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Centrifuge test modelling through a 1D computer code for effective stress analysis

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ABSTRACT: Accurate numerical modelling of seismically induced liquefaction requires sophisticated computer codes with advanced constitutive soil models. These models, which incorporate plasticity and volumetric-distortional coupling, are capable of simulating both localized and generalized failure mechanisms. Nevertheless, their practical application is challenging due to the difficulty in reliably calibrating the large number of parameters that define these models. To bridge the gap between scientific accuracy and practical application, simplified models have been developed. Many of these methods operate on a decoupled basis, treating the seismic wave propagation problem independently from the estimation of response parameters (e.g., Cyclic Stress Ratio) via empirical or semi-empirical correlations. While these decoupled methods are valuable for a first, initial site screening, they may not adequately represent the highly nonlinear soil behavior that occurs as failure conditions are approached. The computer program SCOSSA incorporates a simplified stress-based pore water pressure model into a cyclic response framework, along with one-dimensional consolidation theory. This code can perform 1D loosely-coupled effective stress analyses under both fully and partially drained conditions. The objective of this paper is to evaluate the code's predictive performance in simulating a seismic site response for a layered, liquefiable soil profile, as reproduced in a centrifuge experiment. The model parameters calibration has been performed according to a type C prediction based on laboratory test data available in the literature. The results of the numerical analysis lead to a reasonable prediction in the liquefiable layer, while the underprediction of the effects observed in the dense sand is discussed.

1 INTRODUCTION

Centrifuge experiments constitute an essential tool to validate the performance of soil constitutive models, because they can correctly reproduce complex soil configurations and boundary conditions that cannot be explored in running laboratory tests on soil elements. Especially for earthquake-induced liquefaction problems, several projects, such as VELACS (Arulanandan et al. 1995), LEAP-2017 (Manzari et al. 2017), LEAP-Asia-2018 (Ueda et al. 2019), have been imprinted to explore the reproducibility of reference centrifuge tests among several facilities throughout the world and exploring limits and capability on the current numerical models. Further validation efforts based on centrifuge tests can be found in several recent publications (Porcino et al. 2025; Luo and El-Eman 2025; Sadeghi et al. 2025; Zhang et al. 2025; Kutter et al. 2025).

Due to the complexity of the problem, accurate numerical modelling of seismically induced liquefaction requires sophisticated computer codes with advanced constitutive soil models. These models, which incorporate plasticity and volumetric-distortional coupling, are capable of simulating both localized and generalized failure mechanisms. Nevertheless, their practical application is challenging due to the difficulty in reliably calibrating the large number of parameters that define these models.

To bridge the gap between scientific accuracy and practical application, simplified models have been developed. Many of these methods operate on a decoupled basis, treating the seismic wave propagation problem independently from the estimation of response parameters (e.g., Cyclic Stress Ratio) via empirical or semi-empirical correlations. While these decoupled methods are valuable for a first, initial site screening, they may not adequately represent the highly nonlinear soil behavior that occurs as failure conditions are approached.

The computer program SCOSSA (Tropeano and Chiaradonna 2025) incorporates a simplified stress-based pore water pressure model into a cyclic response framework, along with one-dimensional consolidation theory (Terzaghi, 1943). This model was chosen for its simplicity, consistency with established liquefaction frameworks, ease of implementation and straightforward parameter quantification. Originally calibrated using laboratory tests, the model was later extended to use common in situ test data (CPT and SPT) (Chiaradonna et al. 2020a), and, more recently, DMT (Chiaradonna et al. 2024). The code can perform one-dimensional (1D) loosely-coupled effective stress analyses under both fully and partially drained conditions.

The performance of the code in predicting pore water pressure build-up and dissipation has been validated against data from seismic centrifuges (Tropeano et al. 2019, Chiaradonna et al., 2019a) and instrumented field sites (Chiaradonna et al. 2019b, Chiaradonna and Reder 2019, Chiaradonna et al. 2020a,b). Comparisons with more complex codes on real and ideal cases (Chiaradonna et al. 2022a,b), including the numerical benchmark LICORNE (Khalil et al. 2022), have also been performed. Furthermore, post-analysis results have been used to predict both horizontal displacements and consolidation settlements in real-world case studies, with estimates aligning well with observed damage (Chiaradonna et al. 2018, 2019).

