NON-LINEAR FINITE ELEMENT ANALYSIS OF SITE EFFECTS AT LOTUNG (TAIWAN)

Gaetano ELIA[1], Dimitrios KAROFYLLAKIS[2] and Mohamed ROUAINIA[3]

Abstract: This paper presents a fully-coupled non-linear numerical modelling approach for the prediction of the Lotung Large-Scale Seismic Test (LSST) site free-filed response during the recorded event of May 1986. An advanced kinematic hardening soil model is calibrated using resonant column data and cross-hole measurements available at the LSST site. The two horizontal components of the input motion are applied separately at bedrock level. The acceleration time histories predicted at different depths are compared with the down-hole motions recorded in-situ, indicating a very good agreement with the array data in terms of peak ground acceleration and frequency content.

Introduction

Experience from historical (e.g. Niigata, Japan and Alaska in 1964) and recent strong earthquakes (e.g. L'Aquila in 2009, Chile in 2010, Tohoku, Japan and Christchurch in 2011, amongst the others) has demonstrated the significance of local soil conditions on the seismic ground response. The changes in amplitude, frequency 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 infrastructures during earthquakes.

Site response analysis methods allow geotechnical engineers to quantify the effects of soil deposits on the propagation of waves from bedrock to ground surface. These methods can be divided into: i) frequency-domain schemes (using linear equivalent methods) and ii) time-domain schemes (usually performed with finite element codes). The equivalent visco-elastic approach has been widely adopted during the last thirty years in the engineering practice, although its limitations have been highlighted in the literature (e.g. Kwok *et al*., 2007; Amorosi *et al*., 2010). Alternatively, time-domain finite element (FE) schemes are nowadays available to solve the ground response in a more accurate way, accounting for the solid-fluid interaction by means of a coupled effective stress formulation (e.g. Zienkiewicz *et al*., 1999). In those schemes, the behaviour of soil can be described using non-linear constitutive models with different level of complexity. From the validation point of view, the performance of the different numerical approaches to simulate the complex wave propagation process has been tested over the last decades using real vertical array data and/or laboratory data obtained from centrifuge tests (e.g. Borja *et al*., 1999; Elgamal *et al*., 2002; Phillips and Hashash, 2009; Amorosi *et al*., 2011).

This paper presents a validation study using the recordings from accelerometer arrays installed at the Lotung Large-Scale Seismic Test (LSST) site in Taiwan. In particular, the prediction of the free-field response at Lotung during the event recorded in May 1986 is investigated through a fully-coupled non-linear approach.

In the first part of the paper the LSST site is briefly described. Then, the numerical model adopted for the time-domain dynamic simulations is summarized along with the calibration of the soil constitutive model against in-situ and laboratory data. Finally, the comparison between predicted and recorded motions at different depths within the soil deposit is discussed. The results of the investigation are particularly encouraging, but also indicate that further improvements of the numerical model are still necessary to better capture the observed free-field response.

[1] Lecturer, Newcastle University - UK, E-mail: gaetano.elia@ncl.ac.uk
[2] Ph.D. Student, Newcastle University - UK, E-mail: d.karofyllakis@ncl.ac.uk
[3] Senior Lecturer, Newcastle University - UK, E-mail: mohamed.rouainia@ncl.ac.uk

Lotung LSST site

The Large-Scale Seismic Test (LSST) is located in one of the most seismically active region in the North-East of Taiwan, and has been originally established in the 80's to study the dynamic behaviour of two scaled-down nuclear plant containment structures (Tang *et al*., 1990). The site response has been monitored by a number of surface and down-hole accelerometer arrays, together with pore pressure transducers. The down-hole accelerometers have been installed at depths of 0, 6, 11, 17 and 47 m, oriented in N-S, E-W and vertical directions. Figure 1 shows the elevation and plan views of the instrumentation. Of particular interest here is the DHB vertical array which has been considered as representative of the free-field response at Lotung.

