



VALIDATION BY CENTRIFUGE TESTING OF NUMERICAL SIMULATIONS FOR SOIL-FOUNDATION-STRUCTURE SYSTEMS

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SUMMARY

In this paper, an attempt is made to validate the numerical response of a soil-structure system with observations from physical modeling, using data from high gravity centrifuge testing that have been carried out on portal frames founded on uniform soil strata. The preliminary conclusions drawn by the comparative evaluation of numerical and physical testing are related to: a) the identification of the assumptions and system modifications required to achieve an acceptable level of matching between numerical and experimental results, b) an overview of the numerical results in terms and the dynamic response of soil-structure systems

INTRODUCTION

Soil-structure interaction phenomena have been proven to be of major importance in numerous earthquake cases, thus have attracted significant scientific attention. Several approaches, tools and methodologies have been developed including sub-structuring, finite element and finite difference codes as well as hybrid methods. This phenomenon is very complex and its beneficial or detrimental effect on the dynamic response of the system depends on a series of parameters such as the intensity of ground motion, the dominant wavelengths, the angle of incidence of the seismic waves, the stratigraphy, the stiffness and damping of soil as well as the size, geometry, stiffness, slenderness and dynamic characteristics of the structure. Despite the extensive scientific work therefore, the complexity and the ensuing large number of parameters involved, make it difficult to derive conclusive recommendations for practical design.

Nevertheless, case studies from experimental data on instrumented soil-structure systems subjected to natural or man-made excitations (i.e. Euroseis-Test Site <http://euroseis.civil.auth.gr>) are rather limited, thus preventing the quantification and especially the generalization of conclusions drawn analytically and

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the establishment of a confidence level on numerical calculations, On the other hand, physical experiments on scaled geotechnical structures provide a useful alternative to acquire well constrained sets of data. Centrifuge testing in particular, has the advantage of being able to reproduce the initial stress field of soil models of significant depth since it is conducted in a high gravitational environment. The inherent difficulty though, in matching numerical analysis results with physical experiments of scaled systems is related to the understanding of their differences in terms of scaling laws, boundary conditions and soil non-linearity. It is therefore the scope of this paper to perform a preliminary numerical simulation of centrifuge testing on a soil-structure system, in order to identify the assumptions framework of the adaptive procedure that can be followed analytically to achieve an acceptable level of agreement between numerical and physical modeling. Having obtained the required level of confidence, the study of the dynamic response of the complete soil-foundation-structure system can then be studied.

DYNAMIC CENTRIFUGE MODELLING

Principles of centrifuge modelling

The principles of centrifuge modelling are well established. By increasing apparent self-weight body forces, a reduced scale geotechnical model may be made to replicate the stress-strain behaviour of a larger prototype. Centrifuging causes the required gravity increase to achieve this. A discussion of scaling laws may be found in many sources, e.g. Schofield [1].

Cambridge centrifuge apparatus

Centrifuge tests were carried out on the Cambridge 10m diameter beam centrifuge. The Cambridge geotechnical beam centrifuge was designed by Philip Turner and built locally during the early 1970s. It is capable of almost 175 g-tonnes and has a radius of 4.125 m to the base of the swinging platform. Full specifications and design issues are described by Turner in a document on the Cambridge Geotechnical group website: http://www-g.eng.cam.ac.uk/125/achievements/geotech_intro.htm .

The current earthquake actuator used at Cambridge is the stored angular momentum actuator (SAM). This operates by drawing energy from flywheels motored to a set speed. A hydraulically operated clutch connects the model to the flywheels for the required shaking duration. Shaking is delivered at the speed of the motor (tone-burst), or the motor may be stopped with the clutch on to produce a shake that decreases in frequency over time as the flywheels decelerate. Detailed operation is described in Madabhushi et al [2]. Shaking is applied to the model in one direction, along the long axis of the box.

The other major issue is satisfying boundary conditions. The container used for this research is the larger “equivalent shear beam” (ESB) box described by Brennan and Madabhushi [3]. Consisting of layers of aluminium alloy and hard rubber, the principle is that under earthquake motion the box vibration is similar to that of free-field soil, so little relative force is generated at the boundary. For these tests, a “shear sheet”, a thin aluminium alloy sheet with sand glued on, was used at each end of the container to maintain complementary shear stresses during shaking. It is accepted that the middle-third of the container does not experience significant boundary effects.

Experimental program

This paper describes results from 3 earthquakes applied to a single centrifuge model, selected as representative of part of a wider experimental program (refer to acknowledgement). The model in question consists of a dense ($e = 0.67$, $D_r \approx 80\%$) level bed of Hostun S28 sand. The layer is 340 mm thick, corresponding to 17 m at prototype scale when the full 50-g is applied. A model structure sits at the surface, with nominal embedment to prevent sliding when the model is being transported around the lab. This consists of two aluminium floors joined by two thin aluminium alloy walls whose thickness and height have been selected to produce a single degree of freedom structure with the desired natural frequency. The actual natural frequency was 77 Hz (1.54 Hz prototype) and the structure behaved as a single degree of freedom oscillator.

Accelerations are measured within the soil and on the structure by miniature accelerometers manufactured by D.J. Birchall in the UK. These have an excellent frequency response above about 4 Hz, but low-frequency drift means care must be taken if signals are to be integrated to provide velocities/displacements. Surface displacements and structural movements are measured by LVDTs, contact transducers whose high frequency response (>60 Hz) is poor.

Figure 1a shows a schematic cross-section of the model layout, with the accelerometers to be discussed in this paper shown as rectangles. These are oriented to measure acceleration along the long axis of the symbol. The structure and the laminations in the box are also shown in this figure. Figure 1b shows the model prior to loading on the centrifuge, with a similar (albeit 2 degree-of-freedom) structure.

