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# Suction bucket response under seismic liquefaction: numerical simulations versus centrifuge experiments

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**ABSTRACT:** Multi-bucket foundations are increasingly used to support offshore wind turbines (OWTs) due to their potential for high overturning resistance. However, the seismic response of such foundation systems in granular soils is not sufficiently studied in the literature, while there is no relevant design methodology of these systems against earthquake-induced liquefaction that may appear in such soils. Therefore, a calibrated 3D numerical methodology is presented in this paper for simulating the response of suction buckets as OWT foundations under seismic action and liquefaction. It employs 3D dynamic fully-coupled finite difference analyses with FLAC3D and the use of a state-of-the-art constitutive model (LiPa) for simulating the liquefiable soil response. The methodology is hereby employed for simulating the reference test from the series of dynamic centrifuge tests of Yu et al., 2014, which refers to a single suction bucket foundation supporting an OWT in a thin saturated sand layer. Given that the imposed shaking is very intense in these tests, the saturated sand layer liquefies, thus leading the single bucket to seismic settlements and residual rotation. The analyses simulate accurately the measured response of the bucket and the foundation soil with a set of LiPa model constants that was calibrated beforehand on the basis of element tests from the literature. This good agreement serves as a validation for the proposed methodology and implies that it constitutes a reliable tool for the performance-based-design of such foundation systems against seismic liquefaction.

**Keywords:** Suction buckets; Offshore wind turbine; Seismic loading; Liquefaction; Numerical methodology

## 1 INTRODUCTION

Over the last few years, the expansion of wind farms towards deeper waters and areas of high seismicity has become a necessity, due to the rapidly growing demand for green energy. In this context, economic and safe Offshore Wind Turbine (OWT) foundation designs are deemed necessary, in order to reduce construction costs as well as the potential seismic hazard (e.g., due to seismic liquefaction).

Recently, suction bucket foundations for OWTs are being favored by the industry, due to their low cost and ease of installation. Moreover, when combined in groups as the foundation of a jacket tower carrying an OWT they offer the potential for reduced displacements and rotations in case of seismic liquefaction. However, as the use of these OWT foundation systems has so far been limited to non-seismic areas, there is a significant gap in the literature regarding their seismic response in granular soils and the potential liquefaction hazard. The few relevant studies use 3D coupled non-linear analyses either for buckets in fine-grained non-liquefiable soils (e.g. Kourkoulis et al., 2014), or for other foundation methods (e.g., monopiles) in coarse-grained liquefiable soils. Concurrently, there are few centrifuge experiments in the literature investigating this problem (e.g. Yu et al., 2014, Ueda et al., 2020). These experimental efforts can serve for calibration of numerical methodologies with sophisticated constitutive models (e.g. Dafalias and Manzari, 2004, Limnaiou and Papadimitriou, 2023) that can capture the complex aspects of the response of coarse-grained soils under dynamic loading. So far, such numerical methodologies for suction buckets under seismic loading are scarce in the literature (e.g. Cheng et al., 2023) and this scarcity constitutes the main motivation for this research.

In light of the above, a calibrated 3D numerical methodology is presented in this paper for simulating the response of suction buckets as OWT foundations under seismic action and liquefaction. Aiming to validate the proposed methodology, its results are compared with those of a centrifuge experiment (Yu et al., 2014) studying the dynamic response of a single suction bucket OWT foundation.

## 2 CENTRIFUGE TESTS

The objective of the hereby selected centrifuge experiments (Yu et al., 2014) was to evaluate the seismic response of an OWT with a single suction bucket foundation resting on liquefiable soil.

The wind turbine model was designed according to the characteristics of a prototype structure, but due to centrifuge size limitations, it was chosen to simulate a scaled down (1:10) model. The scaled wind turbine model consisted of three parts: the wind turbine tower, the tower head and the suction bucket foundation (see Figure 1a). More specifically the tower head was simulated as a lumped mass of 10.6t (representing the weight of the rotor, nacelle, blades and gearbox) on top of the wind turbine tower, while the foundation consisted of a single suction bucket with a diameter of D = 4m and a skirt length of L = 1.75m (i.e., denoted as Test 1).

