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## Title:

Modelling the non-linear site response at the LSST down-hole accelerometer array in Lotung

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#### 1 Abstract

2 Down-hole array observations are extremely useful to investigate site amplification effects and 3 to validate numerical modelling techniques for site response. In this paper the ground response of 4 the Lotung experiment site (Taiwan), measured along a down-hole accelerometer array during a 5 weak and a strong motion event, is simulated using different numerical techniques of increasing 6 level of complexity: 1) a simple equivalent-linear visco-elastic procedure, 2) a total stress time-7 domain scheme using a pressure-dependent hyperbolic model and 3) a fully-coupled approach 8 implementing an advanced elasto-plastic soil model. The numerical models are calibrated against 9 resonant column data and in-situ cross-hole measurements. The two horizontal components of 10 the input motion are applied separately at bedrock level. The results of the simple and advanced 11 numerical simulations are compared with the down-hole motions recorded in-situ during the 12 investigated seismic events in terms of acceleration time histories and response spectra. The 13 comparison between in-situ measurements and predicted results highlights the well-known 14 limitations of the frequency-domain technique. It also shows some improved predictive 15 capabilities of the total stress time-domain scheme and demonstrates the excellent performance 16 of the fully-coupled advanced non-linear approach.

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### 19 Keywords:

Seismic ground response analysis; Lotung experiment site; Equivalent-linear visco-elastic
approach; Elasto-plastic constitutive models; Non-linear time-domain analysis

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## 1 **1. Introduction**

2 Experience from historical and recent strong earthquakes has demonstrated the significance of local soil conditions on the seismic ground response. The changes in amplitude, frequency 3 4 content and duration of the seismic motion during its propagation in soil deposits, commonly referred to as "site effects", have a crucial impact on the response of buildings and 5 6 infrastructures during earthquakes [e.g. 1]. The analysis of site effects has been the object of many national and international research projects during the past decades. Following the 7 8 devastating effects of surface geology on shaking damage observed during the 1985 Mexico City 9 earthquake, the Turkey Flat (California, USA) and Ashigara Valley (Japan) strong motion arrays 10 have been established within the 1992 ESG-IASPEI/IAEE project to help determine the state-of-11 practice in estimating the effects of surface geology on earthquake ground motion [2-5]. In 1993 the European Commission funded the initial development of the EuroSeisTest<sup>1</sup> programme in 12 13 Northern Greece for the analysis of structural response to ground motion and the effects of 14 surface geology, non-linear soil response and soil-structure interaction on ground motion. Since 15 then, the monitored site has provided researchers with high-quality multidisciplinary data for 16 conducting theoretical studies on ground response analysis. Wave propagation problems in natural soil deposits have also been studied during the ReLUIS<sup>2</sup> projects (ReLUIS-DPC 2005-17 2008 and 2010-2013) funded by the University Network of Seismic Engineering Laboratories in 18 19 Italy. More recently, an international benchmark on the uncertainty assessment associated to non-linear simulation of one-dimensional (1D) site effects has been set up in 2013 within the 20 project PRENOLIN [6] as part of two larger projects: SINAPS@<sup>3</sup>, funded by the French 21 National Research Agency (ANR), and SIGMA<sup>4</sup>, funded by a consortium of nuclear facility 22 operators (EDF, CEA, AREVA, ENEL). These are only few examples that confirm the ongoing 23

<sup>&</sup>lt;sup>1</sup> http://euroseisdb.civil.auth.gr/

<sup>&</sup>lt;sup>2</sup> http://www.reluis.it/

<sup>&</sup>lt;sup>3</sup> http://www.institut-seism.fr/en/projects/sinaps/

<sup>&</sup>lt;sup>4</sup> http://projet-sigma.com/index.html

interest in site response analysis of the international research community working in the broad
 field of geotechnical earthquake engineering.

3 Ground response problems can be studied by geotechnical engineers using different numerical 4 approaches, ranging from simplified frequency-domain analyses based on visco-elasticity [7, 8] to time-domain schemes whereby the solid-fluid interaction can be accounted for by means of a 5 6 coupled effective stress formulation and sophisticated non-linear constitutive models can be 7 adopted to describe the soil cyclic mechanical behaviour [9-12]. For validation purposes, the 8 performance of these different numerical approaches has been tested over the last decades using 9 real vertical array observations. In this case, the input motion is reasonably well defined, as it is 10 directly recorded on rock. As an example, since the establishment in 1996 of the KiK-net array 11 monitoring system in Japan, different researchers have effectively used the available array data 12 for their numerical site response investigations [e.g. 13-19]. In particular, Aguirre and Irikura 13 [20] simulated the response of the Port Island site in Kobe during the 1995 Hyogo-ken Nanbu 14 earthquake using a finite difference program implementing a hyperbolic non-linear model and 15 comparing the results with the recorded down-hole array data. The acceleration time histories 16 were reasonably predicted at depth but the dynamic response of the site was overestimated at 17 surface, as liquefaction occurring in the top 16 m could not be captured by the adopted 18 constitutive model. Another interesting numerical validation is presented by Huang et al. [21], 19 who simulated the site response of the Dahan down-hole array in Taiwan subjected to a 20 foreshock weak motion and the main shock of the 1999 Chi-Chi earthquake using the Haskell 21 matrix method based on linear elastic shear wave propagation equations in multi-layered solid 22 media. The horizontal accelerations were accurately predicted when the weak motion was 23 considered. However, the seismic response during the main shock was not satisfactorily 24 predicted due to the inability of the adopted linear elastic model to reproduce non-linear soil behaviour. Also located in Taiwan, the Lotung Large-Scale Seismic Test (LSST) site has been 25

