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Influence of soil parameters on the seismic response of offshore wind turbines in liquefiable soils

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ABSTRACT: The goal of this study is to investigate the effect of model parameters on the behaviour of offshore wind turbines in liquefiable soils under earthquake loading. Numerical analyses were conducted using an advanced soil constitutive model for liquefaction behaviour, the P2PSand model, available in FLAC3D. A previous study was chosen from the literature to verify the created numerical model, comparing pore water pressure in the soil and horizontal displacement of the monopile. The results indicate that the model can accurately predict both soil and pile behaviour. After validation, a new model was created to assess the effect of liquefiable soil parameters. Three soils were selected for comparison: Ottawa sand, Karlsruhe fine sand, and standard cyclic resistance field (SCRF) sand. Calibration of the model parameters for these soils is well-documented in the literature. A single earthquake record was applied to the model base, and the responses of free-field ground acceleration at the surface, superstructure (tower) acceleration, and pile head rotation were compared. Results showed that offshore wind turbine response in liquefiable soils is strongly influenced by soil parameters. Particularly, the parameters of SCRF sand led to higher ground and tower accelerations, resulting in greater monopile head rotations.

Keywords: offshore; earthquake; monopiles; liquefaction; soil

1 INTRODUCTION

Offshore wind turbines (OWT) have become widespread globally, including in seismically active regions, as the demand for renewable energy grows. A major concern with OWT's founded on monopiles is permanent rotation, or tilt. Earthquake loading could significantly increase the tilt in liquefiable soils due to the stiffness and strength reductions.

Several researchers have investigated the behaviour of monopiles and OWTs in liquefiable soils through centrifuge tests (Español-Espinel et al., 2023; Seong et al., 2023; Yu et al., 2015) and numerical analyses (Esfeh & Kaynia, 2020; Zhang et al., 2022). The primary factor in seismic soil-pile-structure interaction is the free-field soil behaviour, which largely governs the response to earthquake loading.

Several constitutive models have been developed by researchers to simulate sand liquefaction; such as PM4Sand (Boulanger & Ziotopoulou, 2015), UBCSand (Beaty & Byrne, 2011), DM04 (Dafalias & Manzari, 2004), Pressure-Dependent Multi-Yield 02, PMDY02 (Yang et al., 2003). More recently, a new constitutive soil model called P2PSand was introduced by Cheng and Detournay (2021) to enhance the capabilities of the original DM04 model.

In this study, the P2PSand model was used to investigate the effects of liquefaction on monopile and tower behaviour in offshore wind turbines. A numerical model was created in FLAC3D and verified using a study from the literature. The influence of the model parameters on excess pore water pressure development, monopile rotation, and tower behaviour was shown.

2 NUMERICAL ANALYSES

The NREL 5-MW offshore baseline wind turbine, with properties listed in Table 1, was used in the numerical analyses. This same baseline OWT was also employed in numerical studies by Esfeh and Kaynia (2020), which considered the combined effects of wave, wind, and earthquake loading. However, in this study, only the effects of earthquake loading were examined. A schematic view of the NREL 5-MW turbine founded on a monopile in a two-layered soil profile is shown in Figure 1.

Table 1. Properties of NREL 5-MW baseline wind turbine

Property	Value
Rating	5 MW
Hub height	90 m
Rotor mass	110 000 kg
Nacelle mas	240 000 kg
Tower mass	347 466 kg
Tower height	87.6 m
Tower top diameter, wall thickness	3.87 m, 0.019 m
Tower base diameter, wall thickness	6 m, 0.027 m
Substructure base diameter, wall thickness	6 m, 0.06 m
Embedment depth of monopile	25 m
Structure steel density	8500 kg/m^3
Steel Young's modulus	210 GPa
Tower height	87.6 m
Tower top diameter, wall thickness	3.87 m, 0.019 m
Tower base diameter, wall thickness	6 m, 0.027 m