The scaled OWT model was installed in a rigid box with a length of 53.3cm, a width of 24.1cm and a depth of 17.7cm. Subsequently, a uniform soil layer of Toyoura sand was constructed in the rigid box at 1g, by constantly pouring the sand from a specific height of 80cm in order to achieve the target specific density of $D_r$=68%. The saturation process was accomplished by using de-aired water and by applying vacuum for at least 24 hours. The thickness of the soil deposit was chosen to be 4.5m (in prototype scale), while the water table was maintained 1.5m above the ground surface in order to simulate offshore conditions. The adopted centrifugal acceleration level was 50g and the centrifuge model was subjected to a very intense acceleration time-history that is depicted in Figure 1b. Its very large peak acceleration $a_{max} \approx 0.6$g, implies a cyclic stress ratio CSR = 0.65 $(a_{max}/g)$ $(\sigma_{vo}/\sigma'_{vo})$ $r_d \approx 0.75$ at mid-depth of 2.25m, that, in turn, means immediate liquefaction (in the first 1-2 significant cycles). The centrifuge model was instrumented with pore pressure sensors, accelerometers and LVDTs, the positions of which are shown in Figure 1a, along with the general configuration of the centrifuge model.

The experimental program of Yu et al., 2014 consisted of 4 tests in total, which differed in terms of foundation dimensions (Tests 2 & 3) or foundation weight (Test 4). This study, however, due to lack of space, will focus solely on centrifuge Test 1, which was regarded as the reference experiment and the only one for which a more detailed presentation of its results is provided in Yu et al., 2014.

## 3 NUMERICAL METHODOLOGY

The proposed numerical methodology involves 3D dynamic fully-coupled finite difference analyses with FLAC3D (Itasca, 2017) and the use of the LiPa constitutive model (Limnaiou and Papadimitriou, 2022, 2023) for simulating the coarse-grained soil response. It is underlined here that the employed LiPa model's calibration is not centrifuge test- specific, but (Toyoura) sand-specific, as deduced based on element tests (Limnaiou and Papadimitriou, 2023). The main aspects of the 3D methodology are presented in detail in the following sub-paragraphs.

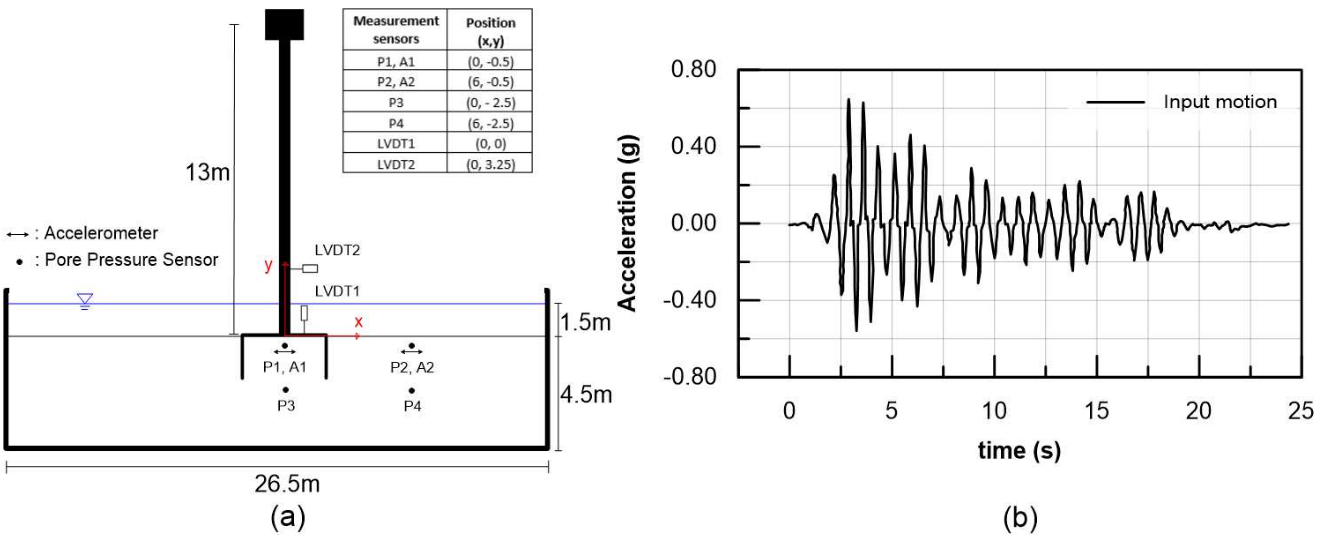

*Figure 1 – (a) Centrifuge model configuration with monitoring points and (b) time-history of input acceleration.*

## 3.1 Geometry and Soil Modelling

A typical 3D-mesh was developed in the finite difference code FLAC3D, in order to study the problem at hand. Since the problem is symmetric with respect to the vertical plane that passes from the center of the bucket, only half of the geometry needs to be analyzed (see Figure 2a). The use of a rigid box in these centrifuge experiments dictates the model dimensions to be identical to those of the rigid box at prototype scale. Therefore, the dimensions of the numerical model are 26.5m (length) x 6.03m (width), while it simulates a 4.5m thick sand layer with the water table being 1.5m above the ground surface. Also, in order to simulate the boundary conditions imposed by the rigid box, the base of the model is fixed in all three directions, while the vertical boundaries are fixed in the direction perpendicular to the model. These boundary conditions remain the same for all analysis stages to be described in sub-paragraph 3.3.

