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SITE RESPONSE ANALYSIS FOR VERTICAL GROUND MOTION

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Abstract. *This study initially considers a uniform soil layer subjected to vertically propagating harmonic P-waves. The response of the layer is calculated analytically, considering one-dimensional propagation, and numerically employing time domain coupled consolidation Finite Element analysis for a range of input frequencies. The computed amplification function is shown to significantly depend on the modelling of the fluid phase and the soil permeability. Subsequently, a ground profile corresponding to a downhole array in Dahan (Taiwan) is examined assuming undrained, drained and coupled consolidation behaviour. The numerical predictions are compared with the vertical recorded response during the 1999 Chi-Chi earthquake demonstrating a very good agreement for the coupled consolidation analysis and highlighting the limitations of the other two approaches (i.e. drained and undrained analyses).*

1 INTRODUCTION

Site response analysis is commonly performed to account for local site effects on ground motion propagation during an earthquake. Most site response analyses involve horizontal ground motion, considering vertically propagating shear waves in horizontally layered systems. In reality, the ground is simultaneously subjected to shaking in both the horizontal and vertical directions during an earthquake, but the vertical response has generally received limited attention in the literature. Field evidence from various recent earthquakes indicates though that damage of concrete buildings and bridges can be attributed to high vertical ground motion (e.g. [4]). Therefore there is a need for better establishing the site response to vertical ground motion.

It is widely believed that vertical ground response is dominated by the propagation of compression waves (P-waves) and has been found to depend on the pore-fluid compressibility, permeability, soil stiffness and porosity ([1], [9]). Saturated soils are two-phase materials consisting of a solid phase (soil skeleton) and a fluid phase (pore water filling the voids). Depending on the soil permeability, the rate of loading and the hydraulic boundary conditions, it is often necessary to employ coupled analysis to accurately model the two phase behaviour of soils. This is in particular the case when the response to vertical motion is considered.

This study examines first the response of a uniform soil layer subjected to harmonic P-waves analytically and numerically, with dynamic coupled consolidation Finite Element (FE) analysis, aiming to highlight the importance of appropriately modelling the fluid-phase when vertical ground motion is considered. This is further emphasised in the second part of this study where the vertical seismic response of a ground profile is computed with different assumptions regarding the pore fluid compressibility (drained, undrained and coupled consolidation) and the results are compared with field measurements.

2 UNIFORM SOIL LAYER ON RIGID BEDROCK

The solution of the wave equation for a uniform soil layer on rigid bedrock (see Figure 1) subjected to vertically propagating harmonic P-waves is a vertical displacement of the form:

$$v(z,t) = Ae^{i(\omega t + k^* z)} + B\, e^{i(\omega t - k^* z)} \tag{1}$$

where ω is the circular frequency of the input pulse, k* is the complex wave number ($= \frac{\omega}{V_p^*}$) and A and B are the amplitudes of waves traveling in the upward and downward directions respectively. Considering the equilibrium of stresses at the ground surface and the compatibility of displacements at the soil-rock interface, similarly to the case of vertically propagating shear waves [6], it can be shown that the transfer function between the top and the bottom of the soil layer is given by:

$$F(\omega) = \frac{1}{cos(\omega H / V_p^*)} \tag{2}$$

where H is the thickness of the soil layer and the complex P-wave velocity is given by:

$$V_p^* = \sqrt{\frac{D}{\rho}}(1 + i\xi) = V_p\ (1 + i\xi) \tag{3}$$

where D, ρ and ξ are the constrained modulus, mass density and damping respectively of the soil. For a fully saturated deposit the expression for the P-wave velocity becomes:

$$V_p^* = \sqrt{\frac{D+\frac{K_f}{n}}{\rho}}(1+i\xi) = V_p\ \ (1+i\xi) \tag{4}$$

where K_f is the bulk modulus of the pore fluid and n is the porosity. Based on Equations (2) & (4) the vertical site response depends significantly on the pore fluid stiffness. This is clearly demonstrated in Figure 2 which plots the transfer function for a typical soil layer for ideal undrained conditions with undrained Poisson's ratio equal to 0.5 and K_f=∞, drained (K_f=0) and for K_f=2.2GPa, which is the value of the bulk modulus of water. Assuming undrained behaviour results in no amplification, while employing the bulk modulus of the water shifts the response to higher frequencies without though affecting the amplitude of the transfer function with respect to the drained case.

