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# Seismic Response of Large Diameter Monopiles for Offshore Wind Turbines in Liquefiable Soils

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**ABSTRACT:** Offshore wind energy is rapidly expanding to earthquake prone regions, with design to seismic loading becoming a primary concern. This is especially relevant to monopiles, which are the preferred option in shallow to medium water depths at present. This paper presents centrifuge tests of a monopile foundation and turbine tower subjected to low and high frequency earthquakes. The experiment reproduces calm air conditions, meaning that no lateral wind load was considered. Low frequency earthquakes were found to cause excitation of the topmost part of the superstructure, whereas high frequency earthquakes generated strong resonance effects at the base. Further data analysis suggests that high frequency earthquake shaking can trigger liquefaction of the soil surrounding the monopile, which may in turn lead to differential settlement of the turbine. This is translated into significant rotation of the structure that can potentially exceed the allowable SLS rotation recommended by the technical guidelines.

## 1 Introduction

Offshore wind energy has crossed the borders of Europe to other regions of the world such as Southeast Asia and the West coast of the U.S. These areas are expected to accommodate the highest growth in the offshore wind sector over the coming years, though they involve a significant risk due to high seismicity. Hence, earthquake loading is increasingly becoming a primary concern for the design and construction of Offshore Wind Turbines (OWTs). This is especially relevant to foundations, which are meant to support the tower and rotor – nacelle assembly during earthquakes. The monopile is the preferred foundation type for OWTs representing 81.9% of the substructures in Europe (Walsh, 2019). This trend will probably prevail in future developments located in earthquake prone areas with appropriate water depth and ground conditions, given the expertise gained in monopile installation across Europe.

Earthquakes generate specific effects on OWTs based on their intrinsic characteristics. This research distinguishes between two types of OWT structural responses depending on the frequency of shaking. Natural frequencies ( $f_n$ ) of current OWTs range between 0.17Hz and 0.35Hz approximately (Arany et al., 2016). Earthquake frequencies that fall within this range, are likely to excite the OWT superstructure significantly. These are called low frequency earthquakes.

Earthquake shaking may as well lead to the development of excess pore pressure ( $U_{excess}$ ) in saturated sand. The effective stress of sandy soils is progressively reduced as  $U_{excess}$  develops. Sufficient accumulation of  $U_{excess}$  may cause the complete loss of effective stress, meaning that the soil has fully liquefied. However, this phenomenon is only triggered by high frequency earthquakes, which contain frequencies ranging between 1Hz and 5Hz (Madabhushi and Haigh, 2012). These are relatively far from typical OWT natural frequencies ( $f_n$ ). Hence, high frequency earthquakes are not expected to cause strong dynamic effects on the OWT superstructure. Liquefaction though, would still represent a major hazard arising from these earthquakes.

In traditional long-slender piles, the complete loss of effective stress due to full liquefaction translates into a substantial reduction of shaft friction and end bearing. Thus, the piled foundation may no longer be capable of supporting its self-weight and that of the superstructure. As a result, the entire structure keeps settling into the soil until enough shaft friction and end bearing are recovered to withstand the self-weight again (Madabhushi et al., 2010). A similar behaviour could potentially be attributed to monopiles. However, their reduced aspect ratios and larg-

er diameters could lead to a different failure mechanism. This being the case, the subsequent settlement could widely differ from that of long-slender piles subjected to the same vertical load. In addition, the serviceability of OWTs could be severely compromised by differential settlements and the resulting rotation. Hence, current design standards restrict the allowable tilt angle to  $0.5^\circ$  (DNV-GL, 2021 and DNV-RP-0585, 2021).

This paper investigates the effects of both, low and high frequency earthquakes on OWTs supported by monopiles in liquefiable soils. Earthquake interaction with the soil and the structure has been reproduced using centrifuge modelling. The study sought to represent calm air conditions, and therefore, the effects of lateral load are not addressed. Nevertheless, future work is in progress to study the combined influence of lateral wind load and seismic loading on monopiles. The research by Esfeh and Kaynia (2020), is one of the few currently published studies that assesses the impact of liquefaction on offshore wind monopiles under no wind load. Esfeh and Kaynia (2020) modelled a 25 m long and 6 m diameter monopile utilising a 3D Finite Difference code. Their findings suggest limited vertical displacement and rotation such that the operational limit states of the OWT would not be compromised. It is noted, however, that the monopile was modelled wished in place and therefore, soil stresses generated during installation were not considered. The dynamic response was also excluded from the analysis since the superstructure was modelled as a lumped mass on top of the monopile. Furthermore, numerical results are in general, subjected to the behaviour described by the constitutive model. Although these constitute a very reasonable approximation to reality, validation against centrifuge data is required.

Seong et al., (2022) conducted one of the first published studies on centrifuge testing of monopile supported OWTs subjected to seismic loading. The experiments determined the first excitation mode or natural frequency ( $f_n$ ) of the system. Following this, the turbines were subjected to the first, second and third mode frequency earthquakes. The dynamic response of the superstructure was measured combining dry and saturated soil profiles. Strong dynamic responses were detected, especially for the first mode earthquake. The latter also excited the second and third modes of the OWT, although only to a minor extent. It is noted that the study involved small diameter piles of aspect ratio relevant to the offshore wind industry, but of restricted overall dimensions. Additionally,  $f_n$  of the structure was 0.45Hz, far from typical OWT natural frequency values.