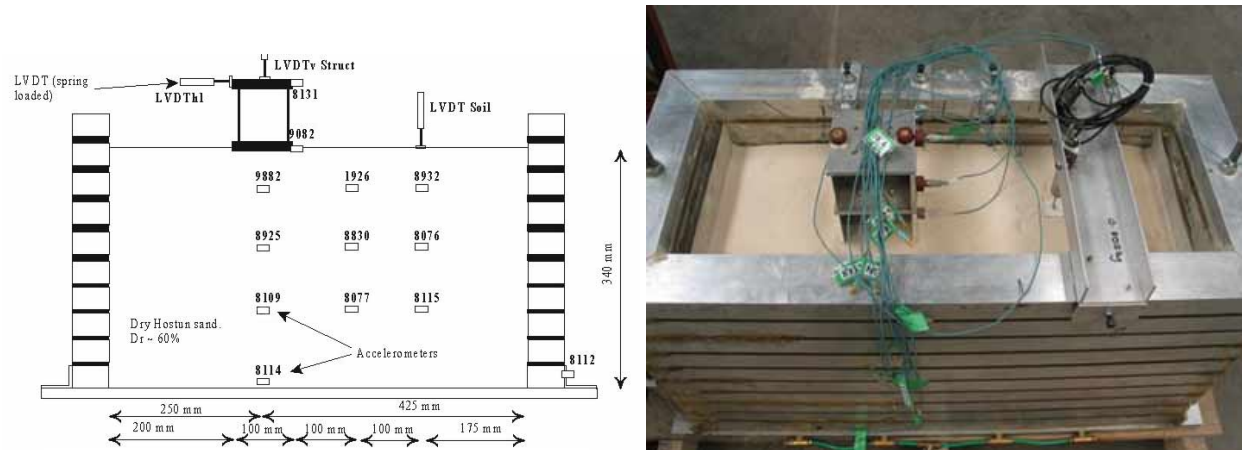


Figure 1a-1b. Schematic cross-section and centrifuge model prior to testing

NUMERICAL MODELLING

Numerical code aspects

Numerical modelling of the centrifuge experiments is performed using 1D and 2D analysis implementing two finite element codes. Finite element code ADINA (Automatic Dynamic Incremental Nonlinear Analysis) is used for two-dimensional analysis (ADINA Users manual [4]) being able to perform linear and non-linear static and dynamic analysis, acoustic, contact and crash analysis as well as complex fluid flows with structural interactions. Transient analysis can be performed using mode superposition, explicit or implicit time integration.

Additionally, one-dimensional analysis is performed with FE code Cyberquake (Modaressi and Foerster [5]), a well known 1D soil-response-oriented FE code that accounts for the linear elastic, equivalent linear or elastoplastic soil behaviour. Within the framework of the research presented herein, two-dimensional linear elastic analysis with ADINA and one-dimensional equivalent linear analysis with Cyberquake are performed, for simulating the centrifuge experiments. Proposed methodology and calibration as well as limitations of each analysis type are discussed in detail in the following paragraphs.

Validation of finite element representation of wave propagation

In order to perform two-dimensional wave propagation analysis in a general-purpose FE code such as the one selected for this study, it is essential to establish a certain degree of confidence in the ability of the code itself to represent site response under earthquake motion. Thus, a detailed validation scheme was employed including the comparison of FE wave propagation results with numerical solutions from soil-modelling oriented numerical codes. The soil profile of Euroseis-Test site mentioned previously was selected in the final stage of the validation procedure. Euroseis-Test is a well documented test site (Pitilakis et al [6]), funded by several European research programs, that is located at the epicentral area to provide detailed information on soil layering and behaviour; G-D-gamma curves are also available. A

simplified soil profile comprising of 8 soil layers was selected as a study case to validate the finite element code in wave propagation problems (Figure 2).

Several simple and complicated bedrock motions were used to validate the 2D analysis of the FE code. In the validation procedure the aforementioned 1D soil-response-oriented FE code Cyberquake was employed. Despite several variations such as 1D versus 2D analysis, different numerical background or damping implementation, good matching was observed between the results obtained by the two codes; i.e. motion amplification as well as dispersion were similarly reproduced (Figure 2).

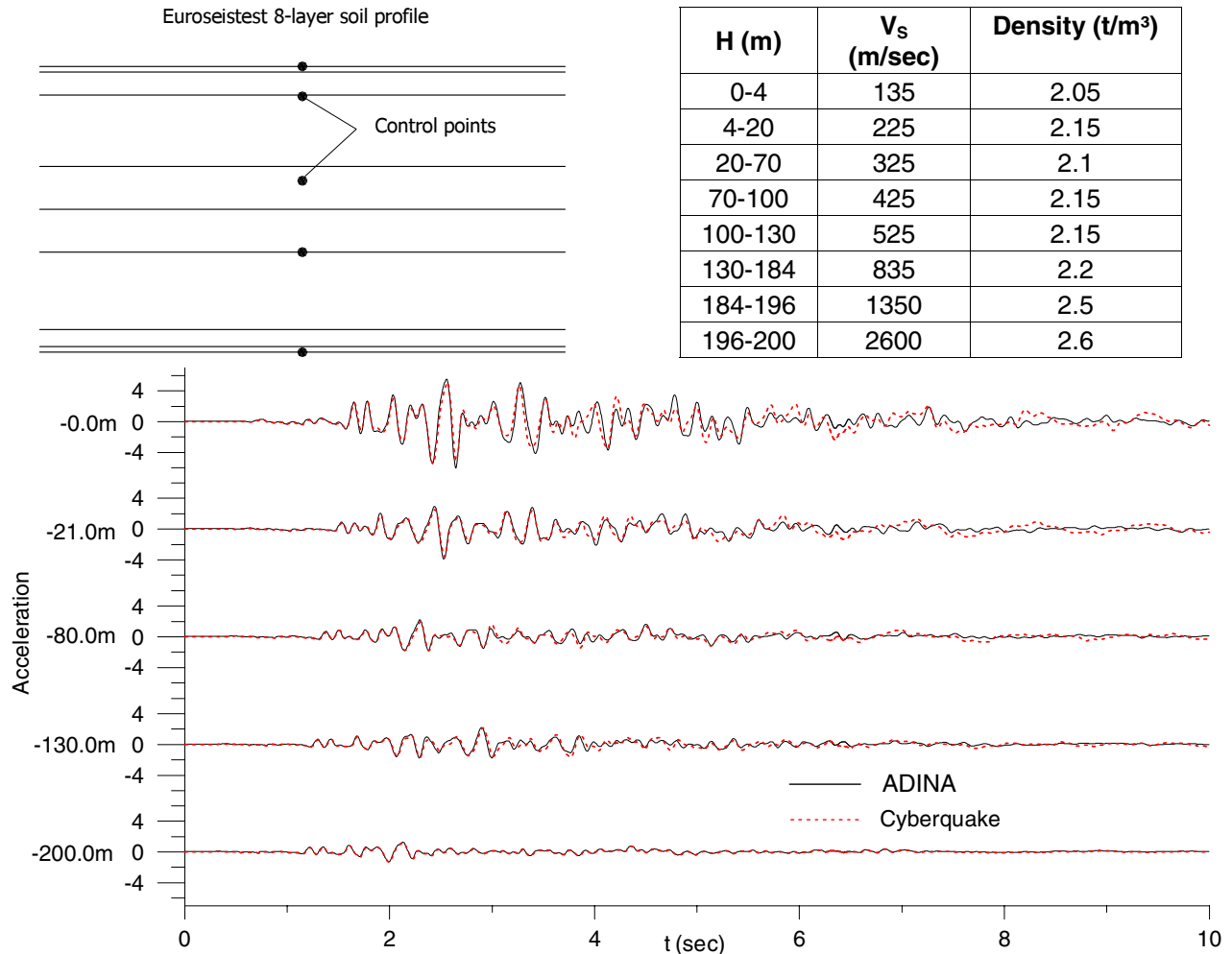


Figure 2. Validation of wave propagation analysis through FE representation (the case of Euroseistest)

Numerical modelling of the centrifuge experiments

Numerical simulation of centrifuge tests can be divided into two main tasks; the first is the proper consideration of geometry, boundaries and scaling issues of the centrifuge apparatus and the second is the calibration of numerical model properties with respect to specific centrifuge tests; both of them are discussed below.

