Boundary effects on dynamic centrifuge modelling of wind turbines on liquefiable soils

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Abstract

Centrifuge modelling is an effective tool to assess the response of reduced-scale structures subjected to earthquakes under increased gravity. Indeed, this experimental technique allows to obtain experimental results under simple and controlled conditions. However, obvious space limitations force the model to be located into relatively small boxes, such as the Equivalent Shear Beam (ESB) container, whose boundaries may adversely affect the seismic performance of the structure under consideration. In this paper, influence of proximity of the ESB box boundaries during dynamic centrifuge tests of an onshore wind turbine resting on liquefiable soils is evaluated. To this end, numerical modelling of the ESB box was implemented in the Finite Element framework OpenSees, so as to fairly capture the results observed in the experiment: hydro-mechanical soil parameters were therefore calibrated against *far-field* centrifuge results only. From this calibration, the seismic performance of the raft foundation turned out to be in a good agreement with the experimental results, for a seismic input capable of triggering liquefaction. Then, a larger numerical model, where boundaries do not play any role, was built, to compare its outcomes with those coming from the small model: this allowed the effect of ESB boundaries to be assessed.

Keywords

Centrifuge modelling; Numerical modelling; Earthquakes.

List of notations

$a_x{}^{inp}{}_{max}$	is the peak acceleration of the seismic input
$a_x{}^{top}{}_{max}$	is the peak acceleration at the top of the wind turbine
D	is the diameter of the raft foundation
D_R	is the relative density
e_{max}	is the maximum void ratio
e_{min}	is the minimum void ratio
f_s	is the fixed-base natural frequency of the wind turbine (=0.30 Hz)
g	is the gravitational acceleration (= 9.81 m/s^2)
G_0	is the small-strain shear modulus of soil
G_s	is the specific gravity
H_2	is the thickness of the loose sand layer
H_3	is the thickness of the dense sand layer
h_s	is the height of the wind turbine
J_{HEAD}	is the rotational inertia at the tip of the wind turbine
k	is the isotropic hydraulic conductivity of soil
m_{HEAD}	is the mass at the tip of the wind turbine
m_{tot}	is the total mass of the wind turbine

N	is the scaling factor adopted in the centrifuge test (= 80)
p	is the pore water pressure
PSa	is the pseudo-acceleration
q	is the bearing pressure exerted by the structure on the loose sand layer (= 58.8 kPa)
s	is the thickness of the raft foundation
$V_{S,0}$	is the small-strain shear wave velocity of soil
w_{ff}	is the *far-field* settlement
Δp	is the excess pore water pressure
Δt	is the time increment
Δy	is the vertical distance between two adjacent nodes in the FE model
θ	is the rigid rotation of the raft foundation
λ_{min}	is the minimum wavelength travelling into the FE model
ν	is the Poisson's ratio
ξ	is the damping ratio
ρ	is the mass density
φ'_{cv}	is the constant-volume friction angle

Introduction

Dynamic centrifuge modelling is recognised as a powerful tool for evaluating the seismic performance of structures subjected to strong seismic events. This is particularly true when assessing the liquefaction hazard of structures resting on loose saturated sandy soils, as pointed out by several Authors (Dashti et al., 2010; Karimi, 2016; Manzari et al., 2018; Esfeh and Kaynia, 2020; Adamidis and Madabhushi, 2021). Centrifuge testing is also often adopted to calibrate numerical Finite Element (FE) of Finite Difference (FD) models that are typically used to perform extensive parametric study (Ramirez et al., 2018; Ramirez, 2019; Chen et al., 2021).

Although very useful and widely adopted, reliability of dynamic centrifuge testing still needs to be carefully evaluated and understood. Indeed, major concern about centrifuge tests is about the presence of boundaries of container where the reduced-scale model is placed into, as they could play a role and affect the results obtained in the laboratory. In this context, Teymur and Madabhushi (2003) performed an experimental study where boundary effects generated with a previous version of the Equivalent Shear Beam (ESB) container used at Cambridge University (Schofield and Zheng, 1992) were investigated, showing that these effects can mainly induce amplification of motion caused by P-wave generation at the edge walls, due to the stiffness contrast between the boundaries and the soil sample. In their study, boundary effects turned out to be minimal for dry and medium-dense sands, (relative density D_R = 50 %), that is when the sandy sample is characterised by a stiffness similar to that the ESB container was designed for. However, in the presence of loose and saturated sand layers subjected to strong seismic shaking liquefaction may occur, thus providing a quite soft soil whose stiffness contrast with the end walls has increased: in this case, boundary effects may play a major role. Hence, the Authors concluded that, in the presence of loose and saturated sandy soils, estimation of boundary influence is necessary.

