4th International Conference on
Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. 1651

EVALUATION OF SITE CHARACTERISTICS IN LIQUEFIABLE SOILS

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ABSTRACT

This paper investigates the ability to simulate ground response at low and high acceleration amplitudes taking into account non-linear effects such as liquefaction. Emphasis is given on the correct simulation of liquefiable soil layers and the calibration of the geotechnical parameters that determine non-linear behaviour of soils prone to liquefy. Accuracy of procedures adopted hereafter is checked by comparison of registered and computed ground motions, with liquefied soil layers.

Keywords: non-linear, soil, seismic, response, liquefaction

INTRODUCTION

The goal of this paper is to gain the knowledge that will allow the successful simulation of non-linear and soil liquefaction effects under strong ground motion with one-dimensional non-linear analyses using Cyclic1D software.

For this to be achieved it is required to use data from sites where down-hole / vertical arrays are installed, with sufficient soil properties and ground water table depth information. There should also be at least two pairs of accelerograms – with one accelerogram at a certain depth and another on the surface of the ground – from witch one pair should dispose PGA < 0.1g while the other > 0.2-0.3g.

Since all required data are collected the procedure witch will be followed for each site can be described as follows: 1) selection of appropriate Vs for Cyclic1D’s material models, b) selection of materials’ strength parameters, c) analysis (transient) using as base input motion the recorded accelerogram within the soil with low acceleration amplitude to sufficiently model soil’s elastic properties (criteria used at this purpose: straightforward comparison of PGA value, acceleration time history and Fourier transform of the acceleration time history at the surface between recorded and computed motion), and finally d) analysis using the accelerogram/s with higher amplitude values in an attempt to simulate non-linear behavior and soil liquefaction effects.

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SOFTWARE OVERVIEW

Materials overview

The materials of this software fall into two main categories: the default materials and the user-defined ones, named “U-Clay/Rock”, “U-Sand” and “U-LiqSand” [4]. The later one stands for liquefiable sand, while the first ones’ name is an early declaration that clays and rocks are defined in the same manner in the model building process. The default materials have fixed properties that cannot be altered, while the user-defined demand the definition of a series of parameters.

Shear wave velocity profile

In Cyclic1D all “Clay/Rock” materials have a constant shear wave velocity while “U-Sand” and “U-LiqSand” materials’ shear wave velocity varies according to the following rule: $V_s = V_{sr} (p / p_r)^{n/2}$ where V_{sr} is the reference shear wave velocity, $p = ⅓ \text{ tr } [\sigma'] = ⅓ (\sigma'_{11} + \sigma'_{22} + \sigma'_{33})$ is the effective mean confinement ($p = ⅔\sigma'_{11}$ if $\kappa_0 = 0.5$), p_r is the reference mean confinement and n is the confinement dependence coefficient [4].

When using user-defined materials, V_{sr} and p_r are user-defined constants and their values should be as close as possible to the mean values of the soil layer they refer to. “U-Sand” and “U-LiqSand” materials can have a virtually constant shear wave velocity throughout their layer depth by having their confinement dependence coefficient “n” equal to 0.01 or 0.001.

Constitutional law

Materials in Cyclic1D follow the constitutional law depicted in Figure 1. On the first yield surface the shear modulus is equal to $G = G_{max} = \rho \times V_s^2$ (ρ is the mass density of the material), which means that the elastic behavior of a material is solely defined by its shear wave velocity and its mass density. Sand materials (“U-Sands” and “U-LiqSands”) have a shear strength equal to $\tau_{max} = c + p \sin\varphi$ and it is recommended that the cohesion c is set to 1 or 2 kPa at least [1, 4, 6]. The developed constitutive model is based on the framework of multi-surface plasticity, where a number of similar yield surfaces with a common apex and different sizes, form the hardening zone. The outmost surface is the envelope of peak shear strength – failure envelope.

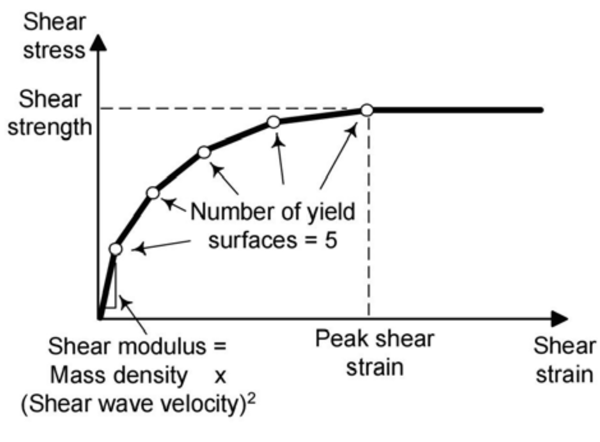


Figure 1 Constitutional law of materials in Cyclic1D [4]

Pore pressure effects

When using the default cohesionless materials and the "U-LiqSands" pore-pressure effects, such as liquefaction, are calculated during analysis. Parameters that define the behavior of such a material under pore pressure build-up are those listed in Table 1, the permeability coefficient and the dilation angle. The constitutional law of the above materials is shown in Figure 2.

Dilation angle divides the domain of shear-induced volume contraction response from that of volume dilation (phase transformation or PT surface). The dilative phase is the part of the diagram between points 2 and 3 (Figure 2). Contraction behavior is removed completely by setting this angle to zero. Dilation behavior is removed completely by setting this angle larger than the friction angle [4].

Table 1 Pore pressure effects suggested parameter values

	Cohesionless Material	
Parameter	Very Loose	Very Dense
Contraction #1	0.3	0
Contraction #2	0.2	0.6
Dilation #1	0	0.6
Dilation #2	10	10
Liquefaction	0.025	0

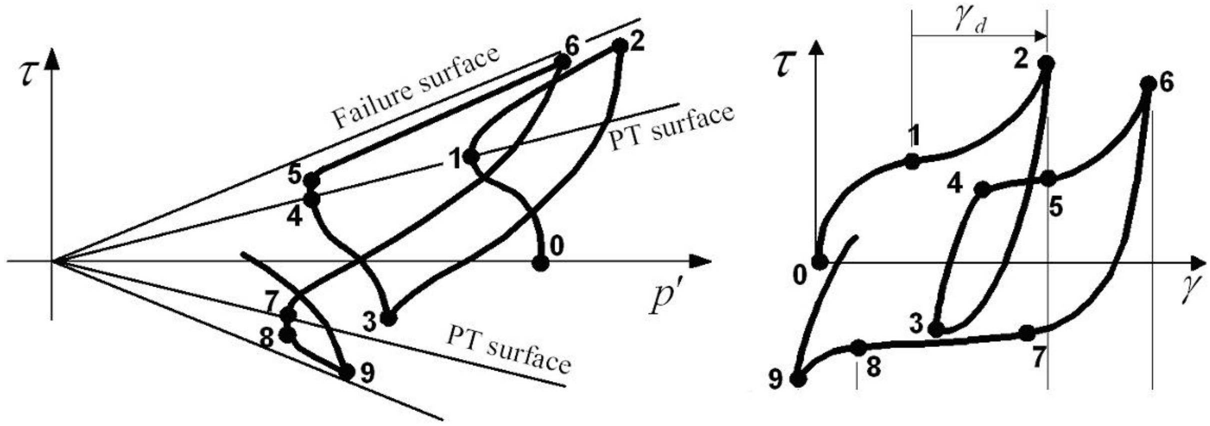


Figure 2 Constitutional law of materials with pore pressure effects [1, 4, 6]

By increasing the difference $\varphi_{friction} - \varphi_{dilation}$ the material gains extra strength against liquefaction. Bolton (1986) [2] has suggested the following way to estimate the above difference:
$\varphi_{friction} - \varphi_{dilation} = 5I_R$, $I_R = D_r (Q - \ln p) - 1$, D_r is the relative density of the material
$D_r = (e_{max} - e)/ (e_{max} - e_{min})$, p is the effective mean confinement, for Q see Table 2.