Overview of the centrifuge experiment modeling strategy

Taking into consideration that the centrifuge experiments target the response of the complex soil-structure interaction system, it was deemed necessary to employ a two-dimensional analysis. At this

particular stage, The analysis soil behaviour is considered as linear elastic after an appropriate calibration of soil elastic characteristics. Extensive investigation of boundary effects, wave propagation parameters, damping implementation etc, is also performed to increase the reliability of modelling.

At the same time, one-dimensional modelling is performed with a soil-response-oriented code in order to validate the simulation of soil behaviour of the two-dimensional case. Indeed, implementation of equivalent linear soil behaviour of the 1D code presents a twofold advantage: first, verifies the adoption of equivalent elastic soil characteristics and second to demonstrate the importance of soil nonlinearities under each earthquake scenario. Clearly, nor modification of soil response due to the influence of the structure existence neither container box effects can be introduced with 1D analysis. As a result, their potential implication is thoroughly examined before reaching to any conclusion from reviewing the results obtained by 1D analysis.

Centrifuge apparatus modelling approach

The model geometry adopted in the two-dimensional analyses presented in this paper corresponds to the original apparatus dimensions. Since analysis methods comprise of linear and equivalent linear soil behaviour, the initial stress field does not interfere with wave propagation through the model, suggesting that simulation of initial soil stresses is not necessary. Moreover, bearing in mind that the FE code used is a general purpose one, hence it is capable of modelling such small dimensions and earthquake motions of high frequencies, it was preferred to simulate the 'exact' physical experiment in terms of centrifuge geometry without further processing the input and recorded signals. Nevertheless, presentation of results takes place after scaling them from the centrifuge to the prototype scale, so as to make their review more efficient.

Due to the high propagating frequencies, an adequately refined mesh to accurately reproduce wave propagation phenomena is adopted. The resulting 2D model consisting of approximately 2300 quadratic finite elements has the ability to reproduce frequencies up to 1000Hz (20Hz in the prototype scale).

Several alternative approaches for simulating the boundary effects of the ESB container have been also considered; the first (Model A, presented in Figure 3) assumes that wave propagation in the middle region of the centrifuge model is not affected by wave reflections and refractions at the side boundaries. Model A implements artificially extended lateral boundaries and appropriate point viscous dampers, that avert undesirable effects from refracted waves during numerical analysis. Model B (Figure 3) on the other hand, considers the case that soil-container interaction affects severely the wave propagation in the model; the soil nodes are tied to the container nodes, thus being able to move along with the container horizontally, while preventing movement along the vertical direction. Finally, Model C (Figure 3) employs a fictitious vertical interface elastic layer with low shear modulus that reduces the vertical constraint of the container to soil nodes and allows partially vertical soil movement (Gajo and Muir Wood [7]). Moreover the interface layer allows for debonding and recontact behaviour between the soil and the container edges (Al-Homoud and Whitman [8]).

All nodes of the side boundaries at equal depths are enforced to exhibit the same horizontal displacements during the analysis. The same applies for the neighbouring nodes in the vertical direction, which represent the successive aluminium container sheets that move as rigid body. Six control points were selected to compare the experiment recordings with results from the numerical analysis. All six of them coincide with the locations that the miniature accelerometers were placed during the physical experiments. In the case of the one-dimensional analysis, no influence of the container box is studied, hence, 1D modelling can be related to Model A of the 2D case, but the lack of boundary effects consideration on the soil response during 1D analysis is taken into account when reviewing the results.

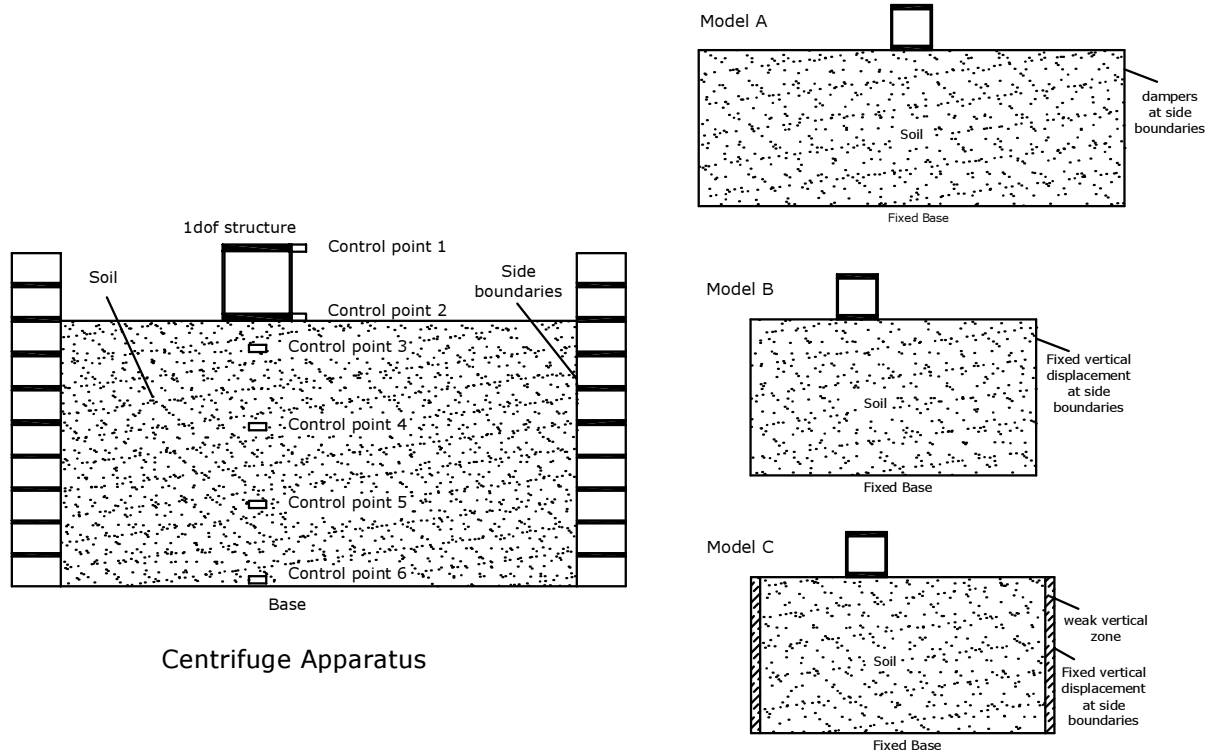


Figure 3. Alternative model configurations for centrifuge apparatus

Calibration of model properties

The soil elastic characteristics and the corresponding V_s profile were determined according to the empirical formula for G_{max} of sand that was proposed by Hardin & Drnevich [9] based on experimental results on many soil types