In this paper, boundary effects of the most recent ESB container adopted at University of Cambridge (Brennan and Madabhushi, 2002) are assessed. Results of a centrifuge test where an Onshore Wind Turbine (OWT) resting on liquefiable soils through a raft foundation was subjected to one-direction ground motions were first taken as a reference. Numerical modelling of the ESB box containing the OWT and soil deposit was then performed through a 3D Finite Element (FE) numerical model implemented in the open-source OpenSees framework v 3.3.0 (McKenna et al., 2000; Tarque Ruiz, 2020), so as to accurately reproduce the results obtained in the centrifuge. Cyclic sand behaviour was described through the advanced constitutive model SANISAND04 model (Dafalias and Manzari, 2004); moreover, the bi-phase nature of saturated soils was accounted for through the *u-p* formulation, based on the assumption of negligible soil-fluid relative acceleration (Zienkiewicz et al., 1980). Soil mechanical and hydraulic parameters were calibrated against the *far-field* centrifuge results, these providing a fair estimate of the seismic performance of the structure. This was needed as calibration of SANISAND04 parameters against centrifuge tests is not available in the literature so far, although Hostun sand is widely adopted for research purposes (Tsinidis et al., 2015). Then, a larger numerical model was built, where boundaries do not affect the system's response, to compare its outcomes with those coming from the small model and therefore allowing to assess the effect of ESB boundaries in a quantitative manner.

Findings presented in this study may be useful for engineers to interpret the results coming from centrifuge tests where liquefaction is triggered, more confidently and with an increased awareness. Moreover, calibration of SANISAND04 parameters for Hostun sand against centrifuge tests is a further novel aspect of the paper.

2. Problem layout

The schematic layout of the problem is given in Figure 1. An onshore wind turbine of height h_s = 48 m rests on a circular raft foundation with diameter D = 15.4 m and thickness s = 1.6 m. The tower is characterised by a total mass m_{tot} = 435.8 Mg and a fixed-base natural frequency $f_s \approx$ 0.3 Hz, while the raft foundation lies on a fully-saturated loose sand layer (D_R = 43 %) of thickness H_2 = 15 m underlain by a dense sand layer (D_R = 90 %) of thickness H_3 = 12 m. The bearing pressure exerted by the structure on the sand layer is q = 58.8 kPa. Superficial layer is constituted of a partially-excavated clay (H_1 = 3.2 m) and a gravel layer. The above-mentioned properties were selected to represent a typical configuration for an OWT on liquefiable soils.

The system is subjected at the base (y = -30.2 m) in the horizontal x-direction to a sinusoidal motion, whose characteristics will be discussed in the following: here it is worth mentioning that this ground motion is intense enough to trigger liquefaction into the loose sand layer. The input is applied in terms of horizontal acceleration time history: the assumption of indefinitely-rigid bedrock is therefore made (fully-reflecting boundary).

3. Dynamic centrifuge testing

Scaled model of the OWT was produced to simulate the prototype structure behaviour through the Turner beam centrifuge of the Schofield Centre at University of Cambridge, UK. The centrifuge model was prepared and spun at a nominal centrifugal acceleration of 80g. Unless otherwise indicated, all units presented in this paper are in prototype scale.

The model container used was the most recent ESB box. In the reduced-scale model, the raft foundation is simulated through an aluminium circular plate, while the OWT is modelled through a steel hollow tube with a lumped brass mass at the top. At model scale, the raft foundation is characterised by a diameter equal to 192 mm and a thickness equal to 20 mm, whereas the steel hollow tube has an outer diameter D_{out} = 17.5 mm and a wall thickness s_w = 2.5 mm; the head mass is m_{lump} = 300 g. To produce the composite layer of sand, clay and gravel, the soil model was made in three steps. First, sand layers were produced using sand pourer. Second, the clay layer was created using pre-cut clay blocks. Third, gravel was placed in the gap where the raft foundation was located.

Hostun HN31 sand was adopted for the preparation of the sand layers, whose physical properties are as follows: specific gravity G_s = 2.65; maximum and minimum void ratio, e_{max} = 1.011 and e_{min} = 0.555, respectively; and constant-volume friction angle φ'_{cv} = 33°. The target relative densities were obtained with air pluviation using the sand pourer available at the Schofield Centre (Madabhushi et al., 2006). After the pouring, the sand layers were fully-saturated through a high-viscosity aqueous solution of hydroxypropyl methylcellulose (Adamidis and Madabhushi, 2015), characterised by a viscosity equal to 80 MPa·s.

