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An innovative approach in assessing the residual tilt for the design of an offshore wind farm in Taiwan

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ABSTRACT

With the offshore wind development at its peak across Europe, other regions turn to this sustainable source of energy. In recent times, South East Asia and more specifically Taiwan has been at the core of this trend with a new set of challenges in comparison to North European projects regarding seismic design.

This paper concerns the geotechnical design and foundation performance of an offshore wind farm in Taiwan supported by monopile foundation. The existence of liquefiable soils in this area demands a complex framework to provide useful information on the potential risks. The present paper describes the methodology implemented to investigate the threshold value of residual tilt following a seismic event, as set out by the turbine supplier. A three-dimensional soil-structure interaction model was created using the Finite Element Software Plaxis3D together with innovative user-defined models, to define the impact of liquefaction and soil damping for the ground model under consideration. Aerodynamic and hydrodynamic loads were accounted for in the model as well as the impact of a possible turbine shutdown during the earthquake event.

Keywords: residual tilt, monopile, seismic, liquefaction, offshore wind, wind turbines, shutdown.

1 DESIGN OF A MONOPILE FOUNDATION

The design of wind turbines in Europe has developed over the last decades and specific standards have been developed to dictate most of the aspects of the design, such standard are IEC 61400 series and DNVGL ST-0126 (2018). For monopile foundation design, as stated by Achmus et al. (2017), the usual checks include primarily the Ultimate Limit State (ULS) lateral capacity and the Serviceability Limit State (SLS) permanent tilt of the monopile. The result should comply with a tilt threshold given by the turbine manufacturer. The lateral behaviour of the pile is modelled in usual engineering practice by sets of springs representing the soil reaction on the pile with respect to its lateral displacement, the so-called p-y curves concept by the Winkler model. For seismic design of wind turbines, the IEC standards, require an analysis based on spectra, frequency domain or time domain for a 475-year return period. Earthquake loads are determined on the structure following a decrease in soil strength and stiffness via the p-y curves, the load transfer therefore accounts for the liquefaction of the soil around the structure.

The integrity of the structure is verified through the ULS verification that considers factored loads, factored soil strength parameters and seismic effects on the underlying soils. Equilibrium must be satisfied to fulfill this ULS criterion. The permanent tilt is however deemed more complex to evaluate as the triggering of liquefaction gives uncertainties on the pile response and eventually could lead to a significant tilt at pile head.

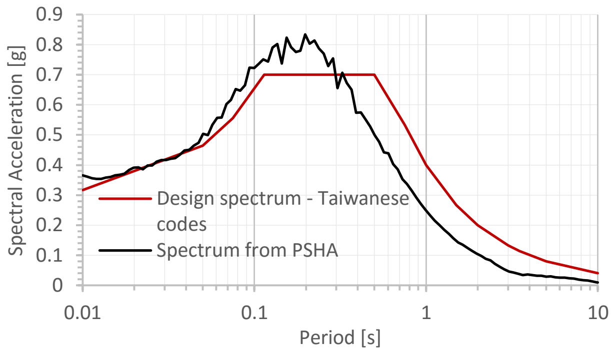

Fig. 1. Difference between design spectra from Taiwanese code (MOI, 2011) and PSHA.

The present study concerns the assessment of permanent tilt of a monopile in time-domain with Plaxis3D during the design process of an offshore wind farm, following the work done at an earlier design stage of the project, Li Destri Nicosia (2016). Standard Penetration Tests and lab tests were used to define the liquefaction potential in accordance with the methodologies of both Youd and Idriss (2001) and the Japanese Road Association (2002). The FE model does not incorporate direct input from these methodologies but the UBCSAND3D used for this analysis was developed to be in line with theoretical cyclic resistance ratios.

2 SITE CONDITIONS

2.1 Loads

Environmental loads. IEC 61400-1 stipulates that frequently occurring loads must be accounted in combination with the seismic loads. The thrust on the Wind Turbine (WTG) induced by the wind is applied as a quasi-static load at hub height based on the reference data from the NREL 5 MW wind turbine (Jonkman, 2009), the methodology is detailed in Li Destri Nicosia (2016). A factor, based on the area covered by the blades is used to scale up the value from the reference WTG. The meteorological data in the region gives an expected (average) wind speed at hub height equal to ~10m/s. This results in a thrust force of ~960kN.

