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3D nonlinear dynamic Finite Element analysis of onshore wind turbines on pile foundation resting on liquefiable soils

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ABSTRACT: The latest development of wind turbines in seismic areas, such as East Asia and Californian coast, has increased the importance of their seismic design. High-intensity ground motions can strongly affect serviceability of wind turbines due to earthquake-induced settlements and rotations. In the presence of loose, saturated sandy soils, liquefaction may also become a concern. In such circumstances cyclic soil behaviour needs to be carefully simulated by considering coupled *u-p* formulations. In this paper, the main results of fully-coupled Finite Element nonlinear dynamic 3D analyses are discussed, where a typical onshore wind turbine on a piled raft foundation resting on liquefiable soils, with piles penetrating into dense sand, is subjected to high-intensity motions triggering liquefaction. Soil behaviour was described through the constitutive model SANISAND. The numerical results were compared to those obtained through a reduced-scale model tested in the centrifuge available at University of Cambridge, thus providing useful insight on the efficacy and reliability of 3D numerical modelling of such systems, which is anticipated to become more and more common in the near future.

Keywords: Wind Turbines, Liquefaction, Dynamic Centrifuge Tests, 3D Finite Element Analyses, *u-p* formulation.

1 INTRODUCTION

Wind turbine structure is inherently susceptible to dynamic loading due to its tall, flexible, and top-heavy structural characteristics. Apart from conventional considerations about mechanical vibration and environmental loading, protecting wind turbines from seismic loading becomes an emerging issue as the wind energy market expands toward the seismically active areas such as East Asia and the Californian coast, also referred to as the 'Ring of fire'. As modern wind farms consist of up to hundreds of turbines in the same design concentrated in one area, failure to provide sufficient seismic capacity for wind turbines might cause catastrophic damage to the entire wind farm.

However, despite its importance, research in this field is significantly restricted due to the lack of field seismic data. Earlier research (Esfeh and Kaynia, 2020) has suggested numerical analysis models to evaluate the seismic behaviour of wind turbines: however, these models need realistic field data to verify and calibrate the result to fit into the realistic condition. As the real earthquake event is unpredictable and unrepeatable, the geotechnical centrifuge has been considered as an alternative to providing such data. Geotechnical centrifuge tests can provide realistic seismic behaviour of geotechnical structure by reproducing the soil field stress condition using centrifugal force while providing sufficient controllability and repeatability from using a scaled model.

It would be possible to establish a numerical analysis model to evaluate wind turbine's realistic seismic behaviour by combining the numerical model with experimental data from the geotechnical centrifuge test.

When evaluating the seismic capacity of the wind turbine, liquefaction of the soil under its foundation is an important factor of consideration since the characteristics of soil change greatly once the liquefaction is triggered, hence making the analysis based on non-liquefiable soil invalid. Previous research (Seong *et al.*, 2022) shows that the wind turbine raft foundation can be subjected to excessive settlement and rotation once the liquefaction is triggered. Following-up research (Gaudio *et al.*, 2023) developed a numerical model to predict the seismic behaviour of wind turbine pre- and post-earthquake considering liquefaction. Then the result was calibrated to reflect the experimental result.

This research focuses on comparing the numerical Finite Element (FE) and geotechnical centrifuge results for the seismic behaviour of wind turbines supported by a group pile foundation on liquifiable soil. Results are examined with respect to soil and structure accelerations, excess pore water pressure generation, and foundation settlement. The main goal of the research is to verify the predictability of the numerical analysis model by evaluating the agreeability of the numerical analysis result with the experimental data provided by the geotechnical centrifuge test.

2 EXPERIMENTAL SETUP

In this section, a brief description of the centrifuge tests carried out by Seong *et al*. (2021) is given, which was then simulated in the FE code *Plaxis 3D CE v22.01* (Bentley, 2022).