*Table 1. Values of model parameters for Toyoura sand.*

| Constitutive part | Parameter | Values |
|---|---|---|
| Elasticity | $G_o$ | 650 |
| | $\nu$ | 0.15 |
| CSL | $e_{ref}$ | 0.934 |
| | $\lambda$ | 0.019 |
| | $\xi$ | 0.7 |
| | $M_c^c$ | 1.25 |
| | c | 0.712 |
| Plastic modulus | $n^b$ | 1.1 |
| | $h_o$ | 60 |
| | $c_h$ | 12 |
| Dilatancy | $n_d$ | 2 |
| | $A_o$ | 1.5 |
| Fabric | $N_o$ | 1550 |
| Post-liquefaction | $L_o$ | 2500 |

The employed LiPa model is a state-of-the-art constitutive model that belongs to the SANISAND family of constitutive models (e.g., Dafalias et al., 2004) and its advantage, among others, is that it achieves simulative accuracy in both monotonic and cyclic loading, without the need for recalibration. Thus, the LiPa model is implemented in the finite difference code FLAC3D and used with a single set of model parameters for Toyoura sand (see Table 1) that was derived independently, via calibration on the basis of element tests from the literature (Limnaiou and Papadimitriou, 2023). Given the extreme intensity of the applied motion at its beginning ($a_{max} \approx 0.6$g for two successive cycles; see Figure 1) abrupt liquefaction is expected, which renders the fabric evolution function that the LiPa model considers irrelevant. Hence, a value of $N_o$ = 0 was assumed here, while all other parameter values of Table 1 were retained.

The soil deposit consisting of Toyoura sand is modeled using hexahedral elements with increasing discretization in the vicinity of the bucket, for a more accurate prediction of its response. Finally, as hysteretic soil damping is captured by LiPa, only 2% damping was assigned to the soil elements artificially to take account of small-strain soil damping. Figure 2a shows a general overview of the numerical model geometry along with that of the foundation-wind turbine system, whose characteristics are discussed in sub-paragraph 3.2.

The fully-coupled numerical analyses required a value for the permeability coefficient $k$ of Toyoura sand used by Yu et al., 2014 in their experiments. Such a value was not reported in their paper. Moreover, it is well known that the use of a constant value of Darcy's permeability coefficient is a rather controversial issue for liquefaction related problems. The reason is that this parameter has been established for quantifying steady state flow through the pores of a stable soil skeleton. These conditions are not met in the analyzed problem, since the loading is dynamic and the soil skeleton is far from being stable when liquefied. Therefore, a variable permeability coefficient $k$ (Equation 1) was adopted in the analyses, as proposed by Shahir et al., 2012:

$$k = k_{ini} \cdot [1 + (a-1) \cdot r_u^{\beta}] \quad (1)$$

where $k_{ini} = 2 \cdot 10^{-5}$ cm/s is the initial permeability coefficient value that was adopted from Farahani and Barari, 2023 who attempted a numerical simulation of the same centrifuge experiment via 2D and not 3D analyses as in this paper. The remaining parameters of Equation (1) are: α = 1000, β = 1 (Shahir et al., 2012).

Considering a variable permeability coefficient $k$ based on the $r_u$ ratio simulates the expected increase of permeability due to an increase of the $r_u$ ratio leading to liquefaction ($r_u$ = 1.0). This simulates the gradual loss of contact of soil particles in the granular medium approaching liquefaction. In this way, a better prediction of the excess pore pressure buildup and dissipation during the vibration is attained in comparison to the use of a constant $k$ value throughout the shaking. For completeness, it is mentioned here that the lateral boundaries and the base of the mesh were considered impermeable simulating the rigid box used in the test. The same holds for the plane of symmetry of the problem, since only half of the mesh is considered in this analysis.