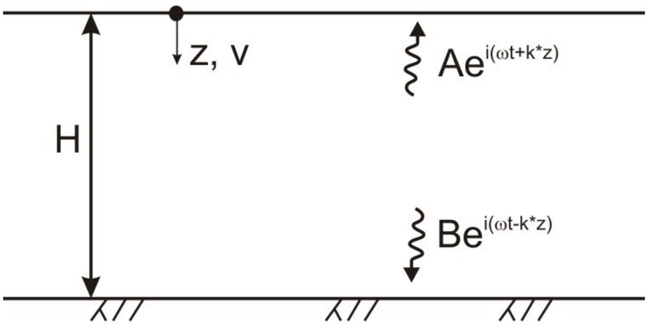

Figure 1: Uniform soil layer of thickness H underlain by rigid bedrock.

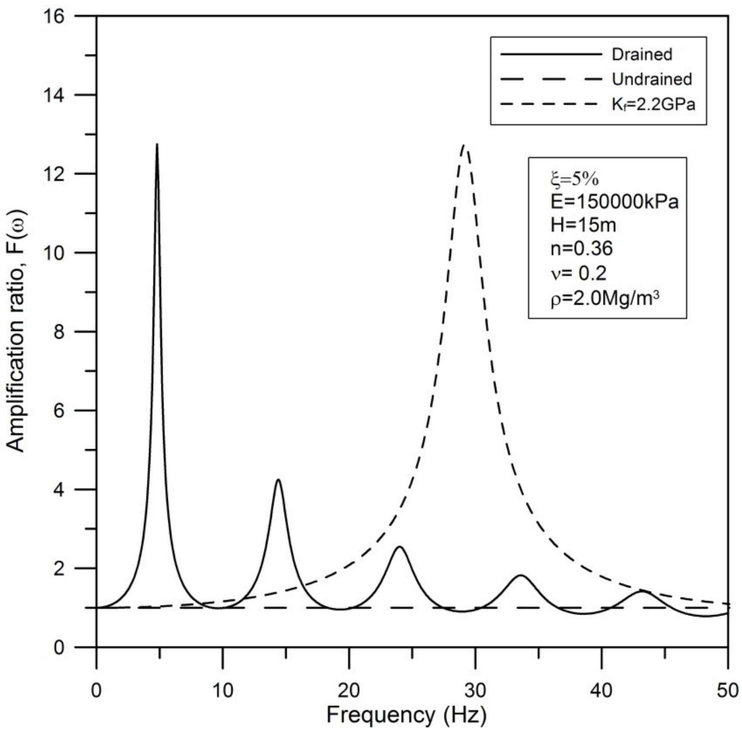

Figure 2: Transfer functions for a uniform elastic soil layer underlain by rigid bedrock for various values of the pore fluid bulk modulus

3 DYNAMIC COUPLED CONSOLIDATION ANALYSIS OF A UNIFORM SOIL LAYER

The simple analysis of the previous section, which is based on one dimensional wave propagation within a one-phase material, gives some insight into the layer response, but it cannot account for the coupling in the dynamic response between the solid and the fluid phase. In order to investigate the effect of the solid-fluid interaction on the vertical ground response, dynamic coupled consolidation analyses are carried out with the Finite Element method in this section.

3.1 Analysis arrangement

To analyse the response of the uniform soil layer illustrated in Figure 1, a 15m high soil column of was analysed in plane strain conditions with the Imperial College Finite Element program ICFEP [7]. In all analyses full coupling of the fluid and solid phases was considered by a adopting a “u-p” formulation, in which the primary unknowns are the solid phase displacements and the pore fluid pressure. The FE mesh comprised of 60 8-noded quadrilateral elements (1m wide and 0.5m high). The water table assumed to be at the ground surface and the soil was treated as fully saturated with hydrostatic pore water pressure distribution. The column was subjected to a set of harmonic pulses of unit amplitude which were applied as a vertical acceleration along the base of the mesh. The tied degrees of freedom boundary condition was applied along the vertical boundaries to ensure one-dimensional propagation and the horizontal movement was restricted along the bottom boundary. The ground surface was treated as a free draining boundary, while the bottom and side boundaries were treated as impermeable ones. A target damping ratio of 5% was adopted using the Rayleigh damping formulation. The time integration was performed with the generalised-α method ([3], [5]) which is an unconditionally stable implicit method, with second order accuracy and controllable numerical damping. The remaining material properties are listed in Table 1. The time step of each analysis was taken as a fraction ($\Delta t= T/40$) of the pulse period. The soil layer was subjected to 13 harmonic pulses which covered a wide range of frequencies (f=5,10,15,20,25,27,28,29,30,40,45,50 Hzs) and each set of analyses was repeated for the four permeability values listed in Table 1.