However, validation of soil models tested in centrifuge tests was limited to models with a single layer of sand, while layered configurations have not yet been considered.

The objective of this paper is to evaluate the code's predictive performance in simulating a seismic site response for a layered, liquefiable soil profile, as reproduced in a centrifuge experiment by Ramirez et al. (2018). A class C prediction (Lambe 1973) is presented, with the model calibration performed independently from the results of the centrifuge experiment. Literature data on the same soils tested in centrifuge but reported by different research groups have been indeed used for the calibration of the soil model parameters.

Next section 2 summarizes the main features of the SCOSSA code, while section 3 syntheses the centrifuge experiment reported in Ramirez et al. (2018). Input data adopted in the numerical simulation are described in section 4 and then critically discussed in section 5.

2 BACKGROUND ON SCOSSA CODE

The non-linear code SCOSSA (Tropeano et al. 2016; 2019; Tropeano and Chiaradonna 2025) models the soil profile as a system of consistent lumped masses, connected by viscous dampers and springs with hysteretic nonlinear behaviour (Figure 1).

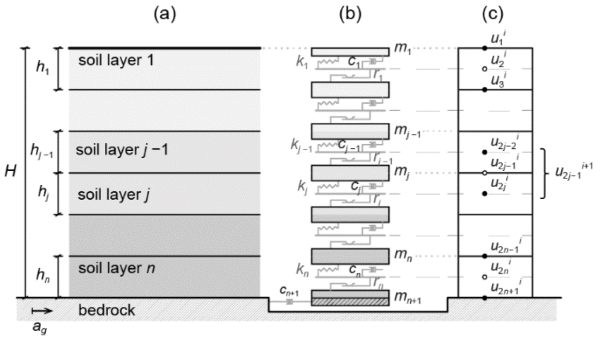

Figure 1. (a) Layered soil profile as modelled in SCOSSA: (b) discretisation for finite element analysis, and (c) discretisation for finite difference analysis of the consolidation equation.

A cyclic response model described by the Modified Kondner–Zelasko (MKZ) hyperbolic model (Matasovic and Vucetic 1993) and the modified Masing rules (Hashash and Park 2002) are implemented in the code together with a stress-based pore water pressure model (PWP) in conjunction with a shear strength degradation model (Matasovic and Vucetic 1993). The PWP model permits to compare the irregular time-history of shear stress-induced by the earthquake with the soil liquefaction resistance. The comparison is expressed through the so-called 'damage parameter', κ, which can be computed for any loading pattern. The damage parameter is an incremental function of the applied load that takes into account the cyclic strength of the soil (Chiaradonna et al. 2018). This latter is expressed in terms of cyclic resistance curve, analytically described by the equation:

$$\frac{(\mathrm{CSR}-\mathrm{CSR_t})}{(\mathrm{CSR_r}-\mathrm{CSR_t})}=\left(\frac{15}{N_L}\right)^{\frac{1}{\alpha}} \quad (1)$$

where CSR is the shear stress amplitude normalized by the initial effective confining pressure; N_L is the number of cycles at liquefaction, $\mathrm{CSR_t}$ is the asymptotic value of CSR as the number of cycles tends to infinite, $\mathrm{CSR_r}$ is the ordinate of the curve for $N_L = 15$ (usually adopted as a reference number of cycles) and α a fitting parameter. The model adopts a criterion for which liquefaction is attained when r_u (ratio between the excess pore pressure and the initial effective confining pressure) exceeds 0.95. For a regular shear stress history, κ is proportional to the number of cycles, N; it is, therefore, possible to express the pore pressure ratio, r_u, as a function of the number of cycles, through the relationship proposed by Chiaradonna et al. (2018):

$$r_u = a\left(\frac{N}{N_L}\right)^b + (0.95 - a)\left(\frac{N}{N_L}\right)^d \qquad (2)$$

where a, b and d are best-fitting parameters that control the shape of the curve.

