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ARTICLE

Dynamic Assessment of OWT under Coupled Seismic and Sea-wave Motions

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ABSTRACT

The effect of soil-monopile-structure interaction is of great importance in the design of offshore wind turbines (OWTs). Although sea waves play the most effective role in the performance of OWTs, the coupled effect of sea-wave loads and seismic motion on the performance of the OWT system in seismic-prone areas is a factor that is less investigated and should not be ignored. In this regard, a 2-D porous model based on Biot's poro-elastic theory is considered to capture the pore water pressure generation in the soil domain surrounding the OWT foundation. The coupled effect of sea waves and seismic motion through a comparative study is considered for the reference OWT system based on the monopile foundation by using the FE program, OpenSees. The results of the analyses are presented in specific locations. Upon the obtained results, the dynamic behavior of the OWT system and the possibility of liquefaction in the soil surrounding the OWT during applied loads are investigated and compared. This comparison is a good representative of the effect of the seismic motion on the performance of the OWT system and the soil medium by considering soil-monopile-structure interaction in seismic-prone areas.

1. Introduction

According to the demand of the world for green energy resources, wind energy is one of the examples that is of great interest in recent years. Wind energy is a secure type of energy and is environmentally friendly. Because of the considerable number of coastal areas in the world and the high speed of the offshore wind, the Offshore Wind Turbine (OWT) compared to the onshore one, is of great interest nowadays. According to the recent agreement by the countries to achieve net-zero emissions over the coming decades, OWT farms will be increasingly employed in seismically active regions. The reasonable performance of these structures both functionally and economically is one of the main concerns in the OWT projects. It needs to be noted that for these giant structures with a design life of 25 years, the design of the foundation which costs about 25-34% cost of the whole project is one of the most

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costly parts of the whole system [1]. In this regard, the current study concerns the performance of the offshore wind turbine system during environmental loads (cyclic sea-wave and earthquakes) with a focus on the geotechnical part of the system (foundation and the surrounding soil) which has been less investigated. Monopile, which is the most common foundation for OWTs, is considered for the numerical analyses in the present study.

The present design standards and guidelines for OWTs are dependent on *p-y* curves that are defined for offshore pile foundations in the oil and gas industry. Because of larger numbers of load cycles, the larger diameter of monopiles for OWTs, and the possibility of pore water pressure generation in the soil domain, the accuracy of their application for the OWT system is now questionable among researchers. The main aim of the current study is to evaluate the possibility of liquefaction in the soil medium in the vicinity of the monopile in seismic-prone areas by considering soil-pile-structure interaction.

In the present research, the Finite Element (FE) program OpenSees (The Open System for Earthquake Engineering Simulation) is employed for numerical modeling and analysis of the OWT system [2]. The consequences such as stresses, strains, excess pore water pressure ratio, and deformations are evaluated in specified locations of the model.

Among studies in the field of OWT, some studies investigated the structural response of the OWT system (e.g., [3,4]), and only a few concentrated on the geotechnical performance of the system under seismic actions (e.g., [5-7]). Despite the numerous recent studies in the field of OWT, the number of items relevant to the investigation of the liquefaction possibility in the soil surrounding the OWT is very limited [7]. The traditional *p-y* curves method, used for the current design of the OWTs suggested by the guidelines and standards (API and DNV GL), greatly overestimates the initial stiffness for the liquefied soils. This procedure leads to an unconservative estimation of foundation tilting and the dynamic performance of the overall system [8-10]. In 2020, Kazemi Esfeh and Kaynia evaluated the possibility of liquefaction in the soil surrounding the OWT foundation during the combined action of wind and seismic loads. According to the authors, liquefaction is a phenomenon that has not been studied sufficiently for the case of the OWT foundation and the soil domain in the vicinity of the foundation. The OWTs based on monopile and caisson foundations were analyzed in this research. FLAC 3D software was employed and the soil medium was formed by using SANISAND constitutive model through nonlinear dynamic analyses. They resolved that liquefaction incident has considerable effects on the rotation of both monopile and caisson-type foundations [7].

There are no reliable guidelines or codes for the design of OWTs in seismic-prone areas, and the available codes for seismic design are mainly developed for classic structures. It is required to check and approve their practicality to offshore wind turbines [10]. Bhattacharya et al. (2021) explained the main steps and challenges in the seismic design of offshore wind turbines. They mentioned that OWTs are designed for a lifespan of 25 to 30 years. So, major seismic events are less probable but they are high-risk events. Nowadays, most standards use a 475-year return period (corresponding to a 10% probability of exceedance in 50 years) for the seismic design of OWTs. The amount of allowable tilt for OWT structures is one of the significant design factors of monopile-supported OWTs. The allowable tilt is specified as 0.5 to 0.75 degrees currently, and the effect of the P-delta moment initiates a rise in the foundation loads by the increase in tilt values. They noted that in sites with loose cohesionless soil deposits (e.g., sand), the occurrence of liquefaction is the most critical condition for the seismic design of a monopile foundation. The liquefaction phenomenon may lead to an excessive permanent tilting of the foundation [10]. Zheng et al. (2015) investigated that the joint effect of earthquake and sea-wave loads is essential to be considered for the structural response of OWT through experimental tests. They mentioned that further numerical FE analysis is required to investigate the effect of seismic and hydrodynamic loads on OWT by considering soil-structure interaction [11].