2.1 Geotechnical centrifuge test

The behaviour of soil is heavily dependent on the effective stress it receives. The geotechnical centrifuge testing method is developed to reproduce prototype field stress conditions within a 1/N scaled model using centrifugal acceleration N times higher than the Earth's gravitational acceleration, thus representing the full-scale field geotechnical structure behaviour with the scaled model test. In this research, tests were conducted under 80 g centrifugal acceleration. All dimensions and results presented in this paper were converted into the prototype scale by applying relevant scaling laws, unless otherwise stated. The scaling laws for dynamic centrifuge tests were derived by previous researchers such as Schofield (1980) and Madabhushi (2014).

2.2 Testing model preparation

The scaled wind turbine and group pile model were produced for the test. The target wind turbine structure was simplified into mass on a pipe model, and the rotor-nacelle assembly (RNA) was simplified into a lumped mass. The simplified model was designed to match the overall dimension, the centre of gravity and the natural frequency of the target wind turbine. The simplified model was then scaled down by following the scaling law to produce the model scale dimension used for the test. Table 1 lists the model's structural details in model and prototype scale. The group pile used for the test consists of 25 hollow piles, each 26.0 m in length and 1 m in diameter. Piles were arranged into three ringed layers, each composed of 5, 8, and 12 piles. Figure 1 shows the scaled wind turbine and pile model used for the test.

The soil model is composed of two fully saturated sand layers: a 15-m-thick loose sand (relative density D_R = 40 %) overlying a 14.4 m-thick dense sand layer (D_R = 90 %). Bedrock was present below the dense sand layer at which the seismic loading was applied. The soil model was located into the new Equivalent Shear Beam (ESB) container (Brennan and Madabhushi, 2002) whose dynamic boundary effects in the presence of liquefiable soils have been recently assessed by Gaudio *et al*. (2023). The soil deposit was produced using Hostun HN31 sand topped by gravel in the vicinity of the foundation and clay at both sides. An automatic sand pourer was used to ensure uniform sand density. Two arrays of miniature piezoelectric accelerometers and pore pressure transducers (PPT) were installed inside of the soil layer, one near the foundation (centre array) and the other in the far field between the foundation and model box sidewall (far-field array). The soil was saturated with ~ 80 cSt viscous fluid to satisfy the dynamic centrifuge scaling law for viscosity (Adamidis and Madabhushi, 2015). The pile group was installed using a hydraulic press after the saturation; then, two clay blocks were placed on each side of the soil to simulate the partially excavated clay layer. The gap between the two clay blocks was filled with gravel.

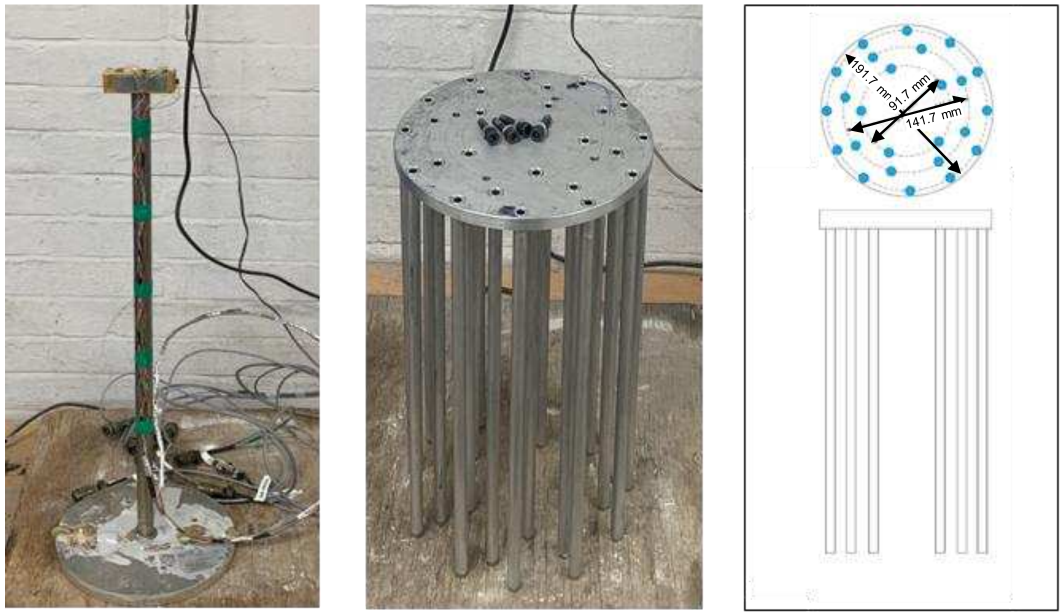
The wind turbine model was firmly connected to the head of the group pile top cap. Horizontal loading was added to the head of the wind turbine model using a pulley system to simulate the operational wind loading. MEMS accelerometers were attached to the head and foundation of the model to measure the horizontal and vertical acceleration of the structure. LVDT sensors were installed at each side of the foundation to measure the vertical displacement. Figure 2 shows overall testing model schematics with sensor placement information, together with a picture of the model as loaded on the beam centrifuge.