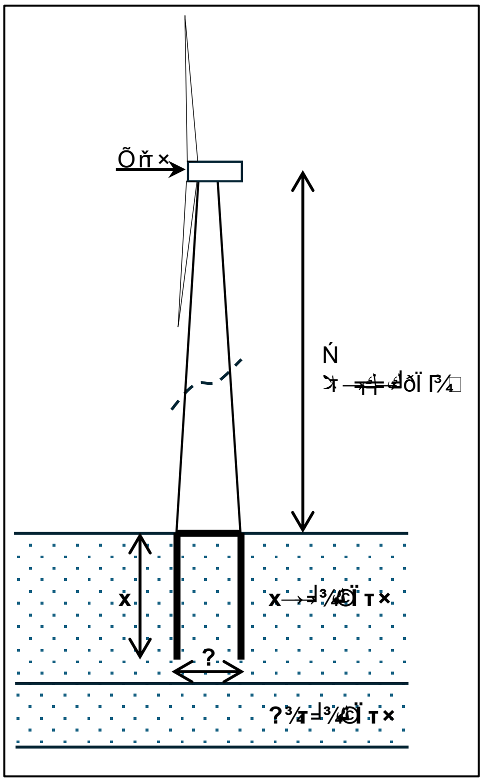

Figure 1. Schematic view of NREL 5-MW offshore wind turbine founded on a monopile in two layered soil profile

2.1 Numerical Model

The numerical analyses were conducted in FLAC3D. The soil domain was modelled with 8-noded brick elements, while the structural beams elements were used to represent the pile and superstructure behaviour. The soil profile consists of a 26 m thick loose sand overlaid by an 8 m thick layer of dense sand, with relative densities of 35% and 80%, respectively.

Both the tower and the monopile were modelled using two-noded structural beam elements with six degrees of freedom per node. The tower beam was divided into 2-m segments to account for the tapered cross-section. Variations in diameter and wall thickness along the height of the tower were represented by assigning an appropriate moment of inertia to each segment. A separate free vibration analysis, conducted to determine the first natural frequency of the fixed-bottom tower, resulted in a value of 0.29 Hz, which is in good agreement with the 0.32 Hz reported by Jonkman et al. (2009).

The monopile behaviour was simulated using structural beam elements with an interface between the pile and soil grid. Following Esfeh and Kaynia (2020), a fully-bonded interface was assumed by assigning a high interface strength with the stiffness in the normal and shear directions equal to the small-strain stiffness of the surrounding soil. The structural beam elements used for the monopile were divided into 1-m segments. The mass of the tower and pile was defined by applying the density of steel. The rotor and nacelle mass were assigned as a lumped mass at the top node of the tower. The model created in FLAC3D is shown in Figure 2.