## 3.2 Structure Modelling

The suction bucket is simulated using isotropic elastic shell elements. These elements interact with the surrounding soil through interfaces necessary to simulate both the slippage between the bucket skirt and the soil, as well as the separation below the bucket lid. Its properties correspond to a steel suction bucket ($\rho_s$=7850 kg/m$^3$, $E_s$=210 GPa, $\nu_s$=0.3), while its dimensions are selected according to those of centrifuge Test 1 (Yu et al., 2014), as shown in Figure 2. In lack of information on the bucket thickness, the bucket was assumed to have a rather small thickness of $t_s$ = 0.05 m for computational efficiency reasons. This selection results in a more flexible bucket response than anticipated, as in practice suction bucket lids are essentially rigid. To compensate for this, the bucket lid is reinforced with radial elastic beams of increased stiffness and negligible weight, so that the kinematic response of the bucket is not affected. This reinforcement of the bucket lid with beams is also shown in Figure 2a, while the bucket's simulation details are depicted in Figure 2b which offers a zoomed-in view of the bucket area. Note that the additional row of soil elements (thickness of 0.4m) under the lid is employed for numerical stability of the skirt-lid-soil interaction. Use of thinner elements gives identical results, but at increased computational cost.

For a proper simulation of the centrifuge experiment, the whole wind turbine must be modelled in FLAC3D, to capture the dynamic soil-structure interaction effects that typically occur in such problems. Consequently, the wind turbine tower is also modelled with the use of isotropic elastic beam elements and has a height of 13m. In the absence of information on the tower stiffness or the fundamental vibration period of the superstructure, the tower properties are chosen so that it represents a structure with a high slenderness ratio and a total weight of 6.25t (as reported in Yu et al., 2015 where the same wind turbine prototype structure was assumed). Moreover, the tower head is simulated as a high stiffness steel rectangle, using isotropic elastic beam elements with properties corresponding to a 10.6t concentrated mass on top of the wind turbine tower. Finally, 5% damping is assigned to all shell and beam elements, considered typical for structural elements.

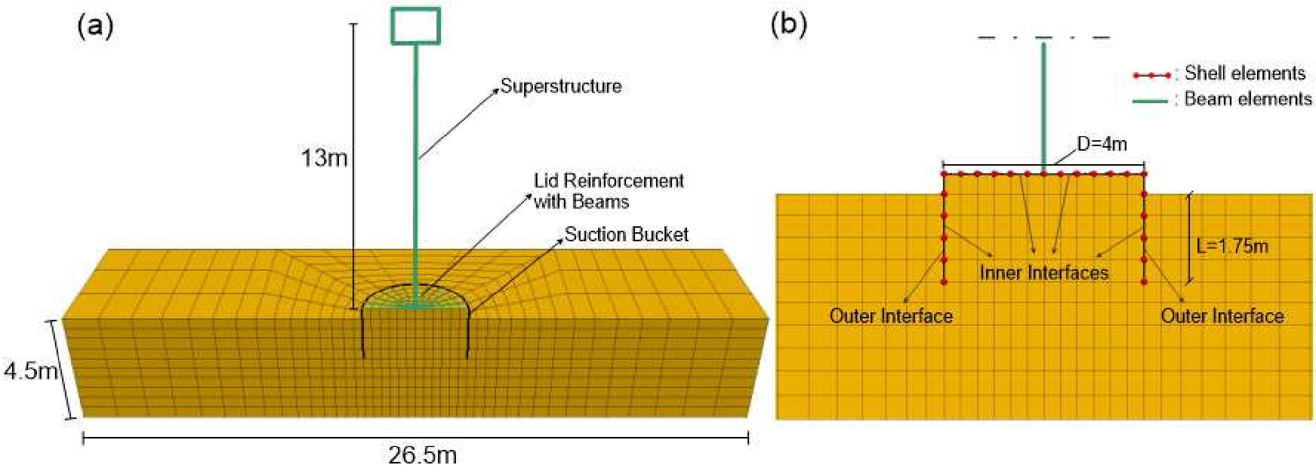


*Figure 2 – (a) Numerical model configuration with finite difference mesh and (b) suction bucket simulation details (zoom in the bucket area).*

## 3.3 Modelling Sequence

The analysis of the foundation response is performed in two steps. First, a static, fully drained analysis is performed which simulates the establishment of the geostatic stress field and hydrostatic pore pressures, and the application of the static loads due to the structure. The latter include the weight of the bucket, as well as the weight of the superstructure that is transferred to the suction bucket foundation. The bucket was modeled as "wished-in-place", thus, soil disturbance and alteration of initial stress or void ratio conditions due to the 1g installation process was not accounted for in all subsequent analyses. Drained conditions and no excess pore pressure generation are assured by assigning a small value to the water bulk modulus during the first stage of the analysis.

