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Prediction of non-linear soil behaviour in saturated sand: A loosely coupled approach for 1D effective stress analysis

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ABSTRACT: Since the catastrophic failures due to soil liquefaction occurred during Niigata (Japan) and Good Friday (Alaska) 1964 earthquakes, increasing efforts have been devoted to the understanding and modelling the phenomenon as well as to the development of effective methods for predicting non-linear soil behaviour at large strains. In this paper, a simplified constitutive model, implemented in a one-dimensional code, is proposed, following a loosely coupled approach. The most important advantage of the adopted approach is that it is at the same time easy to be applied as a semi-empirical method and reliable as an advanced constitutive model. The performances of the proposed procedure, are validated on a well-documented test-site in the north-east of Japan where acceleration records along a vertical array are available. The site was chosen because its geological soil configuration is essentially 1D and detailed site and laboratory investigation are available. Both the accuracy and effectiveness of the procedure are assessed by comparing calculated predictions and field records, in terms of acceleration time histories and spectral accelerations at surface. Moreover, the results of the above simulations were compared with those obtained carrying out total stress analyses and several constitutive models, showing the importance of a coupled approach for a reliable prediction of seismic response of saturated sand.

1 INTRODUCTION

Seismic strong motion events that occurred during past years, such as the 2018 Palu (Indonesia), the 2012 Emilia (Italy), the 2010-2011 Canterbury (New Zealand) and the 2011 Tohoku-Oki (Japan) earthquakes, demonstrated that coupled phenomena in sandy/silty soils cannot be neglected in the analysis of site response; nevertheless, this issue is still a demanding task for the design of infrastructures and facilities.

The soil response of saturated soils under strong motion seismic loading is ruled by rather complex mechanical processes, which may basically be ascribed to hysteretic behaviour and volumetric-distortional coupling. The former consists of the decrease in the shear modulus associated with an increase in energy dissipation (damping); the latter is related to plastic irreversible strain that takes place at high shear loads and induces either volumetric strains under drained conditions or pore pressure changes under undrained conditions. These two processes may evolve into liquefaction in loose sandy soils.

To correctly predict the soil non-linear behaviour, different methods for site response analysis were defined according to the expected strain levels mobilized by the reference input motion. Equivalent linear method involves a linear computation in the frequency domain, coupled to an iterative process that update the value of the shear modulus and damping ratio according to the maximum shear strain calculated at each iteration. Even though this is the most common method used in the engineering practice, the equivalent linear approach should not be used beyond strain levels around 0.2%, as largely verified by Régnier et al. (2018; 2016). For higher

strain levels, non-linear site response analysis in the time domain are recommended, as actually possible through several common codes and constitutive models (Hashash et al. 2016; ITASCA 2016; Brinkgreve et al. 2016; GEOSLOPE 2012). Neverthless, total stress analysis could be unsatisfactory in predicting the dynamic behaviour of saturated loose soil deposits.

A representative example is the Japanese site of Sendai Port, where a pair of accelerometers at surface and depth were located by Port and Airport Research Institute (PARI).

This site was selected during the PRENOLIN project - Prediction of non-linear effects – (Régnier et al. 2016; 2018) to verify the code-to-code variability on 1D nonlinear soil response analysis and validate the performances of the codes on real instrumented sites. During the simulation exercise, the initial plan was to use the shear modulus reduction and damping ratio curves as measured in laboratory tests. However, the curves were modified to ensure a better fit to the strong-motion data during the calculations in total stress (Régnier et al. 2018).

As a matter of fact, the modelling of volumetric/distortional coupling remains a challenging issue due to the limitations of simplified approaches and to the specific expertise required for calibrating advanced constitutive models.

In this study, a simplified pore water pressure model, following a stress-based approach, implemented in a 1D computer code was adopted in order to carry out effective stress analyses (Tropeano et al. 2019). The model does not need either the evaluation of the number of equivalent uniform stress cycles (usually required by simplified models) or the assessment of complex soil parameters (as required by sophisticated plasticity-based approaches), representing a useful tool for engineering practice, since it requires only a few parameters that are clearly defined and easy to calibrate.