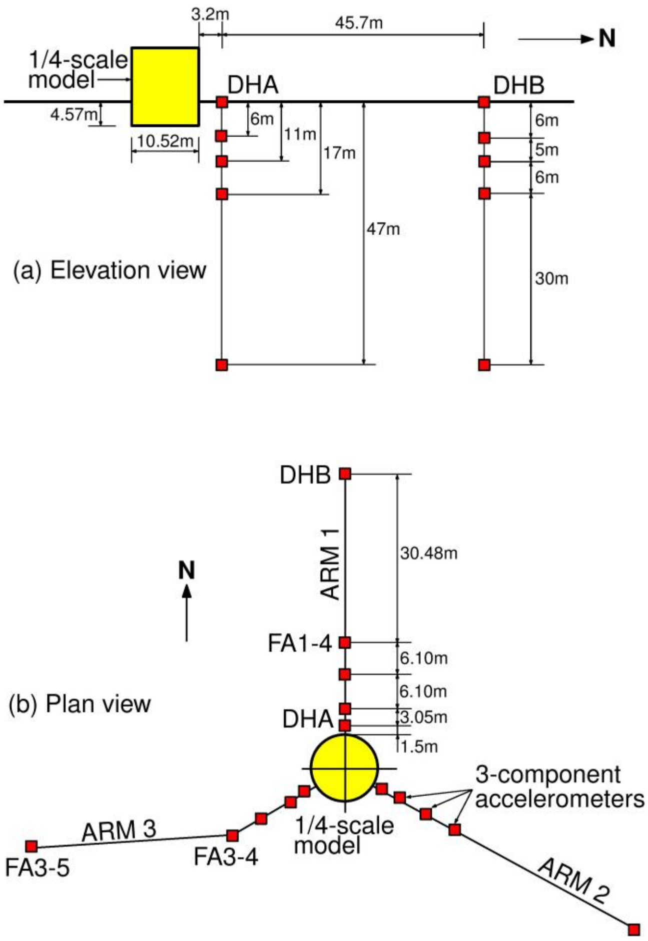

Figure 1. Location of instrumentation: (a) elevation view; (b) plan view (modified from Elgamal *et al*., 1995)

The site geology consists of recent alluvium and Pleistocene materials over a Miocene basement. The upper alluvial layer, 30 to 40 m thick, consists mainly of clayey-silts and silty-clays (Anderson, 1993). The water table is located approximately at a depth of 1 m. The local geological profile shows a first layer of grey silty-sand and sandy-silt about 20 m thick underlain by about 10 m of more gravelly layer resting on a thick deposit of silty clay, as indicated by the SPT log profile reported in Figure 2(a).

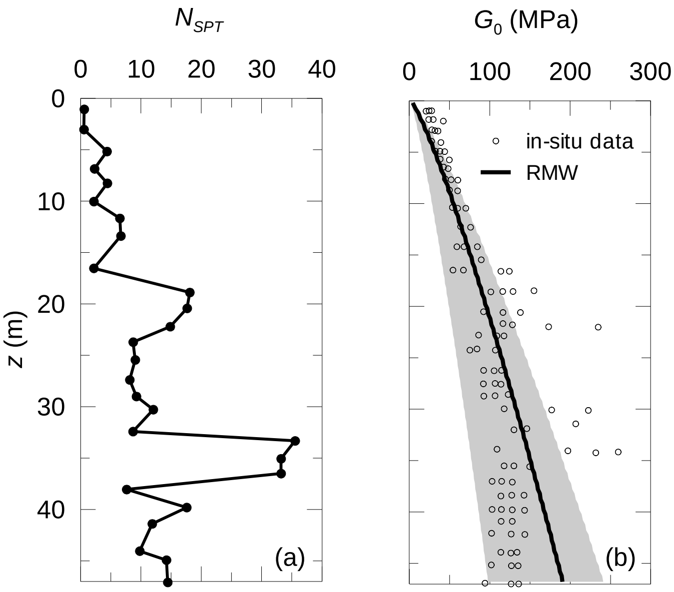

Figure 2. Local soil profile at LSST site: (a) SPT log; (b) elastic shear modulus