1 extensively studied since its establishment in 1985 by a number of authors [22-33]. In particular, 2 Li et al. [22] modelled the multi-directional site response at Lotung by employing a fullycoupled finite element (FE) method with a bounding surface hypoplasticity soil model. 3 4 Satisfactory predictions were obtained for the response in the horizontal direction, whereas the vertical site response was significantly underestimated by the analysis. Similarly, Borja et al. 5 6 [23] and Borja et al. [26] studied the dynamic response at Lotung under the simultaneous 7 application of the three components of the earthquake by employing a 3D FE program and a 8 bounding surface elasto-plastic soil constitutive model. Good agreement was observed for all 9 three directions of loading between predicted acceleration time histories and recorded data. 10 However, the adopted soil constitutive model was formulated in terms of total stress, not 11 accounting for solid-fluid interaction effects. Amorosi et al. [29] attempted to predict the 12 horizontal site response at Lotung using a fully-coupled effective stress FE program 13 implementing an advanced kinematic hardening multi-surface model. The numerical predictions 14 were in good agreement with recorded data in terms of frequency content, although a significant 15 underestimation of the peak ground acceleration at surface was observed for both components of 16 the input motion. Very recently, Amorosi et al. [33] have adopted a three-dimensional FE non-17 linear approach and applied the two horizontal components of the seismic motion singularly or 18 simultaneously to investigate the role of the input multi-directionality effects on the site response 19 at Lotung. A good agreement with the free-field measurements has been obtained when a single 20 input motion is considered, whereas the simultaneous application of the two horizontal 21 components has produced a slight overestimation of the peak accelerations and the rise of 22 spurious high frequencies.

Even though extensive research has been conducted in recent years on the analysis of site effects,
as demonstrated by the mentioned benchmark projects and validation studies, fundamental
questions, such as the importance of multi-directional loading and the predictive capabilities of

1 effective stress advanced soil constitutive models, still need to be answered.

2 This paper presents a new validation study of the LSST site in Taiwan using the recordings from the down-hole accelerometer array installed in-situ. In particular, the free-field horizontal site 3 4 response at Lotung during the weak and strong motion events LSST11 and LSST07 is investigated using the simple equivalent-linear visco-elastic method, a total stress non-linear 5 6 scheme and a fully-coupled effective stress non-linear approach. In the first part of the paper the geological and geotechnical properties of the LSST site are briefly described. The numerical 7 8 models adopted for the frequency-domain and the time-domain dynamic simulations are then 9 summarised along with the calibration against in-situ and laboratory data of the soil models used 10 in the analyses. The direct comparison at various depths within the soil deposit between recorded 11 and predicted motions obtained with the different numerical approaches, adopting similar 12 profiles of the small-strain shear modulus consistent with the measured data, is presented in the 13 main part of the paper. Finally, the influence of variation in the initial shear modulus profile of 14 the top layers is explored and conclusions are drawn at the end.

15

## 16 2. The Large-Scale Seismic Test at Lotung

17 The Large-Scale Seismic Test (LSST) is located in one of the most seismically active region in 18 the North-East of Taiwan. It was originally established in the 1980's to study the dynamic 19 behaviour of two scaled-down nuclear plant containment structures [34]. The site response has 20 been monitored by a number of surface and down-hole accelerometer arrays, together with pore 21 pressure transducers. The down-hole accelerometers have been installed at depths of 0, 6, 11, 17 22 and 47 m, oriented in N-S, E-W and vertical directions. Figure 1 shows the elevation and plan 23 views of the instrumentation. Of particular interest here is the vertical array named DHB in 24 Figure 1a which has been considered representative of the free-field response at Lotung. The site geology consists of recent alluvium and Pleistocene materials over a Miocene basement. The 25

upper alluvial layer, 30 to 40 m thick, consists mainly of clayey-silts and silty-clays [35]. The water table is located approximately at a depth of 1 m. The local geological profile (Figure 2a) shows a 17 m thick silty sand layer above a 6 m thick layer of sand with gravel resting on a stratum of silty clay interlayered by an inclusion of sand with gravel between 29 m and 36 m, as indicated by the SPT log profile reported in Figure 2b.

