Proceedings of ISFOG 2025
5TH INTERNATIONAL SYMPOSIUM ON
FRONTIERS IN OFFSHORE GEOTECHNICS
Nantes, France | June 9-13 2025
© 2025 the Authors
ISBN 978-2-85782-758-0

Seismic assessment of jacket supported offshore wind turbine in liquefiable soil subjected to bidirectional shaking

U. Nath*, S. Haldar
Indian Institute of Technology Bhubaneswar, Bhubaneswar, Odisha, India
**un13@iitbbs.ac.in*

ABSTRACT The increasing demand for renewable energy sources has grown substantially due to the inherent issues associated with the use of fossil fuels, including concerns about their long-term reliability and adverse environmental effects. The growing importance of wind energy as a promising renewable energy source has motivated the rapid development of offshore wind turbines (OWTs). Consequently, the OWTs are installed in deeper water depths in areas prone to seismic activity. Hence, it is essential to study the performance of the OWTs while subjected to seismic motions. This study examines the seismic behaviour of a 4-legged jacket-supported 10 MW OWT in liquefiable soil under the effect of a bi-directional earthquake. A three-dimensional numerical modelling of the structure is done in the PLAXIS 3D program, which is based on the finite element approach. The soil was modelled using the UBC3DPLM constitutive model. The rotor-nacelle assembly (RNA) was considered as a lumped mass in the present study. Finally, the responses of the structure obtained from the bi-directional analysis were compared with the unidirectional shaking. This study provides a valuable perspective on the design of OWTs constructed in the liquefiable ground to ensure the structural safety of the turbines.

Keywords: Bi-directional; 4-Legged jacket ; Liquefaction; Offshore wind turbine; Uni-directional

1 INTRODUCTION

The shift toward replacing fossil fuels with renewable energy sources has gained significant importance over the past few years. Offshore wind energy is regarded as a promising and sustainable energy source due to the higher and more stable wind speed available offshore compared to onshore locations. It is worth noting that offshore wind energy achieved a cumulative capacity of 64.3 GW, with an additional 8.8 GW installed in 2022, anticipating a substantial increase to 300 GW by 2030 (GWEC 2022). Monopiles are the common foundation type in shallow water depths (less than 50 m), whereas jacket foundations have been considered more appropriate for deeper water depths of 60 m (Natarajan and Madabhushi, 2022).

The recent deployment of offshore wind turbines (OWTs) in seismically active areas prone to soil liquefaction (e.g., Southern Europe, East Asia, and the USA) requires a more detailed assessment of seismic loads and liquefaction effects caused by significant shaking on these structures (Kaynia 2018). Current regulations (IEC 614000-3 2009; DNV-OS-J101 2014) for offshore wind turbines also specify that they must be designed to resist seismic loads in areas that are seismically susceptible. Numerous studies on the dynamic responses of OWTs under the combined effects of environmental and seismic loading have also demonstrated the significant impact of earthquake loads on these structures (Bisoi and Haldar 2014; De Risi et al. 2018; Ali et al. 2019). Wang et al. (2017) performed a dynamic model test on a multi-pile offshore wind turbine subjected to seismic and environmental loads. The study demonstrated that seismic loads significantly amplify the responses of offshore wind turbines under the combined effect of wind, wave, current, and seismic loads. James and Haldar (2022) examined the impact of seismic loading in addition to wind, wave, and current loading on a 10 MW jacket-supported OWT. Their study revealed that the vertical component of earthquake loading significantly increases the dynamic responses of the structure. Recent studies have also been conducted to understand the influence of soil-structure interaction (SSI) on the seismic response of monopile-supported OWTs (Yang et al. 2019).