The description of the numerical code is briefly presented in the following section 2 and the performance of the numerical code to successfully model the seismic response of Sendai site is widely described and discussed in section 3. Comparison with total stress analysis and other effective stress simulations performed during the PRENOLIN project permits to gain awareness about the usefulness of an effective stress analysis, even though by a simplified approach.

2 PORE WATER PRESSURE MODEL FOR EFFECTIVE STRESS ANALYSIS

A simplified pore water pressure model, here called 'PWP model' (Chiaradonna 2016; Chiaradonna et al. 2018) was implemented in the non-linear code SCOSSA which models the soil profile as a system of consistent lumped masses, connected by viscous dampers and springs with hysteretic behaviour (Tropeano et al. 2016; 2019). The non-linear shear stress-strain relationship is described by the MKZ model (Kondner and Zelasko, 1963) and the modified Masing rules (Phillips and Hashash, 2009). The code permits to select both options of total and effective stress analyses.

The proposed 'PWP model' permits to compare the irregular time-history of shear stress induced by earthquake with the soil liquefaction resistance, evaluated in stress-controlled cyclic laboratory tests. The comparison is expressed through the so called 'damage parameter', which can be computed for any loading pattern. The damage parameter, k, is an incremental function of the applied load that considers the cyclic strength of the soil. This latter is expressed in terms of cyclic resistance curve, analytically described by the equation:

$$\frac{(CRR - CSR_t)}{(CSR_r - CSR_t)} = \left(\frac{N_r}{N_L}\right)^{1/\alpha} \tag{1}$$

where CSR is the shear stress amplitude normalized by the initial effective confining pressure; N is the number of cycles, CSR_r is the ordinate of the curve for $N_L = 15$ (usually adopted as a reference number of cycles). The parameters α and CSR_t respectively describe the steepness and the horizontal asymptote of the curve, as shown in Figure 1a.

For a regular shear stress history, k is proportional to the number of cycles, N; it is therefore possible to express the pore pressure ratio, r_u (ratio between the excess pore pressure and

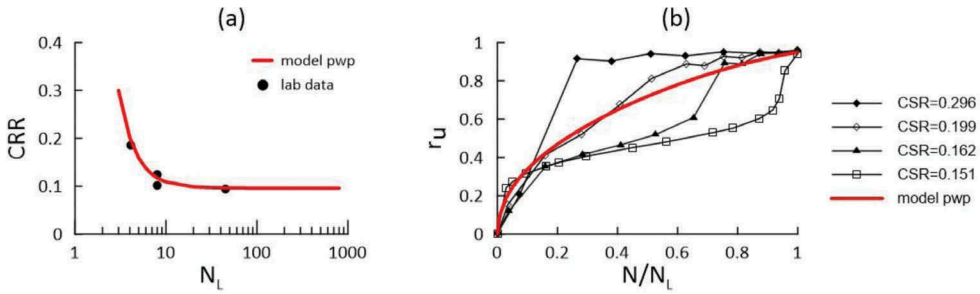

Figure 1. (a) Cyclic resistance and (b) excess pore pressure ratio curves for Sendai sand.

the initial effective confining pressure), as a function of the damage parameter, through the relationship proposed by the authors (Chiaradonna et al. 2016; 2018):

$$r_u = a\left(\frac{\kappa}{\kappa_L}\right)^b + c\left(\frac{\kappa}{\kappa_L}\right)^d \tag{2}$$

where *a, b* and *d* are parameters that control the shape of the curve, which can be easily obtained by best-fitting the data measured in cyclic laboratory tests (Figure 1b).

Eq. (2) is used to compute the generated excess pore pressure in perfectly undrained conditions. To account for the dissipation of the excess pore pressure, the one-dimensional consolidation theory has been adopted (Terzaghi, 1943), thus reducing the excess pore pressure produced at each time step in function of the consolidation coefficient of the soil. The normalized damage parameter is also reduced according to eq. (2). Consequently, the damage parameter is computed as a balance between the damage generated by the load and the reduction induced by the consolidation process.

More details about the numerical implementation of the simplified PWP model in the computer code SCOSSA can be found in Tropeano et al. (2019).