$$G_{max} = 3230 \frac{(2.973 - e)^2}{1 + e} \sigma'_o{}^{0.5} \quad (1)$$

where e is the void ratio of soil (herein $e=0.67$ measured before the experiment) and σ'_o is the mean effective confining pressure that is related to effective vertical stresses according to the following equation:

$$\sigma'_o = (1 + 2K_0) \frac{\sigma'_v}{3} \quad (2)$$

where K_0 is the coefficient of earth pressure at rest assumed here to be equal to 0.5. The aforementioned relationships yield a shear modulus value of $G_{max} = 82$ KPa for a mean confining pressure $\sigma'_o = 63.75$ KPa considered at half of the model depth and a void ratio $e = 0.67$. The shear modulus value is in agreement with laboratory testing of several dry sand specimens according to Kalliglou [10]. The variation of shear modulus G with depth was approximated as a square root function of the mean effective stress level as:

$$G = G_{ref} \left(\frac{p}{p_{ref}} \right)^\alpha, \quad \alpha = 0.5 \quad (3)$$

where G_{ref} is the reference shear modulus at any depth where the mean effective confining stress is p_{ref} .

Consequently, the shear wave velocity at each depth can be calculated from:

$$V_s = \sqrt{\frac{G}{\rho}} \quad (4)$$

The soil profile calculated from the aforementioned relationships refers to the maximum-initial shear modulus state. However, soil shear modulus is strongly dependent on the shear strain during the earthquake event. In the case of the 1D code Cyberquake, this reduction is internally performed following specific $G/G_{\max} - \gamma$ curves that prescribe the dynamic behaviour of the soil material during the equivalent linear analysis.

In order to appropriately reproduce this effect within 2D linear elastic modeling, the following procedure is adopted; first, the maximum strain level at each depth is back-calculated from a preliminary numerical analysis. An effective value equal to 0.65 of the maximum strain is then considered in the next step of the methodology. Then shear modulus values at various depths are appropriately modified according to $G/G_{\max} - \gamma$ diagrams proposed by Seed et al [11] and by Tika et al [12]. The new G variation with depth is then employed and analysis is performed once again taking this time into account the consistent elastic characteristic values. The application of this methodology, results to the equivalent shear wave velocity profiles of Figure 4. It is obvious that shear wave velocity values decrease along with shear modulus, depending on the intensity of the input earthquake motion in each experiment.

Soil damping on the other hand, during the 2D analysis is of Rayleigh type thus defined as a combination of the mass and stiffness matrices as $C = \alpha M + \beta K$. Since Rayleigh damping is frequency dependent, the selection of proper coefficients α and β is required in order to achieve approximately constant damping conditions in the frequency range of interest. Damping modification due to induced shear strains is also taken into consideration following the same methodology as in the case of shear modulus.

RESULTS

Comparative evaluation of modelling results

Description of results in the time-domain

The various earthquake scenarios of increasing ground motion intensity adopted, as well as the identity of each earthquake in terms of acceleration amplitudes and motion predominant frequencies, are summarized in Table 1. It should be noted that acceleration amplitude values refer to the prototype and

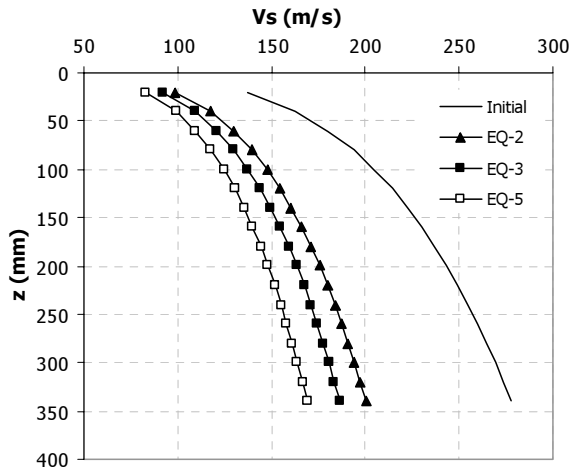


Figure 4. Corresponding equivalent V_s profile for each earthquake input motion scenario

Table 1. Input earthquakes

Earthquake	a_{\max} (%g)	predominant frequency of each input motion
EQ2	13.5	0.8 Hz
EQ3	24.0	1.0 Hz
EQ5	37.1	1.0 Hz

not the centrifuge scale. The results in the time domain for two of the earthquake scenarios are presented in Figures 5-6.

The duration of each centrifuge experiment was approximately 70 seconds (1.4s in centrifuge scale) from which only 30-35 seconds correspond to strong motion duration. In order to reduce the computational cost and increase the efficiency of the numerical analysis, only the strong motion part of the recording was used as input motion for the finite element code model.

The assessment and evaluation of the 2D analyses results obtained, revealed several drawbacks of Models A and B (Figure 3). The effect of the shear box sides was found to be quite important on the dynamic response calculated, since a constraint to the soil vertical movement was actually imposed in some extent by the container. Moreover, horizontal movement of soil at the vicinity of the boundaries was found quite different from deformation pattern assumed for the FE model since the shear box sides exhibited partial debonding with the neighbouring soil.

In Model A in particular, with the use of laterally extended side boundaries in the numerical analysis, no influence of the side boundaries to the response in the middle of the model is accounted for. Consequently, simulation of the centrifuge experiment using Model A does not consider the constraint that in some extent the shear box container imposes on the soil model. On the other hand, the constraint imposed in Model B by the implemented boundary type, completely prevents vertical movements of the soil at that location. Moreover, soil and box side nodes are numerically tied, ignoring the potential debonding during the strong motion duration. Contrary to the first two model approaches, Model C considers a transitional vertical zone of small shear modulus, releasing efficiently the constraint imposed by the box sides. Adopting this technique, all aforementioned issues of soil-box interaction can be taken into account. Therefore, only the results of the Model C 2D linear elastic analysis are presented herein. With respect to the 1D analysis scheme, ignoring the boundary effect was by definition inevitable, hence, it has to be taken into consideration when assessing the simulation results.

The recorded and calculated acceleration time histories in the case of earthquake scenario 2 presented in Figure 5, highlight satisfactory matching between the physical experiment and numerical analysis both in the case of 1D and 2D analysis. No filtering of the measured accelerations or the numerical results is applied, in order to examine motion amplification in a wide range of frequencies. The comparison at all depths where control points exist, shows only minor differences, whereas the most striking one corresponds to the structure's response during the second and third strong motion cycle.