Table 1: Material Properties

Parameter	Value
Young’s modulus, E (MPa)	150.0
Mass density, ρ (Mg/m3)	2.0
Poisson’s ratio, ν	0.2
Bulk modulus of water (MPa)	2.2E3
Porosity	0.36
Permeability, k (m/sec)	Varied, 10^{-6}, 10^{-4}, 10^{-3}, 10^{-2}

3.2 Results

The steady-state response at the ground surface was established for each considered pulse by examining a large number of cycles. The amplitude of the steady-state response normalised by the amplitude of the input motion for the four permeability values is plotted in Figure 3. The computed amplification ratios are also compared with the analytical transfer functions. The analyses for the very impermeable soil layer ($k=10^{-10}$m/sec) compare very well with the analytical transfer function with K_f=2.2GPa for the entire frequency range. For such a low value of permeability there is no relative movement between the solid and the fluid phase and hence the main effect of the fluid phase in the elastic response is to increase the stiffness of the soil layer in compression. It should be noted that the numerical analyses slightly overestimate the amplification close to resonance (f=29Hz), which is probably due to inability of the Rayleigh damping to maintain the target damping ratio constant across the entire frequency range. It is interesting to note that for the range of frequencies which are close to the natural frequency (f_o=29.14Hz) of the layer, the amplification ratio decreases as the permeability increases. This is more pronounced for the highest value of permeability (k=10^{-2} m/sec). Keeping in mind that all analyses were conducted for the same value of damping ratio (ξ=5%), the additional damping in the response results from the relative movement between the solid and the fluid phases which is more evident for the analyses with the higher permeability values. Bardet [1] also concluded that the solid-fluid damping within fully saturated sands can be comparable in magnitude to damping developed by hysteresis.

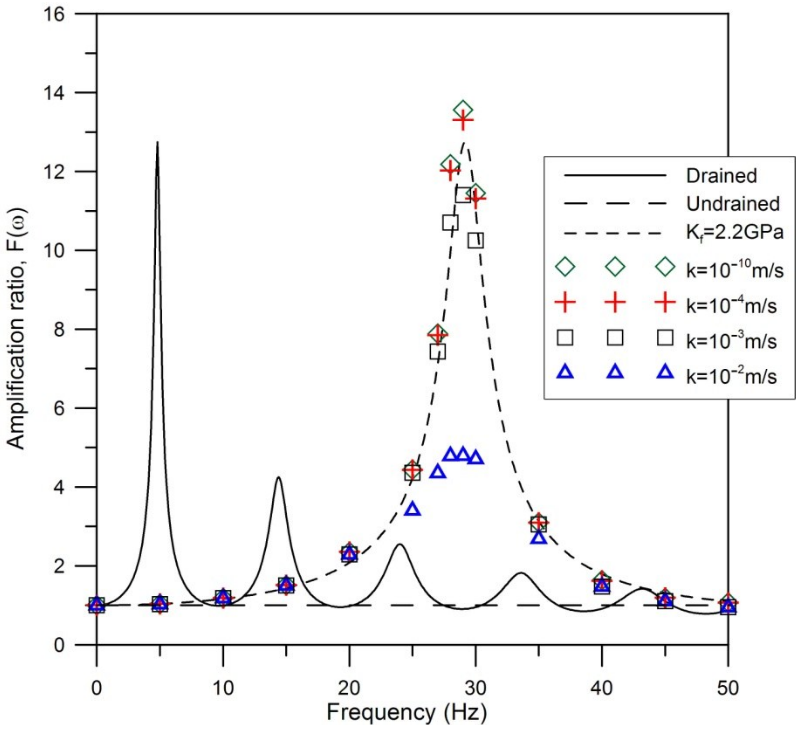

Figure 3: Amplification ratio for various values of permeability.