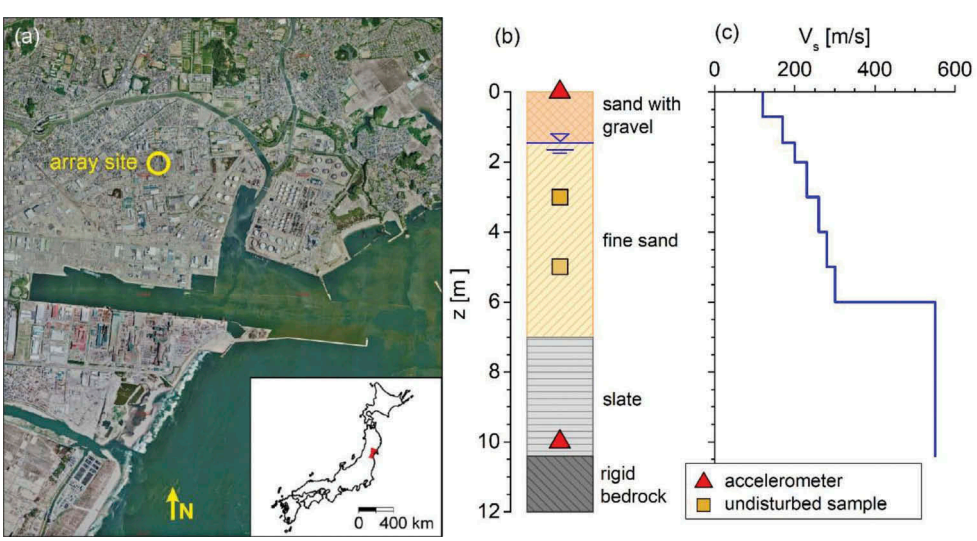

Figure 2. Position of Sendai in Japan with location of the Sendai vertical array (a), soil model (b) and V_S profile (c) (mod. after Tropeano et al. 2019).

3 SENDAI SITE (JAPAN)

A validation benchmark of the numerical code is proposed with reference to a seismic recording array located near Sendai port in northeast Japan (Figure 2a). The recording station, part of the PARI network, consists of a surface accelerometer and a downhole geophone at 10.4 m depth. The array is placed in a Holocene sedimentary soil, called "beach ridge", consisting of gravel and sand of marine origin. This surface deposit is underlain by the Pliocene Geba Formation, which forms the northern and eastern hills and consists of gravel stone, sandstone, tuff, tuffaceous siltstone and lignite (OYO Corporation, 2014).

The Sendai array site was selected as a validation benchmark in the framework of the PRENOLIN project because the subsoil layering is approximately horizontal (as checked by several multichannel analysis of surface waves (MASW) tests) and the S-wave propagation direction of the selected seismic records is approximately vertical.

The upper part of the soil column is composed of loose gravel with a thickness of 1.25 m, overlying 5.9 m of a moderately dense fine sand (Figure 2b). From 7.15 m in depth, a stiff slate formation can be considered the seismic bedrock. The groundwater table is located 1.45 m below the surface.

3.1 *Recorded motions*

In the framework of the PRENOLIN project, six recorded input motions were selected, representing three different peak ground acceleration (PGA) levels (≥ 0.06, 0.02-0.03 g and ≤ 0.01 g) and three distinct frequency ranges (Régnier et al. 2018).

For sake of simplicity, only the three records with low frequency content are reported in this paper, while the other simulations can be found in Tropeano et al. (2019).

The characteristics of the downhole and surface records are listed in Table 1, where the numbering of input motions corresponds to decreasing PGA amplitudes. Figure 3 shows the acceleration response spectra of the recorded downhole and surface records.

3.2 *Geotechnical model*

Downhole PS logging permitted obtaining the in situ shear wave velocity profile; the stress-strain and strength properties were measured by laboratory tests on undisturbed samples.

Table 2. EW components of the downhole and surface records.