Table 2 Values of parameter Q as suggested by Bolton (1986) [2]

Q	Grain Type
10	Quartz and feldpsar
8	Limestone
7	Anthracite
5	Chalk

Contraction parameter 1 dictates the rate of pore pressure build up under undrained conditions. Contraction parameter 2 reflects the effect of overburden pressure on contraction behavior. Dilation parameter 1 dictates the rate of volume expansion (or reduction of pore pressure). Dilation parameter 2 reflects the effect of accumulated shear strain on dilation behavior. Finally, the liquefaction parameter dictates the extent of shear strain accumulation [4].

Damping

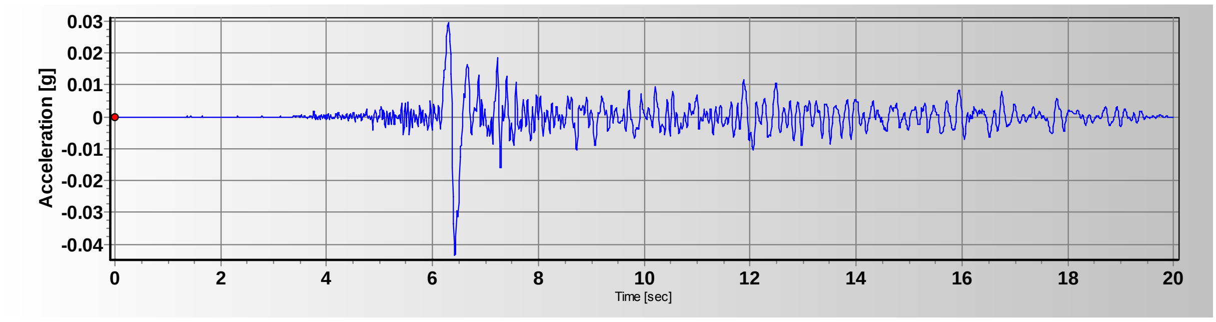
The software uses Rayleigh damping for its analyses [4]. A 5% damping ratio is applied as common practice but if large shear strain levels are expected a higher ratio may be used.

CALIBRATION AT LOW ACCELERATION AMPLITUDES

In this chapter the linear elastic part of the tested code is checked. For calibration needs of the examined 1D code at low acceleration amplitudes, Melloland array site was selected, because of the installed down-hole array where acceleration recordings <0.1g exist at both, bottom and surface of the soil column. Moreover, all necessary geotechnical and dynamic properties of the soil column at Melloland array are well documented [7].
Bearing in mind that all necessary data exist for analysis purposes, the following procedure is proposed in order to calibrate code results in the linear elastic domain:

a) Choose the appropriate V_S values of the soil column. Those values must be available in the way they are entered in the examined code, b) choice of appropriate strength soil parameters, and c) transient analysis using as base input motion the low amplitude accelerogram registered at the bottom of the soil column [8], in attempt to simulate the elastic soil properties (shear wave velocity, mass density), in the best way, by comparison of registered and theoretical surface acceleration time series, Fourier transform amplitude acceleration, amplification function between surface and bottom Fourier transform amplitude accelerations.

For the needs of linear elastic approximation the recorded PGA=0.043g, whereas the theoretical analysis PGA=0.04g. Figure 3 represents recorded acceleration time series at depths of 0 and 100m.



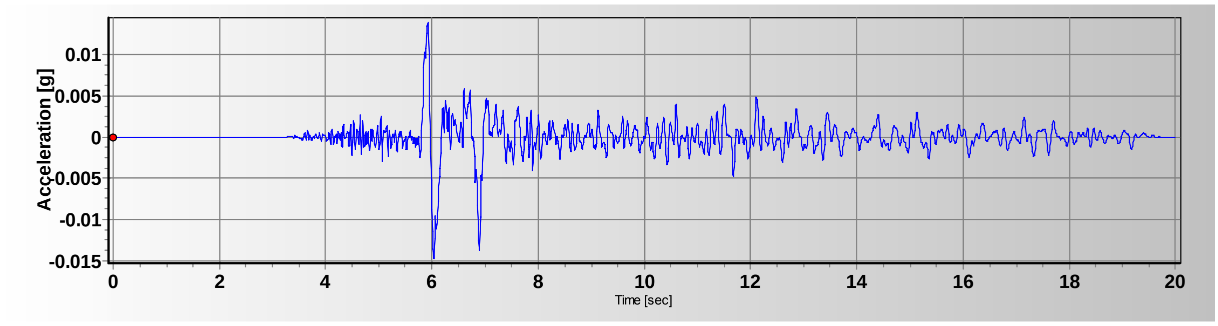


Figure 3 Acceleration recording - PGA=0.043g, 9th of April 2000 (depths: 0 and 100m @270°)

Model profile and model material properties are presented in Tables 3, 4 and 5, whereas in figure 4 shear wave velocity profile is depicted, as measured in situ (green line) [7] and as approximated – calculated by the examined code (red line).

Figures 5 and 6 depict direct comparison between registered and calculated Fourier transform amplitude accelerations and spectral amplification of accelerations of surface over bottom of the soil column. The conducted analysis is linear elastic due to the low amplitude acceleration input motion at the bottom of the soil column. Theoretical results, calculated via Cyclic1D code, in terms of PGA values, acceleration time series, spectral parameters (FFT acceleration amplitudes and spectral amplifications), are very close to the one registered by a weak ground motion. It is therefore concluded that the linear elastic approximation, as modeled by the examined code, is considered successful and reliable. However, analysis with another weak motion of significantly different frequency content, has turned out to be less convincing as comparison of registered and calculated results were not that close. It is authors' belief that the frequency content of the weak input ground motion might influence in a non-uniform way soil layers of the column and by consequence, seismic response is strongly subjected to this effect.

Table 3 Model profile

Soil Column Height (m)	100	Soil Descriptions	Elements
Number of Elements:	100	U-Sand #1	1-5
Water Table Depth (m)	2	U-Clay #1	6-11
Inclination Angle (degree)	0	U-Sand #2	12-17
Bedrock	Rigid	U-Clay #2	18-23
		U-Sand #3	24-35
		U-Clay #3	36-47
		U-Sand #4	48-59
		U-Clay #4	60-79
		U-Clay #5	80-100

Table 4 Model material properties (1/2)

	U-Clay #1	U-Clay #2	U-Clay #3	U-Clay #4	U-Clay #5
ρ (kg/m^3)	2000	2000	2000	2000	2000
V_S (m/sec)	160	280	230	320	360
p_r (kPa)	-	-	-	-	-
n	-	-	-	-	-
K_0	0.5	0.5	0.5	0.5	0.5
c (kPa)	40	50	40	55	75
φ (degrees)	-	-	-	-	-
γ_{peak} (%)	3	3	3	3	3
Number of yield surfaces	20	20	20	20	20

Table 5 Model material properties (2/2)

	U-Sand #1	U-Sand #2	U-Sand #3	U-Sand #4
ρ (kg/m^3)	2100	2100	2100	21200
V_S (m/sec) (m/sec)	130	240	270	280
p_r (kPa)	29	122	264	464
n	0.5	0.5	0.5	0.5
K_0	0.5	0.5	0.5	0.5
c (kPa)	2	2	2	2
φ (degrees)	35	35	35	35
γ_{peak} (%)	2	2	2	2
Number of yield surfaces	20	20	20	20