4 DAHAN DOWNHOLE ARRAY

4.1 Ground profile and analysis arrangement

In this section, acceleration time histories from the 1999 Chi–Chi earthquake in Taiwan recorded at the Dahan downhole array site [2], are used to validate the numerical model. The Dahan downhole array is part of the SMART-2 strong-motion array, located at a distance of about 80 km from the epicentre, and consists of three accelerometers at the depths of 50m, 100m and 200m and one at the ground surface. In this study, only the upper 50m of the stratigraphy were considered. The vertical acceleration time history recorded during the main shock of the Chi–Chi earthquake at a depth of 50m (Figure 4) was used as the input motion for the numerical model and the computed response at the ground surface is compared with the actual recording.

The Dahan site consists of alluvial deposits of silty sands, gravels and silty clays. The profile described in Table 2 was adopted based on information provided by the National Centre for Research on Earthquake Engineering (NCREE) and the Central Weather Bureau (CWB) in Taiwan. Rontogianni [8] performed equivalent linear analysis for the same site and examined its response to the horizontal components of the Chi-Chi earthquake. The converged values of stiffness and damping ratio for each layer obtained from the equivalent linear analysis of [8] were used in the analyses of this section and are also listed in Table 2 (denoted as E_{eq} and ξ_{eq} respectively). The water table was taken at 2m below the ground surface and the Poisson ratio was assumed to be 0.3 for all layers. The acceleration time history illustrated in Figure 4a was applied in the vertical direction along the base of the mesh with a time step, Δt=0.01sec. All other boundary conditions and analysis details are identical to the ones described in the previous section.

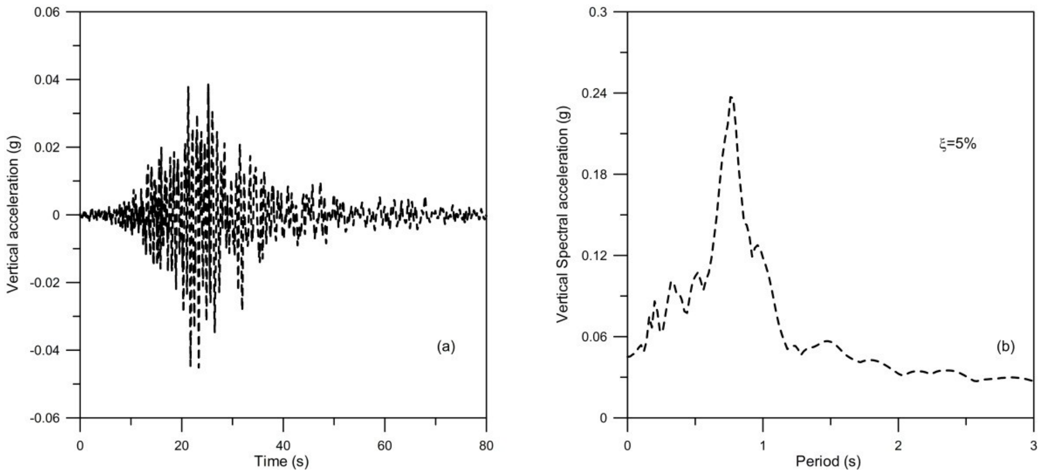

Figure 4: Acceleration time history (a) and the corresponding response spectrum (b) recorded at a depth of 50m at the Dahan site during the main shock of the Chi-Chi earthquake (Chiu [2]).

Table 2: Material properties assumed for the Dahan profile

Soil layer thickness (m)	ρ (Mg/m^3)	E (MPa)	E_{eq} (MPa)	ξ_{eq} (%)	k (m/s)
6	1.76	531.0	480.0	3	5.5E-5
2	1.82	1621.0	1516.0	3	1.5E-4
6	2.1	2333.0	2189.0	3	6.4E-5
4	2.1	1641.0	1467.0	3	6.4E-5
12	1.95	594.0	414.0	3	4.4E-5
20	1.95	670.0	496.0	3	2.0E-6

4.2 Numerical results

Figure 5 compares the recorded surface acceleration in the vertical direction against the computed ones, for undrained, drained and coupled consolidation analyses. The undrained analysis underestimates the ground motion and gives a response which is very similar to the input acceleration of Figure 4a. Since the water table is very close to the ground surface, most of the layers respond in an undrained manner and therefore, based on the transfer function of Figure 2, they do not amplify the response. On the other hand the drained analysis significantly overestimates the ground motion as it amplifies the lower frequencies of the input excitation. The coupled consolidation analysis gives the best prediction out of the three analyses and is in very good agreement with the recorded motion.