Record #	Moment Magnitude M_w	Epicentral distance R_{epi} [km]	downhole PGA PGA_d [g]	surface PGA PGA_s [g]
1	9	163	0.257	0.491
4	6.8	169	0.025	0.091
8	6.4	208	0.005	0.007

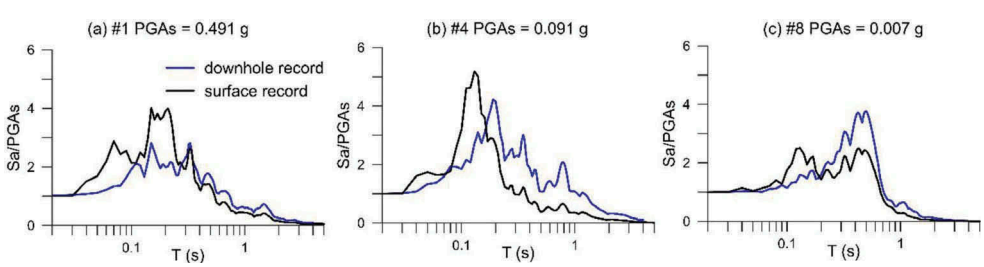

Figure 3. Acceleration response spectra of low-frequency motions recorded downhole (green lines) and at the surface (black lines).

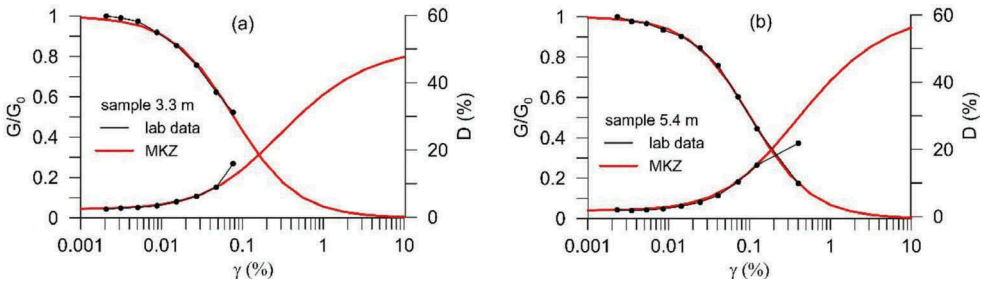

Figure 4. Results and interpretation of pre-failure cyclic triaxial tests on undisturbed samples retrieved from (a) 3.3 m and (b) 5.4 m from the surface.

The shear wave velocity profile adopted in the analysis (Figure 2c) was obtained by adjusting the in situ measurements so that they were consistent with the surface/borehole transfer function empirically obtained from weak-motion seismic records.

The soil profile was divided in sublayers and the maximum allowable sublayer thickness was determined at each depth based on the V_s profile and the maximum frequency content, f_{max}, of the base motion, set equal to 25 Hz. The minimum wavelength ($\lambda_{min} = V_S/f_{max}$) was divided by 8 at each depth to obtain the maximum allowable thickness ($h_{max} = \lambda_{min}/8$) required by Kuhlemeyer and Lysmer (1973) criterion, which was further divided by a factor of 2 to consider soil softening (i.e., reduction in V_S) in the model during simulation, as suggested by Ramirez et al. (2018).

Undisturbed sand samples were retrieved from depths of 3.3 and 5.4 m to carry out laboratory tests. Both samples were subjected to cyclic undrained triaxial tests to determine the pre-failure stress-strain properties of soil; cyclic and static strength were measured on the shallowest sample by means of stress-controlled cyclic undrained and consolidated-drained triaxial tests, respectively. A cohesion $c' = 1.2$ kPa and a friction angle $\varphi' = 43.6°$ were obtained from the triaxial tests.

The pre-failure cyclic triaxial tests were used to define the variation in normalized axial stiffness, E/E_0, and damping with strain level, ε_a. Following the equivalence criteria between triaxial and simple shear tests (Silvestri et al. 2001), the axial strain was converted into shear strain with $\gamma = 1.5 \cdot \varepsilon_a$, while the ratio E/E_0 was assumed equal to G/G_0. Figure 4 shows the good agreement between the MKZ model and the experimental data at both depths.

The cyclic resistance and pore pressure parameters were assigned to the fine sand layer on the results of the cyclic triaxial liquefaction tests (Figure 1).

The cyclic axial strength ratio was converted to an equivalent simple shear ratio through a correction factor equal to 0.64, i.e., the mean value of the factors computed adopting the procedures suggested by Castro (1975) and Finn et al. (1971).

The experimental r_u: N/N_L curves for the Sendai fine sand is not univocal but strongly dependent on the cyclic stress ratio applied during each test (Figure 1b). However, a unique mean curve was defined as the best fit of the whole data set.