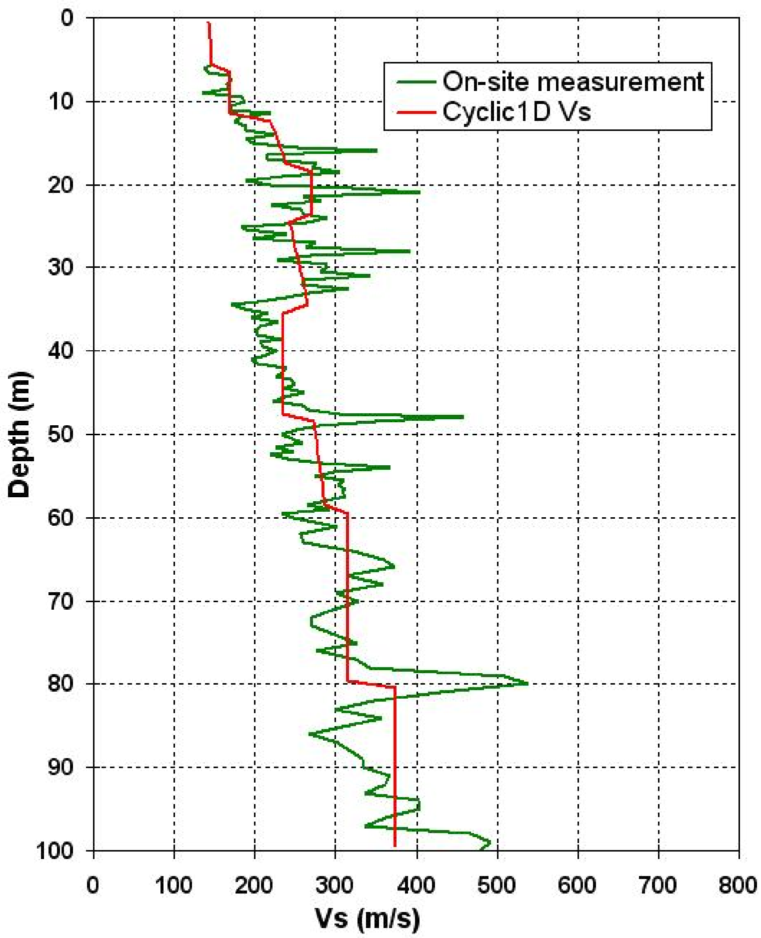


Figure 4 Shear wave velocity as measured on-site [http://geoinfo.usc.edu/rosrine/] [7] and as calculated by Cycilc1D

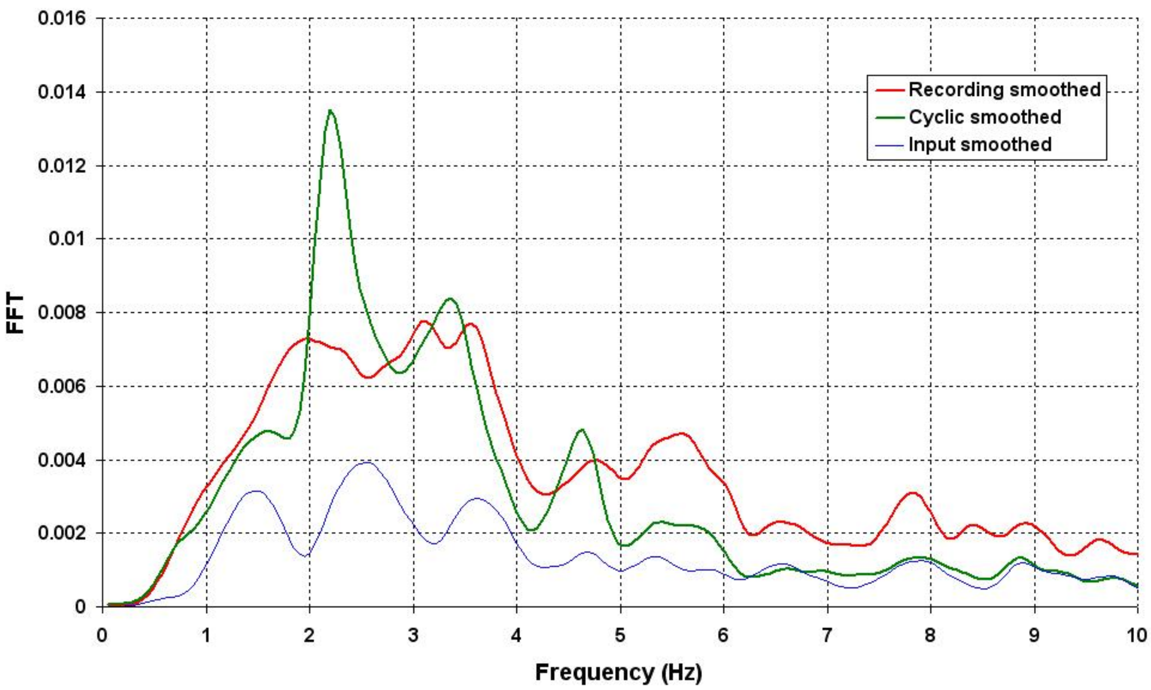


Figure 5 Fourier transform amplitude of acceleration (m/sec^2)

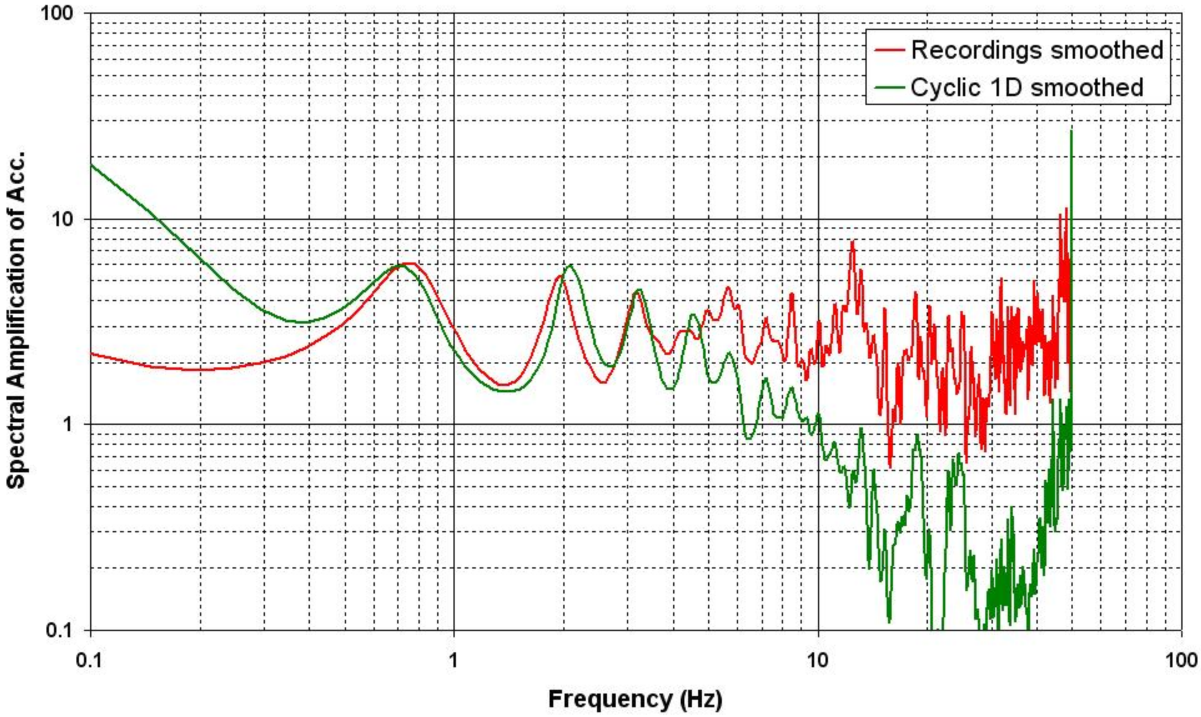


Figure 6 Spectral amplification of accelerations between surface and bottom of the borehole from recorded accelerograms [8] and theoretical non-linear calculations