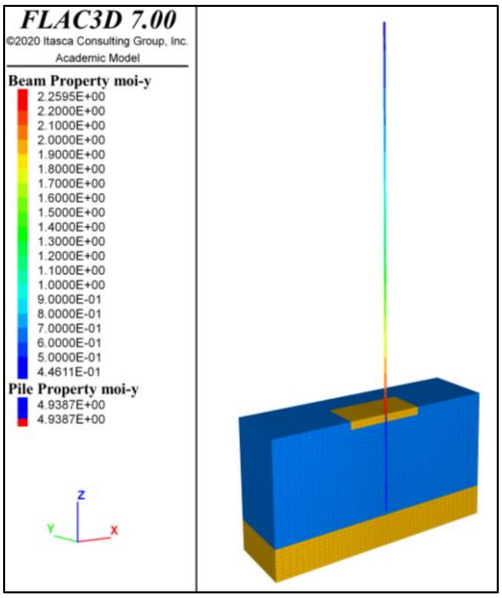

Figure 2. 3D view of the numerical model

The dimensions of the numerical model were 60 m x 18 m x 34 m in the x, y, and z directions, respectively. The model consisted of 36720 zones, with 60, 18, and 34 elements in x, y, and z directions, respectively. The element size in the y and z directions was kept constant at 1-m. To ensure accurate wave transmission, Kuhlemeyer and Lysmer (1973) recommended a minimum element size criterion of one eight of the wavelength ($\Delta l<\lambda/8$) in the vertical direction, where λ is the wavelength, calculated as the shear wave velocity divided by the maximum frequency component of the input motion ($\lambda=V_s/f_{max}$). Assuming a minimum V_s of 75 m/s near the ground surface and an f_{max} of 10 Hz based on the earthquake record (Esfeh & Kaynia, 2020), the resulting wavelength is 7.5 m, making a 1-m zone size sufficient to meet the wave transmission criterion. An aspect ratio of 1.05 was assigned to the zone sizes in the x direction, where the dynamic motion is applied. This approach allows for finer zone sizes near the pile (0.5 m) to better capture the near-field behaviour, while the element size gradually increased to approximately 2.0 m toward the lateral boundaries.

In the initial stage, the lateral boundaries were fixed to generate the geostatic stress conditions, with a K_0 value of 0.5. In the second stage, the tower and monopile were modelled using beam elements. In the final stage, the dynamic analysis was carried out by applying the acceleration-time history of the Kobe-L record (Kirkwood & Dashti, 2018), shown in Figure 3, to the bottom boundary. During this stage, the lateral boundaries were tied, and the bottom boundary was fixed. This setup follows the approach used in the reference study (Esfeh & Kaynia, 2020), which simulated a centrifuge test performed in a laminar box rigidly connected at the base.

A small amount of Rayleigh damping (0.5 %) was applied to the soil domain to filter out the high-frequency components. The dynamic time step was automatically calculated as 4.5×10^{-5}, based on the stiffness of both the soil and structural elements, which was sufficiently small compared to the time step of the input motion.

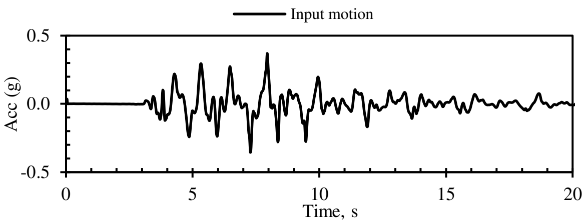

Figure 3. Acceleration-time history of the input motion

2.2 Soil constitutive Models

Two constitutive soil models were used in this study: DM04 and P2PSand. The primary objective was to examine the effects of input parameters in the P2PSand model, while the DM04 model was also employed to verify the numerical model created in FLAC3D. The analysis results were then compared with those from the reference study by Esfeh and Kaynia (2020).

The parameters for the Sanisand (DM04) constitutive model for Ottawa sand and Monterey sand, calibrated by Ramirez et al. (2018), and provided by Esfeh and Kaynia (2020), are given in Table 2. Ottawa sand was used throughout the soil profile, while a 2-meter deep layer of dense Monterey sand, with a 10-meter radius, was assigned around the monopile as protection against scour.

Table 2. SANISAND constitutive model parameters calibrated for Ottawa sand F65 (Ramirez et al., 2018)

Parameter	Ottawa sand	Monterey sand
G_0	125	130
ν	0.05	0.05
M	1.26	1.27
c	0.735	0.712
λ_c	0.0287	0.02
e_0	0.78	0.858
ξ	0.7	0.69
m	0.02	0.02
h_0	5	8.5
c_h	0.968	0.968
n^b	0.6	1.05
A_0	0.5	0.6
n^d	0.5	2.5
z_{max}	11	4
C_z	500	50
$\bar{e}_{eq}^{p}$	0.01	0.01
N	1	1

The second soil constitutive model used in this study is the P2Psand (Practical TWO-surface Plastic SAND) model, developed by Cheng and Detournay (2021) to simulate the behaviour of liquefiable soil. This model extends the DM04 model, developed by Dafalias and Manzari (2004), incorporating revisions to enhance its performance in earthquake applications. While preserving the key features of the DM04 model, P2PSand modifies void ratio-related internal parameters to be based on relative density, making it more suitable for in-situ applications.