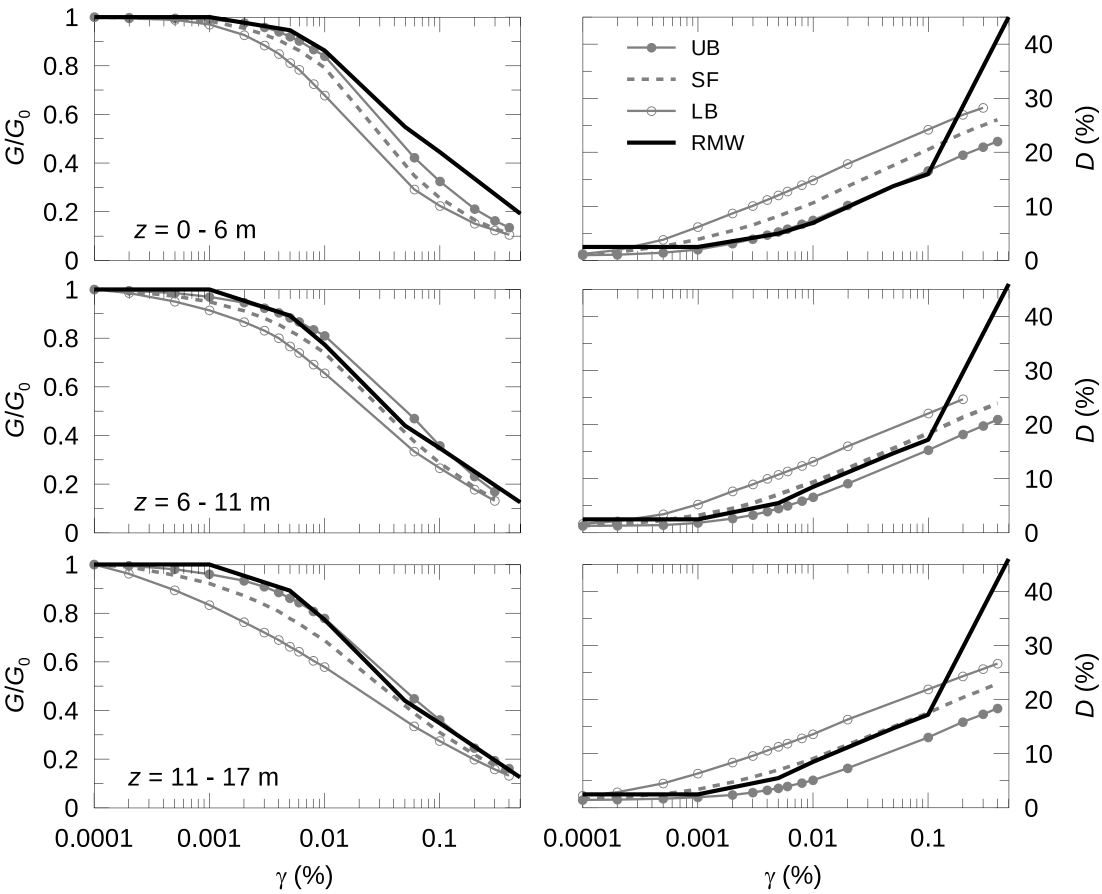

Figure 3. Shear modulus and damping curves developed by Zeghal *et al.* (1995) and RMW predictions

A series of geophysical seismic tests have been performed to measure shear and compression wave velocities at the LSST site. Figure 2(b) shows the elastic shear modulus data obtained from seismic cross-hole tests (Anderson and Tang, 1989). The bedrock formation can be assumed to be at a depth of 47 m. Shear modulus and damping ratio curves have been measured through resonant column and cyclic torsion tests on undisturbed specimens (Anderson and Tang, 1989; EPRI, 1993). Alternatively, Zeghal *et al.* (1995) have proposed to back-figure the in-situ moduli ratio curves for Lotung soil based directly on its 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 and D-γ curves have been developed for the depths of 0-6, 6-11 and 11-17 m, producing for each depth a least-square best fit (SF) as well as an upper (UB) and a lower bound (LB) curve indicative of the possible variations in the material dynamic properties (Figure 3). The shear modulus and damping ratio curves of the soil between 17 and 47 m have been assumed to be the same as those from 11 to 17 m.

Numerical model

The ground response analysis of the Lotung experiment site has been undertaken using the two-dimensional fully-coupled finite element code SWANDYNE II (Chan, 1995). The code allows to perform linear and non-linear dynamic analyses, using the Generalised Newmark method (Katona and Zienkiewicz, 1985) for time integration. In particular, the values of the Newmark parameters selected in all the FE analyses illustrated in this note are β_1 = 0.600 and β_2 = 0.605 for the solid phase and β_1^* = 0.600 for the fluid phase. These values ensure that the algorithm is unconditionally stable, while being dissipative mainly for the high frequency modes (Zienkiewicz *et al*., 1999). A 5 m wide, 47 m high FE mesh composed by 235 isoparametric quadrilateral finite elements with 8 solid nodes and 4 fluid nodes has been used in the dynamic simulations. The base of the mesh has been assumed to be rigid, while equal displacements have been imposed to the nodes along the vertical sides (i.e. tied-nodes lateral boundary conditions). Base and lateral hydraulic boundaries have been assumed as impervious, while drained conditions have been imposed at the top of the mesh. A small amount of viscous damping, equal to 3%, has been introduced into the model through a standard Rayleigh formulation (e.g. Clough and Penzien, 2003) to reduce the high frequency spurious spikes.