CALIBRATION AT LARGE ACCELERATION AMPLITUDES – LIQUEFACTION SIMULATION

At Port Island extensive liquefaction was observed after the earthquake of Kobe at the 17th of January '95. From that earthquake there are 8 accelerograms available (at 0, 16, 32 and 83m depth), 4 per direction (000^o and 270^o) [9], from which those on Figure 7 are used to perform a non-linear and an equivalent linear analysis. What should be pointed out about them is the significant reduction of acceleration between the surface and the recording point at 83m of depth. This should be attributed to the liquefaction of the loose sand/gravel layer at the top of the soil profile.

The recorded ground movement characteristics and those resulting from the analysis (Figures 10-14) are quite close. From 15 to 20m of depth, effective confinement is significantly reduced approaching zero value for a number of cycles of loading - unloading, shear modulus is very low to almost zero, resulting to shear strains more than 4%. In this way, liquefaction is simulated through the analysis.

Another effort to simulate the response of Port Island's soil profile under Kobe's earthquake was made by means of an Equivalent Linear Analysis using the application "EERA" [5]. The comparison between the computed motion of this method and that of the non-linear analysis can be seen in Figures 10 to14. The ELA fails to simulate the higher frequencies of the motion but peak ground acceleration and shear strains seem to be of the same magnitude with the registered ones.

Equivalent linear modeling seems to produce a systematic shift in resonant peaks towards lower frequencies, as the level of strain increases and also to predict a more dramatic reduction in amplification factors at higher frequencies [Nonlinear Site Response - SCEC/PEER Seminar and Workshop Report] [3]. This is in accordance with the results presented in Figures 10 to 14.

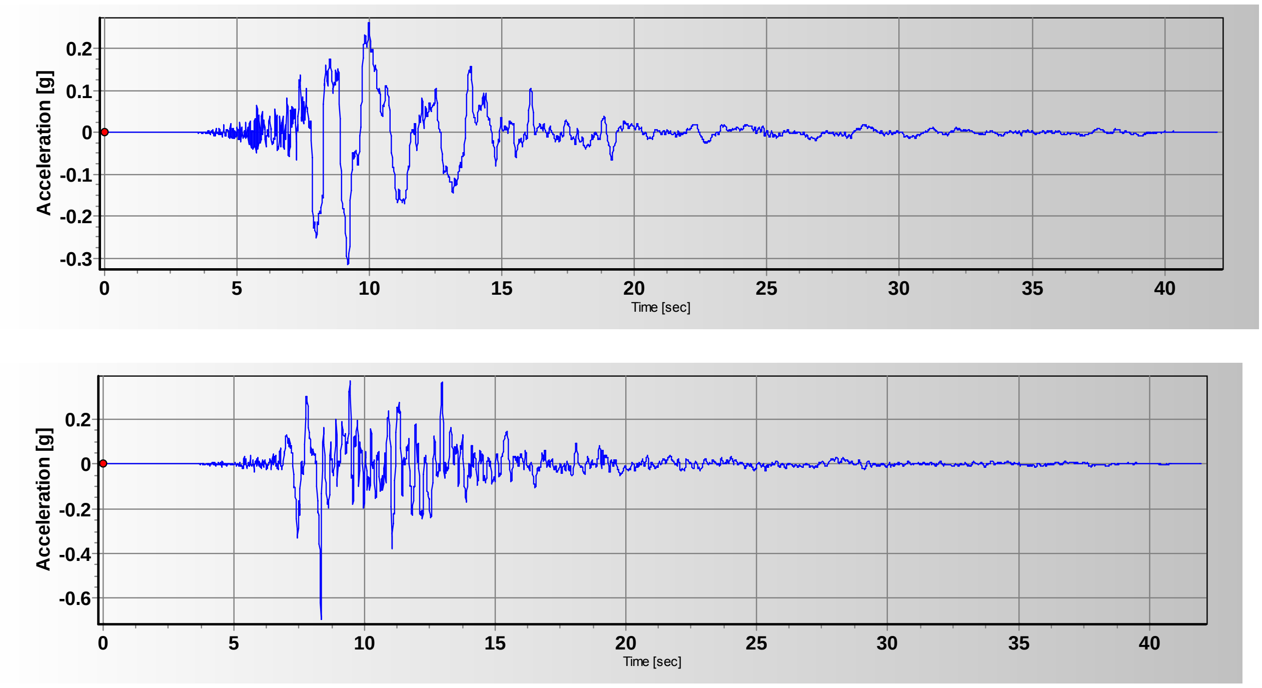


Figure 7 Acceleration recordings - Kobe 01/16/95 (depths: 0 and 83m @ 0°) [9]

Table 6 Model profile

Soil Column Height (m)	83	Soil Descriptions	Elements
Number of Elements:	90	U-LiqSand #6	1-5
Water Table Depth (m)	4	U-LiqSand #7	6-13
Inclination Angle (degree)	0	U-LiqSand #8	14-21
Bedrock	Rigid	U-Clay #1	22-29
		U-Sand #4	30-36
		U-Sand #5	37-66
		U-Clay #2	67-85
		U-Clay #3	86-90

Table 7 Model Material Properties (1/2)

	U-LiqSand #6	U-LiqSand #7	U-LiqSand #8	U-Clay #1
ρ (kg/m^3)	2100	2150	2200	1800
V_S (m/sec) (m/sec)	170	210	210	180
p_r (kPa)	33	88	142	-
n	0.01	0.01	0.01	-
K_0	0.5	0.5	0.5	0.5
c (kPa)	1	1	1	100
φ (degrees)	31	33	34	-
γ_{peak} (%)	3	3	3	5
Number of yield surfaces	20	20	20	20
$\varphi_{dilation}$	28	28	33	-
Contraction Parameter 1	0.25	0.2	0.18	-
Contraction Parameter 2	0.3	0.3	0.3	-
Dilation Parameter 1	0.1	0.2	0.25	-
Dilation Parameter 2	10	10	10	-
Liquefaction Parameter 1	0.02	0.015	0.015	-
k (m/sec)	0.01	0.01	0.00001	-

Table 8 Model Material Properties (2/2)

	U-Sand #4	U-Sand #5	U-Clay #2	U-Clay #3
ρ (kg/m^3)	2120	2200	2000	1800
V_S (m/sec) (m/sec)	240	350	300	320
p_r (kPa)	237	365	-	-
n	0.01	0.01	-	-
K_0	0.5	0.5	0.5	0.5
c (kPa)	1	1	300	300
φ (degrees)	40	40	-	-
γ_{peak} (%)	3	3	5	5
Number of yield surfaces	20	20	20	20