The model layout is reported in Figure 2a together with sensor schematics. Arrays of piezo-electric accelerometers (red arrows in Fig. 2) and pore pressure transducers (PPTs, blue ellipses in Fig. 2) were installed in the centre of the box beneath the structure and supposed *far-field* between the structure and side wall. Sensors located in the same position were installed at least 2.5 cm apart with each other to reduce the interference from the sensor body. A linearly varying differential transformer (LVDT) was positioned at the clay surface to measure the *far-field* settlement w_{ff}. Air hammer was installed at the bottom of the box to perform Air Hammer Tests (AHT, Ghosh and Madabhushi, 2002) during the centrifuge experiment, to obtain the shear wave velocity $V_{S,0}$ of the soil deposit and its small-strain shear modulus G_0, through the well-known relation

$$G_0(z) = \rho \times \left[V_{S,0}(z)\right]^2$$

1.

where ρ is the mass density of soil, whose values are given in the following. The experimental values of the small-strain shear modulus obtained at the *far-field* alignment are plotted in Figure 2b (black crosses).

Horizontal accelerations were also measured on both the model structure's head and foundation (H1, 2, 3, and 4 in Fig. 2a), together with the vertical ones (V1, 2, 3, and 4), through Micro-Electrical-Mechanical Systems (MEMS) accelerometers. Two LVDTs were positioned at the foundation, each 7 cm apart from the central axis, to measure both settlements and compute the foundation rigid rotation during the test. Figure 3 shows the finished model mounted on the centrifuge.

4. Three-dimensional Finite Element modelling

In this section, results of FE nonlinear dynamic 3D analyses performed in the time domain are shown and discussed. First, characteristics of the FE model reproducing the experimental setup with the ESB container size (*small* model in the following) are reported: this model was adopted to calibrate both the hydraulic and mechanical parameters of the sand layers, so as to be representative of the centrifuge test. Then, a much larger numerical domain (*large* model), where wall ends do not affect the seismic performance of the OWT, was developed, to quantify boundary effects of the ESB container.

For both models, geometry and mesh were visualised through the pre/postprocessing software GID v. 14.0.3 (Coll et al., 2018); the *.tcl* file needed for the analysis was first generated using the GID+OpenSees interface v.2.7.0 (Papanikolaou et al., 2017) and then modified.

4.1 Small model reproducing the ESB container

The *small* 3D model is represented in Figure 4a. Following the centrifuge test, the seismic input was applied along the *x*-direction only, making it possible to consider half of the domain thanks to problem symmetry. Model dimensions are exactly those of the ESB container at prototype scale (scaling factor N = 80), that is X = 51.6 m ≈ 3.4 x D, Z = 9.12 m and Y = 30.2 m ≈ 19 x s. The FE mesh is made of 2401 elements and 3121 nodes, with a progressively finer mesh approaching the raft foundation, and particularly at the soil-foundation interface, where thin continuum layers were placed (Fig. 4b) to assign materials of "degraded" mechanical properties (Griffiths, 1985). Thickness of the interface layers was imposed equal to 5%D ≈ 0.8 m (Pisanò, 2019).

BrickUP elements were adopted to discretise the whole domain (Yang et al., 2008). These are hexaedral linear isoparametric elements that were developed on purpose for saturated soils, for which the *u-p* formulation (Zienkiewicz and Shiomi, 1984) is adequate when soil-fluid relative motion can be neglected. Size of the finite elements adopted for soils were selected to fulfil the requirement provided by Kuhlemeyer and Lysmer (1973), therefore avoiding numerical distortion of waves propagating into the model. To this end, the vertical distance between two adjacent nodes, Δy, was checked to satisfy, at every depth, the condition

148 $$\Delta y \leq \frac{\lambda_{\mathrm{min}}}{6} = \frac{V_S}{6 \times f_{\mathrm{max}}}$$

2.

where λ_{min} is the minimum wavelength expected to travel into the FE model, V_S is the soil shear wave velocity and f_{max} = 4 Hz is the maximum frequency of the seismic input. Shear wave velocity profile was evaluated at every depth referring to the profile adopted in the SANISAND04 constitutive model, calibrated against the AHT results (Fig. 2b).