The second stage involves the analysis of the bucket under seismic loading imposed to the model, by directly applying the input motion on its (rigid) vertical boundaries as well as its base. At this stage, the dynamic option of the code is activated, and the actual water bulk modulus ($K=2*10^6$ kPa) is assigned. In order to follow both the excess pore pressure generation during shaking and its dissipation in the post-shaking period, the analysis continues for 5 more seconds after the end of shaking. This process allows for following the full response of the suction bucket, both during and immediately after the end of shaking.

## 4 COMPARISON WITH TEST DATA

The proposed numerical methodology is applied for the simulation of the reference centrifuge Test 1 (Yu et al., 2014), that was described in detail in paragraph 2. Figures 3a and 3b compare the experimental and numerical (black and red line respectively) excess pore pressure ratio $r_u$ time-histories at locations P1 (0.5m under the bucket) and P3 (2.5m under the bucket) respectively. Overall, the numerical simulations are consistent with the experimental data. At both locations the numerical analysis simulates satisfactorily the excess pore pressure build-up, leading to $r_u$ values generally less than 1.0, except for a transient attainment of $r_u = 1.0$ at $t \approx 9$ sec appearing only in the analysis. In other words, unlike in the free-field (in both test and simulation), the soil does not essentially liquefy under the bucket, since the initial effective stresses are larger in this region due to the superstructure weight. The relative increase of $r_u$ at $t \approx 17.5 – 20$ sec does not agree with the data, but it does not inverse the overall excess pore pressure dissipation process in the later stages of the shaking that is captured by the analysis.

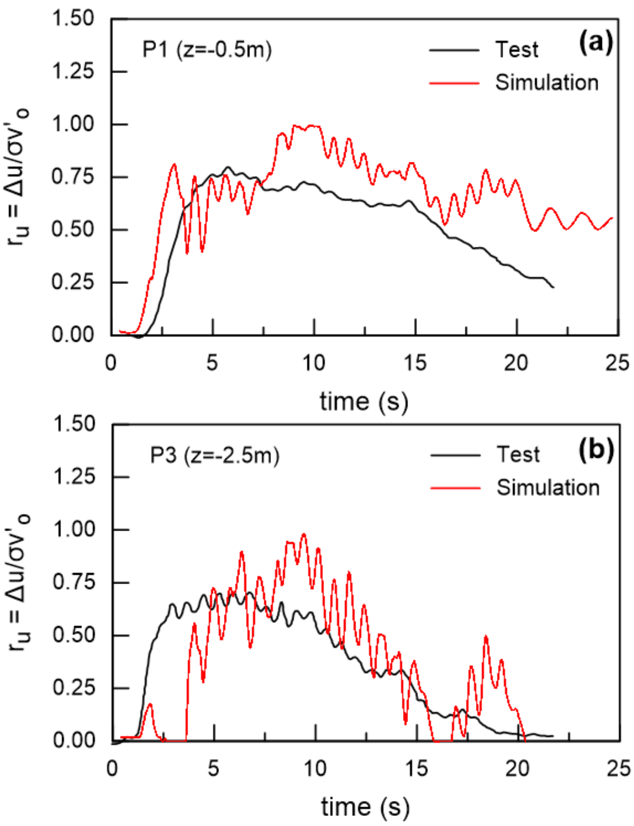


*Figure 3. Comparison of excess pore pressure ratio $r_u$ time-histories from the simulation and the centrifuge test at points (a) P1 and (b) P3.*

At both locations, the numerical simulation predicts dilative behaviour at the beginning of the vibration, which at location P1 is manifested by the appearance of strong fluctuations of $r_u$, while at location P3 by the appearance of a dilation drop for the first 3 seconds of the vibration. This is due to the first two very intense cycles of this shaking ($a_{max} \approx 0.6g$ in Figure 1), which lead to strong shear deformation below the structure resting on such a shallow medium-dense sand layer.