The main objective of the centrifuge test is to provide seismic response data of the soil-structure system during and after the earthquake. To achieve this, a sequence of dynamic motions was applied to the model using a servo-hydraulic shaker. First, a small-scale sine sweep from 0 to 1.5 Hz was applied to confirm the structure's natural frequency with consideration of soil-foundation-structure interaction (SFSI). The maximum input acceleration of the sine sweep was 0.02 g, which is small enough not to cause major soil disturbance. Fast Fourier transformation (FFT) of the tower head response shows f_n = 0.30 Hz. It is worth mentioning that the natural frequency measured during the geotechnical centrifuge test was noticeably lower than that of the fixed natural frequency (0.36 Hz) due to SFSI, as expected.

The main input motion used for the comparison with the numerical analysis is 10 cycles of 1 Hz sine wave with a peak input acceleration of 0.1 g. The sinusoidal motion was used to observe the simple dynamic behaviour of the wind turbine system, and 1 Hz frequency was chosen for higher than $3f_n$ to exclude the effect of harmonic oscillation. During various points of the centrifuge test, air hammer tests were done to monitor the soil stiffness change by measuring the soil's shear wave velocity (V_S).

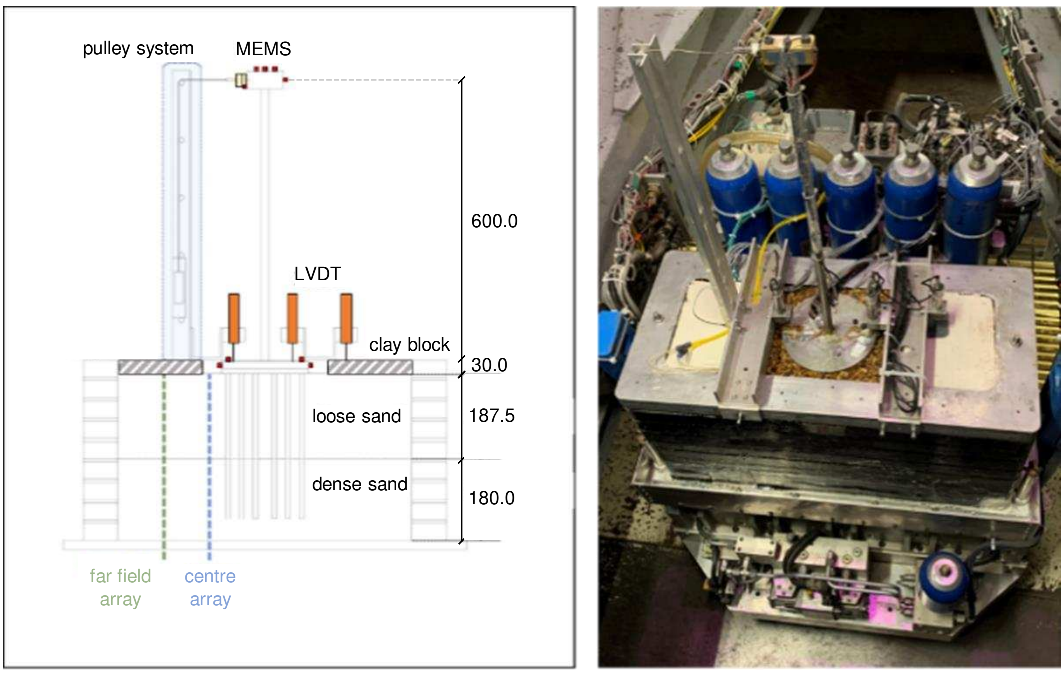
Table 1. Structural details of the wind turbine model

