instead of one element with 10 discrete points in the backbone curve. The superimposition results in stress-strain curve with 120 points in the backbone curve (refer to Bolisetti (2013) for details). This approach is only adopted for the WUS sites, since only small strains are expected in the CEUS sites for the chosen ground motions. The 10-point backbone curve and 120-point backbone curve are hereafter referred to as LS-DYNA 10pt and LS-DYNA 120pt, respectively.

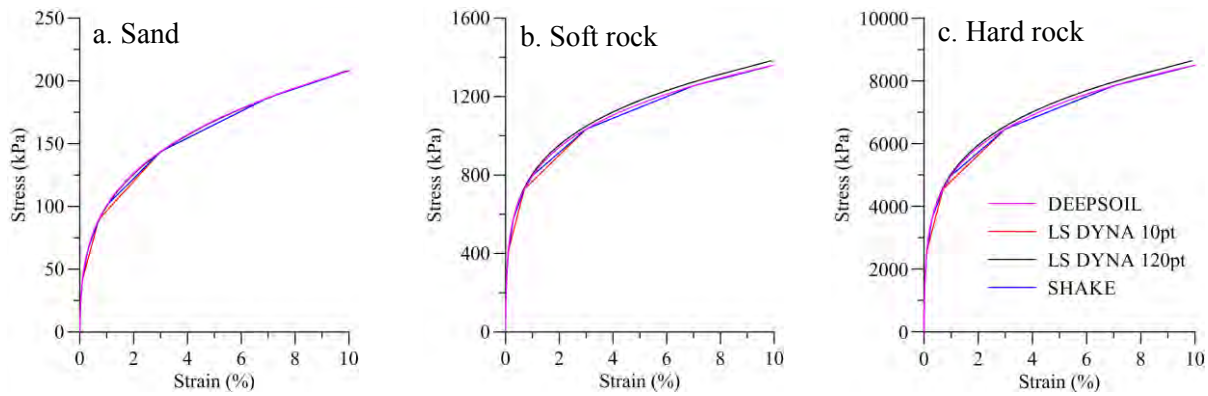

Figure 2. Backbone curves used for site response analyses

Analyses and results

The site response analyses described above are performed using SHAKE, DEEPSOIL and LS-DYNA 10pt. Analyses for the WUS soil profiles are also performed using LS-DYNA 120pt. The time-steps of the input ground motion records are presented in Table 1. The responses are calculated with the same time-step discretization as the input for each ground motion. A small strain damping of 1.1% is incorporated in DEEPSOIL (frequency-independent damping) and LS-DYNA (using the *DAMPING_ FREQUENCY_RANGE option from 2 to 50 Hz). Layer thicknesses of 4m, 2m and 1m are chosen for the hard rock, soft rock and sand layers respectively, for all the programs. These thicknesses result in cut-off frequencies of 63Hz, 50Hz and 30Hz in the three materials at small strains (Bolisetti, 2013). Responses of the soil profiles are output at three different depths as acceleration response spectra, acceleration response histories and stress-strain loops. Spectral amplifications at the surface, peak strain profiles, and peak acceleration profiles are also calculated. These results are presented and examined in detail in Bolisetti (2013); only a sample of the results is presented here. Figure A-1 presents the peak shear strain profiles for Site E1 and Site W1, and Figure A-2 presents the surface acceleration response spectra for Site E1,

Site E2 and Site W1 predicted by all the programs. The following observations can be made from the peak strain profiles presented in Figure A-1:

1. Peak strain profiles calculated by the different programs match well for Site E1.
2. The peak strains calculated by LS-DYNA 10pt and LS-DYNA 120pt match well for all cases presented in Figure A-1. This indicates that the increase in the number of points in the backbone curve causes negligible changes in the hysteretic behavior.
3. SHAKE significantly underestimates the peak strains, especially in the lower layers of Site W1, which experiences much larger strains than Site E1. This is likely because the equivalent linear method accounts for nonlinear behavior in an 'averaged' sense by decreasing the shear modulus based on the effective shear strain, which is taken as a measure of the extent of nonlinearity induced by the earthquake. In contrast, nonlinear analysis accounts explicitly for softening and load history effects.
4. The peak strain profiles calculated using DEEPSOIL show large variations in the peak strains of consecutive layers. This result is clearly unrealistic; Bolisetti (2013) found that the variations arise due to one of the extended Masing rules employed in DEEPSOIL. The LS-DYNA analyses do not show these variations, because the Arup model does not employ the extended Masing rules.
5. Differences between the peak strains calculated by DEEPSOIL and LS-DYNA are larger for Site W1 than Site E1. Since the backbone curves for the two programs are almost identical the differences are likely a result of the different hysteresis rules employed in the programs.