As for mechanical boundary conditions applied to the soil domain, in the initial static (gravity) calculation phase the condition $u_z = 0$ was applied to the x-y plane at the boundaries, while all three displacement components were impeded at the base of the model ($u_x = u_y = u_z = 0$). A periodic boundary condition was applied in the direction of the seismic input to all nodes belonging to the y-z plane, thus enforcing the nodes at same depth to displace by the same amount ($\Delta u_x = 0$) and therefore imposing free-field pure shear conditions at the lateral boundaries, such as those applied by the end walls of the ESB container. When switching to the dynamic calculation phase, the restraint on the horizontal displacement at the base was removed and the seismic input was applied in terms of the horizontal acceleration time history plotted in Figure 5c. This seismic input is exactly that fired in the centrifuge test, representing a high-intensity sinusoidal acceleration time history characterised, at the prototype scale and in its stationary part, by a peak acceleration $a_x^{inp}{}_{max} \approx 0.2\ g$, a frequency f = 1 Hz and a total duration of 10 s (i.e. 10 cycles).

Hydraulic boundary conditions were set as well: as the water table was located at the top of the loose sand, pore water pressures were allowed to fluctuate freely for all nodes into the sand layers ($y \leq -3.2$ m), while both steady and excess pore water pressures were inhibited above ($p = \Delta p = 0$).

Soil constitutive models and relevant parameters were calibrated against the *far-field* centrifuge results, as discussed in the following section (§ 4.1.1), while mechanical behaviour of both the aluminium raft and the steel tower was described through an isotropic linear-elastic medium, whose values assumed for parameters are listed in Table 1, where ρ is the mass density, E is the Young's modulus and ν is the Poisson ratio. The tower was modelled through sixty 0.8-m-long *Timoshenko beam* elements, with a nodal mass at the tip simulating half (for symmetry) of its lumped mass (m_{HEAD} = 153.6/2 = 76.8 Mg). Moreover, the node at the tip was assigned the rotational inertia of the brass mass as well (J_{HEAD} = 480.06/2 = 240.03 Mg·m^2), in order to reproduce the centrifuge test as closely as possible.

The 3D nonlinear dynamic analyses were performed selecting a maximum time increment Δt = 0.0133 s, equal to that adopted in the centrifuge test, and prolonging the dynamic calculation phase up to 15 s. Newmark's time stepping method (Newmark, 1959) was used to integrate equations of motion with values β = 0.60 and γ = 0.3025, while the Krylov-Newton solution algorithm (Scott and Fenves, 2010) was selected to handle nonlinear soil behaviour. A tolerance of 10^{-3} was chosen for the convergence test, based on the norm of the incremental displacement.

4.1.1 Soil constitutive models and calibration of hydro-mechanical parameters

Mechanical behaviour of soils was described by adopting three different advanced constitutive models: Pressure Independent Multi-Yield (PIMY) model for the clay layer, Pressure Dependent Multi-Yield (PDMY) (Yang et al., 2003) model for the gravel layer and SANISAND04 for both the loose and dense sand layers.

For the clay and gravel layer, suggested values of model parameters provided by Yang et al. (2008) were adopted, assuming a soft clay and a medium dense sand (D_R = 65-85 %)-like behaviour, respectively (Tabs. 2 and 3).

As for the loose and dense sand layers, SANISAND04 model parameters adopted were first retrieved from Salvatore et al. (2017), who calibrated the constitutive parameters against triaxial tests on samples of Hostun sand, and then further calibrated to match the excess pore water pressure build-up and dissipation measured during the centrifuge test along the *far-field* array (Tab. 4). Hydraulic conductivity k, as a property of the BrickUP finite elements and not of the SANISAND04 constitutive model, was reduced about six times with respect to Kassas et al.

(2020) to match the excess pore water pressure dissipation after the end of the earthquake, thus obtaining $k = 1.48 \times 10^{-4}$ and 6.74×10^{-5} m/s for the loose and dense sand, respectively. For both gravel and loose sand interfaces, stiffness moduli were reduced by a multiplying factor = 2/3, while the shear strength was reduced by a factor = 3/4, (Kementzetzidis et al., 2019).

A small amount of damping (ratio ξ_{soil} = 1 %) was added through the Rayleigh formulation to attenuate the effect of spurious high frequencies that may arise in the domain. The raft foundation and the turbine were assigned damping ratios ξ_{raft} = 1 % and $\xi_{turbine}$ = 3 %, respectively, the latter being calibrated against experimental free vibrations.