The wave load is derived for a severe 1-year sea state. As it is estimated from a load time history, the overturning moment at mudline is computed and the maximum peak is chosen for implementation in the model. These loads represent maxima and are not representative of an average over the structure lifetime. Consequently, an arbitrary and elementary assumption is to multiply the wind and wave load component by a factor of 0.5 and assume this resulting load as constant along the dynamic simulation.

Seismic input. The time history used for the analyses is developed by spectral matching and represents the most critical horizontal motion component, with the vertical ground motion being neglected. Fig. 2 shows the acceleration time history for outcropping bedrock with $V_s = 760$ m/s. This time series corresponds to the spectrum in Fig. 1. The peak ground acceleration at the site is 0.35 g. The motion input is given through a stress field calculated through the displayed acceleration time history.

Shutdown. As stated by IEC (2005), a possible shutdown of the Wind Turbine Generator (WTG) must be taken into consideration. In this specific analysis, it is expected that the shutdown procedure is triggered at t = 10 seconds, i.e. a brutal stoppage of the rotor shaft within the hub of the WTG leading to a decrease of the wind thrust on the turbine. The wind thrust varies throughout the dynamic stage of the analysis. At t = 10 s, this thrust factor is deactivated, it is conservatively assumed that no wind thrust then acts on the structure to reduce to a maximum the damping of the system. One could however clearly imagine that wind is still present and pushes the turbine to a lesser extent.

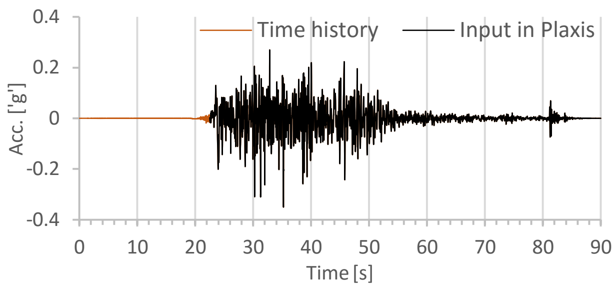

Fig. 2. Acceleration time history with motion along North-South direction.

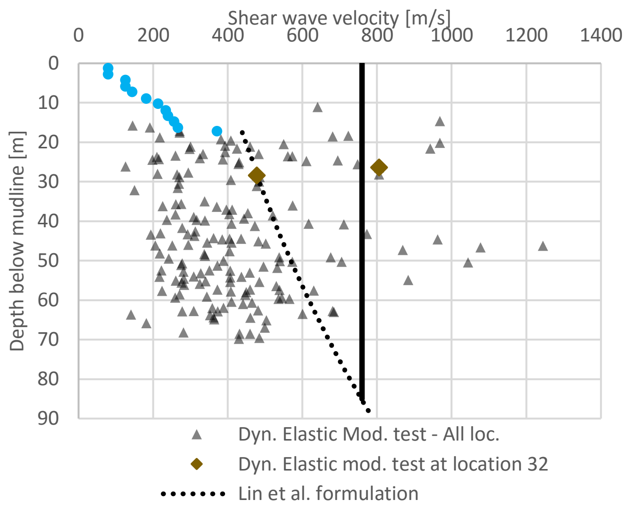

Fig. 3. Stiffness profile at WTG position.

Table 1. Soil profile - stiffness and strength parameters.

Depth [m]	Layer	Cu [kPa]	φ' [°]	Vs [m/s]	PI [-]
0.00-3.35	Silt	5	25	80	-
3.35-4.50	Silty clay	12	21	103	18
3.35-9.70	Silty clay	12	21	144	17
9.70-15.15	Silty sand	-	28	234	-
15.15-17.50	Silty sand	-	29	267	-
17.50-25.00	Gravel	0.1	30	453	-
25.00-30.60	Sandstone	49	32	478	-
30.60-39.20	Gravel	-	32	504	-
39.20-50.00	Gravel	-	33	545	-
50.00-60.00	Gravel	-	33	593	-
60.00-80.00	Gravel	-	33	685	-
80.00-85.00	Gravel	-	-	760	-

2.2 Soil profile

The soil profile at the site is composed of a rather thick layer of sediments, approximately 17 metres, followed by weak weathered rock and gravel layers. The sediment layer is stratified with layers of silty sand, silty clay and sand. More information on the assessed position is displayed in Fig. 3. The main stiffness and strength parameters used for the assessment are listed in Table 1.