The following observations can be made from the acceleration response spectra presented in Figure A-2:

1. The acceleration response spectra calculated by the different programs match well for Site E1, except for slight differences at periods less than 0.04 sec. The response spectra also match reasonably well for Site W1 subjected to the ordinary ground motions. However, differences exist at periods less than 0.2 sec, which are greater than those observed for Site E1.
2. The differences in the acceleration response spectra are much greater at the surface of Site E2, where the strains are larger due to the presence of a sand layer. Similarly for Site W1, the acceleration response spectra calculated using different programs are much different for the pulse motions than for the ordinary ground motions. This indicates that the responses calculated using different programs are increasingly different for larger strains.
3. Analysis using SHAKE often results in near constant spectral accelerations in the short period range, suggesting that the corresponding frequencies do not contribute to the response (see bottom row of Figure A-2). Additionally, it is observed that the period below which the spectral accelerations are near constant increases with the peak strain induced in the soil profile, indicating a decrease in the maximum frequency that contributes to the response. This is because as the peak strain in the soil profile increases, the modified shear modulus (and therefore the shear wave velocity) used in the SHAKE linear analysis decreases thereby decreasing the maximum frequency that can be propagated through the soil profile. However, in a nonlinear analysis, the shear modulus of each layer changes through time according to the strain in the layer. In each load cycle, while the strain is zero, the shear modulus is at its maximum, and while the strain is at its maximum, the shear modulus is at its minimum. As a result, a range of frequencies propagate through the layers, which include the higher frequencies that propagate when the shear modulus is higher. This accounts for the high-frequency (short period) response captured only by the nonlinear analyses and not by the SHAKE (linear) analyses. Although this may not be an important observation for traditional structures, it could be critical for very stiff structures such as nuclear power plants, where higher frequencies (typically much greater than 15 Hz) contribute substantially to the response of both the structure and the safety-related secondary systems.

4. Analysis using LS-DYNA 10pt sometimes results in an overestimation of the short period accelerations (see response of Site W1 to ground motions WO2 and WP2; refer to Bolisetti (2013) for more examples). This also manifests in the corresponding acceleration time series, which show an unusual number of spikes as compared to the response of DEEPSOIL and LS-DYNA 120pt (see Figure 3 below). A significant reduction of the short period accelerations is observed when the number of points in the backbone curve is increased to 120 in LS-DYNA 120pt, indicating that the high-frequency noise in the LS-DYNA 10pt response is primarily due to the significant changes in stiffness in its multilinear backbone curve. Increasing the number of points in the backbone curve significantly decreases the stiffness increments and therefore results in smoother accelerations. This observation could play a critical role in estimating the seismic demands on stiff structures as noted above.

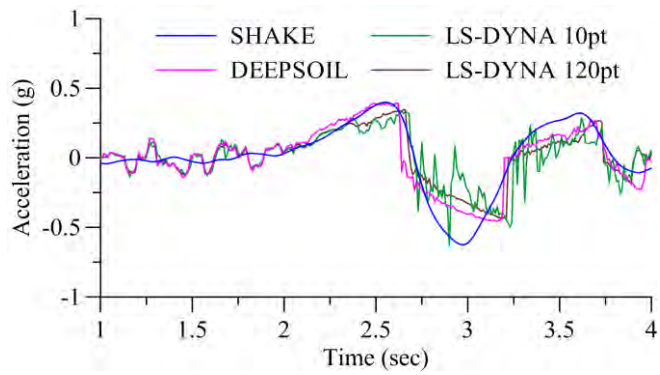

Figure 3. Acceleration time series at the surface of Site W1 subjected to ground motion WP2

5. Despite using very similar backbone curves and the same viscous damping ratio for DEEPSOIL and LS-DYNA 120pt, considerable differences are still noticeable in the short period range (see responses of Site E2 and Site W1 in Figure A-2). This is attributed to the differences in the cyclic hysteresis models and possible differences in the damping formulations in the two programs. Furthermore, this indicates that small changes in the hysteretic material model or damping formulation can affect the high-frequency response.