6 A series of geophysical seismic tests have been performed to measure shear and compression 7 wave velocities at the LSST site. The shear wave velocity obtained from seismic cross-hole and 8 up-hole tests [36] showed a value of about 100 m/s at the surface reaching a value of about 300 9 m/s at 60 m depth. The corresponding elastic shear modulus ( $G_0$ ) data are shown in Figure 2c with open dots. A total unit weight of 19.6 kN/m<sup>3</sup> has been adopted as average value for the 10 11 entire deposit [37]. In all previous studies, the bedrock formation has been assumed to be at a 12 depth of 47 m, corresponding to the location of the accelerometer DHB-47 shown in Figure 1a. 13 Shear modulus and damping ratio curves have been measured through resonant column and 14 cyclic torsion tests on undisturbed specimens [36, 38]. Alternatively, Zeghal et al. [39] have 15 proposed to back-calculate the in-situ moduli ratio curves for Lotung soil based directly on its 16 seismic response recorded along the down-hole arrays during 18 earthquakes which occurred between 1985 and 1986. In particular, different sets of  $G/G_0-\gamma$  and  $D-\gamma$  curves have been 17 18 developed for the depths of 0-6, 6-11 and 11-17 m, producing for each depth a least-square best 19 fit (SF) as well as an upper (UB) and a lower bound (LB) curve indicative of the possible 20 variations in the material dynamic properties (Figure 3). The shear modulus and damping ratio 21 curves of the soil between 17 and 47 m are assumed to be equal to those from 11 to 17 m, as 22 more detailed data relevant to the deeper material were missing. Therefore, three different 23 materials can be identified along the deposit in terms of dynamic properties, although the 24 inclusions of sand with gravel between 17 and 23 m and 29 and 36 m are characterised by higher 25 values of effective friction angles (according to the SPT data reported in Figure 2b). It is noted that the lack of a proper characterisation of the deeper materials (i.e. below 17 m) in terms of soil dynamic properties at the LSST site can affect the accuracy of the site response simulations and, therefore, its influence on the seismic wave propagation process has been investigated in the last part of the paper.

5

## 6 **3. Numerical simulations**

The ground response of the Lotung LSST site has been studied during one weak (i.e. LSST11) and one strong motion event (i.e. LSST07) recorded along the array. As summarised in Table 1, the LSST11 event was characterised by a shallow focal depth, a relatively small magnitude and peak ground accelerations (PGA) of 0.07 g and 0.10 g respectively in the E-W and N-S direction. On the contrary, the LSST07 event was characterised by a higher magnitude and maximum accelerations recorded at ground surface equal to 0.16 g and 0.20 g in the E-W and N-S direction, respectively.

14 Figures 4a and 4b report the response spectra of the accelerations recorded during the LSST11 15 event at bedrock (DHB-47), 11 m below surface (DHB-11) and at ground surface (FA1-5) along 16 the E-W and N-S direction. The response spectra of the acceleration recorded at the same depths 17 during the LSST07 event are shown in Figures 4c and 4d. It is important to note that the weak motion LSST11, although characterised by smaller accelerations across the entire spectrum, is 18 19 richer in high frequencies than the strong motion LSST07. Moreover, the analysis of the LSST11 20 array data shows a very little alteration of the motion between the bedrock and 11 m depth, with 21 a significant amplification of the seismic wave occurring between 11 m and ground surface, 22 especially in the N-S direction. Similarly, the LSST07 array measurements indicate that in the N-23 S direction a much more pronounced amplification has occurred in the top 11 m with respect to 24 the E-W component.



1 1D equivalent-linear visco-elastic software EERA [40], the 1D non-linear time-domain software 2 DEEPSOIL [41] and the 2D fully-coupled FE code SWANDYNE II [42]. In all the analyses, the 3 two horizontal components of the seismic event have been applied separately as input motions at 4 the bedrock level (i.e. at 47 m). It should be noted that the simultaneous application of the two 5 earthquake components would represent a more realistic scenario, but this still requires a better 6 understanding of soil cyclic behaviour under multi-directional loading, from both the laboratory 7 and constitutive modelling point of view [e.g. 33]. Therefore, the choice of applying separately 8 the horizontal components of the input motion has been preferred. The main features of the 9 adopted numerical models are discussed in this section of the paper, whereas the comparison 10 between results obtained with the different numerical approaches is presented in the following 11 Section 4.