## 2 Test Design and Methodology

Realistic soil stresses and structural response have been achieved using centrifuge modelling at 70g. The soil sample included a 7m loose sand layer with relative density ( $Dr$ )  $\approx$  40% overlying a 14m dense sand layer with  $Dr \approx$  90%. The sand was pluviated inside the Equivalent Shear Beam (ESB) model container using the automatic sand pourer at the University of Cambridge (Madabhushi et al., 2006). The ESB container is designed to minimize boundary effects and to simulate the progressive increase in soil stiffness with depth (Brennan et al., 2006). Both sand layers were subsequently saturated with a viscous methylcellulose solution by means of the Cam-Sat system (Stringer and Madabhushi, 2009). In this paper all dimensions are expressed in prototype scale. The centrifuge scaling laws are described in Madabhushi (2014). The monopile diameter was 7.11m, whereas the embedded length was 17.5m, resulting in an aspect ratio of 2.5. Driving of the monopile into the soil sample was conducted at 1g using a hydraulic press. Following this, the superstructure was assembled on top of the monopile (Figure 1).

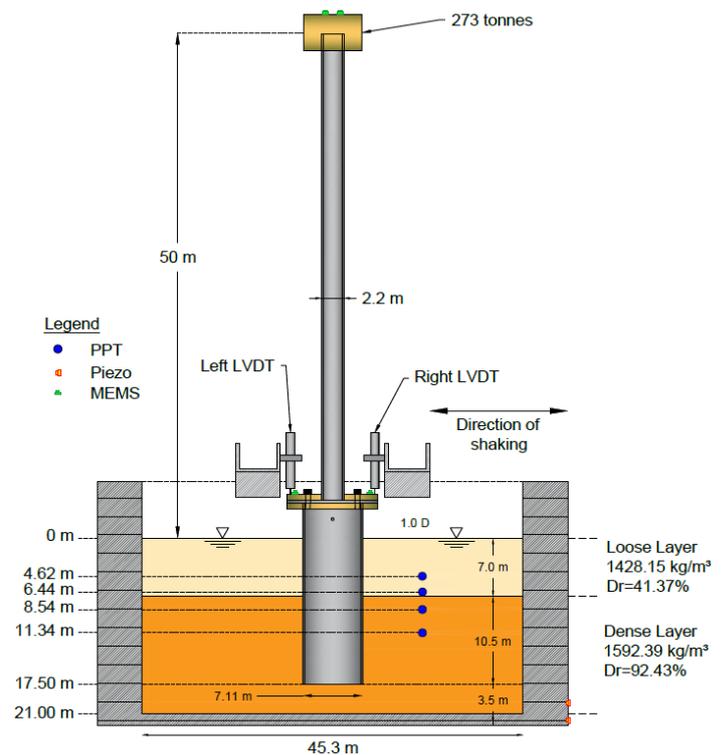


Figure 1. Cross section of the experimental setup (dimensions shown at prototype scale at 70g)

The OWT tower model was designed so that ( $f_n$ ) at prototype scale was 0.2Hz. This is within the range of realistic OWT towers (Arany et al., 2016). The monopile foundation and superstructure were tested against one low frequency (EQ1) and two high frequency earthquakes, EQ2 and EQ3. Acceler-

ation and frequency of EQ2 and EQ3 were specifically selected to generate substantial amounts of  $U_{excess}$ . These were 10 cycles sinusoidal waves with a prototype frequency of 1Hz and a Peak Ground Acceleration (PGA) of 0.11g and 0.21g for EQ2 and EQ3 respectively (Table 1). The earthquake motions are transferred to the model utilising a hydraulic shaker (Madabhushi et al., 2015) which is mounted on the beam centrifuge.

Table 1. Earthquake motions selected for the experiment

| Earthquake No.                     | EQ1           | EQ2            | EQ3            |
|------------------------------------|---------------|----------------|----------------|
| Type                               | Low frequency | High frequency | High frequency |
| Peak Ground Acceleration (PGA) [g] | 0.02          | 0.11           | 0.21           |
| Frequency [Hz]                     | 0.2 ( $f_n$ ) | 1.0            | 1.0            |
| No. of cycles                      | 10            | 10             | 10             |

A broad range of sensors were included in the model to record data on the OWT response and soil behaviour during the test.  $U_{excess}$  accumulation with depth was recorded by a vertical array of four PPTs (Pore Pressure Transducers) located at one monopile diameter distance from the right side of the foundation. This setup allowed for comparison between  $U_{excess}$  generated within the loose and dense sand layers. The settlement experienced by the OWT was recorded using LVDTs (Linear Variable Differential Transformers) mounted on top of the circular base of the OWT tower. Two LVDTs, one installed on each side of the tower base provided a measure of the rotation of the monopile and tower (Figure 1). Measurement of the rotation was conducted in the direc-

tion of shaking. In addition, acceleration of the earthquake as it propagated across the superstructure was determined using MEMS (Micro Electro-Mechanical Systems), installed at the top and base of the OWT tower. Finally, two piezo-accelerometers were attached to the base of the model box to record the input acceleration.