The P2Psand model requires only the relative density and initial stress state as inputs, with the remaining parameters internally calibrated based on the relative density. The internal calibration of parameters is based on the simplified cyclic resistance curves developed by Idriss and Boulanger (2008). The default parameters provided by Cheng and Detournay (2021) for P2PSand, referred to as standard cyclic resistance field (SCRF) sand, are used in this study.

The essential features of a soil constitutive model are its ability to simulate both small-strain and large-strain behaviour. In the P2PSand model, the pressure-dependent small-strain shear modulus of soil is expressed in by the following equation:

$$G = G_r p_{atm} \left(\frac{p}{p_{atm}}\right)^n \quad (1)$$

where G_r is the elastic material parameter, p_{atm} is the reference pressure, p is the pressure, and n is a constant that governs variation with depth.

Table 3. P2PSand model parameters calibrated by El-Sekelly et al. (2022)[*1]* and Cheng and Detournay (2021)*[*2]

Property	Ottawa Sand[*1]	Karlsruhe fine sand[*2]	SCRF sand (internally calibrated-default)[*2]
ϕ_{cs}	33	33	33
g_0	503	206	1240
C_{Dr}	0.73	1.24	0.01
D_{r0}, l_c, e		(-0.0345, 0.2125, 0.3044)	
or (Q, R)	(9, 1)		(10, 1)
n^b	-	0.08	$0.16-\phi_{cs}/400$
n^d	1	0.3	$6n^b$
h_0	0.4	0.68	1.7
A_{d0}	-	0.9	$0.164\ I_R/(M^b-M^d)$
K_c	0.8	$2.99-2.26D_{r0}$	$3.8-7.2D_{r0}+3.0\ D_{r0}^2>0.007$
k_d	-	-	$0.46-0.35D_{r0}$
e_{max}, e_{min}	(0.78, 0.51)	(1.054, 0.677)	(1.0, 0.6)

For large-strain behaviour, the model must accurately represent the shear modulus degradation and damping ratio variation. Cheng and Detournay (2021) demonstrated that this behaviour in the P2PSand model aligns well with generic curves provided in EPRI (1993).The DM04 constitutive model has two main shortcomings: Overlapping of stress paths, which can result in lower shear strains, and an overprediction of damping ratios at large shear strains (Carey & Kutter, 2017). To address these issues, the P2PSand model incorporates a revised plastic modulus formulation.

In this study, three different sets of soil parameters were used to assess their impact on the seismic response of offshore wind turbine (OWT) structures: Ottawa sand, Karlsruhe fine sand, and SCRF (the internally calibrated-default P2PSand) sand. The model parameters are presented in Table 3.

3 RESULTS

The numerical model created in this study was verified by comparing monopile head displacement with that of Esfeh and Kaynia (2020) in Figure 4. Although slight differences were observed, the overall displacement trend was accurately captured. This discrepancy is attributed to different approaches used for structural elements: in this study, the pile and tower were modelled with beam elements for simplicity, whereas solid elements were used in the reference study. Additionally, results from the DM04 model was compared with those from the P2PSand model using Ottawa sand parameters, with the latter resulting in higher permanent displacements, as shown in Figure 4. The higher permanent displacement response in the P2PSand model can be attributed to differences in excess pore water pressure response and damping ratio. As previously reported by Cheng and Detournay (2021), the damping ratio at large strains is overestimated in DM04, whereas the P2PSand model addresses this issue through revised equations in its constitutive formulation. A more realistic representation of the damping ratio at large strains in P2PSand may lead to higher ground accelerations and greater permanent displacements. These aspects will be further discussed later using excess pore water pressure and acceleration response spectra plots.