In the detail illustrated in Figure 7 it is quite clear that both 1D and 2D analysis were equally efficient in the simulation of time histories of the physical test. As a matter of fact the one-dimensional simulation has overcome the lack of side boundaries modelling, whereas the same hypothesis in the two-dimensional code would not lead to results of similar accuracy. Moreover, the absence of the structure in the 1D analysis does not seem to affect the response at the remaining control points 2-5. The latter, which can be probably attributed to the minor SSI effects in the present experiment, will be thoroughly discussed in a following paragraph.

Despite the fact that earthquake scenario 5 corresponds to motion with the higher amplitude, the overall comparison of numerical versus experimental results appears to be equally efficient even for the linear elastic analyses (Figure 6). Indeed, apart from the first 2-3 cycles of the structure's response, the remaining time histories are very similar once again with the recorded motion.

Recorded and computed PGA values from all 3 earthquakes and analysis methods are presented in the diagram of Figure 8. The good agreement between linear elastic numerical calculations and experimental recordings in almost all the earthquake scenarios is not consistent with the documented soil behaviour, which is highly non-linear and allows for hysteretic phenomena under intense dynamic excitations. Linear elastic approximation would not be expected to be able to reproduce wave propagation through soil media that present consumption of significant amount of energy through hysteretic-type high strain cycles. In such a case, only non linear elastoplastic analysis should be able to follow the phenomenon closely. This subject is discussed below.

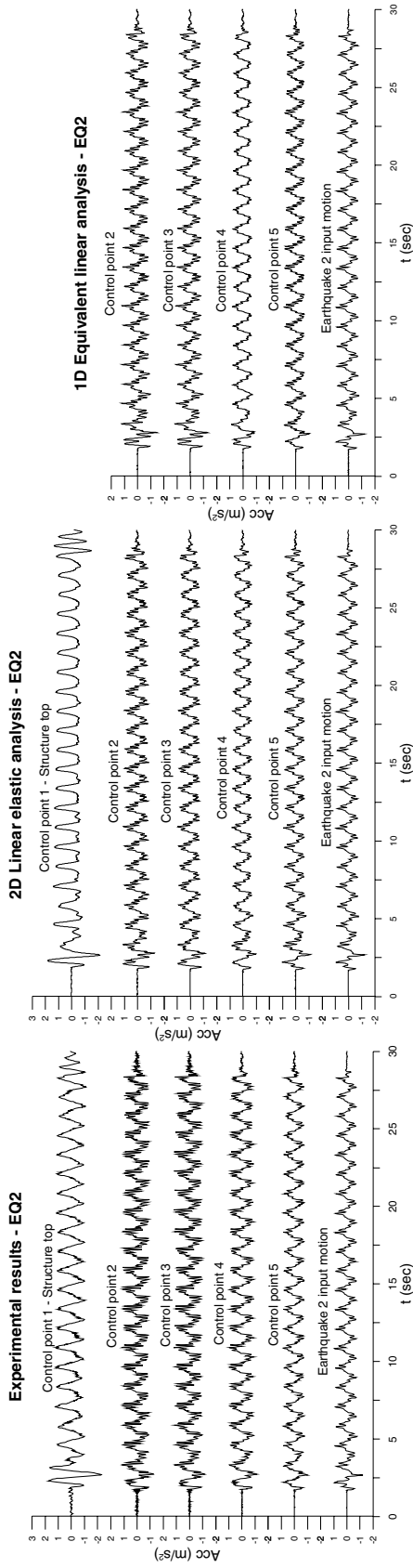


Figure 5. Comparative results at control points (earthquake scenario 2)

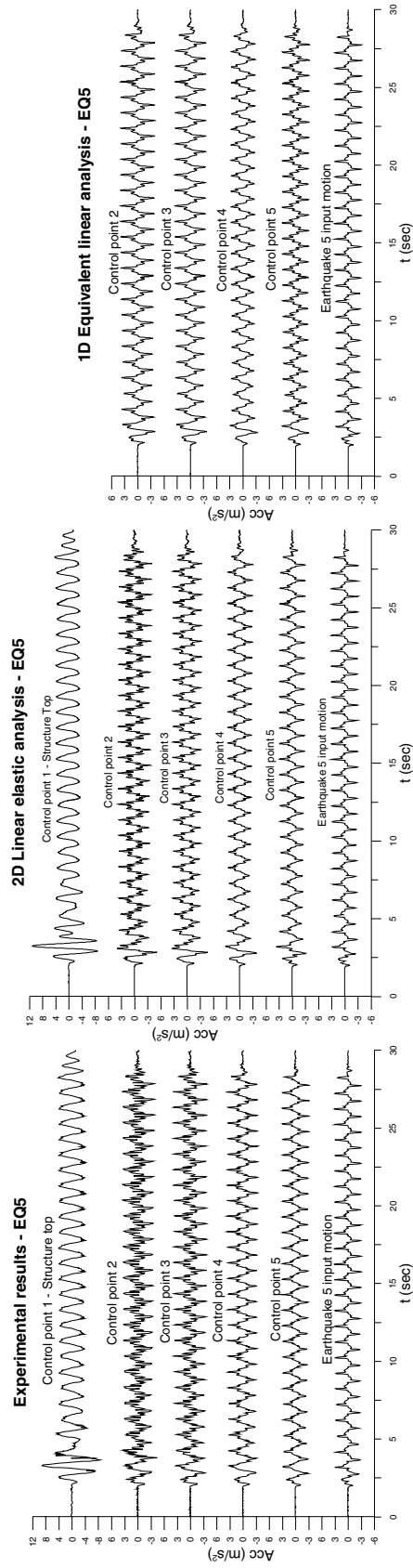


Figure 6. Comparative results at control points (earthquake scenario 5)

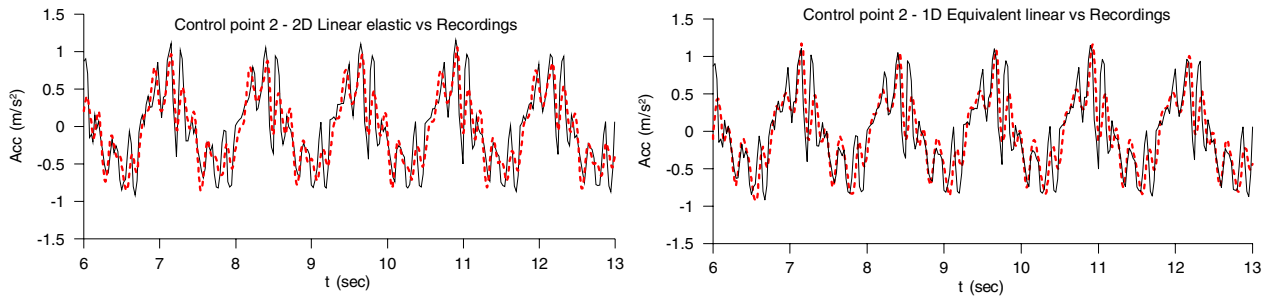


Figure 7. Detail of the comparative results of Figure 6 earthquake scenario 2 (recordings in continuous line)