2. Numerical Modeling

2.1 Soil Medium Modeling

The constitutive model (Drucker-Prager $J2$ multi-surface plasticity model) is generally useful for representing the soil behavior for various cyclic loads [12-16]. This model is mostly determined to simulate cyclic liquefaction response in clean sand and silt numerically. Yang et al. (2003) and Elgamal et al. (2003) developed a multi-surface plasticity model at the University of California, San Diego (UCSD) for both sand and clay [16,17]. This model presents an elastic-plastic material and is implemented as Pressure Depend Multi Yield (PDMY) and Pressure Independ Multi Yield (PIMY) for both sand and clay, respectively, in the OpenSees framework [2]. Later PDMY material was modified by Yang et al. (2008) and defined as PDMY02 soil constitutive model in the OpenSees materials library [17]. After clarification of the model, laboratory and centrifuge test results were considered for calibration of soil parameters specified for the current model, and reasonable outcomes were achieved.

The plastic strain tensors (Q and P) in the specified soil constitutive model contain deviatoric and volumetric components and are presented in equation (1) [12].

$$Q = \acute{Q} + Q''\delta,\ P = \acute{P} + P''\delta \quad (1)$$

$\acute{Q}$ and $\acute{P}$ are deviatoric and Q'' and P'' are volumetric components of the plastic strain. Deviatoric plastic strain is defined based on the associative flow rule ($\acute{P} = \acute{Q}$) and $\acute{P}$ can be expressed as $\acute{Q}$ which is specified based on the yield surface. The volumetric plastic strain follows the non-associative flow rule ($P'' \neq Q''$) and P'' can be defined based on the phase transformation (PT) surface instead of the yield surface [18]. In liquefaction studies, non-associativity is of great importance. Contractive, and dilative behaviors of soil can be simulated based on this factor properly [2,16,19].

Contractive and dilative parameters are defined within the soil constitutive model and are inputted to the program as the terms c_1, c_2, c_3 (contraction) and d_1, d_2, and d_3 (dilation), respectively.

Table 1 presents the calibrated soil parameters for undrained sandy soil used in the current study [20]. For modeling the soil domain, the implemented 9-4 quad-up element (9 nodes for solid deformation and 4 corner nodes for pore water pressure determination) in the OpenSees platform is employed.

2.2 OWT System Modeling

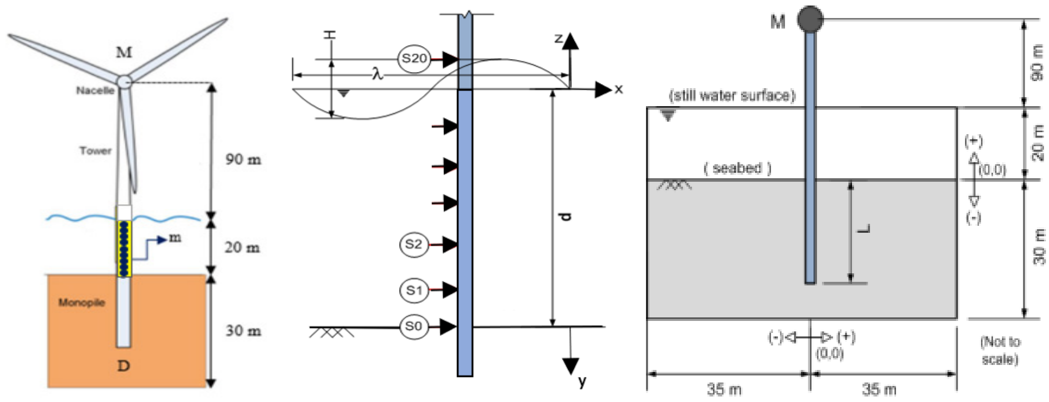
Figure 1 presents the schematic view of the monopile-supported OWT. The still sea-water depth is 20 m, the modeled soil depth is 30 m, and the tower's length above the sea level is 90 m. The mass of the rotor and nacelle is assumed as a lumped mass for the FE model [21,22]. Table 2 gives the properties of the OWT, foundation, and sea-water depth for the reference OWT (5-MW NREL).

After the foundation and superstructure are installed

Table 1. PDMY02 calibrated soil properties for Nevada sand (D_r = 63%).

Parameter	Description	Value	Units
G_r, K_r	Reference shear and bulk modulus	72.5E3, 193.6E3	[kPa]
ρ	Saturated unit weight	2	[ton/m^3]
e	Void ratio	0.66	-
φ	Soil friction angle	34.5	[°]
$φ_{PT}$	Soil phase transformation angle	26.5	[°]
P_r	Reference effective confinement pressure	101	[kPa]
n	Pressure dependent coefficient	0.5	-
Υ_{max}	Peak shear strain	0.1	-
c_1, c_2, c_3	Contraction parameters	0.04, 2.5, 0.2	-
d_1, d_2, d_3	Dilation parameters	0.07, 3, 0	-
NYS	No. of yield surfaces generated by model	20	-
liq_1, liq_2	Account for permanent shear strain (slip strain or cyclic mobility) in sloping ground	1, 0	[kPa]

Source: Karimi, Z., Dashti, S., 2016 [20].