Seismic liquefaction resulting from strong seismic events resulted in the failure of several offshore structures (Sumer et al. 2007; Ye and Wang 2015). However, extensive research on the mechanisms and characteristics of seabed liquefaction and its impact on offshore structures, specially OWT structures, resulting from intense seismic activity, is very limited (Ye and Wang 2015). Moreover, detailed instructions for the construction of offshore wind turbine structures in liquefied soil are also scarce in the available design codes. Hence, a comprehensive study of the dynamic behaviour of offshore wind turbines in liquefiable deposits is crucial for establishing appropriate design guidelines. However, limited studies have been conducted to investigate the implications of liquefaction on the responses of OWT structures. Esfeh and Kaynia (2020) studied the performance of the OWT structures supported by monopile and caisson foundations situated on the liquefied seabed. The finite difference software FLAC3D and constitutive model SANISAND are used to study the effect of liquefaction on the monopile and caisson-supported foundations. Their study revealed that considerable rotation occurs in the case of both foundations when liquefaction occurs. Liquefaction increases the natural period of the OWT structure and also affects the dynamic responses of the OWT (Patra and Haldar 2020). The effect of the vertical component of earthquake motion on the performance of monopile-supported OWT structures in liquefied soil deposits is also examined by Patra and Haldar (2021). The existing literature on liquefaction analysis of OWT structures predominantly focuses on the caisson and monopile types of foundations. However, a few studies have been done to understand the behaviour of jacketed OWT in liquefied soil deposits. For jacket-pile-supported OWT, Natarajan and Madabhushi (2022) found that OWT structures can experience significant settlement and rotation under strong seismic loads, resulting in liquefaction, as demonstrated through dynamic centrifuge testing.

The review of the literature shows that extensive research has not been done on the liquefaction analysis of the OWT, supported by the Jacket Foundation. Hence, this study investigates the effect of liquefaction on the dynamic responses of a 4-legged jacketed OWT subjected to bi-directional earthquake motions. The UBC3D-PLM constitutive model, employed for liquefaction analysis in PLAXIS3D (2023), has been used for this purpose. In addition, the influence of bi-directional earthquake excitation on the seismic performance of the structure is also evaluated by comparing the responses generated through unidirectional shaking. The outcome of the study may enhance the design of jacketed OWT structures situated in liquefiable soil deposits.

2 NUMERICAL MODELLING

A 3D FE analysis of the 4-legged jacketed OWT structure is performed using PLAXIS3D (2023) (cf. Figure 1). A 10 MW reference wind turbine is considered in the properties of the jacket, as adopted by Nath and Haldar (2024). The rotor-nacelle assembly (RNA) is modelled as a lumped mass at the top of the tower. The boundary convergence study was conducted to determine the size of the soil domain. The piles are modelled as solid elements, whereas the jacket and tower are modelled as beam elements. The soil and pile are modelled using 10-node tetrahedral elements. The pile-soil interaction is incorporated by applying a strength reduction factor (R_{inter}). The present study considers the value of R_{inter} as 0.7. Mesh sensitivity analysis has been conducted to define the mesh size of the model. Two-layered Nevada soil, comprised of the loose sand layer at the top and medium-dense sand at the bottom, is considered. The relative densities (D_R) of the topsoil and bottom soil are 55% and 80%, respectively. The depth of the loose sand (z) is considered to be 40 m. However, the total depth of the soil (H) is taken as 90 m. The effective stress elasto-plastic UBC3D-PLM constitutive model is used for liquefaction analysis. This model is originally based on UBCSAND (University of British Columbia Sand), developed by Beaty and Byrne (1988). A non-associated flow rule based on the Drucker-Prager plastic potential function is used in this model. The parameters of the UBC3D-PLM model are listed in Table 1. The parameters presented in Table 1 are primarily correlated to the SPT value $(N_1)_{60}$. The value of $(N_1)_{60}$ is determined using $(N_1)_{60} = 46 \times D_R^2$ (Beaty and Byrne 2011). Stress-dependent stiffness factors such as k_G^e and k_B^e, and plastic shear modulus factor k_G^P required in this model can be calculated using Eqs . 1–3 (Beaty and Byrne 2011).

$$k_G^e = 434(N_1)_{60}^{0.333} \tag{1}$$

$$k_B^e = 0.7k_G^e \tag{2}$$

$$k_G^P = 0.003\, k_G^e(N_1)_{60}^2 + 100 \tag{3}$$

The liquefaction potential in terms of effective vertical stress can be described by means of the excess pore water pressure ratio (r_u) as given in Eq. 4 (PlAXIS3D, 2023),

$$r_u = 1 - \frac{\sigma_v'}{\sigma_{v0}'} \tag{4}$$

where σ_{v0}' is the initial effective vertical stress and σ_v' is the effective vertical stress during the dynamic calculation. The depth up to which $r_u \geq 0.7$ is considered a potential zone of liquefaction (Beaty and Byrne 2011).