Thanks to the calibration of soil parameters, the *small* numerical model can be deemed representative of the ESB container adopted in the centrifuge, as shown in Figure 5 in terms of total horizontal acceleration (a) and excess pore water pressure (b, d) time histories obtained into the loose (depth y = -10.2 m) and dense (y = -30.2 m) sand layers along the experimental and numerical *far-field* array. Calibration of SANISAND04 and hydraulic soil parameters aimed at reproducing the excess pore water pressure time history Δp (t) developed into the loose sand layer (Fig. 5b): although seismic-induced pore pressure build-up turned out to be slightly quicker than in the centrifuge test, calibration allowed to fairly capture both peak values (≈ 80 against 100 kPa, with a difference of about 20%) and frequency content. Moreover, beginning of post-seismic consolidation was adequately reproduced, as demonstrated by the almost parallel time traces. Same conclusions may be drawn for excess pore water pressures into the loose sand layer (Fig. 5d). Acceleration time traces recorded into the loose sand layer were fairly captured as well (Fig. 5a), except for some spares spikes obtained in the centrifuge in between 2.5 and 5 s, which may be attributed to densification occurring into the loose sand and which cannot be reproduced by the numerical model.

The above-mentioned good calibration of soil parameters is supported by the “blind” prediction of the wind turbine seismic performance (Fig. 6). Absolute settlement w of the raft foundation was almost perfectly captured by the FE analysis on the left side (Fig. 6a), in terms of both permanent value (being 0.34 m in the FE analysis and 0.33 m in the centrifuge) and rate of accumulation, whereas slight differences are observed in its frequency content. Conversely, some experimental errors were detected in LVDT_2 measurements (right side) and therefore are not displayed in Figure 6b, where the numerical outcome only is plotted. Seismic performance of the raft is given in Figure 6c in terms of the relative average settlement with respect to the *far-field* one, the latter measured at ground surface. Here the numerical time history shows some deviation from the experimental one, the comparison being still satisfactory though (0.42 with the FE model against 0.45 m obtained experimentally, with a difference of 7%). Finally, the counter-clockwise rotation of the raft foundation (Fig. 6d) is computed as follows

$$\theta(t) = \frac{\left[w_1(t) - w_2(t)\right]}{d_{12}}$$

3.

where w_1 and w_2 are the settlement measured at the left and right side of the foundation and d_{12} = 11.2 m is the distance in between. The experimental rotation time history is fairly captured as well through the numerical analysis: a peak value equal to -0.2° is computed in both the experimental and numerical test indeed, albeit slight difference in the frequency content of the time traces is observed. It is worth mentioning that the experimental rotation time history has been computed from the LVDT_1 measurement assuming as instantaneous point of rotation of the raft foundation the numerical one, to get an estimate of the LVDT_2 measurement.

The satisfactory prediction made though the *small* numerical model comes from a good “blind” forecast of the excess pore water pressures developed beneath both sides of the raft foundation (Fig. 7). Indeed, almost same values of excess pore pressure were obtained at the end of the seismic event in the numerical and experimental tests, this implying that very close permanent settlement and rotation were to be expected. In contrast, the previously-discussed numerical time history of rotation turned out to be ahead the experimental one, and this can be explained by looking at the excess pore water pressure time histories plotted in Figure 7: in fact, a relatively excellent prediction was retrieved on the left side of the raft (Fig. 7a), whereas quite a faster development of excess pore water pressures was computed on the right side by the numerical FE *small* model, this providing a different accumulation rate of the raft rigid rotation.

Finally, comparison between numerical prediction and centrifuge recording is made in Figure 8 in terms of horizontal acceleration time histories (Fig. 8a) and elastic pseudo-acceleration spectra (damping ratio $\xi = 5$ %, Fig. 8b) transmitted to the top of the wind turbine, to get an estimate of inertial forces acting on it. Again, the comparison is quite satisfactory both in terms of the peak acceleration (equal to about 0.12 *g* both in the numerical and experimental test) and frequency content. As for the latter, the pseudo-acceleration spectra show the two peaks at periods $T = 0.44$ s and 1.00 s, being the second eigen-period of the system and of the seismic input, respectively. Fundamental period of the system, equal to 0.30 s, is not excited by the applied seismic input. Modest differences are observed when looking at the acceleration time histories during free-oscillations time frame following the end of the seismic input ($t = 12.5$ – 15 s) and in the pseudo-acceleration spectra at the above-mentioned periods.