### 3 Results

#### 3.1 $U_{excess}$ generation within the soil

The loss of effective stress ( $\sigma'_{v0}$ ) due to generation of seismically induced  $U_{excess}$  in the loose and dense sand layers has been assessed via the excess pore pressure ratio  $r_u$ , defined as:

$$r_u = \frac{U_{excess}}{\sigma'_{v0}} \quad (1)$$

$$\sigma'_{vi} = (1 - r_u) \sigma'_{v0} \quad (2)$$

Each cycle of the earthquake contributes to additional accumulation of  $U_{excess}$  and the consequent reduction in  $\sigma'_{v0}$ . Thus,  $r_u$  ranges from zero, indicating the absence of  $U_{excess}$  to one, where full liquefaction is reached. However, not all earthquakes are sufficiently strong to cause a complete loss of  $\sigma'_{v0}$  within the soil, i.e. full liquefaction. As an example, EQ2 with a PGA of 0.11g produced a 46% reduction in  $\sigma'_{v0}$  at 4.62m depth and a 33% loss at 6.44m depth, both points located within the loose sand layer (Figure 2).  $U_{excess}$  generation also occurred within the dense sand layer, although to a minor extent, with  $r_u$  values of 0.21 and 0.13 at 8.54m and 11.34m depth respectively.