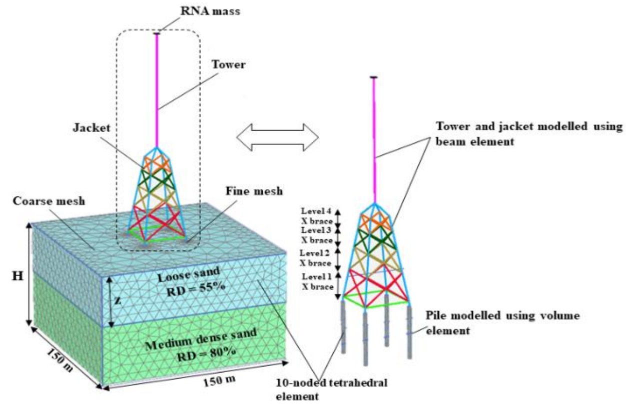

Figure 1. 3D FE model of 4-legged jacketed OWT

Table 1. Parameters of the UBC3D-PLM model used in the present study.

Description	Loose sand layer	Medium dense sand layer
Elastic shear modulus number, k_G^e	1032	1325
Elastic bulk modulus number, k_B^e	722	927
Plastic bulk modulus number, k_G^P	659	3314
Power for stress dependency of elastic bulk modulus, m_e	0.5	0.5
Power for stress dependency of elastic shear modulus, n_e	0.5	0.5
Power for stress dependency of plastic shear modulus, n_P	0.4	0.4
Friction angle at constant volume, φ_{CV}	33°	35.4°
Failure ratio, R_f	0.85	0.7
Corrected SPT value, $(N_1)_{60}$	13.4	28.4

3 VALIDATION OF THE NUMERICAL MODEL

The current numerical model is validated using the centrifuge test results reported by Wilson et al. (2000) to substantiate the accuracy and reliability of the numerical model. A flexible shear beam container was utilised to conduct the centrifuge test. The test was carried out at a centrifugal acceleration of 30 g, employing the acceleration time history of the Kobe earthquake recorded at Port Island, having a peak ground acceleration (PGA) of 0.22 g applied to the model. The test was conducted on a pile installed in two layers of Nevada sand with a water table at the ground surface. The top layer of soil had a thickness of 9.1m with a relative density of 55%, whereas the bottom layer of soil had a thickness of 11.4m and a relative density of 80%.

The numerical analysis results are compared with the results obtained from the dynamic centrifuge test for a single steel pipe pile.

Figure 2 presents the variation of the excess pore water pressure ratio (r_u) with dynamic time at a depth of 4.6 m, as obtained from the finite element (FE) model and centrifuge test data. The result of the FE models shows good agreement with the test result reported based on the centrifuge test.

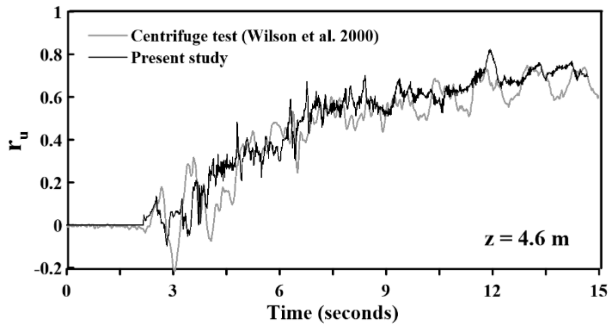

Figure 2. Comparison of the time history of the excess pore water pressure ratio (r_u) at a depth of 4.6 m from the ground surface

4 SELECTION OF GROUND MOTIONS

The three ground motions are selected from the Pacific Earthquake Engineering Research (PEER) ground motion database, having a magnitude (M) of 6.5–7.5. The source-to-site distance (R) varies from 0.5 to 16 km. The two horizontal components of the selected earthquakes are considered in this present study. The peak ground acceleration of the longitudinal and transverse components of the ground motions varies from 0.17g to 0.45g. The properties of the strong ground motions are listed in Table 2. The earthquake motions are selected in such a way that the peak frequency (derived from the Fourier spectrum) of one earthquake is close to the fundamental frequency of the OWT; however, in other cases, it lies away from the fundamental frequency of the structure (cf. Fig. 3). The selected ground motions were given input at the bottom of the continuum soil model. (Barron et al. 2024; Castelli et al. 2024).