Figure 6 compares the computed vertical acceleration response spectra (by undrained, drained and coupled consolidation analyses) at the ground surface with the spectrum of the recorded motion. The undrained analysis significantly underestimates the high-frequency part of the response, but it predicts correctly the main peak of the spectrum. The drained analysis overestimates the response in the medium to low frequency range, while the spectrum of the coupled consolidation analysis compares very favourably with the spectrum of the actual record.

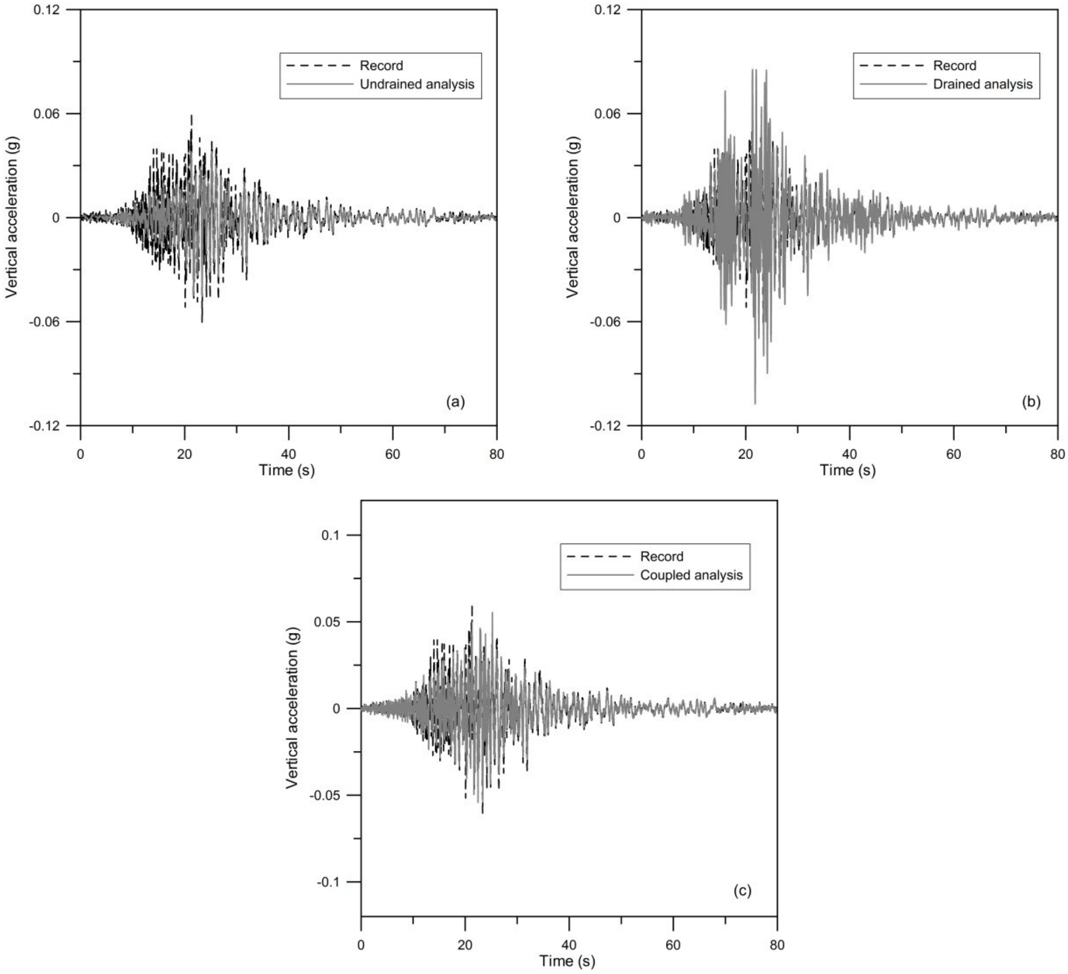

Figure 5: Comparison of the computed acceleration time histories at the ground surface with the recorded one for (a) undrained, (b) drained and (c) coupled analyses.