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

Although a number of studies have been performed to develop numerical models for one-dimensional site response analysis, significant differences still exist in the responses calculated from the industry-standard numerical programs especially 1) in the short period range, and 2) for the cases involving large strains. Robust validation of these programs is therefore needed to increase the credibility of these programs, especially for soft soil profiles subjected to intense ground motions. These validations need to be in the form of large-scale laboratory experiments since the one-dimensional wave field is not produced by a real earthquake. Particular attention needs to be paid to the short-period (high-frequency) response of the soil profiles since it is critical for calculating seismic demands in stiff structures such as nuclear power plants. Additionally, the present material models need to be extended and validated for very large strains (greater than 1%) to accommodate site response analyses for nuclear applications, which involve the calculation of seismic demands for ground motions with very large return periods.

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APPENDIX A: RESULTS OF SITE RESPONSE ANALYSES

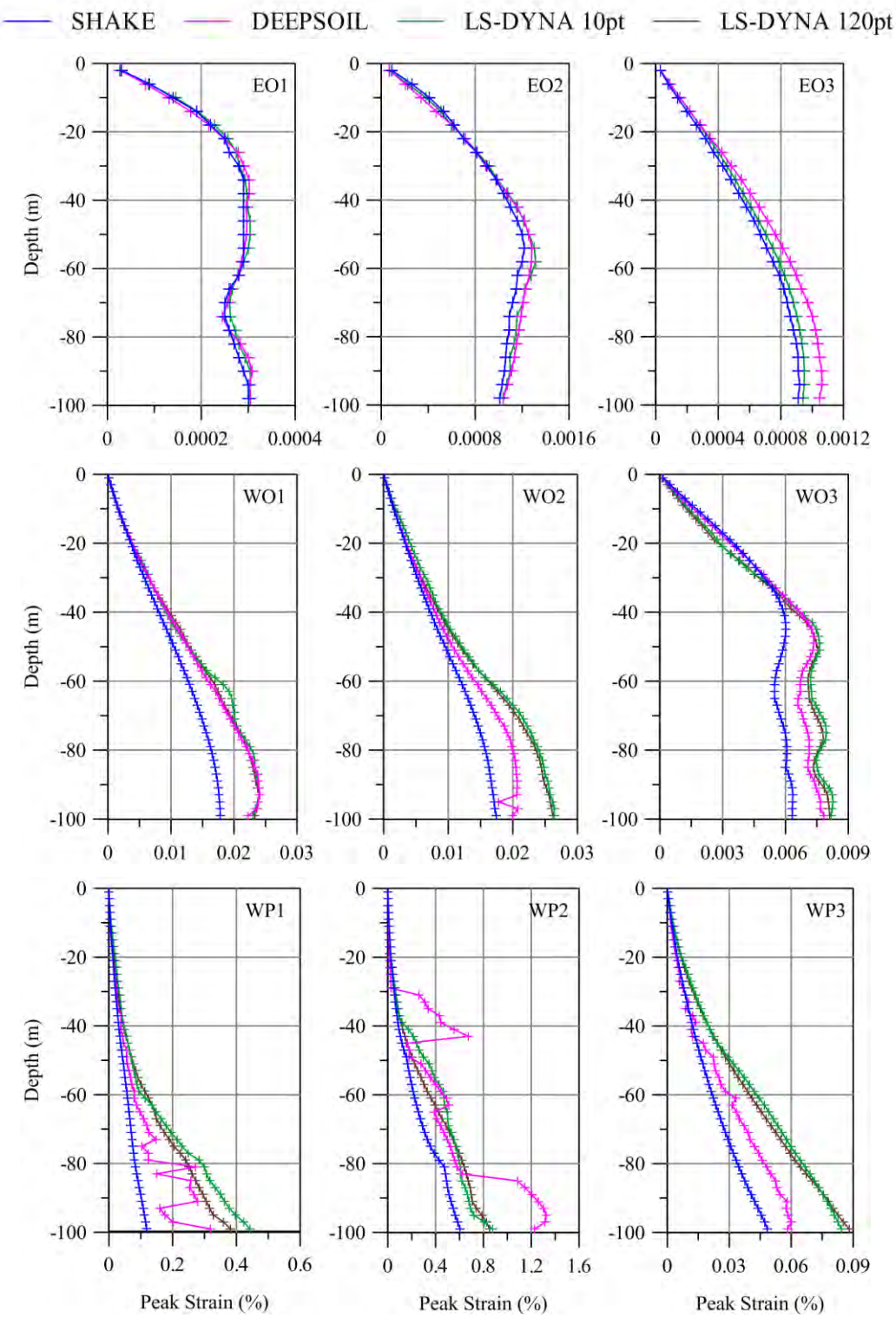

Figure A-1. (Rows top to bottom) Peak shear strain profiles for Site E1 subjected to CEUS ground motions, Site W1 subjected to ordinary WUS motions and Site W1 subjected to WUS pulse motions