Table 2. Properties of strong ground motion parameters.

Ground motion	Year	Station	PGA (g)		*R* (km)	Peak freq. (Hz)	*M*
			Longitudinal	Transverse			
Fruili, Italy	1976	Tolmezzo	0.35	0.31	16.0	1.9	6.5
Imperial valley-06	1979	El Centro Array #6	0.45	0.44	1.35	0.29	6.5
Kocaeli	1999	Izmit	0.23	0.17	7.21	1.5	7.5

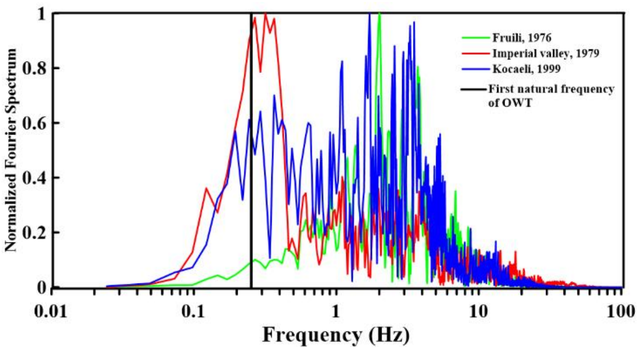

Figure 3. Normalised Fourier spectrum of the longitudinal components of the selected earthquake motions.

5 DESIGN CRITERIA

An OWT supported by a 4-legged jacket foundation must meet various limit state criteria, including the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS), to ensure its safety, stability, and operational efficiency throughout its design lifespan. In this study, the SLS criteria include tower-top displacement (u_{tt}), tower-top rotation (θ_{tt}), and mudline rotation (θ_{ml}). The limiting value of tower-top displacement and tower-top rotation is considered as 1.25% of tower height and 0.5º, respectively (Nath and Haldar 2024; Ali et al. 2020). The permissible limit of mudline rotation is taken as 0.5º, as mentioned in the available design code (DNV 2016). ULS criteria ensure the strength and stability of the structure against failure. The maximum bending moment (M) is checked against the resisting moment (M_R) to satisfy the buckling failure of the structure (Nath and Haldar 2024; Patra and Haldar 2020).

6 RESULTS AND DISCUSSION

6.1 Depth of liquefaction

This study examines the impact of liquefaction on the seismic responses of a 4-legged jacketed Offshore Wind Turbine (OWT) subjected to bidirectional seismic motions combined with environmental loading. The liquefaction depth was evaluated under bi-directional seismic motions and compared with the corresponding depth observed under uni-directional motions. Subsequently, the seismic responses of the OWT were analysed to assess the impact of these conditions.

During dynamic analysis, the excess pore water pressure ratio (ru) was monitored in the near field stress points, i.e., at a distance of 2 m from the pile, considering various depths of the soil domain. Figure 4 illustrates the variation of r_u with dynamic time for the Friuli earthquake (1976), comparing bi-directional and uni-directional motions at z = 5 m, 10 m, 15 m, and 20 m. It can be observed from Fig. 4 (a) that for unidirectional motion, when z = 5m, the excess pore water pressure ratio (r_u) is equal to 1. This shows that the corresponding layer is completely liquefied. However, the r_u decreases with the increase in the soil depth. For instance, when z is equal to 10 m, 15 m, and 20 m, then the value of ru becomes 0.7, 0.65, and 0.66, respectively. A similar trend was also noticed for r_u values with the increase in the soil depth when the structure is subjected to bi-directional motions. However, the soil depth increased to 10 m, where r_u becomes 1, i.e., the soil is in a fully liquefied state (cf. Fig. 4 (b)).