12

#### 13 3.1 Equivalent-linear visco-elastic model

14 The code EERA is based on the assumption of equivalent-linear visco-elastic soil behaviour. The 15 approach makes use of the exact solution of wave propagation in horizontally layered visco-16 elastic materials subjected to vertically propagating transient motions [e.g. 43]. The solution is 17 attempted in the frequency-domain. The non-linear variation of soil shear modulus, G, and 18 damping, D, with shear strain is accounted for through a sequence of linear frequency-domain 19 analyses with iterative update of stiffness and damping parameters. For a given soil layer, G and 20 D are assumed to be constant with time during the shaking. Therefore, an iterative procedure is 21 needed to ensure that the properties used in the linear dynamic analyses are consistent with the 22 level of strain induced by the input motion in each layer.

In the presented EERA analyses, the profile of small-strain stiffness shown in Figure 2c has been discretised using constant stiffness sub-strata of 1 m thickness. The statistical fit (SF) curves reported in Figure 3 have been adopted in the visco-elastic simulations for the top layers (i.e. between 0 and 17 m). From 17 to 47 m the same curves as those relevant to the depth between 11
and 17 m have been considered. EERA simulations have been performed also using the upper
bound (UB) and lower bound (LB) curves shown in Figure 3, but the best results have been
obtained adopting the SF curves, as discussed later.

5

## 6 *3.2 Non-linear time-domain models*

7 To overcome some of the well-known limitations of the equivalent-linear visco-elastic approach, 8 non-linear schemes able to solve the wave propagation problem by direct numerical integration 9 in the time-domain can be employed. The LSST site response has been, therefore, studied with 10 the 1D program DEEPSOIL, which allows to perform time-domain total stress analyses using 11 the MRDF pressure-dependent hyperbolic model [after 44] with extended unloading-reloading 12 Masing rules [after 45]. The non-linear constitutive model allows to capture the influence of 13 confining pressure on shear modulus degradation. In addition, it implements the hysteretic 14 damping reduction factor (MRDF) proposed by Phillips and Hashash [46] to match measured 15 modulus reduction and damping curves simultaneously over a wide range of shear strains. The 16 same profile of small-strain stiffness used in the EERA simulations has been adopted in the DEEPSOIL analyses (Figure 2c). The  $G/G_0 - \gamma$  and  $D - \gamma$  curves predicted by the non-linear model 17 18 for the depths of 0-6, 6-11 and 11-17 m are presented in Figure 3 and the relevant material 19 parameters are reported in Table 2. For more details on the MRDF pressure-dependent 20 hyperbolic model, the reader is referred to [46]. 1% viscous damping has been introduced in the 21 time-domain simulations performed with DEEPSOIL by means of a standard Rayleigh 22 formulation [e.g. 47], using two modes to obtain a frequency dependent dissipation at small 23 strains. In particular, the best simulation results for both horizontal components of the LSST11 24 event have been obtained using the frequencies of 1.14 Hz and 5.7 Hz. For the E-W component 25 of the LSST07 event, frequencies equal to 0.84 Hz and 2.65 Hz have been selected, whereas the

values of 0.95 Hz and 4.2 Hz have been used for the N-S component, according to the procedure
 proposed by Amorosi *et al.* [12].

The ground response analysis of the Lotung experiment site has been also undertaken using the 3 4 two-dimensional fully-coupled finite element code SWANDYNE II. The code performs linear and non-linear dynamic analyses, using the Generalised Newmark method [48] for time 5 6 integration. In particular, the values of the Newmark parameters selected in all the FE analyses illustrated in this note are  $\beta_1 = 0.600$  and  $\beta_2 = 0.605$  for the solid phase and  ${\beta_1}^* = 0.600$  for the 7 8 fluid phase. These values ensure that the algorithm is unconditionally stable, while being 9 dissipative mainly for the high frequency modes [9]. A 5 m wide, 47 m high FE mesh composed 10 by 235 isoparametric quadrilateral finite elements with 8 solid nodes and 4 fluid nodes has been 11 used in the dynamic simulations. The base of the mesh has been assumed to be rigid, while equal 12 displacements have been imposed to the nodes along the vertical sides (i.e. tied-nodes lateral 13 boundary conditions). Base and lateral hydraulic boundaries have been assumed as impervious, 14 while drained conditions have been imposed at the top of the mesh. In order to investigate the 15 effects of soil non-linearity on the wave propagation process, plasticity has been introduced in 16 the SWANDYNE II simulations through the advanced elasto-plastic model (RMW) developed by 17 Rouainia and Muir Wood [49]. The RMW model allows to reproduce some of the key features of the cyclic behaviour of natural soils such as the decay of the shear stiffness with strain 18 amplitude, the corresponding increase of hysteretic damping and the related accumulation of 19 20 excess pore water pressure under undrained conditions. The model has been implemented in 21 SWANDYNE II with an explicit stress integration algorithm and a constant strain sub-stepping 22 scheme. RMW has been successfully employed to simulate both static [50, 51] and dynamic 23 geotechnical problems [52, 53]. For more details on its formulation and implementation the 24 reader is referred to [49] and [54]. In previous versions of the model a classical hypoelastic formulation was employed for the determination of the bulk and shear moduli, K and  $G_0$ . In this 25