	MODEL SCALE	PROTOTYPE SCALE
Tower head mass	0.3 kg	153.6 tons
Tower diameter	17.5 mm	1.4 m
Tower thickness	2.5 mm	0.2 m
Tower height	600 mm	48 m
Raft foundation diameter	192 mm	15.36 m
Raft foundation thickness	17 mm	1.36 m
Raft bearing pressure	59.1 kPa	59.1 kPa
Fixed natural frequency	28.96 Hz	0.36 Hz
Lateral loading	62.8 N	401.8 kN



(a) (b) (c)

Figure 1. (a) Scaled wind turbine model, (b) group pile model, and (c) group pile dimension



(a) (b)

Figure 2. (a) Testing model schematics in model scale (units: mm) and (b) a photo of the completed model setting

3 3D FE MODEL

The 3D FE model is represented in Figure 3. The seismic input was applied along the x-direction only; however, it was not possible to consider half of the domain only, due to the 3D nature of piles (Fig. 3, right). Prototype, inner dimensions of the ESB container are $X = 51.6$ m $\approx 3.4\text{x}D$, $Y = 25.8$ m $\approx 1.7\text{x}D$ and $Z = 31.8$ m $\approx 13.3\text{x}s$, where $D = 15.4$ m and $s = 2.4$ m are the raft diameter and thickness, respectively, the latter including the pile cap as well. Piles are characterised by a length $L = 26.0$ m and a diameter $d = 0.8$ m, the latter being lower than the one adopted in the centrifuge (at prototype scale) to capture the flexural stiffness $EI = 1.225$ GN·m^2 through solid concrete rather than hollow aluminum piles. The FE mesh is made of 17205 10-noded tetrahedral elements and 29779 nodes. Both soil-pile and soil-raft contact were simulated though special interface finite elements available in the software, which were also applied beneath the base of the piles, as suggested by Bentley (2022).

It is worth mentioning that both the superstructure and piles were modelled through cluster elements. As for the piles, this would allow the user to modify their constitutive properties so as to model drains rather than piles. Moreover, the mesh was built on the purpose of permitting to simulate the "detached piles" condition, thanks to the introduction of 0.8-m-thick cluster elements in between the circular raft and the piles which may be replaced by soil.

Calculation phases are as follows:

- initialisation of soil stress state (k_0 procedure) to establish geo-static stresses;
- static phase where the gravel around and above the raft foundation replaced the equivalent amount of shallow clay;
- static phase with both raft and piles *wished-in-place* installation;
- activation of the wind turbine (tower and head mass);
- application of the wind thrust at the level of the head mass through a horizontal static force F_h = 1414.84 kN. This stage was performed under the assumption of drained conditions for the soils, as the main objective was only that of modifying the conditions preceding the earthquake. The horizontal force was kept constant over the following steps;
- switching the boundary conditions, from static to dynamic ones. The static ones were characterised by fixing displacements along all directions at the base of the model ($u_x = u_y = u_z = 0$) and displacements orthogonal to the vertical edges, while the dynamic ones entailed periodic boundary conditions (*tied degrees of freedom*) which impeded relative displacements along the direction of the seismic input ($\Delta u_x = 0$) on the two planes normal to the direction of the seismic input;
- application of the seismic input along the x direction at the base of the model in terms of acceleration time history. As mentioned above, the seismic input hereby considered is a sinusoidal earthquake characterised by a peak acceleration $PGA = 0.1g$, a frequency of 1 Hz and 10 cycles. This dynamic phase was performed as a *dynamic with consolidation* calculation, which has been recently implemented in *Plaxis* and that allows the user to adopt the *u-p* formulation (Zienkiewicz *et al*., 1980). This kind of analysis permits to accurately analyse the dissipation of seismic-induced excess pore water pressures during the earthquake, which is a key feature of liquefiable sandy soils. The relevant hydraulic boundary conditions were characterised by a drainage contour at the water table level (z = 2.4 m), whereas all remaining boundaries were modelled as impervious (closed). All materials above the water table, such as clay, gravel, and structures, were assigned as non-porous materials.

Cyclic behaviour of both the loose and dense sand (and relevant interfaces) was described through the SANISAND04 constitutive model (Dafalias and Manzari, 2004), whose values of mechanical parameters are given in Gaudio *et al*. (2023), together with their hydraulic conductivity. Gravel and clay were assigned a linear elastic-perfectly plastic behaviour with the Mohr-Coulomb failure criterion, whereas the behaviour of all structural members (piles, raft, tower and head mass) was simulated using a linear elastic model with relevant Young's modulus and Poisson's ratio.

All materials were assigned a Rayleigh (viscous) damping ratio $\xi = 1$ %, while a value $\xi = 3$ % was assigned to the superstructure (both the tower and head mass): the latter stems from previous calibration based on free-oscillation of the fixed-base tower (Gaudio *et al*., 2023).

The output schematics adopted in the numerical analysis was identical to that previously adopted in the centrifuge tests through miniature instruments (Fig. 2a).

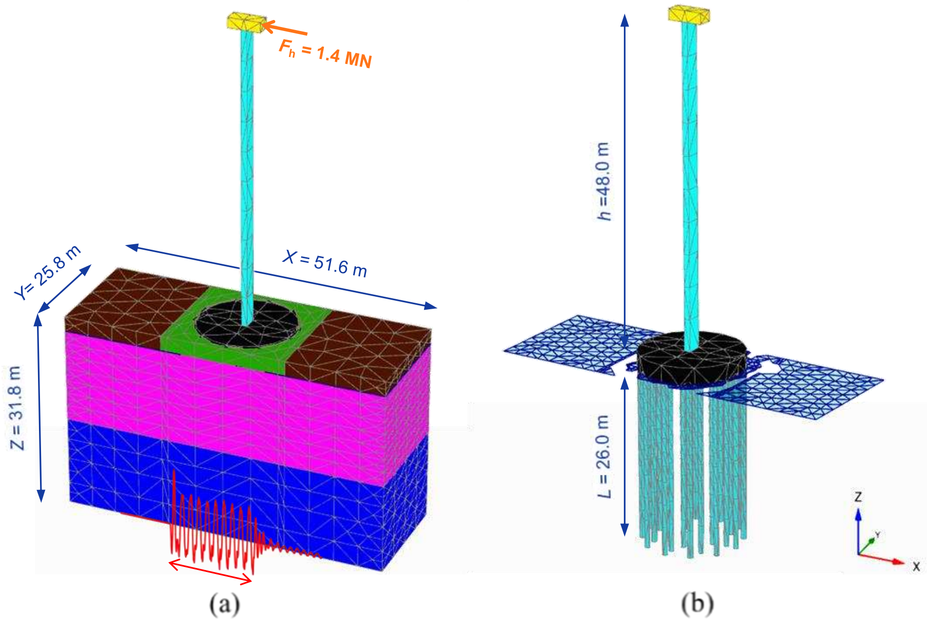


Figure 3. 3D FE model implemented in Plaxis CE 3D v22.01: mesh (left); details of the piles and the water table level (right)

4 EXPERIMENTAL VS. FE RESULTS

This section reports the main results obtained from the numerical analysis and the comparison with the relevant experimental ones.