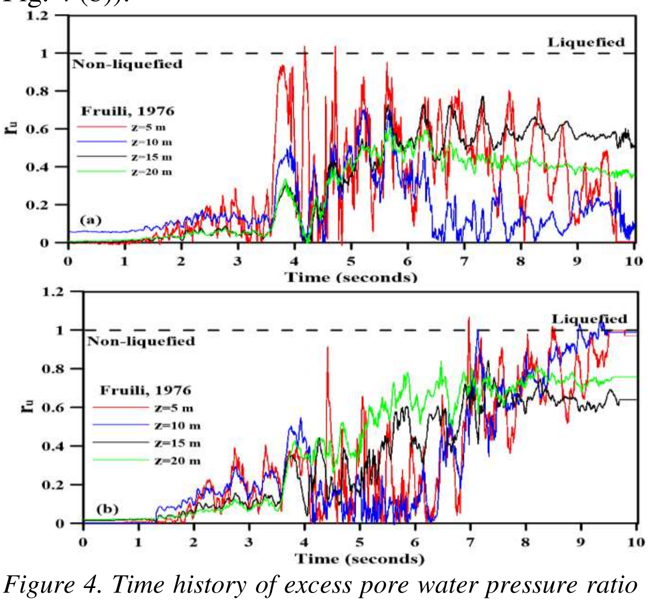

Figure 4. Time history of excess pore water pressure ratio (r_u) at different depths of loose sand under (a) uni-directional, (b) bi-directional shaking of Fruili (1976) strong ground motion.

To illustrate the effect of bi-directional earthquake motion on the depth of the liquefaction zone, the maximum excess pore pressure ratio with the normalised depth of loose soil is shown in Fig. 5. The maximum pore pressure ratio under uni-directional motion is also depicted. The potential zone of liquefaction is identified where $r_u \geq 0.7$ (Beaty and Byrne 2011). It can be clearly noticed from Fig. 5 that the depth of liquefaction increases for bi-directional earthquake motions compared to the unidirectional motions. The value of r_u increases by almost 15% under bi-directional motion when compared to uni-directional excitation at z/H of 0.05 m for the Kocaeli earthquake (1999). Moreover, it has also been noticed that the natural frequency corresponding to the first mode of vibration shifts to 0.194 Hz from 0.23 Hz after the liquefaction for uni-directional motion, while it is further shifted to 0.187 Hz when considering bi-directional components of earthquake motions. This results in a 15.65% and 18.7% reduction in natural frequency in the case of uni-directional and bi-directional motions, respectively, with respect to the fundamental frequency of OWT before liquefaction. The decrease in natural frequency may result in a greater depth of liquefaction under bidirectional motions compared to unidirectional motions. However, a comprehensive evaluation of the liquefaction effects necessitates the inclusion of a larger set of bidirectional seismic motions to accurately assess the liquefaction phenomena during earthquake excitations.

6.2 Seismic responses considering SLS and ULS criteria

The seismic responses of the structure subjected to liquefaction were computed under the bi-directional seismic motions for SLS and ULS design criteria. The responses of the OWT were also compared when the structure was situated in a non-liquefiable deposit. The hardening soil model with small-strain stiffness (HSsmall) is used to model the non-liquefiable soil layer. The input parameters are selected based on the expressions provided by Brinkgreve et al. (2010). Figure 6 shows the responses such as tower-top displacement (u_{tt}), tower-top rotation (θ_{tt}), mudline rotation (θ_{ml}), and bending moment (M) of the tower for liquefiable and non-liquefiable soil cases subjected to bi-directional ground motions. All the responses were normalised against the permissible value, as mentioned in section 5. It is noticed that the responses under the liquefaction case are higher compared to the non-liquefaction case. It is also worth noting that the u_{tt} and θ_{tt} exceed the threshold limit in the case of liquefiable conditions for Imperial Valley (1979), strong ground motion. The u_{tt} and θ_{tt} for liquefiable soil increase by 56% and 76% for the same earthquake compared to the non-liquefaction case. However, the mudline rotation and maximum bending moment remain below the permissible limit due to all the ground motions for both the liquefaction and non-liquefaction cases. It is evident that the Imperial Valley earthquake motion gives higher responses compared to other motions in both cases. This can be attributed to the fact that the peak frequency of the Imperial Valley earthquake (1979) is very close to the natural frequency of the structure, which enhances the responses of the structure compared to other seismic motions.