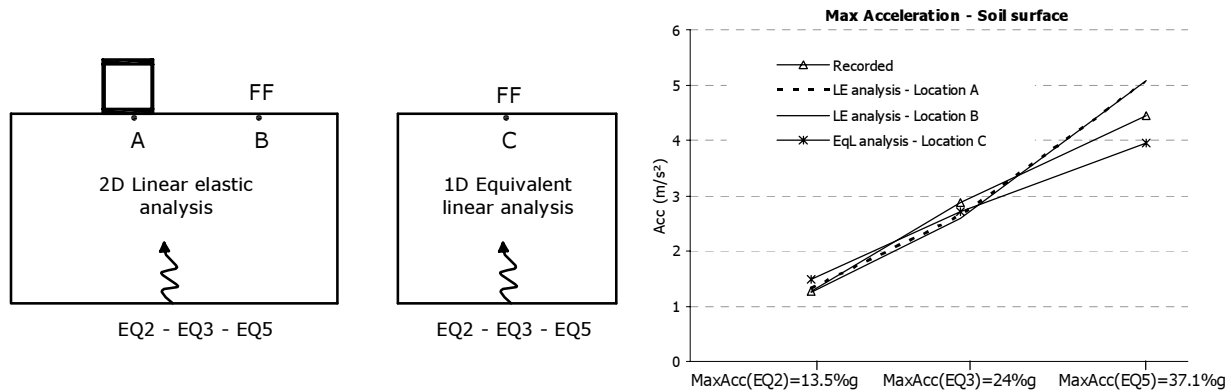


Figure 8. Recorded and computed (1D and 2D) PGA values

Evaluation of results in the frequency domain

Fourier spectra of recorded and calculated accelerations at the soil surface (control point 2) presented in Figures 9a-9b, indicate, similarly to the time-histories, that wave propagation was efficiently captured even with linear elastic numerical analysis. However, it was of particular interest to focus on the transfer functions between the soil surface and base (control points 2 and 6 respectively in Figure 3) in order to scrutinize ground motion amplification in the frequency domain. Indeed, significant differences were observed between linear elastic analysis and experimental data (Figure 10-left). A possible explanation was at first based on the fact that the shear strains developed during the earthquake events were relatively high, indicating that non-linear soil response and inelastic phenomena have taken place to a certain extent. Indeed, by employing the equivalent linear analysis which more accurately represents soil shear modulus modification during the earthquake event, significantly improved matching was achieved between analytically and experimentally derived transfer function, as seen at the right side of Figure 10. Very reasonably, the difference in the case of the stronger earthquakes 3 and 5 is greater on account of more profound inelastic behaviour phenomena. Moreover, the frequency at which soil exhibits the larger amplification is now shifted to lower frequencies, due to the subsequently decreased shear modulus (i.e. the highest amplification that was observed at 2.2Hz in earthquake scenario 2, shifts to 1.9Hz for the case of earthquake 5). Linear elastic analysis on the other hand, despite the inefficiency to reproduce hysteretic phenomena, may capture this frequency shift of the peak response amplification, after an appropriate modification of the shear modulus values.