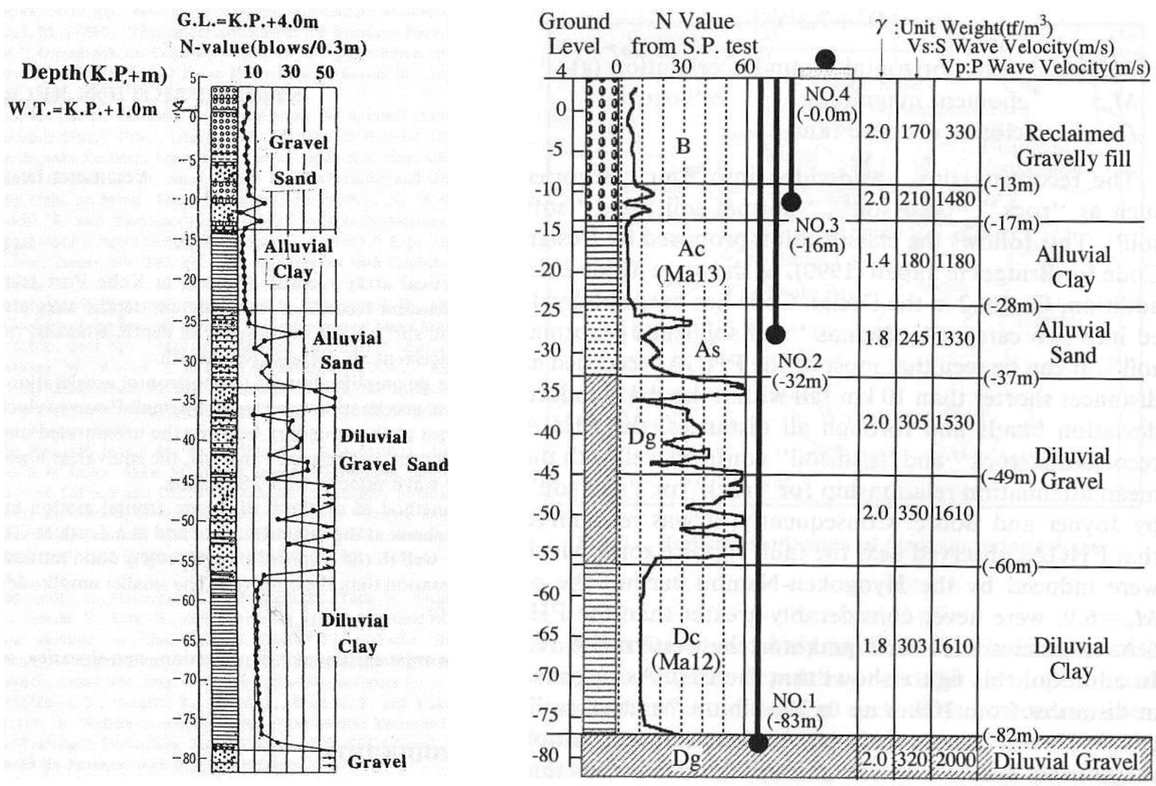


Figure 8 Geotechnical and dynamic properties of soil layers at Port Island [10]

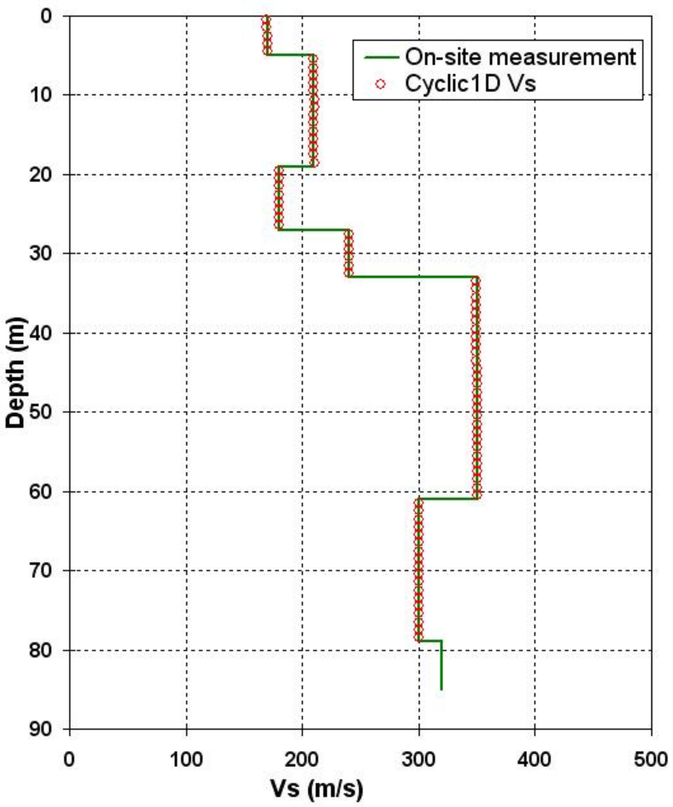


Figure 9 Shear wave velocity as measured on-site [http://cee.ea.ucla.edu/faculty/jstewart/groundmotions/PEER2G02/index.htm] [9] and as calculated by Cyclic 1D

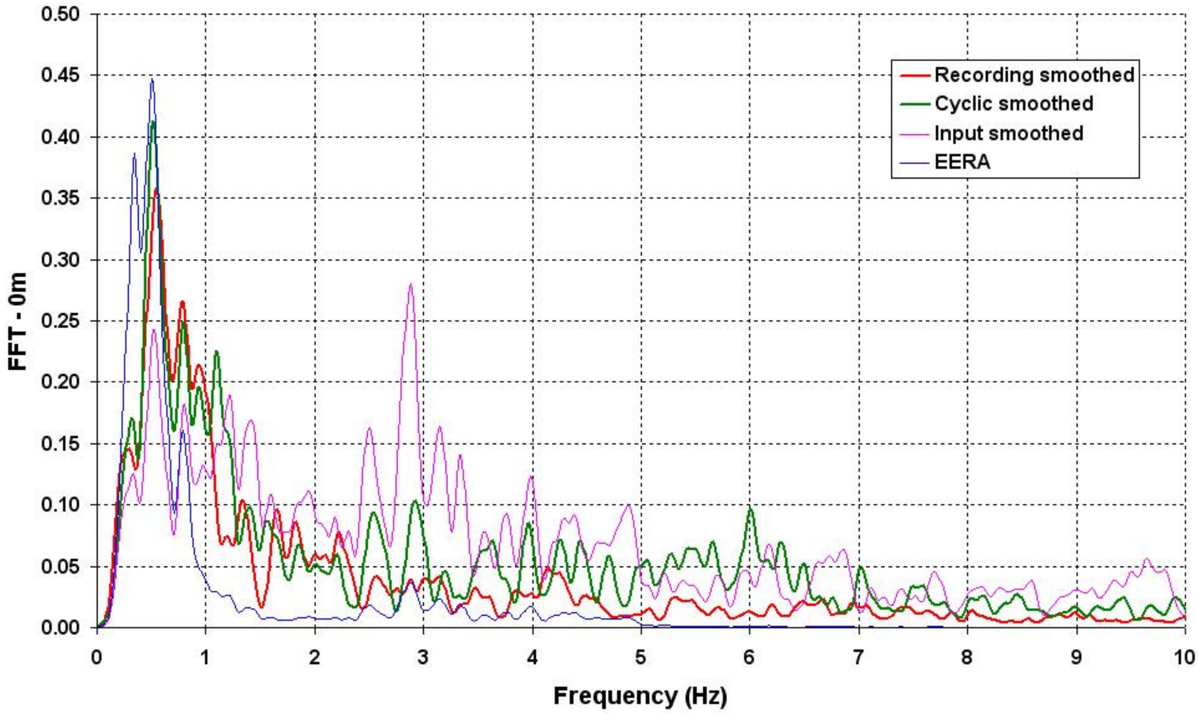


Figure 10 Fourier transform amplitude of acceleration of theoretical calculations via “non-linear” and “equivalent linear” approaches, as well as, registered accelerations (m/sec^2) at surface (0m)