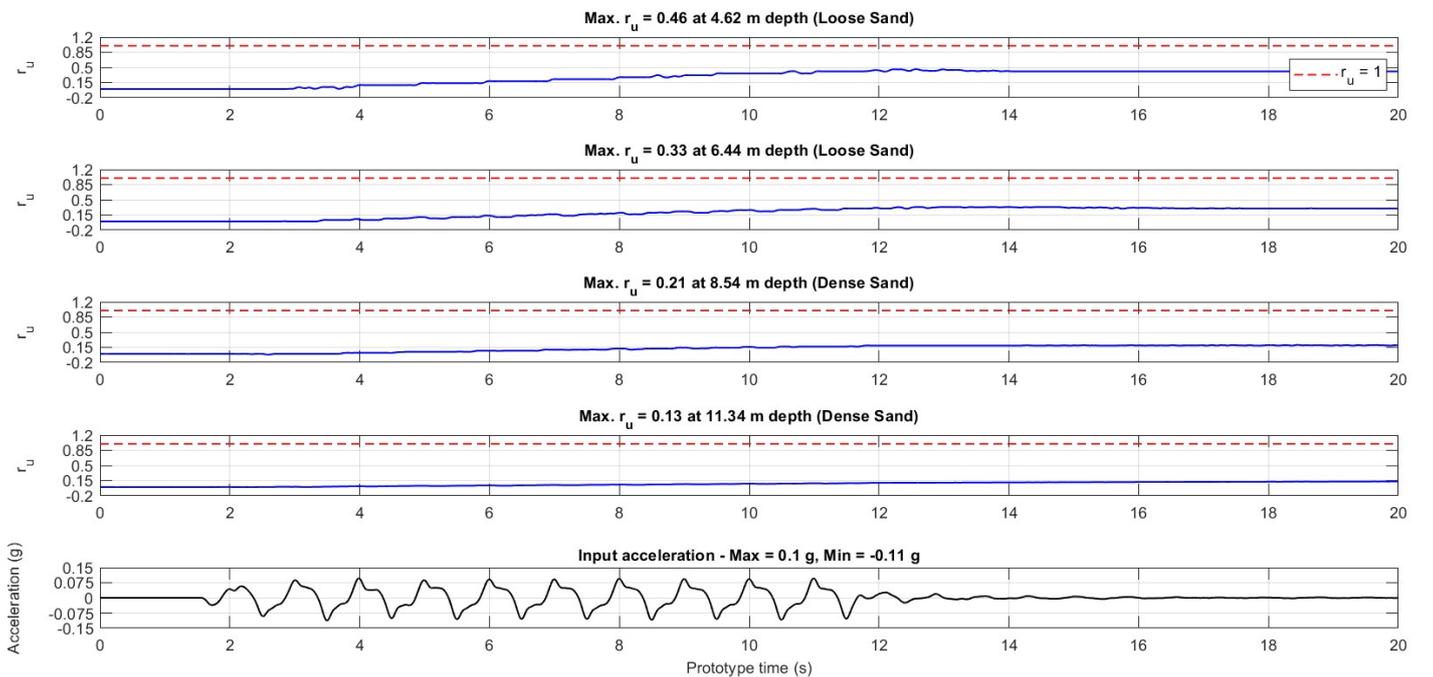


Figure 2:  $r_u$  time history recorded during EQ2

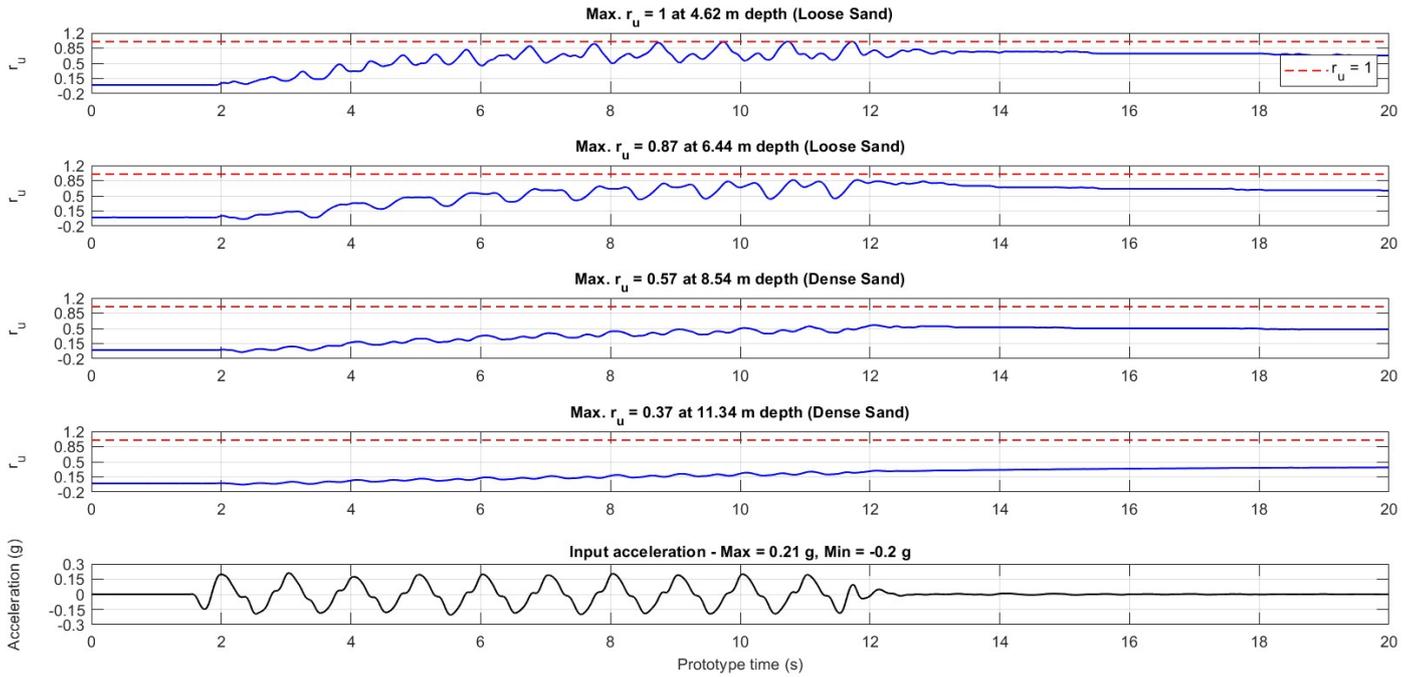


Figure 3.  $r_u$  time history recorded during EQ3

EQ3, with a PGA of 0.21g, was significantly stronger than EQ2 and generated a larger amount of  $U_{excess}$ , especially within the loose sand layer. Liquefaction is evidenced at 4.62m depth with  $r_u$  values up to one and the presence of suction cycles along the  $r_u$  time history. Signs of dilation are also seen at 6.44m, where  $r_u$  registered maximum values of around 0.87. In addition, the residual  $r_u$  value at 4.62m depth reaches 0.8, whereas at 6.44m depth  $r_u$  becomes 0.6. Thus, only 20% and 40% of the initial  $\sigma'_{v0}$  remains available at these depths towards the end of the earthquake. This trend also matches with that identified within the dense sand, where  $r_u$  increased to almost 0.6 at 8.54m depth and to nearly 0.4 at 11.34m depth. The impact of the substantial loss of  $\sigma'_{v0}$  within the loose and dense sand layers on the overall response of the OWT is described in Section 3.2.

### 3.2 Settlement and rotation of the OWT

The substantial loss of  $\sigma'_{v0}$  due to the accumulation of  $U_{excess}$  during earthquake shaking often translates into settlement of the OWT. Generally, the structure does not experience uniform settlement on both of its sides due to inherent eccentricities. This may lead to significant rotation of the monopile and tower. Settlements generated during EQ2 were minor and very similar on the left and right sides (Figure 4). Hence, rotation was very limited ( $<0.2^\circ$ ). In contrast, EQ3 caused strong rocking of the OWT tower, which oscillated  $\pm 0.5^\circ$  during the first six cycles of the earthquake. The residual settlement was 167mm on the left side and 42mm on the right side, leading to a residual rotation of almost  $1^\circ$  in the anticlockwise direction (Figure 5). The results of this study suggest that the OWT that was tested in this research, rotated by more than  $0.5^\circ$ , exceeding the allowable limit recommended in the DNV-GL (2021) and the DNV-RP-0585 (2021) standards.