5 CONCLUSIONS

This study first examined the response of a uniform soil layer subjected to harmonic P-waves analytically, based on one dimensional wave propagation within a one-phase material and numerically, with dynamic coupled consolidation FE analysis. The analytical investigation showed that assuming undrained behaviour results in no amplification of the vertical ground motion, while employing the bulk modulus of the water shifts the response to higher frequencies without though affecting the amplitude of the transfer function with respect to the corresponding drained case. The numerical results showed that the transfer function depends also on the soil permeability and that for high permeability values (greater than $k=10^{-4}$ m/sec) there is evidence of additional non-hysteretic damping in the response which results from the relative movement between the solid and the fluid phases.

In the second part of this study, the vertical seismic response of a ground profile was computed with different assumptions regarding the pore fluid compressibility (drained, undrained and coupled consolidation) and the results were compared with field measurements. The undrained analysis underestimated the ground motion and gave a response which was very similar to the input acceleration. On the other hand the drained analysis significantly overestimated the ground motion as it amplified the lower frequencies of the input excitation, while the results of the coupled consolidation analysis were in very good agreement with the measured ground motion.

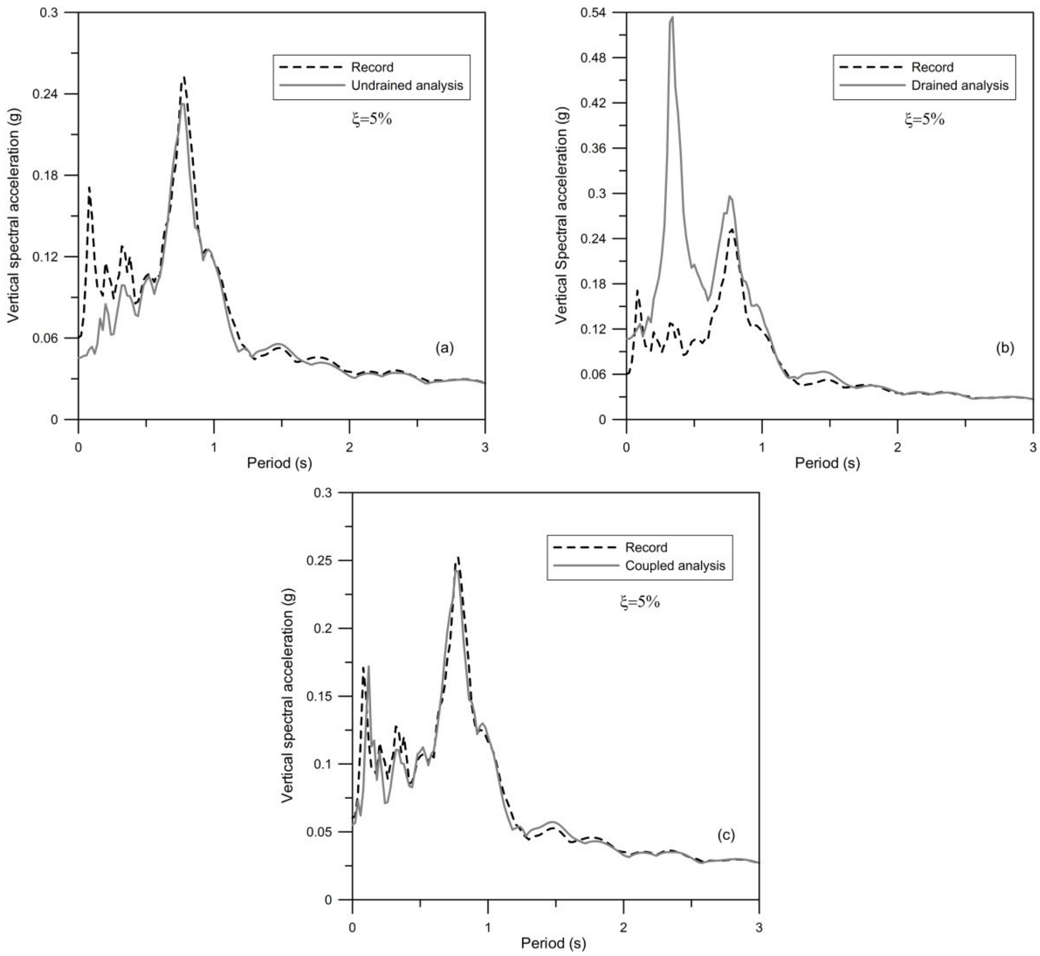

Figure 6: Comparison of the computed acceleration response spectra at the ground surface with the spectrum of the surface record for (a) undrained, (b) drained and (c) coupled analyses.

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