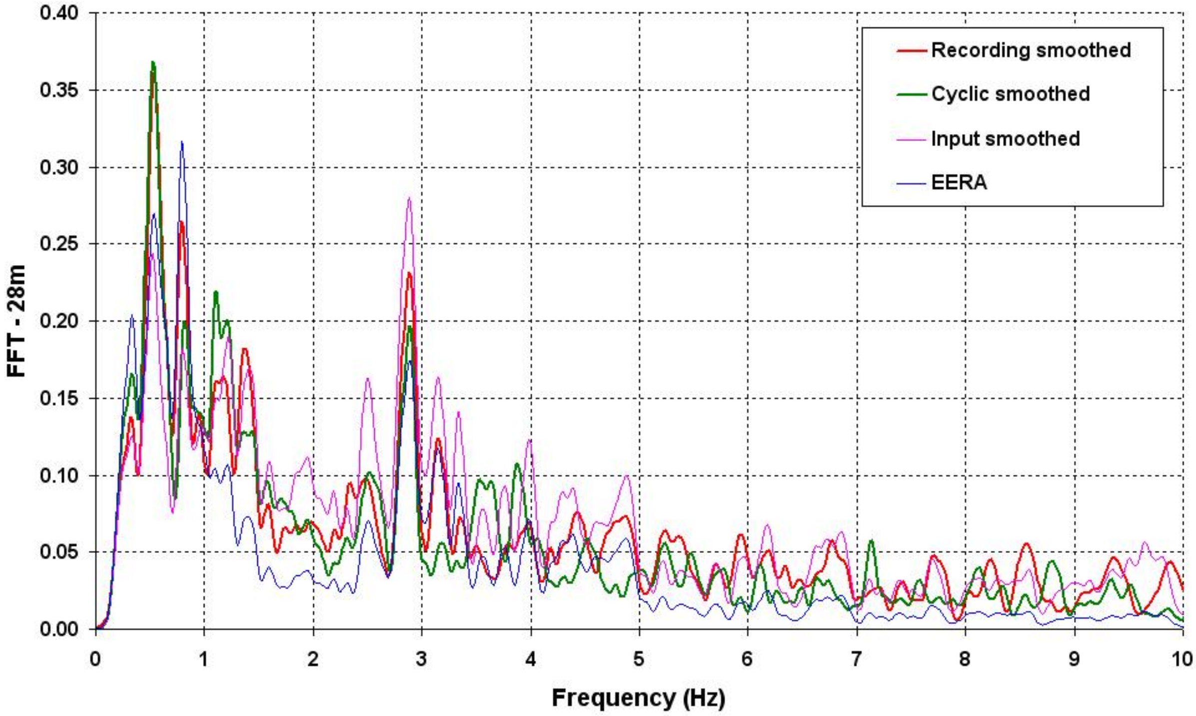


Figure 11 Fourier transform amplitude of acceleration of theoretical calculations via "non-linear" and "equivalent linear" approaches, as well as, registered accelerations (m/sec^2) at a depth of 28m

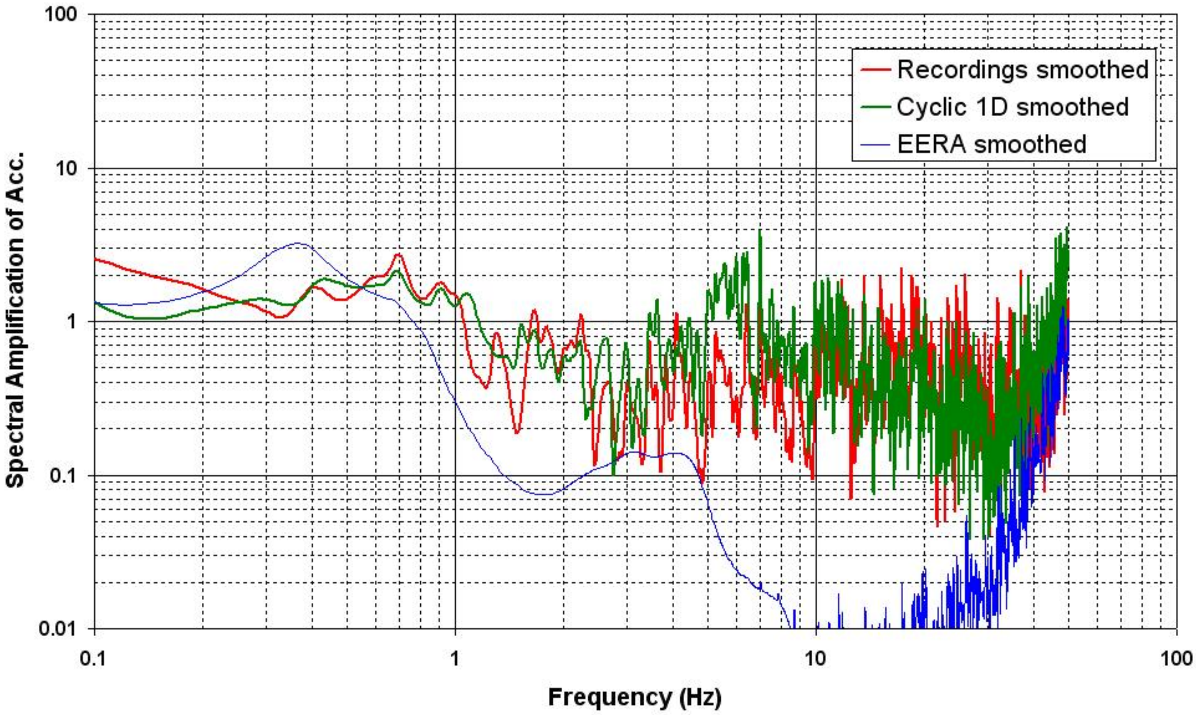


Figure 12 Spectral amplification of surface acceleration over acceleration at depth of 83m: straightforward comparison between theoretical results of ELA and non linear analyses versus registered accelerograms at surface and the bottom of the borehole (83m)