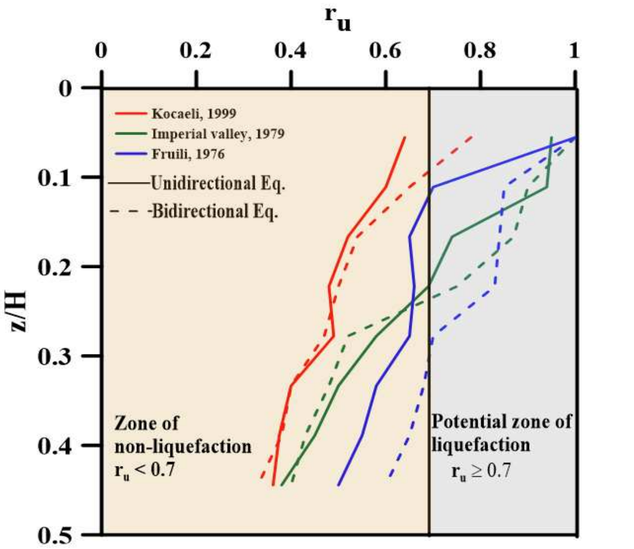

Figure 5 Profile of maximum excess pore water pressure ratio (r_u) at various normalised soil depths for uni-directional and bi-directional ground motions.

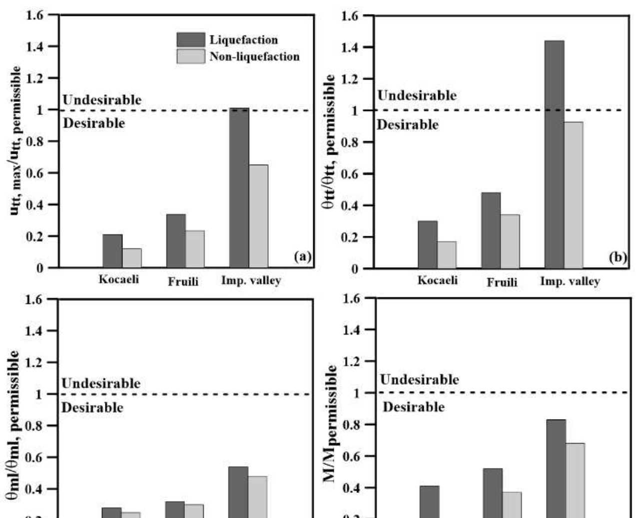

Figure 6. Normalised responses (a) u_{tt}, (b) θtt, (c) θml, and *(d) M for liquefaction and non-liquefaction under bi-directional motions.*

7 CONCLUSIONS

The liquefaction effect on the responses of jacketed OWT is studied under bi-directional and unidirectional earthquake motions. A 3D numerical analysis has been carried out in PLAXIS 3D (2023). The zone of liquefaction is analysed for both uni-directional and bi-directional earthquake motions. Moreover, the seismic responses due to liquefaction analysis are also compared to the responses generated in non-liquefaction cases for both SLS and ULS design criteria. Some useful observations can be made from this current study:

1) The depth of liquefaction increases under the application of bi-directional earthquake motions when compared to uni-directional motions. This highlights the bi-directional component, which has a significant effect on the liquefaction phenomena of the structure.
2) The seismic responses recorded for structures situated in a liquefiable deposit show higher responses than responses generated in non-liquefiable soil deposits. The tower-top rotation gives a higher response of 76% when compared to the responses recorded in non-liquefiable soil deposits in the case of Imperial Valley earthquake motion. However, the outcome of the study shows the necessity of considering an adequate number of earthquake motions when the jacketed OWT is subjected to a liquefied soil deposit.

AUTHOR CONTRIBUTION STATEMENT

Upasana Nath: Methodology, Formal analysis, Visualization, Conceptualization, Writing - original draft. **Sumanta Haldar**: Writing – review and editing, Supervision, Conceptualization.

REFERENCES

Nataranjan K., Madabhushi, G.S.P. (2022). Seismic response of an offshore wind turbine jacket structure with pile foundations. *Soil Dynam. Earthq. Eng.* 162:107427.