The consolidation coefficient was computed, according to its definition, known the permeability coefficient, k, and the oedometric modulus, E_{oed}. In the absence of direct measurements, a permeability coefficient equal to $1 \cdot 10^{-5}$ m/s was adopted and E_{oed} is computed through the elasticity theory, assuming Poisson' ratio, ν, equal to 0.24. The at rest lateral earth pressure, K_0, is estimated from the friction angle through the Jaky (1948) relationship. In the one-dimensional analyses carried out with SCOSSA code, the downhole acceleration records were applied as input motions. Rigid bedrock was therefore assumed at the depth of 10.4 m, where the downhole sensor is located, while the slate rock layer above the downhole sensor (7 ÷ 10.4 m) was modelled as a linear visco-elastic material with a constant damping equal to 1%.

3.3 *Dynamic analyses results*

Figure 5 compares the surface records with the simulations of total and effective stress analyses for the selected input motions. The results are expressed in terms of acceleration time

histories and spectral accelerations, both normalized for the PGA recorded at surface layer, PGA_S.

In total stress analyses, the surface motion predicted for input #1 (PGA_d = 0.257 g) shows high non-linear soil behaviour effects and overestimates the recorded PGA by a factor of 1.5, and the predominant period is attained between 0.1 and 0.2 s (Figure 5a).

The predicted ground motions of total stress analyses are in agreement with the recorded acceleration time history and response spectra at surface layer for the input signals #4 (Figure 5b) and #8 (Figure 5c), characterized by medium-low intensity.

As shown in Figure 5a, the effective stress analysis predicts well the surface ground motion with reference to the maximum recorded spectral accelerations and predominant period, while a satisfactory reproduction can be observed in the all range of periods.

Effective stress analyses were also performed on the whole set of input motions, but excess pore pressure was triggered only for record #1 (i.e., those with the highest PGA_d). For input motions #4 and #8, the results of effective stress analyses are the same of total stress simulations.

3.4 *Comparison with other simulations*

Effective stress simulations were also performed on Sendai site by a group of participants to the PRENOLIN project, identified as W-0 team. The W-0 team (University of British Columbia and University of Washington) used an anisotropic sand constitutive models developed within the framework of critical state soil mechanics and bounding surface plasticity (Taiebat and Dafalias, 2008).

The results of the W-team are reported in terms of response spectra in Figure 6; they are compared with the results of SCOSSA, both on total and effective stress analyses.

The comparison in Figure 6 confirms that effective stress analyses based on laboratory tests can correctly predict the actual soil response without introducing further hypotheses on the cyclical soil behavior. Moreover, the simulation carried out by SCOSSA, nevertheless the

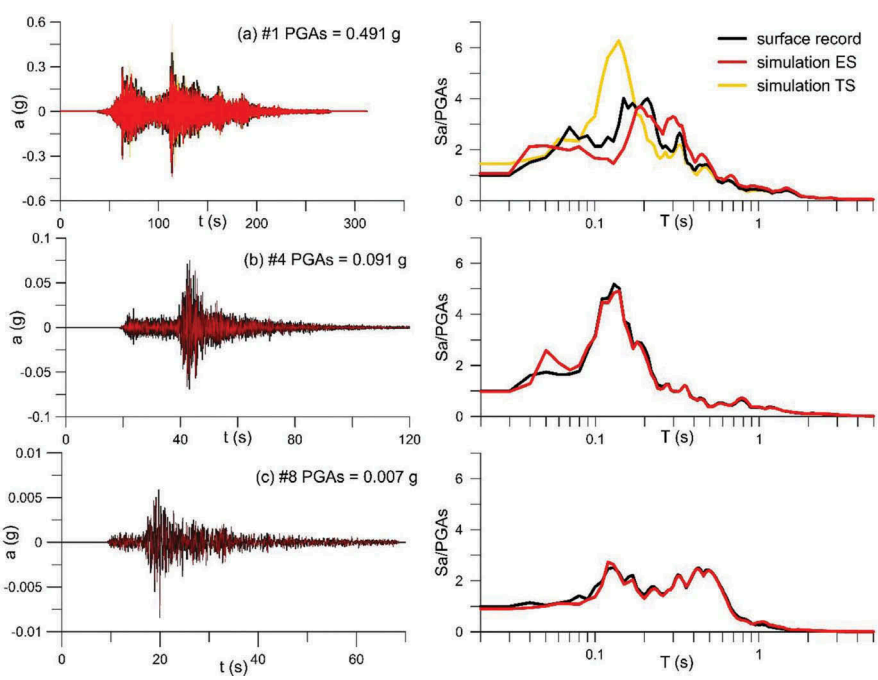

Figure 5. Acceleration time histories and response spectra recorded at the surface vs. those simulated by total (TS) and effective (ES) stress analyses.