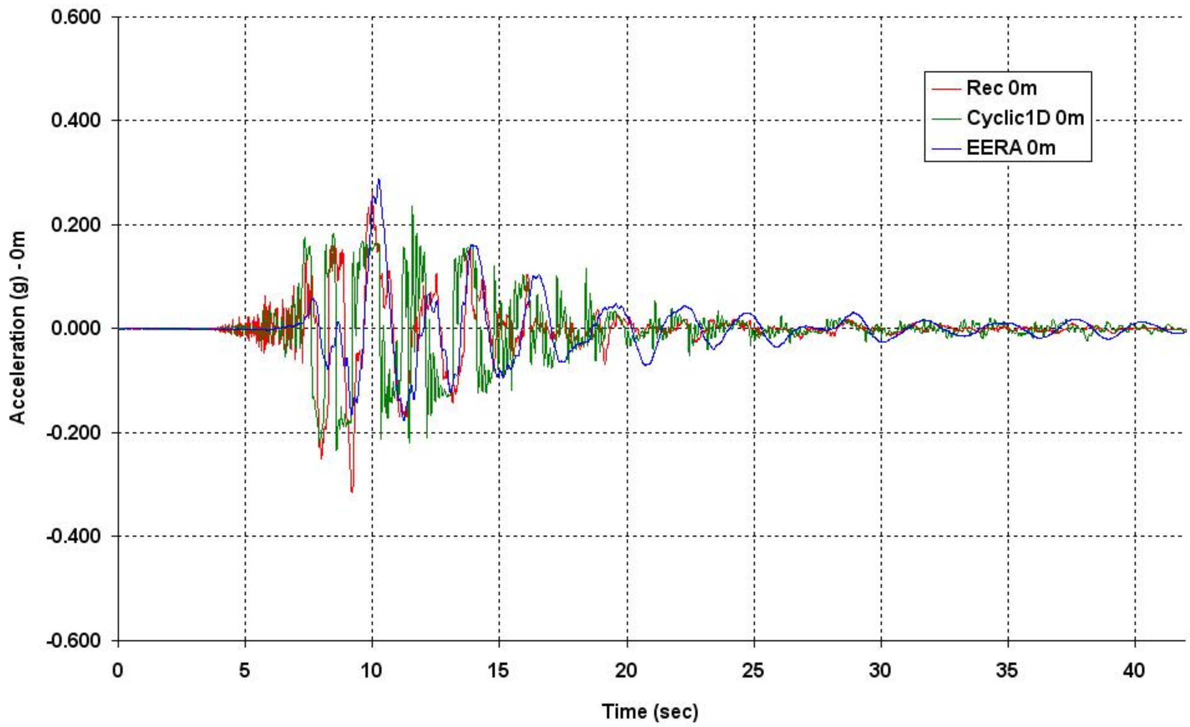


Figure 13 Time-history of acceleration at the ground surface as recorded and as computed (equivalent linear and non-linear analyses)

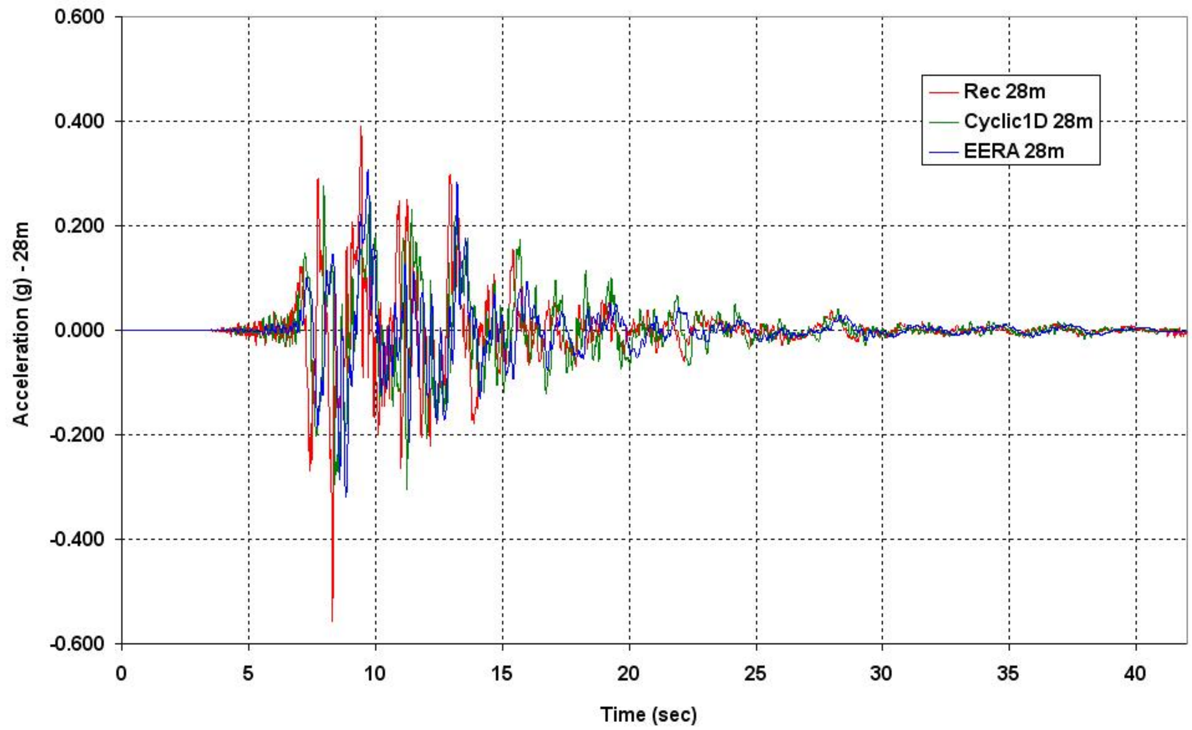


Figure 14 Time-history of acceleration at 28m depth, as recorded and as computed (equivalent linear and non-linear analyses)

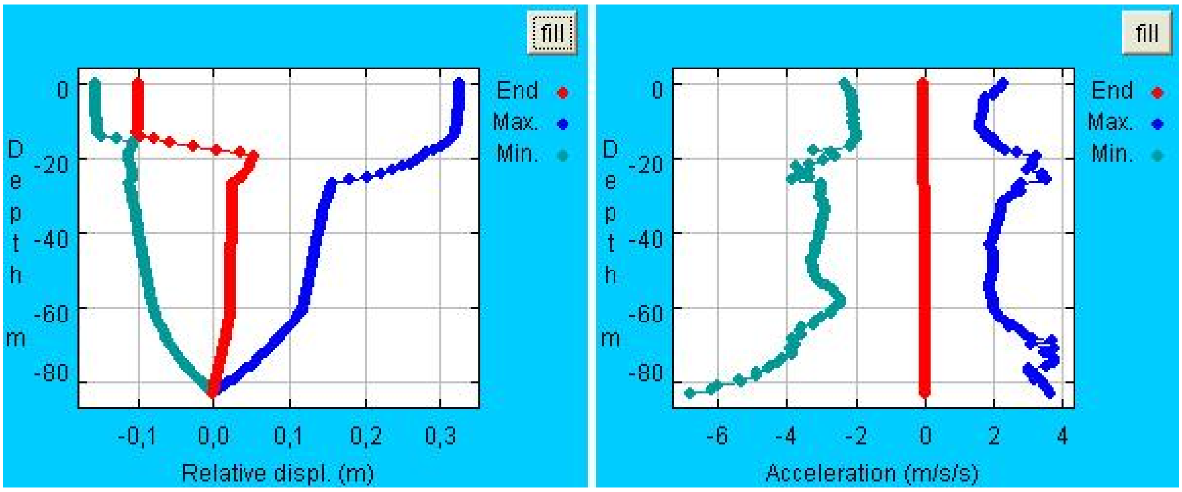


Figure 15 Variation of relative horizontal displacements and accelerations versus depth