In order to investigate the effects of soil non-linearity on the wave propagation process, plasticity has been implemented in the FE simulations through the advanced elasto-plastic model (RMW) developed by Rouainia and Muir Wood (2000). The RMW model allows to reproduce some of the key features of the cyclic behaviour of natural soils as the decay of the shear stiffness with strain amplitude, the corresponding increase of hysteretic damping and the related accumulation of excess pore water pressure under undrained conditions. The model has been implemented in SWANDYNE II with an explicit stress integration algorithm adopting a constant strain sub-stepping scheme. RMW has been successfully employed to simulate both static (Gonzáles *et al.*, 2012; Panayides *et al.*, 2012) and dynamic geotechnical problems (Elia and Rouainia, 2013; 2014). For more details on its formulation and implementation the reader is referred to Rouainia and Muir Wood (2000) and Zhao *et al*. (2005). 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, i.e.:

$$K = \frac{p}{\kappa^*} \qquad\qquad G_0 = \frac{3(1-2\nu)}{2(1+\nu)}K \tag{1}$$

where ν is the Poisson's ratio and κ^* is the slope of the swelling line in a bi-logarithmic compression plane. The bulk and shear moduli obtained with Equations (1) are linearly dependent on the mean effective pressure only. In this work, the well-known equation proposed by Viggiani and Atkinson (1995) for the small-strain shear modulus has been implemented to reproduce the dependency of G_0 on the mean effective stress (p) and the overconsolidation ratio (OCR):

$$\frac{G_0}{p_r} = A\left(\frac{p}{p_r}\right)^n OCR^m \tag{2}$$

where p_r is the reference pressure (equal to 1 kPa). The non-dimensional parameters A, m and n depend on the properties of the soil and can be determined as function of the plasticity index. In the initialisation of the numerical 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 constant *OCR* of 2 has been imposed for the remaining part of the model. The assumed variation of *OCR* with depth is deemed to be realistic in accordance with the G_0 data shown in Figure 2(b), where a non zero elastic shear modulus of about 25 MPa can be observed near the ground surface (corresponding to a measured shear wave velocity of about 100 m/s). Once the FE model has been initialised, a range of possible elastic shear modulus profiles, reported in Figure 2(b) with the shaded area, have been investigated and the final G_0 profile adopted in the FE dynamic simulations is shown in the same figure with a solid black line.

Numerical simulations of undrained cyclic simple shear tests have been carried out with RMW in order to produce normalized shear modulus G/G_0 and damping D curves. The secant shear modulus and the damping ratio for each shear strain amplitude have been assessed after 500 load cycles, a number sufficient to reach steady-state condition. The results of the single element tests performed with the RMW model are reported in Figure 3 with a solid black line for each depth and compared with the corresponding curves presented by Zeghal *et al.* (1995). Table 1 summarises the material parameters adopted for the different soil layers.

Table 1. Material parameters for the RMW model

Depth	λ^*	κ^*	M	ν	R	B	ψ	r_0
0-17 m	0.03	0.0015	0.922	0.3	0.08	0.60	1.0	1.0
17-23 m	0.03	0.0015	1.096	0.3	0.08	0.60	1.0	1.0
23-29 m	0.03	0.0015	0.814	0.3	0.08	0.60	1.0	1.0
29-37 m	0.03	0.0015	0.941	0.3	0.08	0.60	1.0	1.0
37-47 m	0.03	0.0015	0.730	0.3	0.08	0.60	1.0	1.0

Results and discussion

In this section, the non-linear FE dynamic analyses undertaken to predict the free-field response of the Lotung site during the LSST7 event occurred on May 1986 are presented. The earthquake, characterised by maximum accelerations of 0.16 g and 0.21 g respectively in the E-W and N-S direction, has been selected due to its strong-motion characteristics. The two horizontal components of the seismic event have been applied separately as input motions at the bedrock level (i.e. at 47 m).

The results of the advanced FE simulations are reported in Figure 4 in terms of acceleration time histories recorded at depths 0 and 11 m. The corresponding down-hole motions recorded in-situ during the earthquake along the DHB array (Tang, 1987), named FA1-5 and DHB-11, are shown in the same figure. The time-domain analyses, carried out with SWANDYNE II and employing the RMW model, are in good agreement with the recorded data, especially at the depth of 11 m from surface, for both the E-W and N-S component. The peak acceleration of the E-W motion at ground surface is perfectly predicted, while a small under-estimation of the PGA can be observed in the N-S direction. A very good agreement with the array data is also obtained in terms of frequency content.