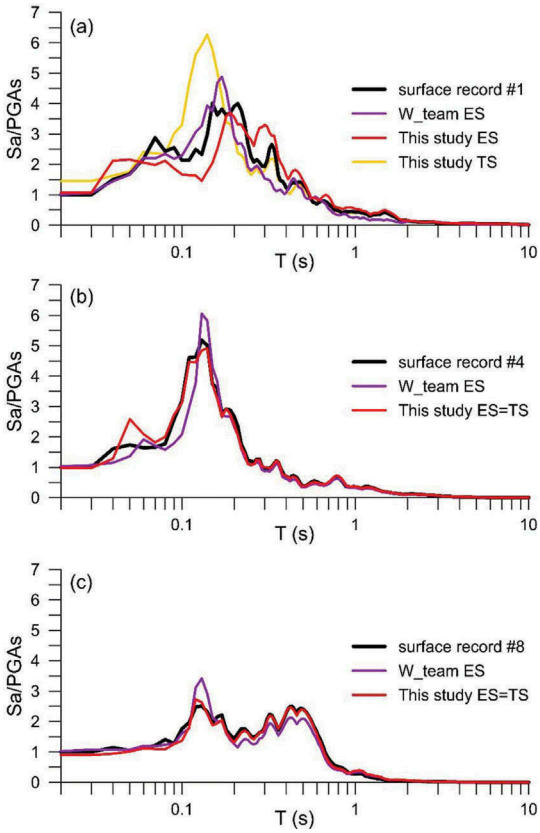

Figure 6. Comparison between recorded and simulated response spectra at the surface performed by W-team and by SCOSSA code in total (TS) and effective (ES) stress analysis.

simplicity of the adopted approach, leads to a realistic acceleration response spectrum at the surface, comparable with those obtained by means of more sophisticated constitutive models.

4 CONCLUSIONS

A loosely coupled approach to simulate the liquefaction of saturated soils in one-dimensional seismic response analyses is presented. The main advantages are that the constitutive model is simple and that the model requires a straightforward calibration of soil parameters with respect to those implemented in other numerical codes performing effective stress analyses.

The study of the Japanese site of Sendai array allows highlighting the limits of total stress analysis and the importance to perform effective stress analysis to adequately predict the dynamic behaviour of saturated loose soil deposits.

The overall good match between numerical and experimental results verifies the reliability of the model implemented in the code, notwithstanding the simplicity of the proposed approach.

Finally, the code was able to yield predictions comparable with more sophisticated and advanced constitutive models and, as consequence, it can be considered a useful tool for engineering practice.

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REFERENCES

Castro, G. 1975. Liquefaction and cyclic mobility of sands. *J Geotech Eng Div* 101(GT6): 552-569.

Chiaradonna, A., Tropeano, G., d'Onofrio, A. & Silvestri, F. 2018. Development of a simplified model for pore water pressure build-up induced by cyclic loading. *Bulletin of Earthquake Engineering* 16(9): 3627–3652. https://doi.org/10.1007/s10518-018-0354-4.

Chiaradonna, A. 2016. Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils. Ph.D. Thesis, Naples, Italy: University of Napoli Federico II.

Finn, W.D.L., Pickering, D.J. & Bransby, P.L. 1971. Sand liquefaction in triaxial and simple shear tests. *J Soil Mech Found Div* 97(SM4): 639-659.

Gingery, J.R., Elgamal, A. 2013. Shear Stress-Strain Curves Based on the G/Gmax Logic: A Procedure for Strength Compatibility. *In Proceedings of the 2th International Conference on Geotechnical and Earthquake Engineering, IACGE, Chengdu, China*, *2013*, p. 1–9.