1 work, the well-known equation proposed by Viggiani and Atkinson [55] for the small-strain 2 shear modulus has been implemented to reproduce the dependency of  $G_0$  on the mean effective 3 stress and overconsolidation ratio:

4

5 
$$\frac{G_0}{p_r} = A \left(\frac{p'}{p_r}\right)^n OCR^m$$
(1)

6

7 where A, m and n are dimensionless stiffness parameters,  $p_r$  is a reference pressure (equal to 1 kPa), p' is the mean effective stress and OCR is the soil overconsolidation ratio. In the 8 9 initialisation of the FE model, a higher overconsolidation ratio has been assumed for the upper part of the FE column (from 0 to a depth of 6 m), with an average OCR equal to 4, while a 10 11 constant OCR of 2 has been imposed for the remaining part of the model. The assumed variation 12 of *OCR* with depth is deemed to be realistic in accordance with the  $G_0$  data shown in Figure 2c, 13 where a non zero elastic shear modulus of about 20 MPa can be observed near the ground 14 surface (corresponding to a measured shear wave velocity of about 100 m/s). Once the FE model has been initialised by applying the gravity load, a range of possible elastic shear modulus 15 16 profiles, controlled by the values of the dimensionless stiffness parameters A, m and n in 17 Equation (1), has been investigated. In previous works by Elia et al. [31] and Karofyllakis et al. [32], a set of stiffness parameters A, m and n constant with depth was assumed. With this 18 19 adopted initial small-strain stiffness profile, the peak acceleration of the LSST07 E-W motion at 20 ground surface was very well predicted, while the PGA in the N-S direction was under-21 predicted. A more realistic  $G_0$  profile, which achieves a better calibration of the cross-hole data, 22 has been assumed in this paper (see Figure 2c). The calibration of the remaining RMW parameters has been performed through single element simulations of undrained cyclic simple 23 shear (CSS) tests, by imposing different shear strain amplitudes and assessing the predicted 24

1 secant shear modulus and the damping ratio for each strain value after a number of cycles (equal 2 to 500) sufficient to reach a steady-state condition. The results of the CSS test simulations are reported in Figure 3 for each depth and compared with the corresponding curves presented by 3 4 Zeghal et al. [39]. A good agreement with the experimental data can be observed at intermediate strain levels, although the model tends to overestimate the damping at large strains. A small 5 6 amount of viscous damping, equal to 3%, has been introduced into the model through a standard 7 Rayleigh formulation [e.g. 47] to reduce the spurious high frequency spikes and to compensate 8 for the RMW underestimation of damping within the small-strain range (Figure 3). Table 3 9 summarises the RMW model parameters adopted for the different soil layers. It should be noted 10 that the anisotropic and structure degradation features of the model have not been considered in 11 the calibration process, given the lack of appropriate experimental data.

12

#### 13 **4. Results and discussion**

In this section, the free-field site response at Lotung predicted during the weak and strong motion events LSST11 and LSST07 by the equivalent-linear visco-elastic method (EERA), the total stress non-linear scheme (DEEPSOIL) and the fully-coupled effective stress non-linear approach (SWANDYNE II with *RMW*) is presented and the numerical results are consistently compared with the corresponding down-hole accelerometer data recorded at the LSST site.

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#### 20 *4.1 LSST11 event*

Figures 5a and 5b report the acceleration time histories predicted by the three numerical approaches at ground surface and 11 m depth, respectively, when the E-W component of the LSST11 event is applied at the bedrock. The results of the numerical simulations for the N-S component of the same input motion are, instead, presented in Figures 5c and 5d. Given the high frequency content of the LSST11 event, only the first 10 and 5 seconds of the acceleration time

histories obtained during the E-W and N-S motions, respectively, are shown in Figure 5. The 1 2 corresponding down-hole motions recorded in-situ during the earthquake along the DHB array are also reported in Figure 5. For the same LSST11 E-W and N-S events, the response spectra 3 4 (5% damping) at ground surface and 11 m depth are presented in Figure 6. Overall, the results obtained with the three different schemes are in good agreement with the measured data, 5 6 although none of the numerical techniques is able to properly predict the propagation of the high 7 frequencies at ground surface in the E-W direction (Figure 6a) and at 11 m depth in the N-S 8 direction (Figure 6d). Moreover, a better prediction of the maximum acceleration at surface and 9 at 11 m depth is obtained with the fully-coupled non-linear approach in the N-S direction.