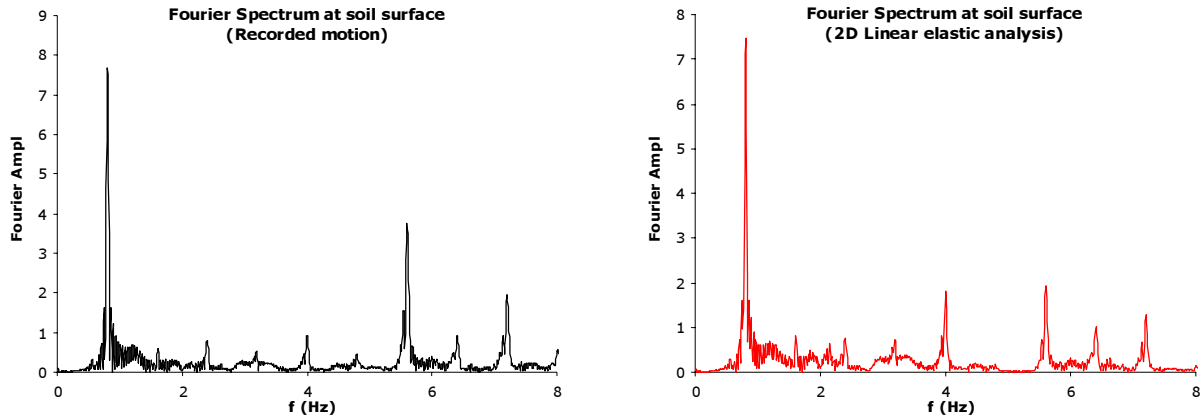


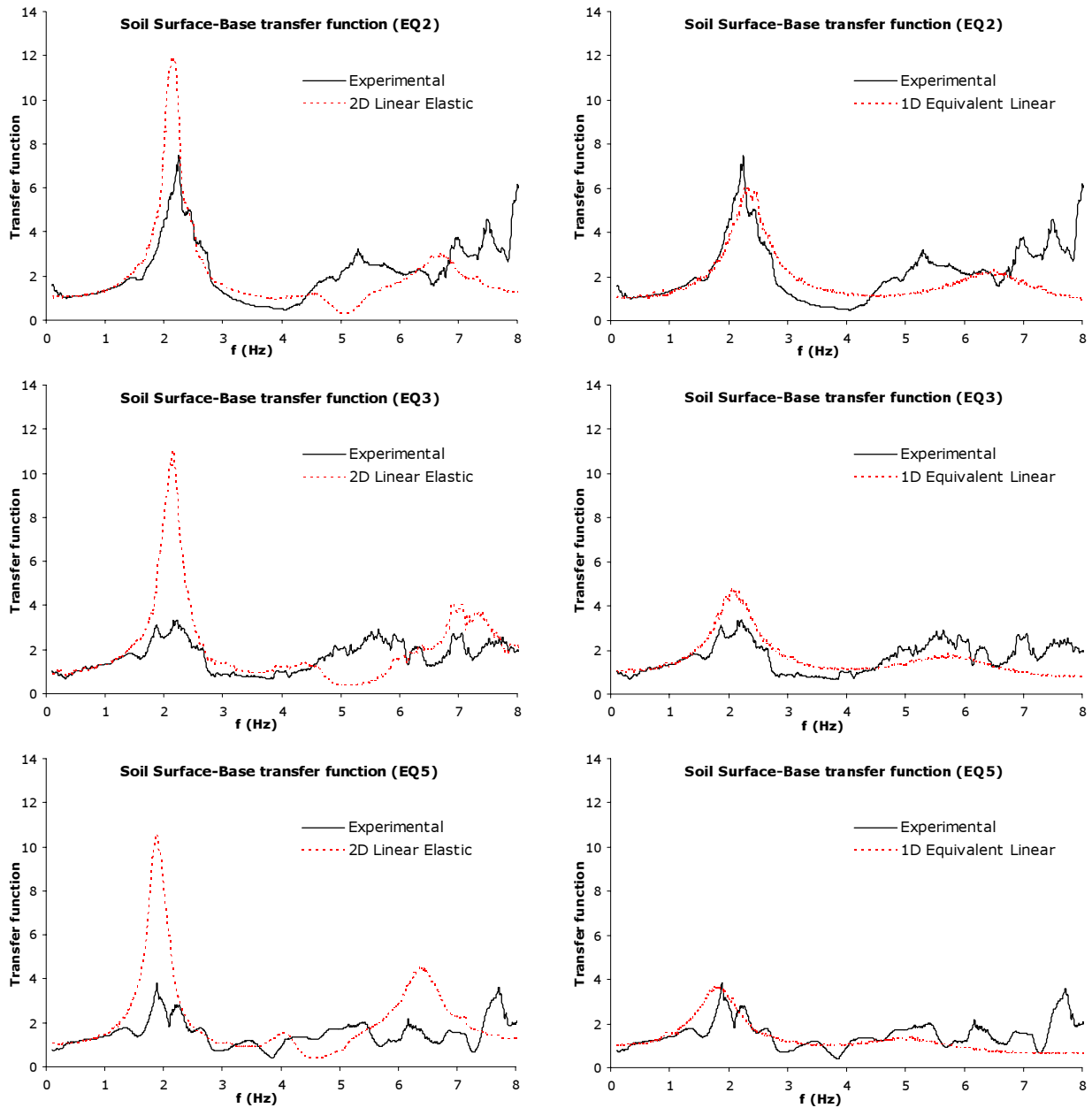
Figure 9a-9b. Fourier amplitude spectra at soil surface (earthquake scenario 2)

An additional verification of that the hypothesis of soil hysteretic response is valid can be obtained by the calculation of the shear stress-strain cycles as they are derived from the recorded data. This procedure is described in detail elsewhere (Zeghal and Elgamal [13]). In Figure 10, a comparative evaluation of the numerically calculated and experimentally recorded stress-strain histories is presented at the middle of control points. Useful conclusions can be drawn observing the area of the stress-strain loops between the recordings and the equivalent linear analysis data. The similarity suggests that equivalent linear analysis has closely approximated the amount of energy dissipation during the experiment. In the case of linear elastic analysis of course, by definition, the conclusions may only concern the inclination of stress-strain diagrams that suggest a shear modulus value in good accordance with the secant shear modulus value of recordings and equivalent linear analysis. This agreement validates the methodology to estimate the soil elastic characteristics that were used in the linear elastic analysis and furthermore, it is obvious that empirical formulas for shear modulus values and $G/G_{\max} - \gamma$ curves from the literature have resulted in G values consistent with the measured, as indicated in Table 2. The peculiar shear modulus value that was calculated between control points 3 and 4 as well as the respective stress-strain diagram of Figure 11, may be probably attributed to small vertical movements of the accelerometers during the experiment or to other problems related to the experiment itself.

Another issue of particular interest is the fact that the deficiencies of linear elastic analyses have only a minor effect on the accuracy of the results in the time domain. This, can be attributed to the substantial difference between the soil natural frequency and the input motion predominant frequency. Indeed, all frequencies up to 4.5Hz seem to be efficiently amplified during linear elastic analysis (left side of Figure 10), except from the range between 2-2.5Hz. Observing the Fourier spectrum of Figure 9a though, leads to the conclusion that the specific frequency range does not dominate the overall response. Consequently, the final time-history is not affected by the incompetence of the linear elastic analysis to accurately predict the motion amplification at the natural frequency of the soil. That explains the reason why time-histories and corresponding Fourier spectra seem to have a very good agreement with the experimental results regardless the intensity level of the input motion and the inelastic phenomena that have taken place.

Table 2. Resulting equivalent shear modulus (recordings – analysis)

Shear modulus G	Experiment	2D Linear Elastic	1D Equivalent Linear
Control Point 2-3	10900 KPa	15900 KPa	13400 KPa
Control Point 3-4	159000 KPa	25200 KPa	17000 KPa
Control Point 4-5	18500 KPa	35600 KPa	21200 KPa
Control Point 5-6	50000 KPa	43600 KPa	27500 KPa



**Figure 10. Computed and recorded transfer functions between soil surface and base
Cases of linear elastic (left) and equivalent linear analysis (right)**