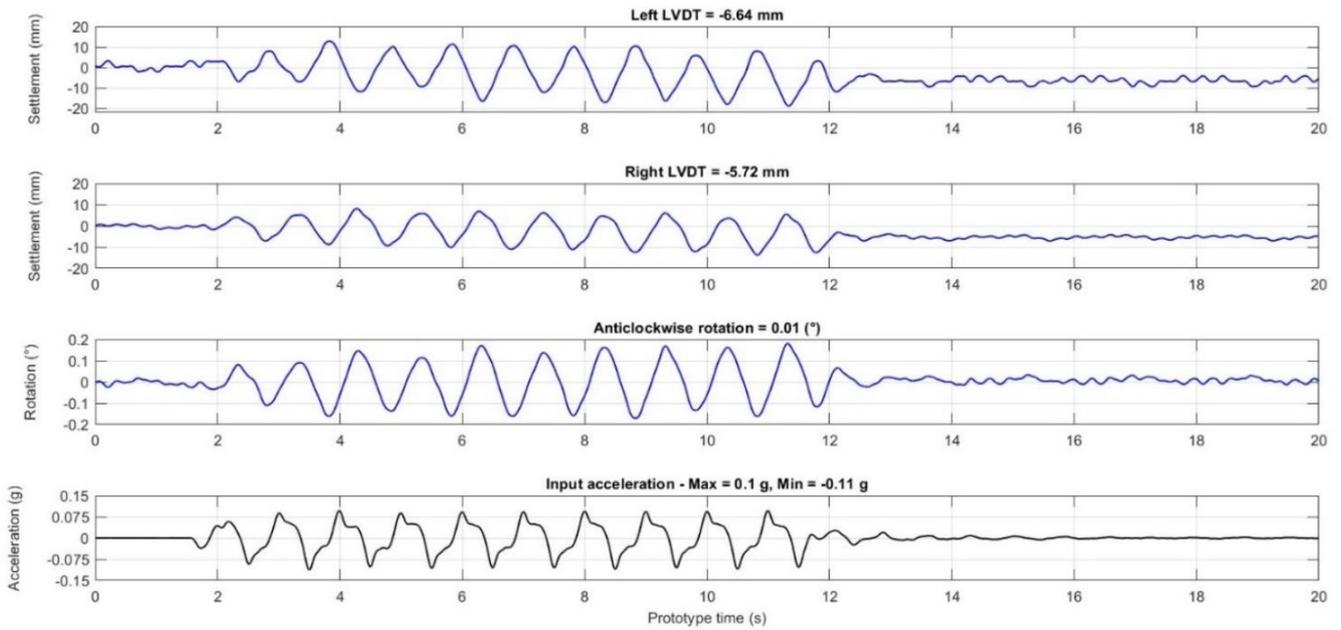


Figure 4. Settlement and rotation experienced by the prototype OWT during EQ2

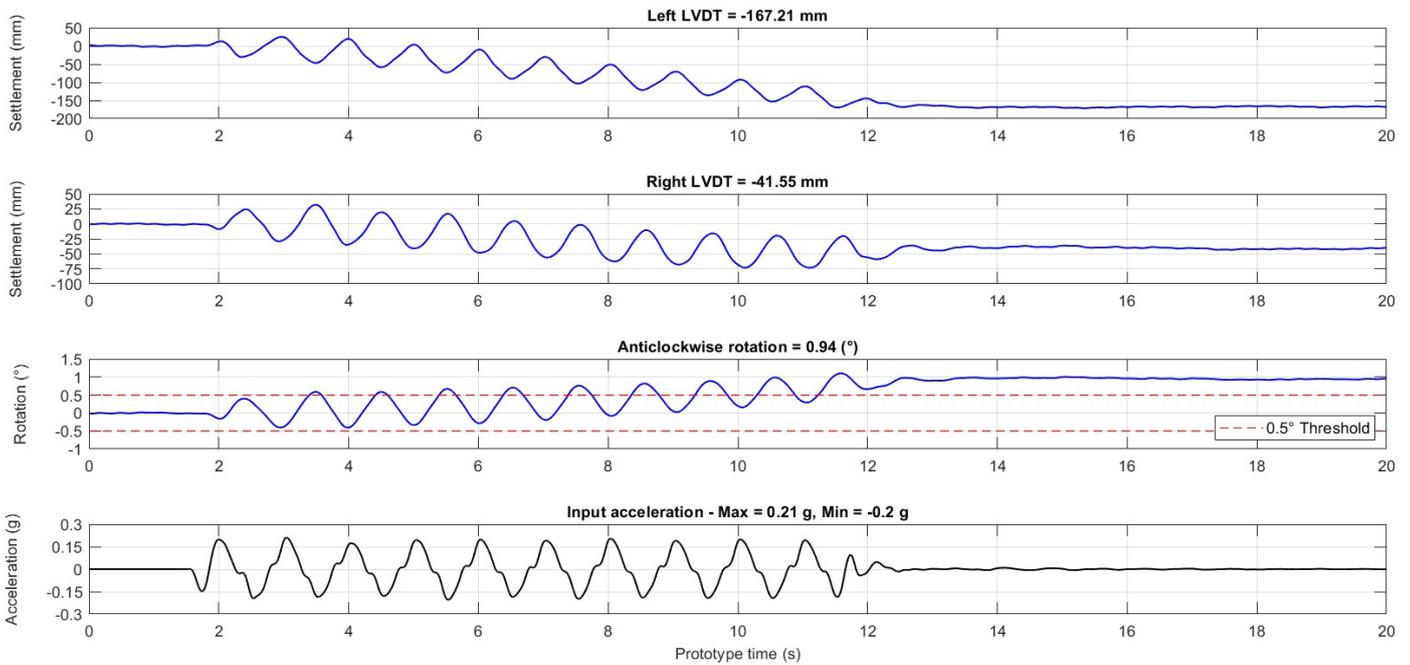


Figure 5. Settlement and rotation experienced by the prototype OWT during EQ3

### 3.3 Dynamic response of the OWT tower

This section compares the dynamic response of OWT towers against low frequency and high frequency earthquakes. An earthquake with PGA of 0.02 and a frequency of 0.2Hz has been chosen as the low frequency motion (EQ1). Moreover, a sinusoidal wave with PGA of 0.21g and 1Hz has been selected to evaluate the high frequency response (EQ3). Accelerations recorded at the top and base of the tower have been used to assess the propagation of the earthquake across the superstructure. Both, top and base signals, together with the input acceler-

ation, have been decomposed into their constituent frequencies using a Fast Fourier Transform (Madabhushi, 2014).

As observed in Figure 6, the low frequency input acceleration (EQ1) is magnified as it propagates across the structure. The acceleration recorded at the top is about two times larger than the input acceleration, whereas at the base this has barely amplified. The first excitation mode is detected at 0.2 Hz, where maximum excitation occurs at the top of the

superstructure. Other secondary frequencies contained in the input signal (0.52Hz, 0.87Hz and 1.23Hz) lead to attenuation at the top of the structure and to slight amplification at the base.

Figure 7 illustrates the dynamic response of the OWT tower when subjected to a high frequency earthquake (EQ3). In this case, the main frequency of the earthquake, 1Hz, is far from the first mode of vibration of the OWT. Hence, the top of the structure does not experience amplification, but attenua-

tion at this frequency. In contrast, the base of the tower is significantly more excited. Additionally, minor excitation of the top and base of the tower is evidenced at 2.9Hz, and at 0.2Hz at the top only. Overall, amplification is significantly larger at the base of the OWT tower rather than at the top. This is evidenced by the acceleration recorded at the base of the tower, which is approximately three times larger than the input acceleration.