The main results obtained along the far-field array are given in Figure 4, where the time histories of the horizontal acceleration, a_x, and pore pressure ratio, $r_u = \Delta p/\sigma'_{v0}$, are plotted, with Δp being the excess pore water pressure and σ'_{v0} the lithostatic vertical effective stress. In the Figure, z-axis is assumed to be directed upwards from the ground surface. As for the horizontal acceleration, the agreement is very good, both in terms of amplitude and frequency content: this holds true for both the loose and dense sand layer. Excess pore water pressures are fairly predicted by the numerical model as well, particularly up to time instant $t = 10$ s. Different maximum values of r_u are obtained in the loose sand layer, namely $r_{u,\,max} \sim 0.6$ and ~ 0.4 in the centrifuge and the numerical FE model, respectively, whereas a very good agreement on the final value of r_u ($= 0.2$) is computed at the model base (dense sand). In the latter case, slightly higher amplitude cycles are computed in the numerical model though.

The slight differences observed in the excess pore water pressures along the far-field alignment affected the prediction of the clay, far-field settlement, as a permanent value $w_{ff} = 0.05$ m was obtained numerically, lower than the one measured in the centrifuge, $w_{ff} = 0.09$ m (Fig. 5). Conversely, the agreement in terms of absolute vertical displacement of the raft edges is satisfactory, with a settlement of about 0.01 m on the left edge and zero on the right one: it is worth noting that this result is in agreement with the direction of application of the static force simulating the operational wind, which was not captured in the centrifuge test.

Finally, the comparison between the numerical and experimental results is reported in Figure 6 in terms of peak acceleration amplification. Here the agreement can be still deemed satisfactory into the soil deposit (height < 0), whereas the amplification is not captured at the tower head. This may be improved by reducing the superstructure damping ratio, which constitutes the ongoing research which has being currently carried out.