10 The profiles of maximum acceleration  $(a_{max})$  and maximum shear strain  $(\gamma_{max})$  induced by the E-11 W motion are shown in Figures 7a and 7b, respectively, whereas the same profiles obtained 12 during the N-S simulations are reported in Figures 7c and 7d. The maximum accelerations 13 recorded along the DHB array during the LSST11 events are shown in Figures 7a and 7c for comparison. The results of EERA and DEEPSOIL are very similar for both components of the 14 15 earthquake and in good agreement with the array data recorded in the E-W direction (Figure 7a). 16 The better SWANDYNE II prediction of the N-S motion previously observed in terms of 17 maximum acceleration at different depths (Figures 5 and 6) is confirmed by the results presented 18 in Figure 7c. In general, the weak LSST11 motions induce small shear strains in the soil column 19 (in the order of 0.01%), not enhancing non-linear effects in the mechanical behaviour of the soil. 20 Therefore, the performance of the simpler numerical approaches is considered to be satisfactory 21 in this case.

22

In contrast with LSST11, the strong motion characteristics of the LSST07 earthquake are expected to induce higher shear strains in the deposit associated with more soil non-linearity,

thus leading to a more demanding test of the predictive capabilities of the numerical techniques
 selected in this work.

The results of the numerical simulations for the E-W and N-S components of the LSST07 3 4 seismic event are shown in Figure 8 in terms of acceleration time histories and are compared with the corresponding down-hole motions recorded along the DHB array. Figure 9 presents the 5 6 same results and comparison in terms of response spectra. EERA is able to predict well the E-W 7 motion, particularly at the depth of 11 m, both in terms of peak acceleration and zero crossing. 8 The peak acceleration at ground surface in the E-W direction is slightly overestimated (Figure 9 8a). On the contrary, a poorer prediction is obtained by applying the N-S component at bedrock 10 level: the equivalent-linear analysis under-predicts the peak acceleration both at ground surface 11 and at the depth of 11 m (Figures 8c and 8d). Moreover, a time shift in the acceleration peak 12 between the recorded and predicted motions can be observed at surface (Figure 8c). It is, in fact, 13 evident from Figures 9c and 9d that EERA is not able to correctly capture the frequency content 14 of the N-S component both at 0 and 11 m depth. A similar performance of the equivalent-linear 15 visco-elastic approach has been observed previously by Borja et al. [23] and Amorosi et al. [29]. 16 With reference to DEEPSOIL, an overall better agreement with the array data is observed both in 17 terms of acceleration time histories and frequency content of the two motions at different depths. 18 Nevertheless, the total stress non-linear approach tends to underestimates the peak values of the 19 accelerations at all depths. This may be due to the choice of the same set of shear modulus and 20 damping ratio curves for the soil between 17 and 47 m, while a more linear behaviour, associated 21 to the increasing confining pressure, could be assigned to the deeper soil layers as suggested by 22 Suwal et al. [30]. The results of the SWANDYNE II simulations show a better prediction of the 23 frequency content of both the E-W and N-S motions than DEEPSOIL and less underestimation 24 of the peak accelerations, a part from the one at 11 m depth in the N-S direction (Figure 9d). 25 With respect to the previous works by Elia et al. [31] and Karofyllakis et al. [32], the assumption

of a more realistic  $G_0$  profile has not affected the already very good prediction of the E-W motion and has slightly improved the results in the N-S direction by reducing the underestimation of the PGA from 31% to 27%. Nevertheless, none of the adopted numerical schemes is able to properly predict the observed considerable amplification of the LSST07 seismic wave occurring in the top layers of the soil deposit in the N-S direction.

The above observations are confirmed by the results plotted in Figure 10 in terms of  $a_{max}$  and 6 7  $\gamma_{max}$  profiles obtained with the three numerical approaches by applying the two components of 8 the LSST07 earthquake at bedrock. The level of maximum shear strains induced by the E-W 9 component is between 0.1% and 0.15%, while it ranges between 0.05% and 0.1% in the N-S 10 direction. Higher shear strains are obtained with the *RMW* model in the top 5 m of the column. 11 This is corroborated by the stress-strain curves shown in Figure 11 for soil elements at ground 12 surface, 11 m and 47 m depth. The stress-strain response of the RMW model is, in fact, softer at 13 ground surface, being then comparable with that obtained through EERA and DEEPSOIL at 14 deeper depths. Given the level of shear strain induced by the strong intensity earthquake (i.e. max 0.15%), the hysteretic damping provided by both the hysteretic model in DEEPSOIL and 15 the RMW model in SWANDYNE II is well within the limits proposed by Zeghal et al. [39] 16 17 reported in Figure 3. Therefore, the under-prediction of the PGAs, especially in the N-S 18 direction, cannot be attributed to over-damping effects.