3 BRIEF SUMMARY OF THE CENTRIFUGE EXPERIMENT

The considered centrifuge experiment was performed at the University of Colorado Boulder's 5.5 m-radius centrifuge facility (Ko 1998). The description of the experiment detailed by Ramirez et al. (2018) is here summarized briefly in view of the numerical modelling illustrated in the following section 4.

The test was conducted to model a one-dimensional (1D) free-field liquefaction scheme, utilizing two uniform, fully saturated sands: Ottawa F65 and Monterey 0/30 sands. The tests aimed to reproduce a soil column of 18 m at the prototype scale, consisting (from the bottom to the top) of: 10 m of Dense Ottawa F65 sand with a relative density of 90% (labeled as DO), 6 m of Loose Ottawa F65 sand with a relative density of 40% (labeled as LO) and 2 m of Dense Monterey sand with a relative density of 90% (labeled as DM). Soil properties of the three layers at the beginning of the tests are listed in Table 1. Ramirez et al. (2018) integrated the physical soil properties with a mechanical characterization carried out in the laboratory consisting of monotonic and strain-controlled cyclic triaxial tests. The sand layers were housed in a flexible shear beam (FSB) container constructed of aluminum and rubber to reduce boundary effects (Olarte et al. 2017).

Table 1. Physical properties of the sand layers (from Ramirez et al. 2018)

Sand/ (Acronym)	Layer thickness, ΔH [m]	Relative density, Dr [%]	Saturated unit weight, γ_{sat} [kN/m^3]
Monterey 0/30 (DM)	2	90	19.8
Ottawa F65 (LO)	6	40	19.1
Ottawa F65 (DO)	10	90	19.9

A pluviation technique with an automated system (Kirkwood and Dashti 2018) was adopted for the preparation of the sand layers to improve the uniformity in the soil layers, which were then fully saturated with a solution of methylcellulose in water with a viscosity 70 times larger than the water. The model was tested under a centrifuge acceleration of 70g while being excited horizontally at the base with a target prototype accelerogram with a Peak Ground Acceleration, PGA, equal to 0.41g (Figure 2). Instrumentation, including LVDT, accelerometers and piezometers, allowed for continuous monitoring of soil motion and the development of excess pore water pressures.

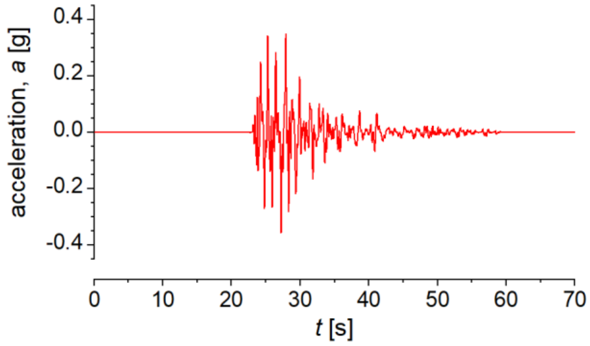

Figure 2. Recorded accelerogram at the base of the central array used as input motion in the numerical analysis.

4 NUMERICAL MODELLING

The numerical model consists of a three-layer soil column with the saturated unit weight reported in Table 1; for the shear wave velocity profile, similarly to what adopted by Ramirez et al. (2018), the simple power relationship proposed by Seed and Idriss (1970) for sand with an exponent 0.25, was adopted.