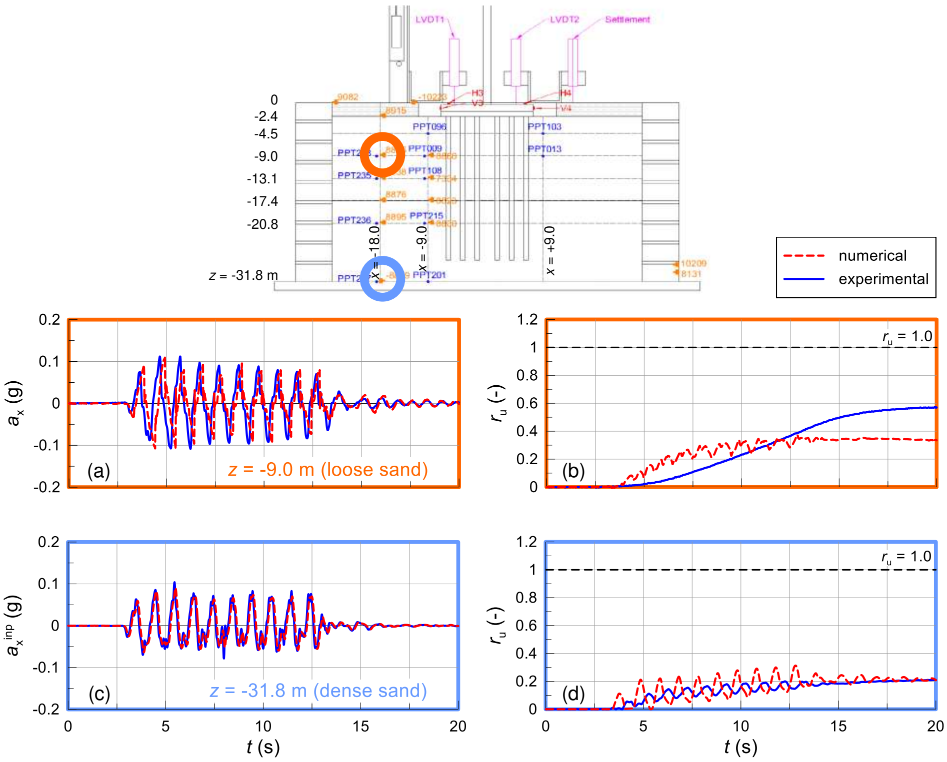


Figure 4. Numerical and experimental time histories of (a, c) horizontal acceleration and (b, d) pore pressure ratio

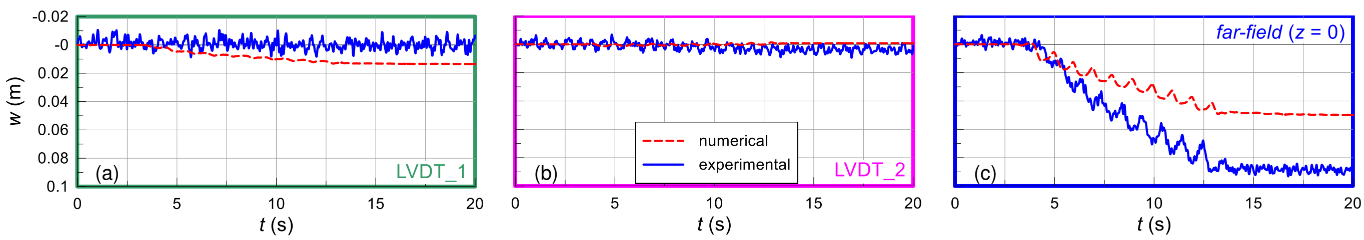


Figure 5. Numerical and experimental time histories of the settlement at the (a) left and (b) right edge of the raft; (c) far-field

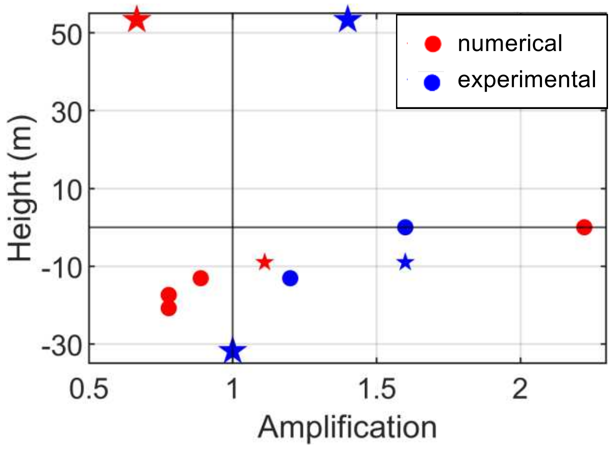


Figure 6. Profile of peak acceleration amplification

5 CONCLUSIONS

This paper illustrated the comparison between the numerical results, which were obtained by performing the 3D nonlinear dynamic analyses in the time domain via the FE code *Plaxis 3D CE v22.01*, and the centrifuge test reported in Seong *et al.* (2021). The onshore wind turbine seated on a piled foundation, embedded in a loose and dense sand layers, the former being susceptible to liquefaction. The sand mechanical behaviour under cyclic loading was simulated through the bounding surface model SANISAND: the hydro-mechanical sand parameters which were adopted in the analyses had been previously calibrated for the Hostun sand (Gaudio *et al*., 2023).