3 NUMERICAL MODEL

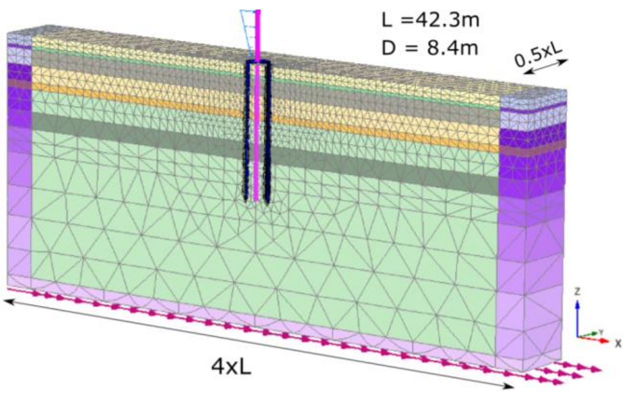

Fig. 4. Mesh size based on pile outer diameter D and embedded length L.

3.1 Model geometry

Regarding the model geometry the focus is put on the model and element sizes so appropriate deformations is calculated. With regards to the model size, boundaries are sought to not impact the behaviour of the monopile. The model used for the analyses is shown in Fig. 4. Verifications of the impact of boundaries has been performed.

Element size. The mesh is composed of 31,467 elements and 48,374 nodes. Calculations of the required minimum element size are carried out accounting for the element type in the model (15-noded tetrahedral elements for soils), the max frequency at which the energy present in the acceleration spectra is negligibly impacting the structure (10 Hz), and the shear wave length (λ_s) within each layer. It is considered that an element size of one eighth of the shear wave length is sufficient to properly model the shear wave (Laera and Brinkgreve, 2015).

Boundary conditions. Static boundaries are fixed in the normal direction with respect to the plane in question while for the dynamic phase, free-field boundaries are assigned to the two planes normal to the seismic motion. A fully reflective boundary for the plane of symmetry and a viscous boundary is given to the rear plane (parallel to seismic motion). The bottom boundary is modelled as a compliant base, i.e. a fully absorbing boundary at a depth of 85 m to reach the needed stiffness, see Fig. 3.

3.2 Soil constitutive relations

The constitutive relations in the FE model are not described extensively in this paper, enough literature being already part of the public domain, the following paragraphs intend to clarify how the different elements are modelled.

Linear Elastic. No considerations of plasticity are accounted for structural elements as this model aim to fulfil one geotechnical design criterion. To prevent numerical convergence issues, the model boundaries are also modeled with a linear elastic model with stiffness parameters defined through the shear wave velocity profile, i.e. the input is a dynamic stiffness for each layer, calculated based on the initial stress state in the middle of the respective layers.

Linear-elastic perfectly-plastic model. Interfaces between soil and structure are modelled with Mohr-Coulomb as failure criterion. Strength and stiffness of these interface elements are given a value of 67% of their respective soil elements.

UBCSAND3D. This model is used to model the sand and silty sand layers. It reproduces the NCEER liquefaction triggering behaviour. The model parameters are derived through a simplified procedure with SPT 'N' values. Parameters implementation is described in Nicosia (2016) and Laera and Brinkgreve (2015). An important state parameter of this model is the pore pressure ratio, r_u , main indicator of the level of liquefaction, a value of 1.0 meaning fully liquefied soil.

$$r_u = 1 - \left(\frac{\sigma_v'}{\sigma_{v initial}'}\right) \quad (1)$$

Generalised Hardening Soil (GHS). This model is assigned to clay, weak rock and gravel layers. The idea behind using this constitutive model instead of Plaxis' HS small model is the deactivation of shear and cap hardening, limiting the damping in the model as well as the computation time. In brief, the model uses Mohr-coulomb or Tresca failure criteria depending on whether the layer respectively is gravelly or cohesive, furthermore it has the particularity of being able to capture strain-dependency of stiffness (hysteresis) and stress dependency of stiffness. To accurately model these layers, proper degradation curves are matched to Lin et al. (2000) for gravels and Ishibashi and Zhang (1993) for clays.

3.3 Validation of 1st eigenfrequency of pile-soil system.

The calculation of the natural frequency in the finite element model is done by a pushover analysis, a small load is prescribed at the top of the tower and subsequently released. This validation is done to ensure a proper tower response under the seismic loads. The spectral displacement of the top node of the structure is then extracted, results are shown in **Error! Reference source not found.**.