Groholski, D.R., Hashash, Y.M.A., Kim, B., Musgrove, M., Harmon, J & Stewart J.P. 2016. Simplified Model for Small-Strain Nonlinearity and Strength in 1D Seismic Site Response Analysis. *J Geotech Geoenvironmental Eng* 142(9). DOI: 10.1061/(ASCE)GT.1943-5606.0001496.

Hardin, B.O., Drnevich, V. 1972. Shear modulus and damping in soils: design equations and curves. *J Soil Mech Found Div* 98(SM7): 667–691.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Groholski, D.R., Phillips, C.A. & Park, D. 2016. DEEPSOIL 6.1, User Manual. Urbana, IL, Board of Trustees of University of Illinois at Urbana-Champaign.

Hashash, Y.M.A., Phillips, C. & Groholski, D.R. 2010. Recent advances in non-linear site response analysis. *In Proceedings of the Fifth Int Conf Recent Adv Geotech Earthq Eng Soil Dyn* 29(6): 1–22.

Jaky, J. 1948. Pressure in silos, In: Proceedings of the second International Conference on Soil Mechanics and Foundation Engineering.

Kondner, R.L. & Zelasko, JS. 1963. Hyperbolic stress-strain formulation of sands. *In Proceedings of the 2nd Panamerican Conference on Soil Mechanics and Foundation Engineering, Sao Paulo, Brazil. Associação Brasileira de Mecânica dos Solos* 1: 289-324.

Kuhlemeyer, R.L., Lysmer, J. 1973. Finite Element Method Accuracy for Wave Propagation Problems. *J Soil Mech Found Div* 99(SM5): 421–427.

Matasovic, N. & Vucetic, M. 1993. Cyclic Characterization of Liquefiable sands. *J Geotech Eng* 119(11): 1805–1822.

OYO Corporation. 2014. Report of soil investigation: Sendai District and Onahama District. Report from the PRENOLIN Project, Nice, France.

Phillips, C. & Hashash, Y.M.A. 2009. Damping formulation for nonlinear 1D site response analyses. *Soil Dyn Earthq Eng* 29(7): 1143–1158.

Ramirez, J., Barrero, A.R., Chen, L., Dashti, S., Ghofrani, A., Taiebat, M. & Arduino P. 2018. Site Response in a Layered Liquefiable Deposit: Evaluation of Different Numerical Tools and Methodologies with Centrifuge Experimental Results. *J Geotech Geoenv Eng* 04018073-04018071 https://doi.org/10.1061/(ASCE)GT.1943-5606.0001947

Régnier, J., Bonilla, L.F., Bard, P.Y., Bertrand E., Hollender, F., Kawase, H., et al. 2016. International benchmark on numerical simulations for 1D, nonlinear site response (PRENOLIN): verification phase based on canonical cases. *Bull Seismol Soc Am* 106(5): 2112–2135. DOI: https://doi.org/10.1785/0120150284.

Régnier J, Bonilla L-F, Bard P-Y, Bertrand E, Hollender F, Kawase H, et al. 2018. PRENOLIN: International benchmark on 1D nonlinear site-response analysis - Validation phase exercise. *Bull Seismol Soc Am* 108(2): 876-900. DOI: https://doi.org/10.1785/0120170210.

Silvestri, F. 2001. Looking for objective criteria in the interpretation of laboratory stress-strain tests. *Prefailure deformation characteristics of geometerials*. Jamiolkowski, Lancellotta e Lo Presti (eds). Swets and Zeitlinger, Lisse, ISBN 9058090752.

Taiebat, M. & Dafalias, Y.F. 2008. SANISAND: simple anisotropic sand plasticity model. *International Journal for numerical and analytical methods in geomechanics* 32: 915–9948.

Terzaghi, K. 1943. Theoretical Soil Mechanics. John Wiley & Sons.

Tropeano, G., Chiaradonna, A., d'Onofrio, A. & Silvestri F. 2016. An innovative computer code for 1D seismic response analysis including shear strength of soils. *Geotechnique* 66(2): 95–105. http://dx.doi.org/10.1680/jgeot.SIP.15.P.017.

Tropeano, G., Chiaradonna, A., d'Onofrio, A. & Silvestri F. 2019. A numerical model for non-linear coupled analysis of the seismic response of liquefiable soils. *Computers and Geotechnics* 105: 211–227. doi:https://doi.org/10.1016/j.compgeo.2018.09.008.