19 One of the main advantages of using a fully-coupled non-linear approach in geotechnical 20 dynamic analyses is its ability to predict the development of excess pore water pressures ( $\Delta u$ ), 21 related to the accumulation of plastic deformations, during the seismic excitation. This is, 22 obviously, not possible with the equivalent-linear visco-elastic and the total stress non-linear 23 schemes. With this respect, the excess pore water pressure distributions predicted at the end of 24 the SWANDYNE II analyses are shown in Figure 12 for both the LSST07 components. The 25 numerical results are compared with a set of data recorded during the LSST16 event [56], which

was the only earthquake during which excess pore water pressures were measured in-situ. This event was characterised by similar values of peak ground acceleration, epicentral distance and magnitude of those relative to the LSST07 earthquake. The comparison shows a reasonable agreement between recorded and predicted excess pore pressures in the first 20 m of the deposit, for which the data are available.

6

# 7 *4.3 LSST07 event with a shortened soil column*

8 In the case of the strong intensity LSST07 earthquake, the analysis of the numerical predictions 9 presented above suggests that the N-S component reaches the depth of 11 m already significantly 10 damped from the deeper soil layers (Figure 9d), for which a proper geotechnical characterization 11 in terms of shear modulus and damping curves is not available. The predicted motion at 11 m 12 obtained in the E-W direction is, instead, in good agreement with the measured in-situ data 13 (Figure 9b). Therefore, to reduce the uncertainty related to the dynamic properties of the bottom 14 layers, further simulations have been undertaken using the fully-coupled non-linear approach and 15 applying the LSST07 motion recorded at 17 m depth (DHB-17 in Figure 1) as the input action at 16 the base of a 17 m FE column. Firstly, a  $G_0$  profile similar to that adopted in the 47 m column 17 fully-coupled FE analyses has been used on the shortened column. This profile is shown in Figure 13 with a solid line (named "RMW") and it matches the average cross-hole data. Figure 18 19 14 shows the acceleration time histories obtained with SWANDYNE II. Excellent agreement 20 with the down-hole data can be observed at all depths (i.e. 11 m, 6 m and 0 m) in the E-W 21 direction, while the acceleration peaks are substantially underestimated in the N-S direction. This 22 result is consistent with the observations made on the 47 m column, indicating that the overall 23 propagation is not significantly influenced by the bottom layers of the soil deposit (i.e. below 17 24 m).

25 The small-strain shear modulus data obtained from the cross-hole tests at Lotung in the top 17 m

1 shows a natural variability with a standard deviation of approximately 10 MPa [57]. Therefore, a 2 reduced  $G_0$  profile, still within the limits indicated by the in-situ measurements, has been used in the final set of simulations using the shortened column. The adopted profile is shown in Figure 3 4 13 with a dashed line (named "RMW mod") and represents a lower bound of the experimental data set. The corresponding SWANDYNE II results are reported in Figure 15 for the two 5 6 earthquake components. Very good agreement with the down-hole data can be observed at 11 m 7 depth in both directions (Figures 15c and 15f) and at 6 m depth in the E-W direction (Figure 8 15b). The maximum acceleration at ground surface in the N-S direction is almost identical to the 9 measured one (Figure 15d), although the PGA of the E-W component is now overestimated 10 (Figure 15a). The analysis of the 17 m column results confirms the idea that, even adopting the 11 most advanced numerical approach, it is quite difficult to perfectly estimate the peak 12 accelerations at ground surface of both the LSST07 input motions: when the PGA of the E-W 13 component is well captured, the PGA in the N-S direction is under-predicted and, vice versa, 14 when the N-S component is well predicted, the PGA of the E-W motion is overestimated.

15

#### 16 **5. Conclusion**

17 The ground response analysis of the Large-Scale Seismic Test site in Lotung has been studied in 18 this work using the simple equivalent-linear visco-elastic method (EERA), a total stress non-19 linear FE technique (DEEPSOIL) and a fully-coupled non-linear FE approach (SWANDYNE 20 II). The incorporation of plasticity, through a hyperbolic model in DEEPSOIL and the advanced 21 *RMW* model in SWANDYNE II, has allowed investigation of the effects of soil non-linearity on 22 the propagation of a weak and a strong motion. The comparison with recorded array data has 23 enabled the assessment of the performance of three numerical schemes and the following 24 conclusions can be drawn:

25

i) While a weak motion is reasonably predicted by all numerical methods, the equivalent-

linear visco-elastic method cannot capture the frequency content and peak acceleration of
 the strong intensity event, especially at ground surface. On the contrary, the predictions
 obtained with the fully-coupled effective stress approach implementing the kinematic
 hardening elasto-plastic *RMW* model are particularly successful.