4.2 Dynamic analysis through the large model

Boundary effects of the ESB container in the presence of loose and saturated sandy soils are assessed through the comparison of results obtained with the *small* numerical model, deemed representative of the ESB container, with those computed through the *large* 3D model depicted in Figure 9. This domain simply constitutes an extension of the *small* model in the *x* and *z* direction: new dimensions are now equal to $X = 150$ m $\approx 10 \times D$ and $Z = 75$ m $\approx 5 \times D$, which were selected to ensure that model boundaries were far enough from the structure, thus not affecting its dynamic behaviour. FE mesh is now made of 12013 elements and 13780 nodes, and the same hydraulic and mechanical boundary conditions as those already discussed in § 4.1 were adopted.

Outputs of the numerical analysis with the *large* model mainly followed the location of instruments set for the centrifuge test (Fig. 2), and schematics adopted in the *large* model is represented in Figure 10. With respect to the *small* numerical model, the *free-field* alignment

(x = -39.0 m) only was added in the *large* model, which is supposed to be affected very little by both the structure and the lateral edges of the *large* numerical model. Conversely, although the *far-field* alignment (x = -17.0 m) can be still judged not being influenced by the vertical edges of the *large* model, it might be affected by the structure: hence, comparison of results obtained along the *free-field* and *far-field* alignments in the *large* model allowed to quantify the influence of the structure on the *far-field* results, while the comparison between outcomes from the *large* and *small* models along the *far-field* arrays will shed some lights on the influence of boundaries on the *far-field* results in the *small* model (and therefore on the ESB box).

Comparison of *free-* and *far-field* results obtained in the large FE model is first shown in Figure 11, where horizontal acceleration and excess pore water pressure time histories are plotted. From the Figure it can be clearly seen that results almost overlap at all depths, with some acceptable discrepancies at the ground surface (Fig. 11a) in terms of peak values of horizontal acceleration, where a difference of about 10% is computed. Profiles of peak values are given in Figure 12, where the peak acceleration ratio (Fig. 12a) conveys the ratio between the peak acceleration at a given depth, $a_{x\,max}$, and the peak acceleration of the input motion, $a_x^{inp}{}_{max}$: comparison is very good even in terms of peak values of acceleration ratio and excess pore water pressure, confirming that the *far-field* alignment is not influenced by the presence of the structure and that is representative of the *free-field* soil response, despite its proximity to the structure (about 10 m away ≈ 0.6 x D, see Fig. 2).

5. Assessment of boundary effects on the OWT seismic performance

Boundary effects of the ESB container are assessed in this section through the comparison of results obtained along the *far-field* alignments into the *large* and *small* numerical model, the former being representative of *free-field* conditions and the latter of the *far-field* array in the ESB box (§ 4.1).

Comparison in first shown in terms of contours of the displacement norm $|u|$ relative to the base (Fig. 13) and excess pore water pressure (Fig. 14) accumulated till the end of the dynamic calculation phase (t = 15 s). Results computed with the *small* and *large* models are quite similar throughout the numerical domain: this implies that, on average, boundary effects did not alter noticeably the OWT response. However, two main exceptions are observed: as for Figure 13, slightly higher displacements are computed in the *small* FE model (i.e. in the ESB container) close to the boundaries, as one could have expected, this causing a bit higher raft rotation (black contour on the left edge of the raft foundation); and higher excess pore water pressures (Fig. 14) in the *small* model at the loose-dense sand interface, again close to the boundaries, where peak values of Δp are caused by higher shear strains γ due to impedance ratio between the two sand layers.

Similarly, a good agreement between the *small* and *large* FE model results is noticed in Figure 15, where the comparison is made in terms of horizontal acceleration (a) and excess pore water pressure time histories (b) measured and computed into the loose sand layer. Prediction made through the *large* FE model strongly resembles that obtained with the *small* one, although some modest deviation is obtained in terms of values of Δp at the end of the dynamic calculation phase, with the excess pore water pressure computed through the *large* model approaching the experimental one. However, the good agreement between the *small* and *large* model is not obtained when looking at the absolute settlement experienced by the clay at the *far-field* ground surface, w_{ff} (Fig. 16a). Indeed, swelling of about 10 cm results from both the *small* model and the centrifuge, whereas a settlement is obtained in the *large* model (≈ 3 cm). This difference can be attributed to boundary effects on the *free-field* array that caused higher excess pore pressures during the seismic event, as already discussed previously. The observed discrepancy in the *far-field* settlement therefore implies a different evaluation of the OWT seismic performance, as shown in Figure 16d in terms of the relative average settlement $w - w_{ff}$. In fact, permanent values attain values equal to 0.42 and 0.31 m for the *small* and *large* FE model, respectively, this providing a difference of about 27 %. Here it is worth mentioning that this difference is to be mainly ascribed to the already-discussed deviation in the *free-field* settlement, as confirmed by the prediction of the absolute raft settlement shown in Figure 16 b-c, where the assessment performed with the *small* and *large* FE models gave almost superimposed results. Moreover, the raft rigid rotation (Fig. 16d) computed with the *large* model are turned out to be lower than that from the *small* one, with a final value of about 0.12°