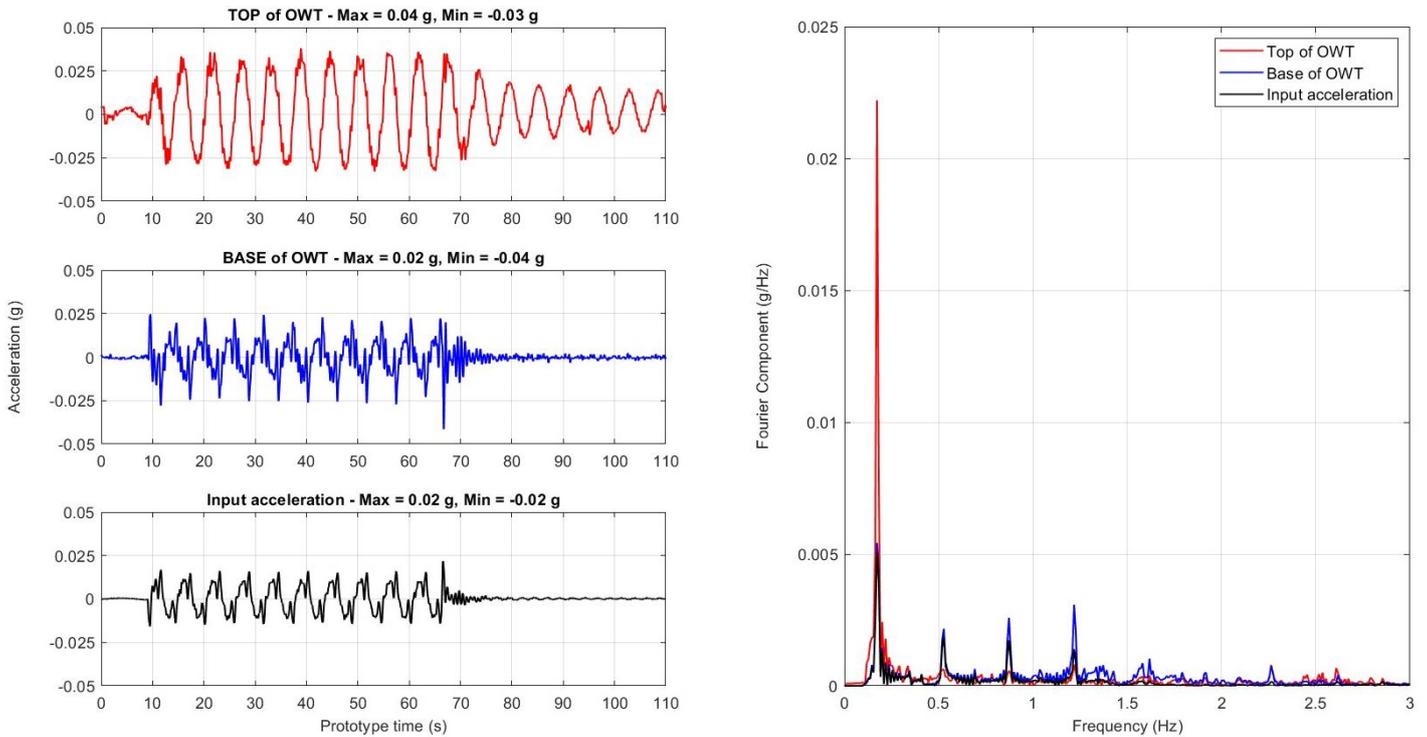


Figure 6. Dynamic response of the OWT tower relative to an input acceleration with frequency equal to  $f_n$  of the OWT (EQ1)

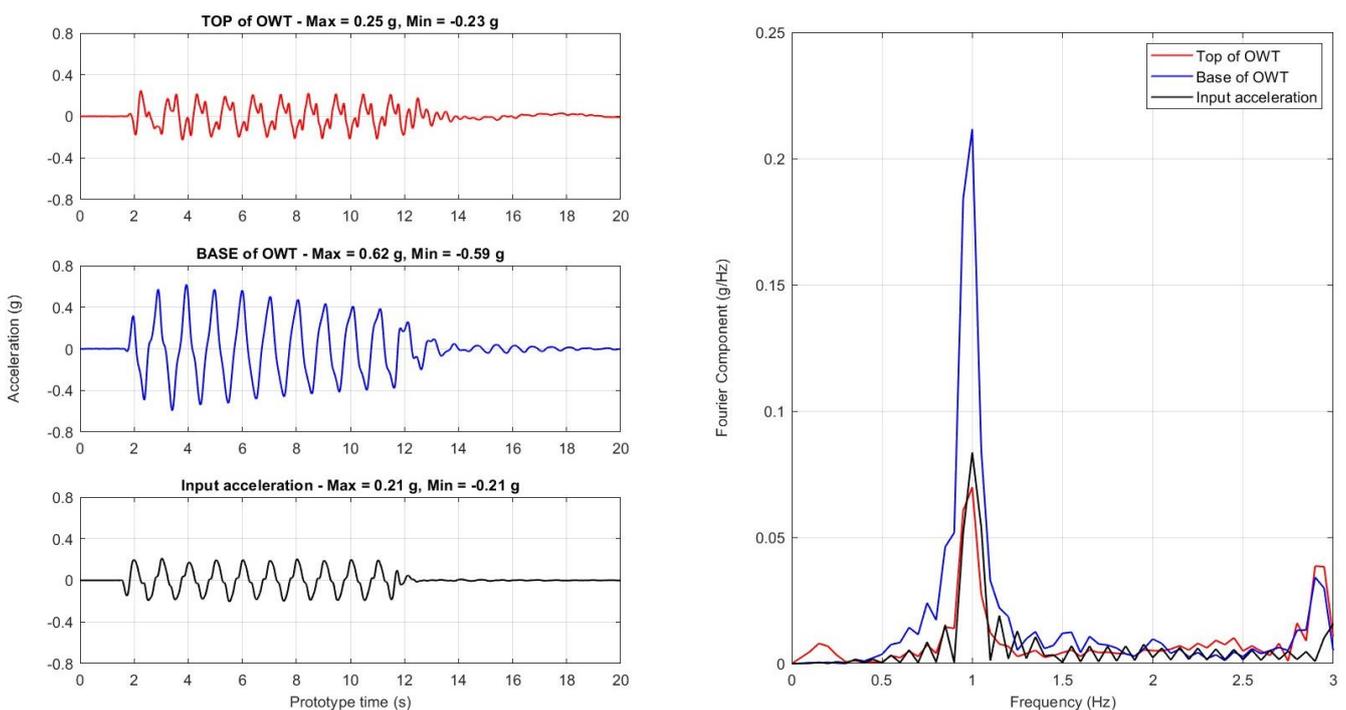


Figure 7. Dynamic response of the OWT tower relative to an input motion of 1Hz and 0.21g (EQ3)

## 4 Discussion

This paper addresses the response of OWTs supported by large diameter monopiles subjected to seismic loading. A full size OWT, including monopile foundation and superstructure, was replicated using centrifuge modelling at 70 g. The soil profile comprised a loose sand layer overlying a dense sand layer, both of which were saturated. The structure was assessed under calm air conditions, meaning that no lateral load was applied. The experiment covered the response of the monopile and superstructure under two well distinguished seismic loading scenarios. Firstly, earthquake shaking with frequencies similar to that of the OWT tower. These generally produce small amounts of  $U_{excess}$ , meaning that  $\sigma'_{v0}$  of the soil remains unaltered. Hence, shear waves can easily propagate through the soil, reach the monopile, and fully interact with the superstructure. EQ1, with a frequency of 0.2Hz, close to the first mode of the OWT, was specifically chosen to reproduce this scenario.