Ali, A., R. De Risi, Sextos, A., Goda, K., Chang, Z. (2019). Seismic vulnerability of offshore wind turbines to pulse and nonpulse records. *Earthquake Eng. Struct. Dyn*. 49 (1): 24–50.

Bisoi, S. and Haldar, S. (2014) Dynamic analysis of offshore wind turbine in clay considering soil-monopile-tower interaction. *Soil Dynam. Earthq. Eng.* 63:19-35.

Beaty, M. H., Byrne, P. M. (2011). UBCSAND Constitutive ModelVersion 904aR. Itasca UDM Web Site 69.

Barron J., Rouainia M., Charlton T., Edwards S., Gibson F. (2024). Seismic performance of shared suction caisson installed in liquefiable sand for floating offshore wind turbines. *Soil Dyn Earthq Eng*., 180:108598.

Castelli F., Grasso S., Lentini V., Sammito M. (2024). Assessment of liquefaction effects on dynamic soil-structure interaction for the 1908 Messina and Reggio Calabria scenario earthquake. *Soil Dyn Earthq Eng*., 178:108445.

De Risi, R., Bhattacharya, S., Goda, K. (2018). Seismic performance assessment of monopile-supported offshore wind turbines using unscaled natural earthquake records. *Soil Dynam. Earthq. Eng*. 109, 154–172.

DNV-OS-J101 (2010). Design of offshore wind turbine structures. Hovik, Denmark: Det Norske Vertias.

Esfeh, P.K., Kaynia, A.M. (2020). Earthquake response of monopiles and caissons for offshore wind turbines found in liquefiable soil. Soil Dynam. Earthq. Eng. 136:106213.

GWEC (2022). Global offshore wind report. Brussels, Belgium: Global Wind Energy Council.

IEC 61400-3 (2009). Design requirements for offshore wind turbines. Switz: Int Electrotech Comm Geneva.

James, M., Haldar, S. (2022). Seismic Vulnerability of Jacket Supported large Offshore Wind Turbine Considering Multi-Directional Ground Motions. *Structures*. 43:407-423.

Kaynia, A.M. (2018). Seismic considerations in the design of offshore wind turbines. *Soil Dyn Earthq Eng*., 124:399–407.

Nath, U., Haldar, S. (2024). Effectiveness of combination rule at predicting the elastic seismic demand of jacket-supported offshore wind turbine subjected to bi-directional motions. *Structures.* 64:106455.

Patra, S. K., Haldar, S. (2020). Fore-aft and the side-to-side response ofmonopile supported offshore wind turbine in liquefiable soil. *Mar.Georesour. Geotechnol*

Patra, S. K., Haldar. S. (2021). Seismic performance of multi-megawatt offshore wind turbines in liquefiable soil under horizontal and vertical motions. *Int. J. Geomech. Vol. 22.*

Sumer, B. M., Ansal, A., Cetin, K. O., Damgaard, J., Gunbak, A. R., Hansen, N. E. O., Sawicki, A., et al. (2007). Earthquake-induced liquefaction around marine structures. *Journal of Waterway,* *Port,Coastal, and Ocean Engineering* 133 (1): 55–82.

Wilson, D. W., Boulanger, R. W., Kutter, B. L. (2000). ObservedSeismic Lateral Resistance of Liquefying Sand. Journal of Geotechnical and Geoenvironmental Engineering 126 (10): 898–906.

Wang, W., Gao, Z., Li, X., Moan, T. (2017). Model test and numerical analysis of multi-pile offshore wind turbine under seismic, wind, wave, and current loads. *J. Offshore Mech. Arct. Eng*. 139(3): 031901.

Yang, Y., Li, C., Bashir M., Wang, J., Yang, C. (2019) Investigation on the sensitivity of flexible foundation models of an offshore wind turbine under earthquake loadings. *Eng Struct*. 183:756–69.

INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 5th International Symposium on Frontiers in Offshore Geotechnics (ISFOG2025) and was edited by Christelle Abadie, Zheng Li, Matthieu Blanc and Luc Thorel. The conference was held from June 9th to June 13th 2025 in Nantes, France.