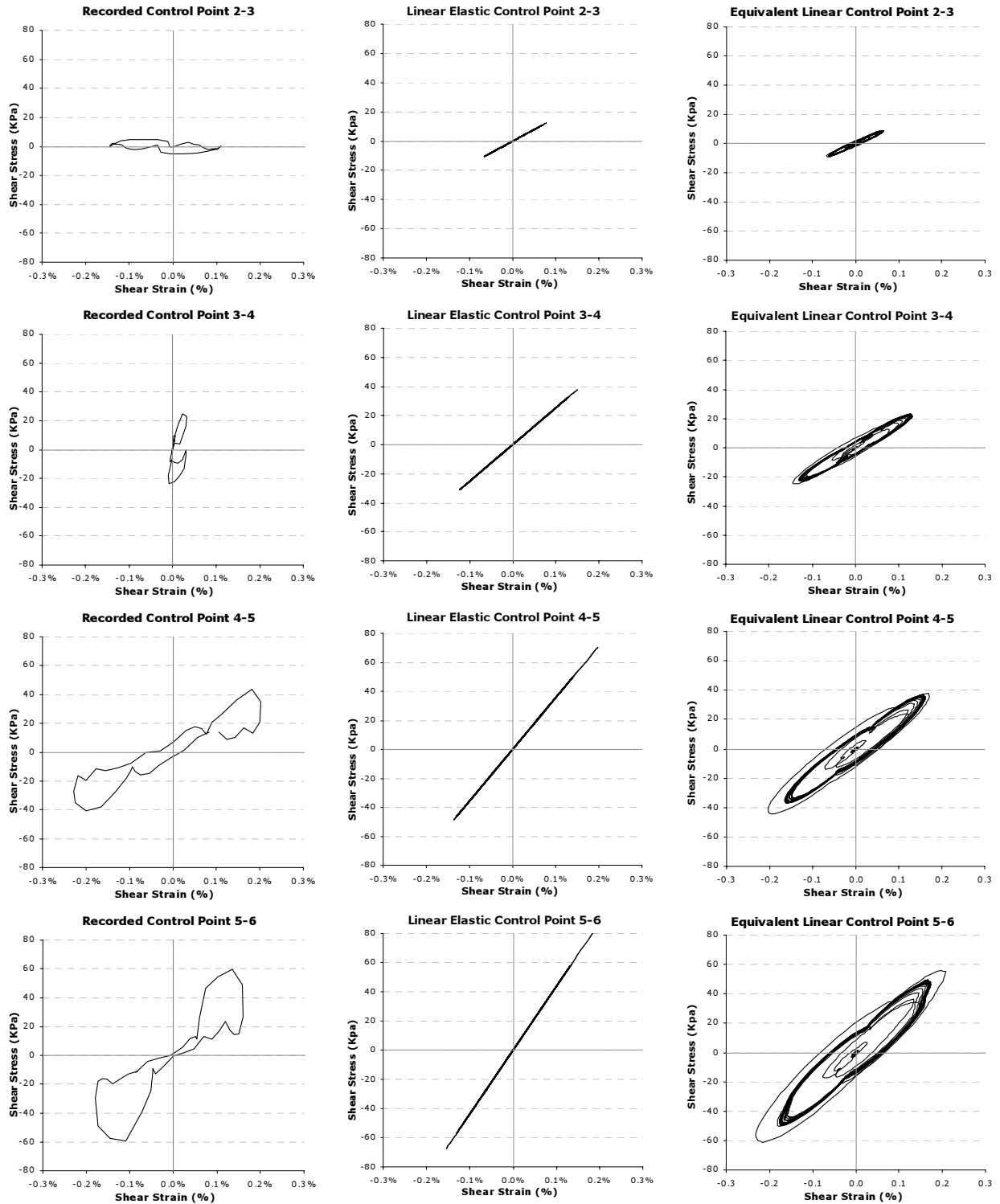


Figure 11. Comparison of shear stress-strain diagrams from recordings and from numerical analysis results (earthquake scenario 5)

Identification of SSI effects

The results of the numerical analysis with respect to structural response are presented in comparison with the measurements in Figures 5 and 6 (Control point 1) and the calculated acceleration time-histories from one-dimensional linear elastic analysis match quite well the recorded in both earthquake scenarios presented. In order to identify the role played by soil-structure interaction, the step-by-step verification scheme was adopted. In particular, a free-field analysis was first performed using 2D linear elastic modelling as well as 1D equivalent linear analysis (cases B and C in Figure 12). The resulting acceleration time history of the soil surface was then used as input motion for the structure model which was considered to be fixed at its base. The response of the fixed-base structure (case B) compared to the response of the full soil-structure system (case A) can reveal potential SSI phenomena. In Figure 13 this comparison is illustrated in terms of maximum acceleration at the top of the structure and maximum bending moment at its base (approximate experimental moment values were obtained after computations). The SSI effect can be estimated after observing the linear elastic analysis (LE) in both full SS system and fixed structure (dotted and continuous line respectively). In the same plots the experimental structural response as well as the fixed-base response that was calculated from the equivalent linear analysis free-field motion, are also presented. Peak ground acceleration values for each analysis case are given in the diagram of Figure 8.

A critical evaluation of the results presented in Figure 13 suggests that SSI phenomena were not particularly highlighted during the specific experiment, as it was anticipated by taking into consideration the fact that the motion frequency content was concentrated at 1Hz whereas soil profile and structural eigen-frequencies were equal to 2.2Hz and 1.54Hz respectively. Such interaction indeed occurred but at frequencies which were apparently related to low motion amplitude. Complementary numerical analysis with appropriately selected ground motion characteristics verified this conclusion. Having therefore obtained the desirable level of confidence on numerical simulation by centrifuge testing, and taking into consideration all the assumptions required in order to ensure the feasibility of such a matching, the subsequent stages of the analysis and physical modeling scheme can take place.

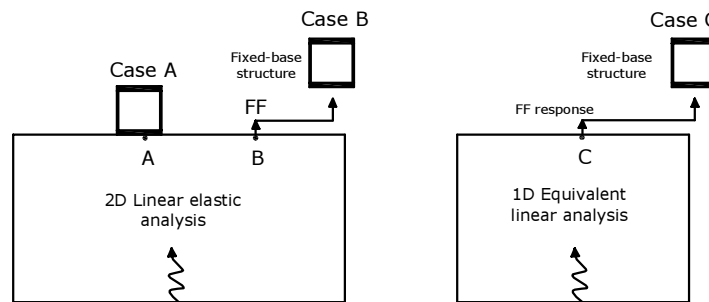


Figure 12. Schematic representation of SSI verification procedure

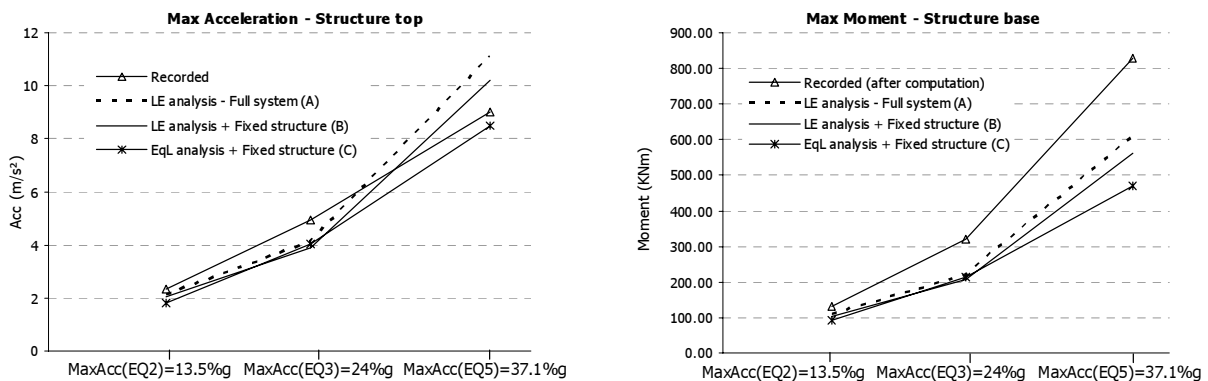


Figure 13. Comparative structural acceleration and moment results

CONCLUSIONS

A centrifuge experiment involving a soil sample and a single-degree-of-freedom structural model was performed under high gravitational environment of 50g. Three earthquake scenarios were applied at the model base and the system response was recorded in several control points. Two-dimensional linear-elastic and one-dimensional equivalent linear analysis were performed to simulate the physical experiment. An equivalent approximation to determine the soil elastic characteristics was implemented during linear elastic analysis. Comparative results presented good agreement between experimental and numerical response particularly in the time domain. In the frequency domain there are some differences attributed mainly to the difference in the predominant frequencies of the model and the input motion and partially to the non linear response of the soil. SSI effects have been discussed in the light of improving the future test program. Given the embedded limitations of each analysis approach, the simulation is considered to have appropriately reproduced the system response.

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