compared to 0.20° and hence a non-negligible difference of about 38 %. Although this difference shows that boundary effects affected the seismic performance of the OWT at hand, it is worth mentioning that the absolute deviation of rotation is equivalent to 0.08° only: such a small difference is usually much less than the accuracy required in the framework of Performance-Based Design (PBD), which is emerging as a new paradigm in the assessment of seismic performance of foundations under strong seismic loading.

Finally, the OWT seismic performance can be assessed in terms of an additional index, such as the peak horizontal acceleration acting on top, $a_x^{top}{}_{max}$, which can be retrieved from both the time history and the elastic pseudo-acceleration spectra plotted in Figure 17, and is equal to 0.11 and 0.10g with the *small* and *large* model, respectively (≈ 10 % difference). Results show that time history and long-period response of the wind turbine predicted using the *small* and *large* model are nearly identical, whereas the short-period (i.e. high-frequency) behaviour is slightly influenced by the presence of the boundaries. Indeed, the horizontal acceleration detected at the second peak of the spectrum (T = 0.44 s) is moderately amplified with the *small* model, turning out to be equal to 0.61g, whereas the value 0.51g is computed with the *large* model: this important outcome confirms that contribution of the boundaries resulted in an increase of high-frequency components of inertial forces transmitted to the wind turbine.

The values attained by the seismic performance indexes discussed above are summarised in Table 5, where the ratio *large*/*small* is also listed to give an insight of the influence of boundaries on the seismic performance. This ratio is plotted in Figure 18 as well, for the raft foundation (a) and the wind turbine (b): in this figure it is pointed out that the best prediction is obtained for the wind turbine, while the performance of the foundation results to be affected by the presence of the boundaries: this is particularly true for the rigid rotation of the raft. However, values from the *small* model (and then from the ESB container adopted in the centrifuge) are always higher than those from the rigorous *large* model, thus providing an estimate of the seismic performance from the safe side.

6. Concluding remarks

In this paper, the influence of flexible boundaries of the Equivalent Shear Beam container has been assessed, so as to gain more awareness of results coming from centrifuge testing, which has established as a powerful tool in the field of physical modelling in geotechnics.

To this end, an example case of an Onshore Wind Turbine resting on liquefiable soils and subjected to a strong sine wave, capable of triggering liquefaction into the loose sand layer, has been first tested in the Turner beam centrifuge available at Schofield Centre, University of Cambridge. Experimental results have been therefore reproduced by a 3D numerical Finite Element model implemented in the OpenSees framework, reproducing the ESB box size and boundary conditions. Mechanical behaviour of foundation soils has been simulated through the advanced SANISAND04 constitutive model, while the bi-phase nature of soils has been reproduced through the *u-p* formulation: hydro-mechanical soil parameters have been calibrated against the *far-field* results obtained in the centrifuge, and this turned out to provide a surprisingly good "blind" prediction of the seismic performance of the structure at hand. Then, boundary effects on the OWT seismic performance have been evaluated by comparing the numerical results computed with the numerical model reproducing the ESB box (*small* model) with those obtained with a much larger and rigorous domain (*large* model), where wall ends do not affect neither the results at the *far-field* array nor the structure behaviour.