The effect of this temporarily dilative behaviour in the first part of the seismic motion is also evident on the bucket settlement time-history. Figure 4 shows the comparison of the experimental and numerical (black and red line respectively) bucket settlement time-histories. The deviation of the two time-histories at the beginning of the vibration is attributed to the aforementioned numerically predicted dilative behaviour of the sand layer, which limits the settlement development for the first 5 seconds of the vibration. Thereafter, when the peak seismic acceleration values are more "typical" (e.g., 0.2 – 0.3g in Figure 1), the differences between the numerical simulation and the experiment diminish and the analysis yields a final settlement value exactly equal to that of the experiment. Performing a LiPa model re-calibration to capture the centrifuge test data even more successfully was not opted for, since it would undermine the use of this methodology in practice, where one relies on pre-calibrating the soil model.

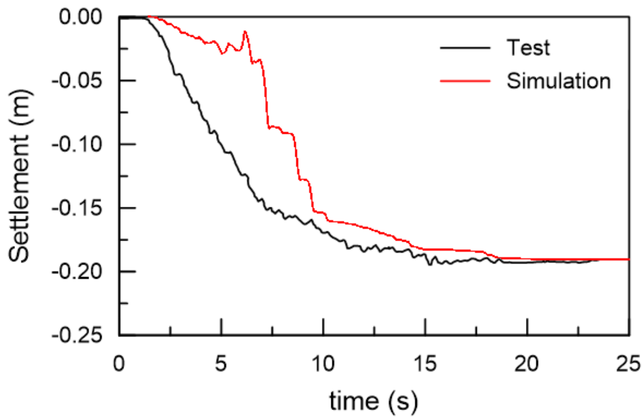


*Figure 4. Comparison of suction bucket settlements time-history from the simulation and the centrifuge test.*

Finally, Figure 5 presents a finite difference mesh (with deformation scaling factor of 1.5) and contours of accumulated shear strain at the end of shaking. Observe that the bucket's response is governed by the development of a shear band extending from its left skirt base towards the bottom right side of the liquefied layer. The existence of a "failure" mechanism below the bucket explains the significant settlement (see Figure 4), while the asymmetry of this mechanism explains the (small) residual rotation of the bucket to the left (seen in Figure 5 and also reported in Yu et al., 2014). The analysis overpredicts the residual rotation angle (in the order of 2º versus 0.6º in the data), but this is attributed to the uncertainty in the superstructure's fundamental vibration period (see Section 3.2). In general, rotation is more sensitive to the superstructure characteristics in comparison to settlement, whose value is lightly affected by the fundamental period of the superstructure when shallow-founded in liquefied soil (Bazaios et al., 2023). Meanwhile, the fact that the shear band extends to the bottom of the sand layer suggests that the results from the centrifuge experiment are affected by the liquefiable sand thickness and shouldn't be considered typical for suction buckets in thicker sand layers.

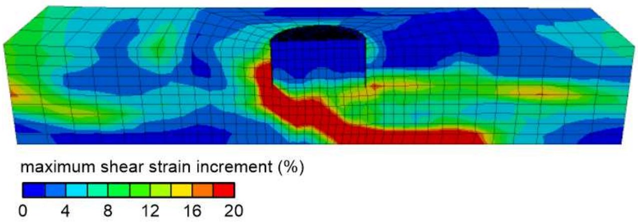


*Figure 5. Finite difference mesh (with deformation scaling factor of 1.5) and contours of maximum shear strain increment at the end of shaking.*

## 5 CONCLUSIONS

This paper introduces a 3D numerical methodology for simulating the response of suction buckets as OWT foundations under seismic action and liquefaction. The soil response is modelled with the LiPa constitutive model, using a single calibration for both static and dynamic loading irrespective of initial conditions, that was based on element tests from the literature (i.e., it was not problem-specific). The proposed methodology is applied for the simulation of a centrifuge test measuring the response of a single bucket foundation undergoing seismic loading and liquefaction, under the limitations of the test configuration (e.g., use of rigid box container). Overall, the numerical simulation shows good agreement with the experimental results in terms of both soil and bucket responses. This confirms the reliability of the proposed 3D numerical methodology, that may now be considered as a reliable tool for the performance-based design of such foundation systems under seismic loading and liquefaction. It should be underlined that using this methodology for design purposes requires that care is taken for simulating the actual site conditions (e.g., by using wide meshes for the soil and by employing boundaries replicating free-field conditions instead of a rigid box).

## AUTHOR CONTRIBUTION STATEMENT

**V.A. Katsoularis**: Data curation, Formal Analysis, Inevstigation, Methodology, Visualization, Validation, Writing - Original draft. **A.G. Papadimitriou**.: Project Administration, Conceptualization, Supervision, Writing - Reviewing and Editing. **Y.K. Chaloulos**: Supervision, Conceptualization, Software, Methodology, Writing - Reviewing and Editing.

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