The cyclic response model has been also defined by using the mean curve for sands proposed by Seed and Idriss (1970), consistently with what reported in Ramirez et al. (2018). Differently, it was not possible to use the laboratory tests results reported by Ramirez et al. (2018) for defining the parameters of the PWP model, because stress-based cyclic tests are required for PWP calibration. The cyclic resistance and pore water pressure relationships to define parameters in Equations (1) and (2) were indeed found in previous published literature. In detail, the cyclic resistance curve for DM was defined according to Kammerer et al. (2004) for $Dr = 80\%$ and effective confining pressure of 40 kPa. For Ottawa F65 sand, the adopted cyclic resistance curve for $Dr = 40\%$ (LO) was that reported by Bastidas et al. (2017) and for $Dr = 90\%$ (DO) was that reported by Manzari et al. (2017). The calibrated cyclic resistance curves compared to the experimental data are reported in Figure 3a.

The pore pressure relationships were not found in literature, so the mean curve for clean sand was defined according to Booker et al. (1976) for an exponent equal to 0.7, then adopted in the determination of the parameters of equation (2). Figure 3b reports the adopted pore pressure ratio curves used for the three soils. The numerical values of the PWP model are also listed in Table 2.

Finally, the parameter of the degradation model proposed by Matasovic and Vucetic (1993) was set equal to 3.5, according to author`s suggestion.

The analysis has been performed under fully undrained conditions by applying at the base of the soil column the input motion shown in Figure 2 as inside motion at the interface of a rigid bedrock.

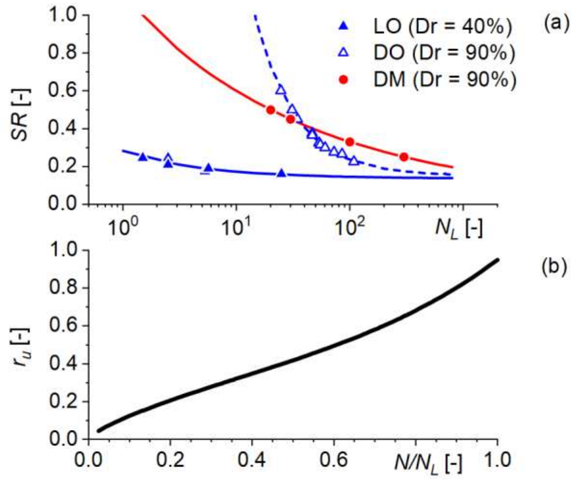

Figure 3. Cyclic resistance (a) and pore water pressure ratio (b) curves used in the numerical analysis.

Table 2. Parameters of the PWP model used in the numerical analysis.

Soil	α	CSR_t	CSR_r	a	b	d
DM	3.686	0.020	0.539	0.664	0.73	4
LO	1.660	0.136	0.165	0.664	0.73	4
DO	0.844	0.150	0.966	0.664	0.73	4

4.1 Results

The results of the simulation, shown in Figure 4, successfully predicted the pore pressure response and response spectra for the liquefiable layer (5 m), while underprediction can be observed for the dense deeper layer.

In fact, regarding the pore water pressure time history (Figure 4c), liquefaction is attained along all profiles in the experiment, while the simulation led to liquefaction only in the upper layers (z = 2 and 5 m). In the deeper layers, the maximum predicted pore pressure ratio is less than 0.25, even though the instant where the build-up is triggered is well matched.

When looking at the acceleration response spectra (Figure 4d), the misprediction for the deeper layers is again a direct consequence of the underprediction of the pore water pressure ratio. Indeed, low values of r_u implied a small reduction of the soil stiffness, so the simulated recorded accelerations are higher than the recorded ones.

5 DISCUSSION

Possible explanations of the reasons of the discrepancy between simulation and centrifuge test results can be found in several aspects. Firstly, the calibration based on literature data could not reflect the actual behaviour of the tested soils, especially with reference to the dense sands. Lack of specific data related to the measurements of shear wave velocity in the FSB, or nonlinear soil properties investigated in literature, increases the uncertainty of the adopted parameters in the numerical model. In addition, partially drained conditions could have played a role in the centrifuge experiment due to the high permeability of the sands. This aspect has not been accounted for in this SCOSSA analysis.