Conclusions

In this paper the ground response of the Lotung experiment site during the LSST7 event has been simulated using a fully-coupled non-linear numerical modelling approach. An advanced elasto-plastic soil model has been calibrated using resonant column data and in-situ cross-hole measurements. The two horizontal components of the input motion have been applied separately at the rigid base of a two-dimensional FE model and non-linear time-domain analyses have been performed. The results of the advanced numerical simulations have been compared with the down-hole motions recorded in-situ during the investigated seismic event in terms of acceleration time histories. The direct comparison with recorded array data has shown a very good performance of the adopted numerical scheme. Further investigation is needed to improve the numerical prediction of the observed ground response, particularly with regard to the peak ground acceleration in the N-S direction.

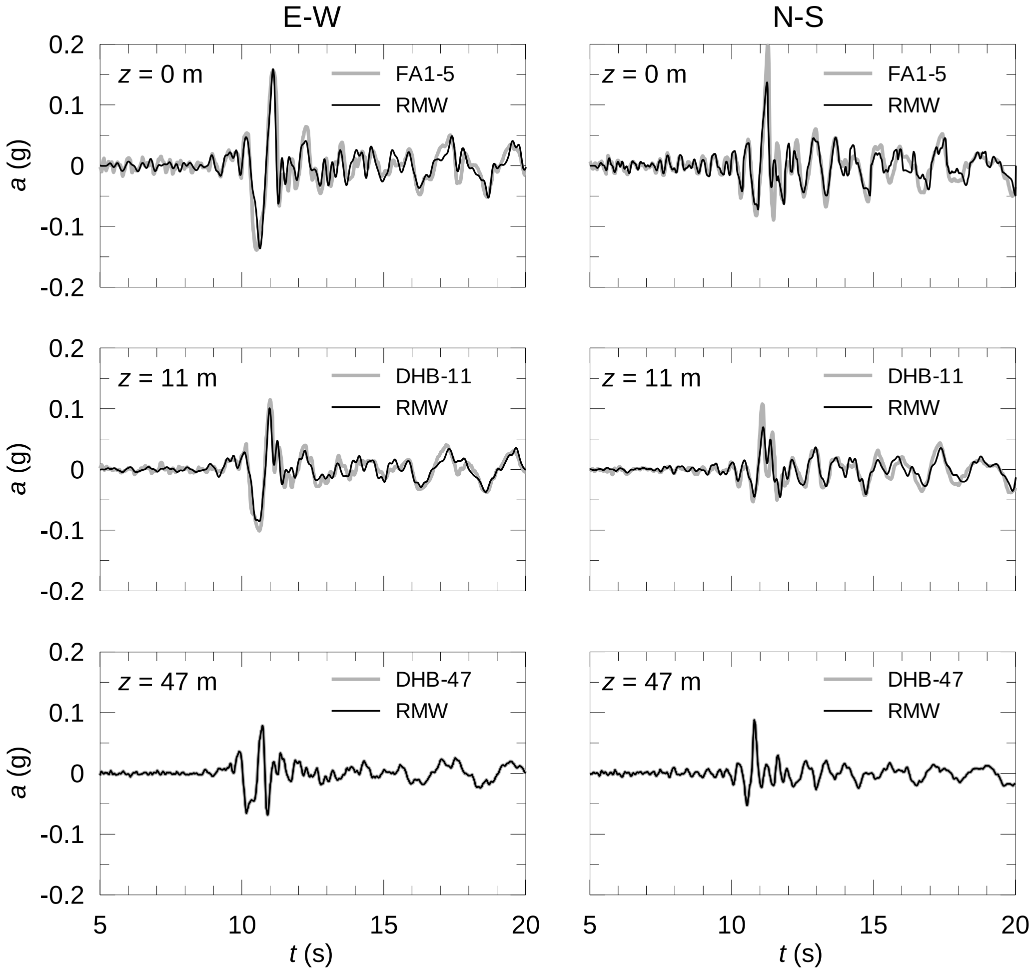

Figure 4. Comparison between predicted acceleration time histories and array data

REFERENCES

Amorosi A, Boldini D, Elia G (2010) Parametric study on seismic ground response by finite element modelling, *Computers & Geotechnics*, 37(4):515-528