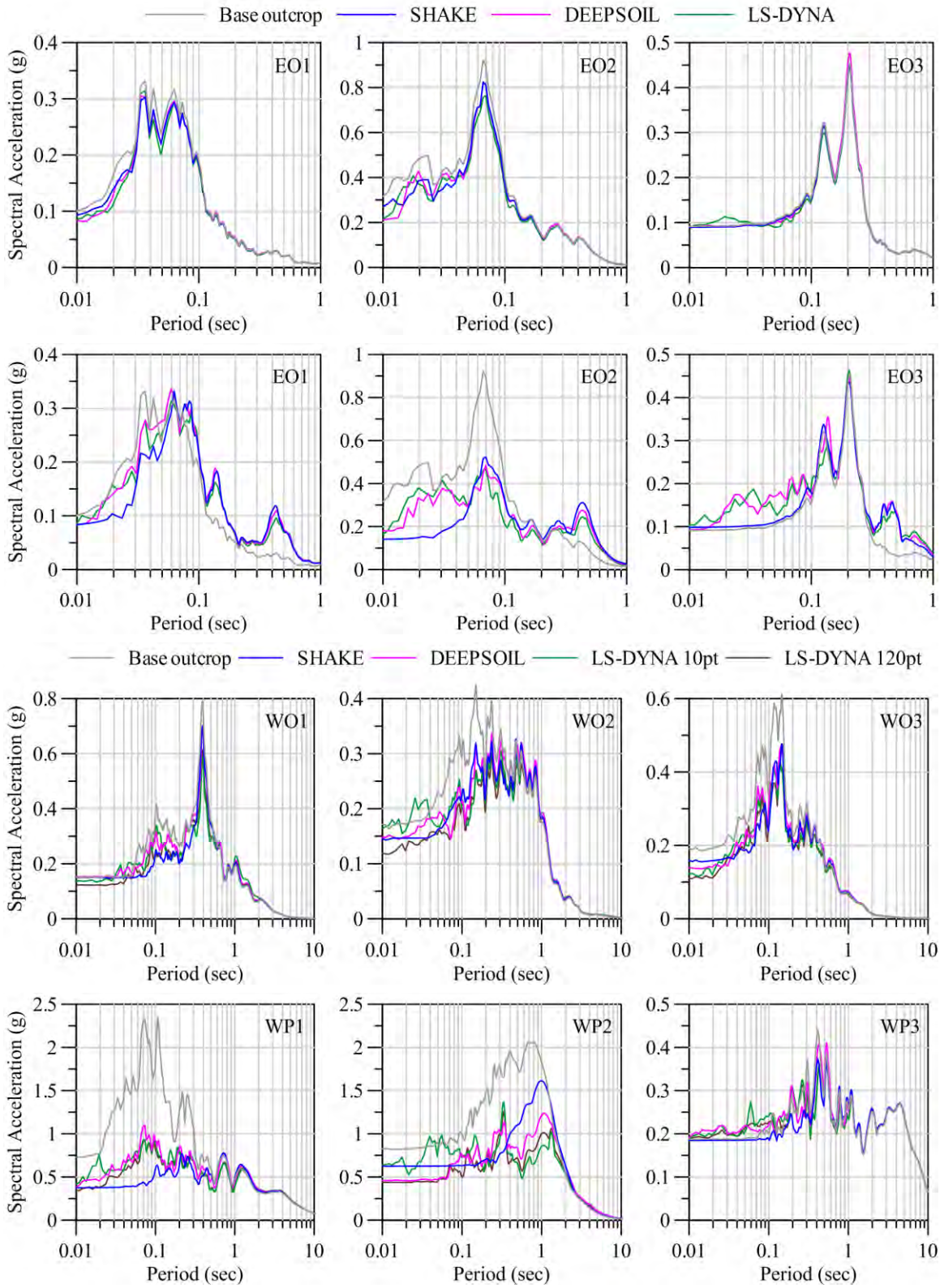

Figure A-2. (Rows top to bottom) Acceleration response spectra at the surface for Site E1, Site E2, Site W1 subjected to ordinary ground motions, and Site W1 subjected to pulse motions