Anyway, focusing on the outcomes of centrifuge test only, the experimental results suggest that the deeper layer of dense Ottawa sand did not behave as a dense soil but more likely as loose one, considering that full liquefaction was attained as in the shallowest layer.

Based on this consideration, a new simulation of the centrifuge test has been carried out, where a unique 16 m-thick layer of Loose Ottawa sand is modelled between 2 and 18 m below the ground surface. The nonlinear and dissipative properties of LO are the same used for the previous analysis, as well as the parameter used for the describing the PWP model.

Figure 5 shows the comparison between the predicted and measured time histories of pore pressure ratio at the different depth along the soil column. The simulated histories well reproduced the maximum attained value at all depths and approximately the time where the triggering of excess pore pressure accumulation starts.

6 CONCLUDING REMARKS

The computer code SCOSSA version 1.0 has been demonstrated to be a valuable tool for the prediction of the seismic response of horizontally layered deposits across the entire range of shear strain levels, from the very small up to liquefaction.

The results of the numerical simulation of the centrifuge experiment were very satisfactory for the loose sand layer, while underprediction of excess pore water pressures was obtained for dense deeper layers.

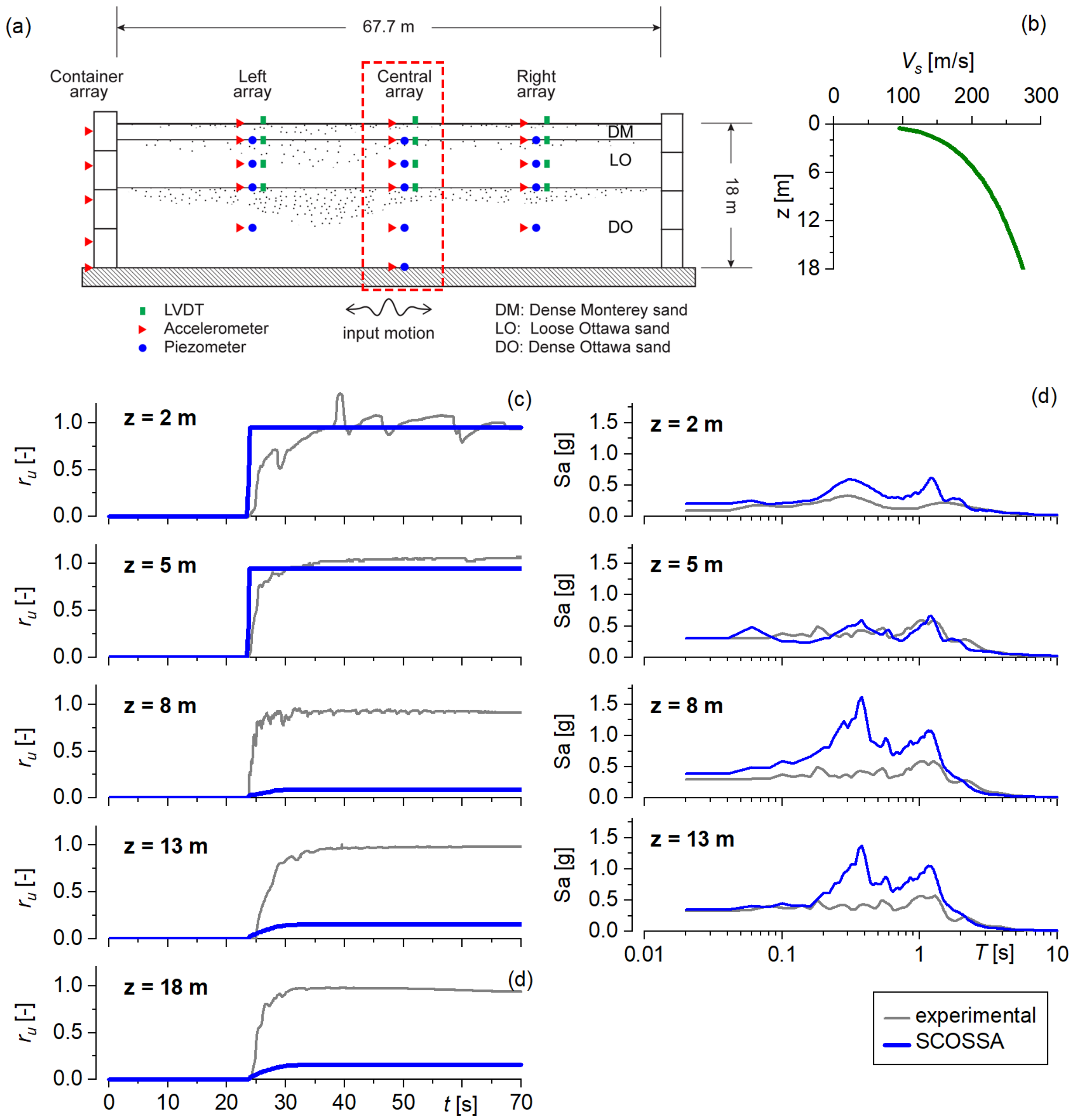

Figure 4. (a) Cross section of the physical model tested in centrifuge with instrumentation and soil layers, (b)Vertical profile of the shear wave velocity adopted in the numerical model, (c) Comparison between experimentally measured and simulated pore water pressure ratio time histories and (d) 5% damped acceleration response spectra at different depths.