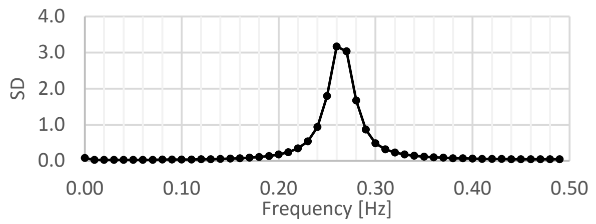

Fig. 5. Spectral displacement at top of tower. The natural frequency of the system is evaluated at 0.26 Hz.

The structural model with p-y curves calculated a frequency of 0.246 Hz against a value of 0.26 Hz, the 3D FE model yielding a stiffer response as expected. This difference is deemed satisfactory for the present study. Besides, detailed comparisons of 1D and 3D analyses are unnecessary due to the different methodologies and different input in each calculation.

4 RESULTS AND CONCLUSIONS

4.1 Liquefaction

The maximum pore pressure ratio during the seismic loading is plotted in Fig. 6. The uppermost layer at seabed shows a rather high liquefaction 70-100% while the second sandy layer, located deeper and with higher relative density, has a pore pressure ratio of about 30-40%. Liquefaction results are in line with what was expected, and the residual tilt is therefore processed.

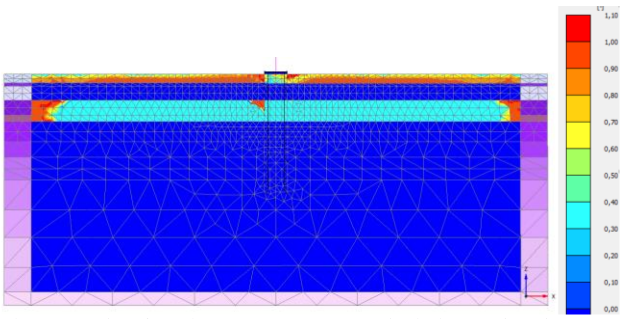

Fig. 6. Results of maximum pore pressure ratio during earthquake.

4.2 Residual tilt

The residual tilt is calculated at the end of the dynamic time-domain simulation, whereby all the loads in the model are released and the residual displacement of the pile is determined. Table 2 **Error! Reference source not found.**and Fig. 7 give more details on the results of the analyses. Three phases may be observed; first a period from t=0 s to t=10 s, where the same response is observed. Subsequently, the shutdown is triggered, hence the wind thrust diminishes to zero, resulting in higher amplitude for the shutdown case scenario. The last phases consists of a period from t=30 s, where the tilt is stabilised for both models. The shutdown does not severely impact the results given by the fully operational WTG model, but this is believed to be highly dependent on the loading direction, for instance a transverse earthquake loading may have much more impact on the residual tilt due to less damping along this direction. Furthermore, the application of quasi-static loads together with dynamic loads in the same plane, is thought to sufficiently characterise the effect of the loading on the structure, which combined with the reversal release of wind thrust provides an envelope of system response.

This paper delivers an overview of the additional seismic component when dealing with monopile foundation subjected to earthquakes. With an allowable tilt angle of 0.25 degrees at the site, 20% are induced by seismic loads. It is advised to consider with care the additional contribution of seismic loads on the monopile residual tilt. High residual tilt from seismic loads may impact the project if not considered at a proper time during the design process.

Table 2. Results of dynamic phase

	Operational	Shutdown
Residual tilt at pile head	0.05 degrees	0.025 degrees
Ratio to allowable tilt	20%	10%

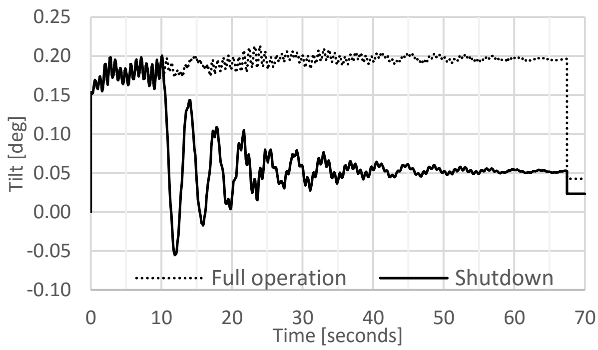

Fig. 7. Residual tilt at pile head along analyses.

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