The comparison showed that the array usually taken as *far-field* in the ESB container is not affected by the presence of the vertical boundaries when looking at horizontal accelerations and excess pore water pressures, whereas settlement at ground surface shows non-negligible deviation caused by the proximity of wall ends. Nevertheless, displacements of the raft foundation and inertial forces transmitted to the superstructure are very slightly influenced by the presence of the wall ends, except for some high-frequency components which can be attributed to P-waves generated at the soil-boundary contacts. Boundary effects have been quantified in terms of the difference between the values attained by some selected seismic performance indexes evaluated with the *small* and *large* model: the peak values of the relative settlement between the raft foundation and the *far-field* ground surface, of the rigid rotation of the foundation, and of the horizontal acceleration transmitted to the top of the OWT. The maximum difference, equal to about 38 %, has been obtained for the raft rotation, this corresponding to the modest deviation of 0.08° which can be deemed negligible in the framework of the Performance-Based Design. As for the remaining seismic indexes adopted in this study, the average difference is slightly beyond 20 %, which confirms the reliability of dynamic centrifuge testing when assessing the seismic performance of structures even on liquefiable soils.

Novelty of this paper relies on the evaluation of boundary effects in the presence of soft and saturated sandy soils subjected to strong seismic events, for which the stiffness ratio between wall ends and foundation soils may strongly increase due to liquefaction. Results obtained in this study may be taken as a reference when interpreting results coming from dynamic centrifuge tests, as they provide a quantitative measurement of boundary effects on the seismic performance of slender structures on liquefiable soils. Moreover, calibration of Hostun sand parameters for the SANISAND04 model against dynamic centrifuge tests constitutes a novel and useful outcome as well, as Hostun HN31 sand is widely used for research purposes.

Clearly, conclusions drawn in this papers should be confirmed by further numerical analyses, where different seismic inputs are applied. This has been already done and not shown here for the sake of brevity, and will be discussed in a coming publication.

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Table captions

Table 1. Values assumed for the isotropic linear elastic media adopted for the raft and the tower

Table 2. Values of PIMY parameters assumed for the clay layer

Table 3. Values of PDMY parameters assumed for the gravel layer

Table 4. Calibrated values of SANISAND parameters adopted for the sand layers

Table 5. Comparison of seismic performance indexes from the *small* and *large* models

Figure captions

Figure 1. Schematic diagram of the problem (dimensions in meters)

Figure 2. Model layout for the centrifuge test (a) and *far-field* small-strain shear modulus profile from Air Hammer Test (AHT) and calibrated SANISAND04 model (b) (prototype units at 80*g*)

Figure 3. Model mounted on centrifuge

Figure 4. *Small* 3D Finite Element domain (dimensions at prototype scale) (a), detail of the tower and raft modelling (b)

Figure 5. Comparison of total horizontal acceleration and excess pore water pressure time histories computed at the far-field alignment into the loose sand layer (depth y = -10 m) (a-b) and into the dense sand layer (y = -30.2 m) (c-d)

Figure 6. Comparison of settlements measured at the foundation edges (a-b), average settlement relative to the free field (c), and rotation time histories (d), obtained with the *small* FE model and in the centrifuge

Figure 7. Comparison of excess pore water pressure time histories obtained beneath the left (a) and the right (b) edge of the raft foundation

Figure 8. Total horizontal acceleration time histories and elastic acceleration spectra obtained at the lumped mass level (a-b) and of the input motion (c-d)

Figure 9. *Large* 3D Finite Element domain (dimensions at prototype scale)

Figure 10. Output schematics adopted in the numerical analyses performed with the *large* numerical domain (dimensions at prototype scale)

Figure 11. Total horizontal acceleration (a, b, d) and excess pore water pressure (c, e) time histories computed along the *far-field* and *free-field* alignments with the *large FE* model

Figure 12. Profiles of peak acceleration ratio (a) and excess pore water pressure (b) computed along the *far-field* and *free-field* alignments with the *large FE* model

Figure 13. Contours of displacement relative to the base computed at the end of the dynamic calculation phase (t = 15 s) close to the raft foundation: *small* (a) and *large* (b) FE models

Figure 14. Contours of excess pore water pressure computed at the end of the dynamic calculation phase (t = 15 s) close to the raft foundation: *small* (a) and *large* (b) FE models

Figure 15. Total horizontal acceleration (a) and excess pore water pressure (b) time histories computed along the *far-field* alignment with the *small* and *large* FE models, together with those recorded in the centrifuge

Figure 16. Time histories of settlements at the *far-field* ground surface (a) and at both foundation sides (b-c), of the average settlement relative to the free field (d), and of rotation time histories (e) obtained with the *small* and *large* FE models, together with those recorded in the centrifuge

Figure 17. Total horizontal acceleration time histories (a) and elastic acceleration spectra (b) obtained at the lumped mass level with the *small* and *large* FE model

Figure 18. Ratio of seismic performance indexes computed for the raft foundation (a) and the Wind Turbine (b) with the *small* and *large* FE model