A better match with the experimental time histories of pore pressure ratio was obtained when a single layer of Loose Ottawa sand is modelled instead of the two layers configuration with loose over dense material. This result shades a light on the uncertainties on the effective performance of the sand layers in addition to the uncertainty related to the model calibration.

The calibration of the PWP model for a specific relative density of the soil cannot be extended to a different relative density, but a new calibration of the model parameters is necessary even though the soil is the same. This makes the calibration process a bit long and repetitive compared to other models, where soil calibration is independent of the relative density. As future perspective, the predictions of the code will be extended to the pore water pressure dissipation phase of the experiment and also performing a type C1 prediction. Future goals will include also the comparison with the results of simulations obtained by using different numerical platforms and advanced soil constitutive models (multiyield and bounding surface) available for the same centrifuge experiment used as a benchmark.

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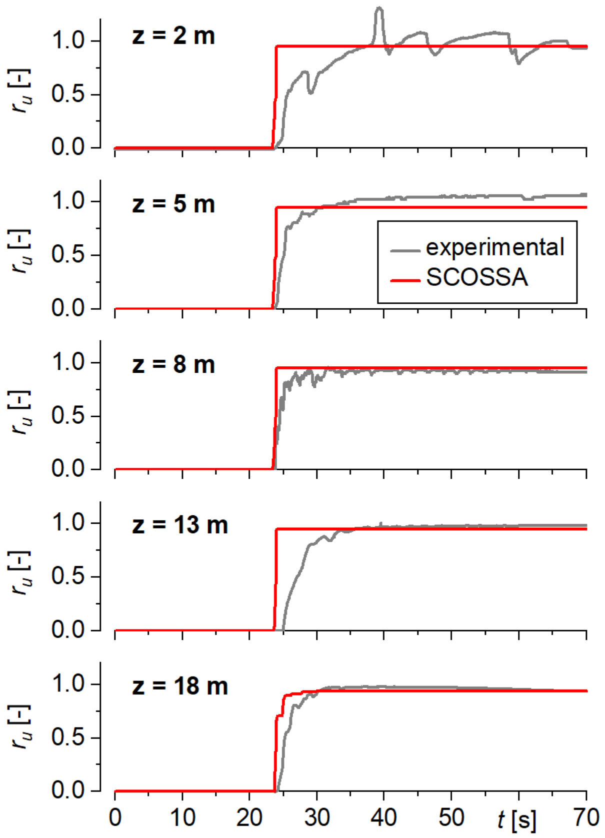

Figure 5. Comparison between experimentally measured and simulated pore water pressure ratio time histories for a soil column where Loose Ottawa sand is considered for the from 2 to 18 m depth.

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