Amorosi A, Elia G, Boldini D, Schiavone F (2011) *Seismic ground response analysis: comparison between numerical simulations and observed array data*, Proc. 5th Int. Conf. on Earthquake Geotechnical Engineering, Santiago, Chile, 10-13 January

Anderson DG (1993) *Geotechnical synthesis for the Lotung large-scale seismic experiment*, Tech. Rep. No. 102362, Electric Power Research Institute, Palo Alto, California

Anderson DG and Tang YK (1989) *Summary of soil characterization program for the Lotung large-scale seismic experiment*, Proc. EPRI/NRC/TPC Workshop on Seismic soil-struct. interaction anal. techniques using data from Lotung, Taiwan, EPRI NP-6154, Electric Power Research Institute, Palo Alto, California, 1, 4.1-4.20

Borja RI, Chao HY, Montáns FJ, Lin CH (1999) Nonlinear ground response at Lotung LSST site, *Journal of Geotechnical and Geoenvironmental Engineering*, 125(3):187-197

Chan AHC (1995) *User Manual for DIANA-SWANDYNE II*, School of Engineering, University of Birmingham, UK

Clough RW and Penzien J (1993) *Dynamics of Structures*, 2nd Ed., McGraw-Hill Book Company, New York

Elgamal AW, Yang Z, Parra E (2002) Computational modelling of cyclic mobility and post-liquefaction site response, *Soil Dynamics and Earthquake Engineering*, 22:259-271

Elgamal AW, Zeghal M, Tang HT, Stepp JC (1995) Lotung downhole array. I: Evaluation of site dynamic properties, *Journal of Geotechnical Engineering*, 121(4):350-362

Elia G and Rouainia M (2013) Seismic performance of earth embankment using simple and advanced numerical approaches, *Journal of Geotechnical and Geoenvironmental Engineering*, 139(7):1115-1129

Elia G and Rouainia M (2014) Performance evaluation of a shallow foundation built on structured clays under seismic loading, *Bulletin of Earthquake Engineering*, 12(4):1537-1561

EPRI (1993) *Guidelines for determining design basis ground motions – Vol. 1: Method and guidelines for estimating earthquake ground motion in Eastern North America*, Rep. No. TR-102293, Electric Power Research Institute, Palo Alto, California

Gonzáles NA, Rouainia M, Arroyo M, Gens A (2012) Analysis of tunnel excavation in London Clay incorporating soil structure, *Géotechnique*, 62(12):1095-1109

Katona MG and Zienkiewicz OC (1985) A unified set of single step algorithms Part 3: the Beta-m method, a generalisation of the Newmark scheme, *International Journal of Numerical Methods in Engineering*, 21:1345-1359

Kwok AOL, Stewart JP, Hashash YMA, Matasovic N, Pyke R, Wang Z, Yang Z. (2007) Use of exact solutions of wave propagation problems to guide implementation of nonlinear seismic ground response analysis procedures, *Journal of Geotechnical and Geoenvironmental Engineering*, 133(11):1385-1398

Panayides S, Rouainia M, Muir Wood D (2012) Influence of degradation of structure on the behaviour of a full-scale embankment, *Canadian Geotechnical Journal*, 49:344-356

Phillips C and Hashash YMA (2009) Damping formulation for nonlinear 1D site response analyses, *Soil Dynamics and Earthquake Engineering*, 29:1143-1158

Rouainia M and Muir Wood D (2000) A kinematic hardening constitutive model for natural clays with loss of structure, *Géotechnique*, 50:153-164

Tang HT (1987) *Large-scale soil structure interaction*, Rep. No. NP-5513-SR, Electric Power Research Institute, Palo Alto, California

Tang HT, Tang YK, Stepp JC (1990) Lotung large-scale seismic experiment and soil-structure interaction method validation, *Nuclear Engineering and Design*, 123:197-412

Viggiani G and Atkinson JH (1995) Stiffness of fine-grained soil at very small strains, *Géotechnique*, 45(2):249-265

Zeghal M, Elgamal AW, Tang HT, Stepp JC (1995) Lotung downhole array. II: Evaluation of soil nonlinear properties, *Journal of Geotechnical Engineering*, 121(4):363-378

Zhao J, Sheng D, Rouainia M, Sloan SW (2005) Explicit stress integration of complex soil models, *International Journal of Numerical and Analytical Methods in Geomechanics*, 29(12):1209-1229

Zienkiewicz OC, Chan AHC, Pastor M, Schrefler BA, Shiomi T (1999) *Computational Geomechanics (with special reference to earthquake engineering)*, Wiley & Sons, Chichester