The results discussed in this paper showed that the numerical prediction can be deemed satisfactory as the far-field soil response, both in terms of horizontal acceleration and excess pore water pressure time histories. Same accuracy was reached for the absolute settlement of the piled foundation, whereas the same cannot be stated as for the far-field settlement, where a difference of about 80 % was obtained. However, it is worth mentioning that, although these differences may need to be reduced, seismic performance of the structure can be still deemed adequate in the Performance-Based Design (PBD) framework, where the order of magnitude of permanent settlement and tilting are the main focus of the analysis. From the foregoing it follows that the difference of about 0.04 m in terms of the relative settlement (corresponding to 0.26 % of raft diameter) is a fairly good estimate of the piled raft performance, bearing in mind the complexity of the analyses at hand. Hence, the numerical model hereby presented may be still deemed reliable.

Further improvement of the numerical model may head towards lowering the viscous damping which was assigned to the OWT at hand (e.g., from 3 % to 1 %). Then, the model may be profitably used to perform an extensive parametric study concerning soil hydraulic conductivity and the properties of inclusions, where cylindrical drains may replace the piles hereby considered and where the detached piles condition may be taken into consideration as well.

6 ACKNOWLEDGEMENTS

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REFERENCES

Adamidis, O., Madabhushi, S.P.G. 2015. Use of viscous pore fluids in dynamic centrifuge modelling, *Int. J. Phys. Mod. Geotech.*, **15** (3): 141–149.

Bentley. 2022. *Plaxis 3D CE v22.01 – Reference Manual*, Delft Univ. Tech. Delft, The Netherlands.

Brennan, A. J., Madabhushi S. P. G. 2002. Design and performance of a new deep model container for dynamic centrifuge testing. *Proceedings, Physical Modelling in Geotechnics – ICPMG '02* (Phillips R, Guo PJ and Popescu R (eds)), 183-188. Balkema, Rotterdam, The Netherlands.

Dafalias, Y.F., Manzari, M.T. 2004. Simple plasticity sand model accounting for fabric change effects, *J. Eng. Mech*., ASCE, **130** (6), 622-634.

Esfeh, P. K., Kaynia, A. M. 2020. Earthquake response of monopiles and caissons for Offshore Wind Turbines founded in liquefiable soil. *Soil Dyn. Earthq. Eng*., **136**, 106213.

Gaudio, D., Seong, J., Haigh, S. K., Viggiani, G. M. B., Madabhushi, S. P. G. 2022. Assessment of boundary effects of the Equivalent Shear Beam container for dynamic centrifuge testing of liquefiable soils. *Proceedings, 10th International Conference on Physical Modelling in Geotechnics (ICPMG 2022)*. Daejeon, South Korea, September 19-23: 1-4.

Gaudio, D., Seong, J., Haigh, S. K., Viggiani, G. M. B., Madabhushi, S. P. G., Shrivatsava, R., Veluvolu, R., Padhy, P. 2023. Boundary effects on dynamic centrifuge modelling of onshore wind turbines on liquefiable soils, *Int. J. Phys. Mod. Geotech.*, **23** (1), 16-34, DOI: 10.1680/jphmg.21.00085.

Madabhushi, S. P. G. 2014. *Centrifuge modelling for civil engineers*, Boca Raton, FL, USA: CRC Press (Taylor & Francis Group).

Schofield, A. N. 1980. Cambridge geotechnical centrifuge operations, *Géotechnique*, **30** (3), 227–268.

Seong, J., Haigh, S.K., Madabhushi, S. P. G. 2021. *Dynamic centrifuge tests for onshore wind turbine with pile group*, Data report AGEL-4, February 2021, University of Cambridge, Department of Engineering.

Seong, J., Haigh, S.K., Madabhushi, S. P. G., Shrivastava, R., Veluvolu, R., Padhy, P. 2022. On seismic protection of wind turbine foundations founded on liquefiable soils. *Soil Dyn. Earthq. Eng*., **159**, 107327.

Zienkiewicz, O.C., Chang, C.T., Bettes P. 1980. Drained, undrained, consolidating and dynamic behaviour assumptions in soils, *Géotechnique*, **30** (4), 385-395.