Secondly, high frequency earthquakes including two 1Hz frequency motions with 0.11g and 0.21g PGA were investigated. Both earthquakes were sinusoidal waves intended to generate enough  $U_{excess}$  to liquefy the soil surrounding the foundation. This may distort shear wave propagation, given that shear waves cannot propagate across a liquid medium. But more importantly, the main frequency of the earthquakes selected, is considerably higher than typical OWT natural frequencies ranges. Hence, the superstructure should in theory be, less responsive to high frequency than to low frequency earthquakes.

Data gathered during EQ1 revealed that the acceleration at the top of the OWT was twice the input acceleration, meaning large amplification of the input motion at the top of the superstructure. The main excitation mode was detected at 0.2Hz, the natural frequency of the OWT tower.

EQ2 produced enough  $U_{excess}$  within the soil to reduce  $\sigma'_{v0}$  by up to a half in the loose sand and by 20% or less in the dense sand only (Figure 2). Although the loss of  $\sigma'_{v0}$  in the shallow loose sand layer was remarkable, the resulting settlement was minimal (Figure 4), and so was the rotation.

In EQ3, full liquefaction was nearly achieved at 6.44m and fully reached at 4.62m depth during shaking. Based on this, it is possible that  $\sigma'_{v0}$  was completely lost within the entirety of the loose sand layer. In addition, the accumulation of  $U_{excess}$  was significant within the dense sand layer as well. In fact,  $r_u$  reached values of 0.57 and 0.37 at 8.54m and

11.34m depth respectively. Therefore, only 40% to 60% of the initial  $\sigma'_{v0}$  was available towards the end of the earthquake (Figure 3). The substantial decay of  $\sigma'_{v0}$  in the loose and dense layers translated into a reduction of the bearing capacity of the monopile. As a result, the OWT and foundation settled into the sand until enough shaft friction and end bearing were restored. The settlement towards the end of the earthquake, although not excessive, was around four times larger on the left side than on the right side of the OWT. Such a large variation in settlement, which may be explained by bias created by the first loading cycle of previous earthquakes, turned into a residual rotation of nearly  $1^\circ$ . This value exceeds the allowable threshold recommended by the technical guidelines (DNV-GL, 2021 and DNV-RP-0585, 2021) for the monopile investigated in this paper for shaking amplitudes of up to 0.2g. This research, therefore, suggests that OWTs could experience higher rotations than  $0.5^\circ$  where soil liquefaction is expected even though no lateral load was applied.

In addition to this, resonance effects were identified at the structure despite the earthquake frequency being far from the first excitation mode of the OWT tower. Substantial amplification was detected at the base of the superstructure, where the input acceleration was three times larger. The top and base of the superstructure experienced minor amplification at 2.9Hz. In addition, the topmost part of the tower was also excited at around 0.2Hz, although to a minor extent.

Full liquefaction occurred in the shallow loose sand layer only, and did not have an influence in shear wave propagation to the OWT. Share stress was still fully transferred from the dense sand to the monopile, which in turn propagated to the superstructure.

Based on the above, high frequency earthquakes may not only be capable of generating liquefaction, but also resonance effects at the base of the superstructure. Moreover, low frequency earthquakes with predominant frequencies similar to that of the first mode of the OWT or  $f_n$ , generate significant excitation of the top of the superstructure.

## 5 Conclusions

EQ2, although relatively small, could generate enough  $U_{excess}$  to reduce  $\sigma'_{v0}$  within the loose sand layer up to half of the initial value. However, the dense sand layer was barely affected. Settlement and rotation experienced by the OWT were negligible in this scenario. In contrast, the stronger high frequency earthquake (EQ3) triggered full liquefaction of the loose sand layer. Moreover, a significant amount

of  $U_{excess}$  accumulated within the dense sand layer, whose  $\sigma'_{v0}$  was reduced between 40% and 60%. As a consequence, the monopile experienced substantial differential settlement that translated into rotation of nearly  $1^\circ$ . Thus, earthquake induced rotation in OWTs could exceed  $0.5^\circ$  (DNV-GL, 2021 and DNV-RP-0585, 2021) even if no lateral load due to wind and waves is considered.

The dynamic response of the OWT tower against low and high frequency earthquakes was also addressed in this paper. EQ1 with a predominant frequency similar to  $f_n$  of the OWT, led to excitation of the topmost part of the superstructure. In contrast with this, input motion EQ3 with a main frequency of 1Hz, produced larger amplification of the input acceleration at the base of the OWT rather than at the top.

The results presented in this paper show that dynamic centrifuge modelling can be successfully employed to investigate OWT foundations subjected to seismic loading when the seabed is vulnerable to liquefaction.

## 6 Acknowledgements

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