- ii) The performance of DEEPSOIL sits somehow between a simpler visco-elastic approach
  and more advanced fully-coupled schemes. The significant underestimation of the peak
  values of the accelerations during the strong motion event can be possibly attributed to
  the choice of stiffness and damping curves for the deeper soil layers, as suggested by
  Suwal *et al.* [30].
- iii) The underestimation of the peak accelerations recorded at ground surface during the
   strong intensity event cannot be attributed to over-damping effects, as the hysteretic
   damping provided by the hyperbolic and *RMW* models is well within the limits of the
   experimental data for the shear strain level induced by the earthquake.
- 14 iv) The influence of the poor geotechnical information regarding the dynamic properties of 15 the deeper soil layers at Lotung has been analysed by reducing the depth of the 16 investigated soil column to the top 17 m, showing a good dynamic characterisation. The 17 results obtained with the fully-coupled non-linear scheme for the LSST07 strong motion 18 event highlight the importance of the  $G_0$  profile adopted in the simulations. Specifically, 19 a shear stiffness profile plotting on the lower boundary of the experimental data produces 20 a better prediction of the PGA in the N-S direction, although the maximum acceleration 21 during the E-W motion may be overestimated.
- v) In this paper, the two horizontal components of the input motions have been applied
  separately at bedrock level. The recent work by Amorosi *et al.* [33] seems to indicate that
  the simultaneous application of the two components of the earthquake in 3D FE
  simulations of the LSST site does not lead to a better prediction of the down-hole data,

possibly due to a less dissipative response of the soil model under multi-directional
 loading conditions. Nevertheless, further laboratory and constitutive modelling
 investigation is needed to prove this conclusion.

In general, the presented work highlights the usefulness of down-hole array data for the validation of site response analyses. The paper emphasises the importance of proper site characterisation in terms of soil dynamic properties and the limits and benefits of the currently available numerical techniques for seismic ground response simulations. A parametric analysis performed to simulate different decay curves for the deeper soil layers has not led to important deviations in the results. Nevertheless, the sensitivity of advanced site response models to statistical variations in soil properties represents an area of future research.

11

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Date	Name	Focal depth	Magnitude	Magnitude PGA (g		)
		(km)	$(M_L)$	E-W	N-S	V
17 July 1986	LSTT11	2.0	4.3	0.07	0.10	0.04
20 May 1986	LSTT07	15.8	6.2	0.16	0.20	0.04

Table 1 Earthquakes recorded by the LSST array and used in the simulations

 Table 2 Material parameters for the MRDF pressure-dependent hyperbolic model

Depth	β	S	γr	$\xi_{small}$ (%)	<i>p</i> 1	$p_2$	<i>p</i> <sub>3</sub>
0-6 m	1.365	0.975	0.0536	1.0228	0.96	0.400	2.10
6-11 m	1.425	0.840	0.0516	1.2051	1.00	0.400	2.70
11-47 m	1.470	0.705	0.0536	1.3147	1.00	0.158	3.25

 Table 3 Material parameters for the RMW model

Depth	$\lambda^{*}$	$\kappa^{*}$	М	R	В	Ψ	$r_0$	Α	т	п
0-17 m	0.03	0.0015	0.922	0.08	0.60	1.0	1.0	1500	0.36	0.82
17-23 m	0.03	0.0015	1.096	0.08	0.60	1.0	1.0	1900	0.36	0.82
23-29 m	0.03	0.0015	0.814	0.08	0.60	1.0	1.0	1350	0.36	0.82
29-36 m	0.03	0.0015	0.941	0.08	0.60	1.0	1.0	1900	0.36	0.82
36-47 m	0.03	0.0015	0.730	0.08	0.60	1.0	1.0	1150	0.36	0.82



Fig. 1. Location of instrumentation: (a) elevation view; (b) plan view (after Elgamal et al. [36]).



Fig. 2. Local soil profile at LSST site: (a) stratigraphy; (b) SPT log; (c) elastic shear modulus.



Fig. 3. Shear modulus degradation and damping curves at different depths.



Fig. 4. Response spectra of the accelerations recorded during the LSST11 and LSST07 events.



Fig. 5. Comparison between predicted acceleration time histories and LSST11 array data.



Fig. 6. Comparison between predicted response spectra and LSST11 array data.



**Fig. 7.** Comparison between LSST11 array data and numerical predictions in terms of (a, c) maximum acceleration and (b, d) maximum shear strain.



Fig. 8. Comparison between predicted acceleration time histories and LSST07 array data.



Fig. 9. Comparison between predicted response spectra and LSST07 array data.



**Fig. 10.** Comparison between LSST07 array data and numerical predictions in terms of (a, c) maximum acceleration and (b, d) maximum shear strain.





**Fig. 12.** Comparison between excess pore water pressure distributions predicted at the end of the SWANDYNE II analyses and the data recorded during the LSST16 event [55].



Fig. 13. Elastic shear modulus profiles adopted in the 17 m soil column analysis.



Fig. 14. Acceleration time histories obtained in the 17 m soil column analysis assuming the "RMW" profile.



Fig. 15. Acceleration time histories obtained in the 17 m soil column analysis assuming the "RMW\_mod" profile.