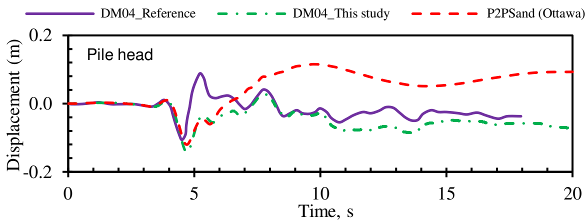

Figure 4. Comparison of monopile head displacements obtained in this study (DM04 and P2PSand models) and DM04 model result provided in Eshef and Kaynia (2020)

The effects of P2PSand constitutive model parameters on the seismic response were examined by comparing the results for three sets of soil parameters. Instead of using the Monterey sand parameters for the 2-m thick scour protection layer, the P2PSand model was assigned with a relative density of 80%. Displacements at the top of the tower are shown in Figure 5. Although peak absolute displacements were similar for Ottawa sand and Karlsruhe sand (approximately 1.0 m), Karlsruhe sand resulted in higher permanent displacements. The higher displacements in Karlsruhe sand could be attributed to the higher K_c parameter used, which influences the pore water pressure development and cyclic behaviour. Both peak displacements during seismic loading and permanent displacements at the end of the time history were highest for SCRF sand.

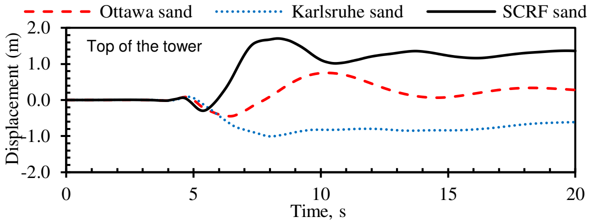

Figure 5. Comparison of lateral displacements at the top of the tower with P2PSand model

The primary concern in offshore wind turbines is the rotation at the monopile head. Figure 6 shows the time history of the rotations for each set of soil parameters used in this study. Similar to the tower displacements, the model with SCRF sand exhibited the highest permanent rotation. The lower response in Ottawa and Karlsruhe sand could be attributed to the lower plastic modulus parameter (h_0) parameter adopted based on the laboratory-based calibration process. In contrast, for SCRF sand, the h_0 parameter was calibrated using the modulus degradation curves provided in EPRI (1993), resulting in lower shear modulus degradation. A higher shear modulus can significantly increase the ground accelerations, particularly under high-intensity ground motions. This assumption will be examined by comparing the acceleration response spectra.

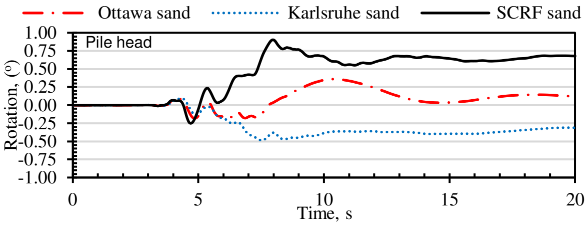

Figure 6. Comparison of pile-head rotations using P2PSand model for different set of soil parameters

The pore water pressure ratio (r_u) variation over time through the depth of soil profile is shown in Figure 7. According to Figure 7a, the DM04 and P2PSand models yield similar responses in loose sand. However, in dense sand (at z=30 m), the r_u values reach as high as 1 in the DM04 model, whereas they remain lower in the P2PSand model, with a maximum value of 0.36. The high r_u values in DM04 model can be attributed to its inability to accurately represent the true cyclic resistance (CRR) versus number of equivalent uniform loading cycles (N) relationship. As reported by Boulanger and Ziotopoulou (2015), the DM04 model significantly overestimates the slope of the N-CRR curves.

Furthermore, the effect of P2PSand model soil parameters on the pore water pressure ratio is shown in Figure 7b. The results indicate that SCRF sand parameters led to the lowest r_u values in dense sand, approximately 0.10, whereas r_u reached as high as 1.0 in Karlsruhe sand. Moreover, the model with SCRF sand parameters showed sharp decreases in r_u at t=4.8 s and t= 5.65 s at relatively shallow depths of z=4 m and z=8 m. A similar sharp decrease in pore water pressure was observed in the centrifuge tests reported by Wilson (1998) during the Kobe earthquake, leading to an increase in effective stress. This behaviour resulted in higher accelerations in the free-field ground, which in turn resulted in greater responses in the superstructure and pile.