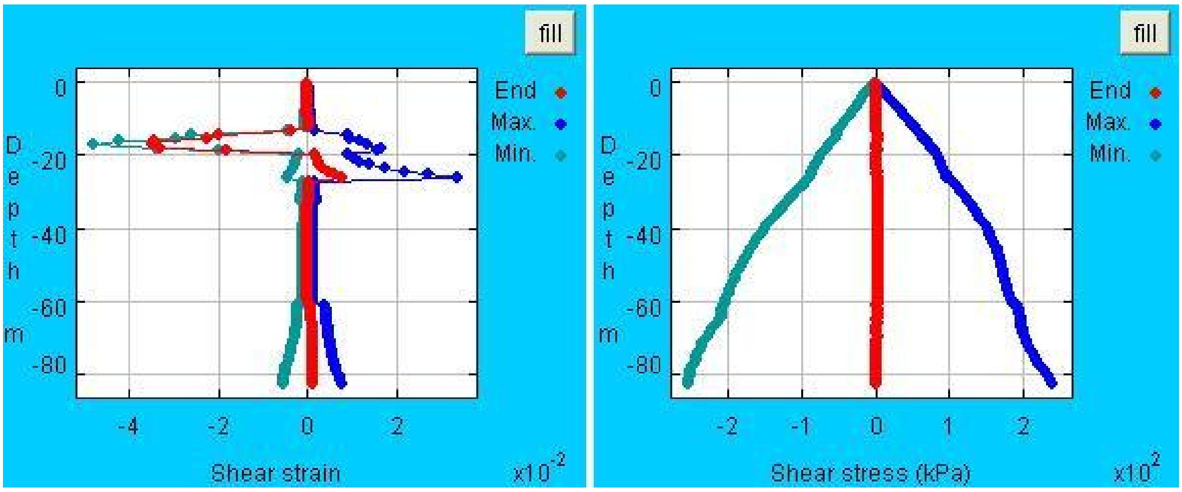


Figure 16 Variation of shear strain and shear stress versus depth

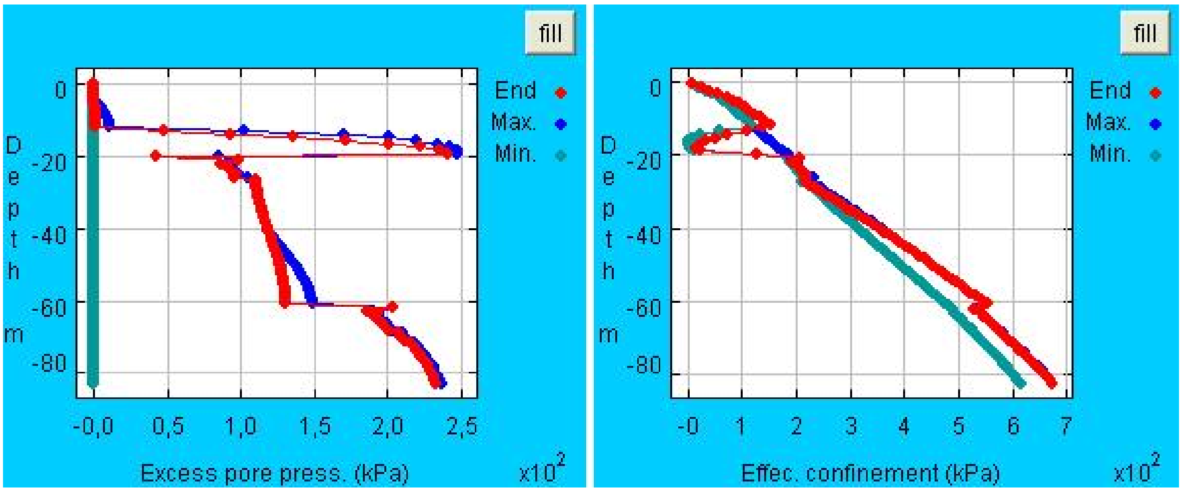


Figure 17 Variation of pore pressure build-up and mean effective confinement versus depth

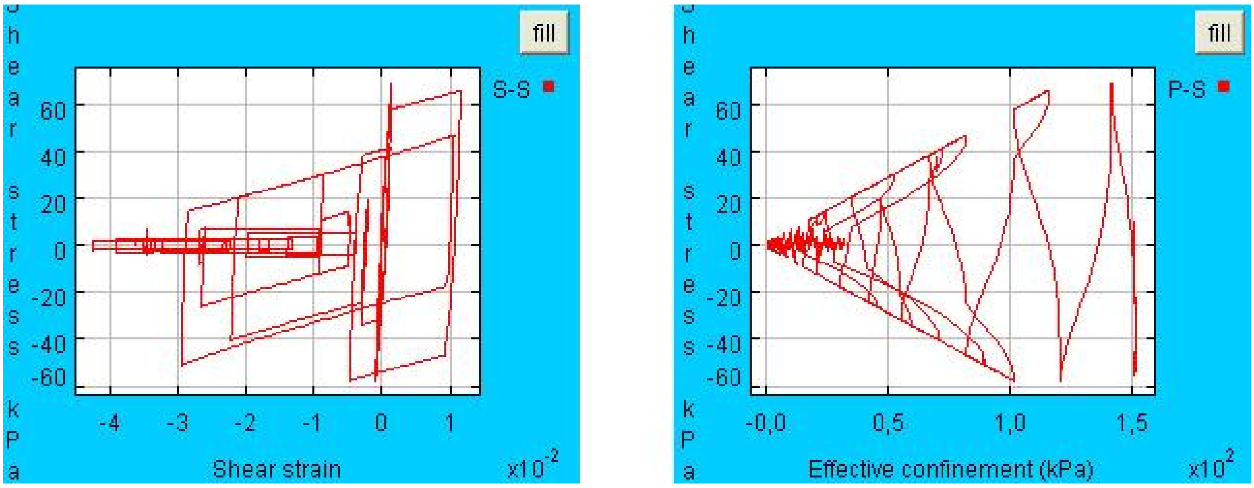


Figure 18 Variation of shear stress versus shear strain at the depth of 16.1m. Mean effective stress and shear modulus result in zero values for a considerable number of cycles loading – unloading (indices of liquefaction). Both graphs are characteristic of the soil layer between 15 and 20m

In figures 10 and 11, the frequency content of registered accelerograms at surface and at a depth of 28m, is well approximated by non-linear analyses conducted by Cyclic1D. On the opposite, equivalent linear analysis exhibited considerable differences for all frequencies >0.8Hz, whereas amplitude of calculated Fourier transform at free surface is almost 30% higher than registered.

Similarities and coincidence of theoretical results coming from non-linear analyses, with registered accelerograms, are equally good for the depth of 28m, as opposed to the equivalent linear approach, which fails to be accurate and reliable for high-level accelerations. In figure 11, spectral amplification of the surface over the bottom of the borehole, at 83m of depth, for registered and computed results via non-linear approach is proved as very successful, whilst equivalent linear approach fails to reproduce the registered spectral amplification.

CONCLUSIONS

The theoretical results of ground response calculated for low and high-level accelerations have been tested with real earthquake results. When peak ground accelerations less than 0.1g are successfully reproduced by Cyclic 1D software, as well as spectral characteristics, the linear elastic part of the constitutive model is checked. The most difficult task of this work was the correct simulation of liquefiable soil layers response bottom excited at high acceleration levels (>0.3g) and the calibration of the geotechnical parameters that determine non-linear behaviour of soils prone to liquefy or undergo extreme conditions, such as, lateral spreading. Linear and non-linear response of soils subjected at low and high accelerations has been thoroughly tested and it has been proved that simulation of soil behaviour via soil parameters described by the examined software was successful. Non-linear behaviour was close enough to reality predicted by the constitutive law of Cyclic 1D for soils that did liquefy during Kobe, 1995 earthquake. Frequency content, peak values, accelerations time series, spectral amplification functions, shear stresses and shear strains developed with depth, pore pressure build-up at prone to liquefy zones, intense decrease of mean effective stresses together with shear modulus values tending to zero, have been calculated as characteristic indices to define a liquefying zone manifested during strong ground motions.

On the opposite, the equivalent linear approach, proved to be not successful in prediction of peak and spectral values when nonlinearities prevail soil behaviour.

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