Figure 8 presents acceleration response spectra plots for the input motion, the free-field ground surface, the top of the tower, and pile head for each set of soil parameters used in this study. According to Figure 8a and 8b, while the peak acceleration of the input motion was approximately 0.38g, there was a notable reduction in peak accelerations (de-amplification) at the ground surface: 0.20 g for Ottawa sand and 0.12 g for Karlsruhe sand. Similarly, the tower response showed significantly lower accelerations, approximately 0.2g. The largest pile head response obtained in Karlsruhe sand explains the greater permanent lateral displacements compared to those in Ottawa sand, indicating the effects of soil-pile-structure interaction on the overall system response.

Figure 8c shows that the SCRF sand model parameters result in amplified ground surface accelerations, reaching 0.5g. The response spectrum at the top of the tower exhibited significantly high spectral accelerations for periods greater than 1.2 s. Overall, the highest superstructure acceleration occurred with SCRF sand, which explains the largest pile-head rotations. Besides, despite lower acceleration demands with Karlsruhe sand, pile-head rotation exceeded the acceptable limits (0.5°) due to significant strength reductions, even in dense sand where the r_u value reached 1.0.

4 CONCLUSION

In this study, the influence of liquefaction on the seismic response of offshore wind turbines was investigated using the P2PSand model with three sets of soil parameters: Ottawa sand, Karlsruhe Sand, SCRF (standard cyclic resistance field) sand. Results indicated that SCRF sand parameters led to higher superstructure (tower) accelerations compared to the laboratory-calibrated Ottawa and Karlsruhe sands, resulting in greater monopile head rotations. The pore water pressure variation was also significantly affected by model parameters, leading to different ground surface responses.

The results of this study show that the liquefaction model parameters calibrated using the laboratory tests (Ottawa and Karlsruhe sands) led to a significant deamplification in the ground response. In contrast, model parameters calibrated based on field observations (SCRF sand) resulted in higher ground and tower accelerations, leading to increased monopile rotations. Furthermore, the significantly high excess pore water pressures obtained in Karlsruhe sand lead to increased pile head acceleration responses, highlighting the importance of soil-pile-structure interaction in liquefiable soils. However, this study was limited to a single soil profile under a single earthquake record. Furher research is required to investigate the effects of relative density, liquefiable layer thickness, and seismic loading characteristics.

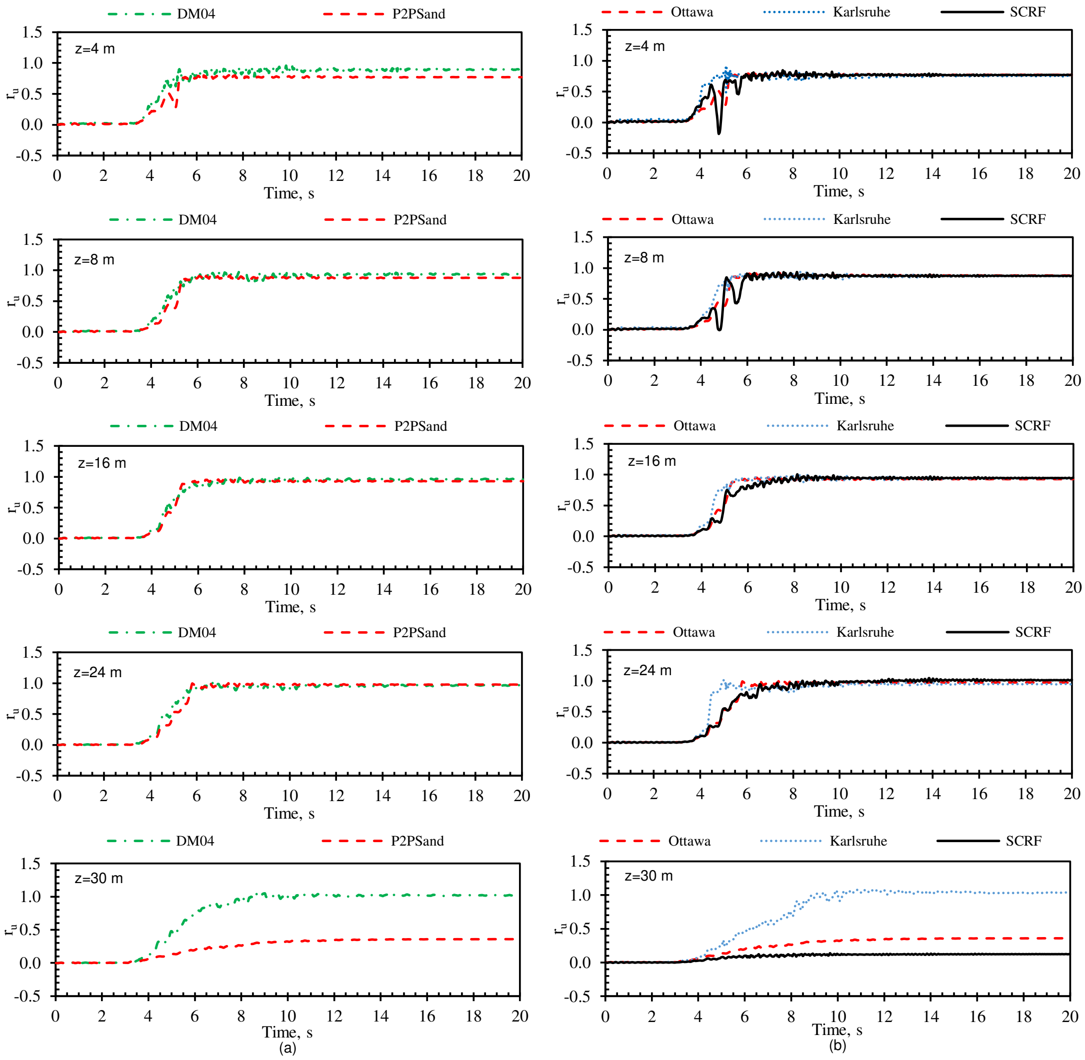

Figure 7. Pore water pressure ratio (r_u) variation over time for various depths: (a) DM04 and P2PSand model comparison for Ottawa sand, (b) Comparison of various sets of soil parameters in P2PSand model

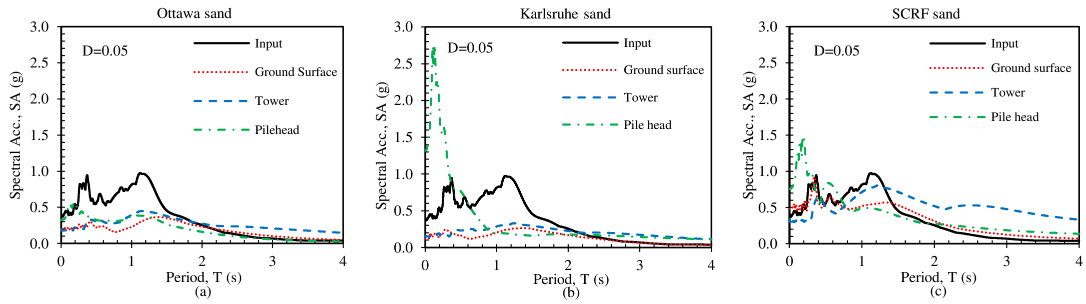

Figure 8. Acceleration response spectra at the ground surface, the superstructure (Tower), and the pile head for various soil parameters of P2PSand model: (a) Ottawa sand (b) SCRF sand (c) Karlsruhe sand

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AUTHOR CONTRIBUTION STATEMENT

Ozan Alver: Conceptualization, Methodology, Software, Writing, Validation, Formal analysis.
Kenneth Gavin: Supervision, Investigation, Validation, Review & Editing.
E. Ece Eseller-Bayat: Supervision, Software, Review & Editing.

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