(a) (b) (c)

Figure 1. (**a**) Schematic outlook for the reference OWT, (**b**) wave force application, and (**c**) numerical modeling of the problem for FE analysis.

Source: Fard, M.M., Erken, A., Erkmen, B., et al [22].

Table 2. Offshore wind turbine parameters.

h (m)	**d (m)**	**L (m)**	**D (m)**	**ρs (ton/m³)**
Tower's length above S.W.L	Water depth	MP embedment depth	MP diameter	Steel density
90	20	20	5	7.85
Es (GPa)	**Asec (m²)**	**Isec (m⁴)**	**IM (ton.m²)**	**M (ton)**
Steel modulus of elasticity	MP cross-section area	MP moment of inertia	Rotational inertia	Mass of rotor+ nacelle
200	0.777	2.38	2600	350

Note: Figure 1 illustrates the parameters in Table 2 and MP is monopile.

Source: Corciulo, et al.; Fard, et al. [21,22].

in the FE domain, the effect of the soil-pile interaction is captured by utilizing modified soil elements as the interface [22]. The wall thickness (t) is considered constant along the foundation and is considered as 1% of the monopile diameter [21]. The viscous boundary conditions for the base and lateral boundaries are defined by using the Lysmer-Kuhlemeyer dashpots (1969) [22,23]. They are used to damp out outgoing waves by reproducing radiation damping and are defined through zero-length elements in the OpenSees platform [2].

2.3 Load Characteristics

The sea-wave loads are estimated using linear wave theory (small-amplitude wave or Airy theory) and calculated based on the determination of the water particle velocity and acceleration using DNVGL [9,24] according to Morison's equation as given in Equation (2) and are applied to the tower of the OWT. For this purpose, the tower elevation between the seabed and the maximum water level was divided into twenty equal-length strips as shown in Figure 1b. The hydrodynamic force acting on each strip was calculated using Morison's theory.

$$f(t) = \rho(1 + C_A)A\dot{v} + \frac{1}{2}\rho C_D Dv|v| \quad (2)$$

where ρ is the mass density of the fluid, C_A is the added mass coefficient (C_A = C_M (inertia coefficient)-1) and C_D is the drag coefficient which is dependent on the Keulegan-Carpenter number and tower diameter calculated based on equations [9], A is the cross-sectional area, $\dot{v}$ is fluid particle acceleration, v is fluid particle velocity, and diameter of the cross-section is defined as D [22]. Based on the equations, C_A and C_D are considered as 0.6 and 1.0, respectively. The variations of surface elevation are considered based on linear wave theory [24].

The distributed sea-wave loads calculated for a 9-sec period and 20-m water depth, are applied to the OWT. The pile diameter and embedment depth are considered 5 m and 20 m respectively. The load applied to the seabed station as a sample for 5-m diameter tower is presented in Figure 2. For detailed information regarding the calculation of sea-wave load through Morison's theory [9,22,24] can be referred.

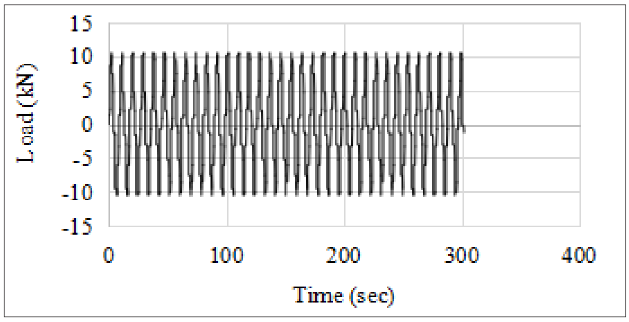


Figure 2. Sea-wave load at y = 0 (seabed (S0)).

Firstly, the sea-wave load is applied for 270 seconds (30 cycles) to the reference OWT system. After this step, the specified seismic motion, Kobe earthquake 1995, with 6.9 M_w (PGA = 0.28 g, PGV = 0.55 m/s, PGD = 0.15 m) is applied to the FE model (at the base of the model) while still, the sea-wave load remains in the system. The time history for the acceleration of the record is presented in Figure 3 (https://peer.berkeley.edu/peer-strong-ground-motion-databases) [25].

3. Finite Element Analysis and Results

In reality, the seismic acceleration and lateral loads of the OWT due to waves will be coupled together to affect the motion and pore water pressure of the soil around the monopile foundation. In the present study, for simplicity, the lateral loads from waves are simplified as a cyclic point force acting on the tower of the OWT [26]. In the present study, the sea-wave cyclic load is obtained based on the common approach used for offshore structures through Morison's theory, linear wave theory (small-amplitude wave or Airy theory) [9,24].

Dynamic excitation is applied as a displacement time history to the base of the soil domain, at the nodes which share equal degrees of freedom with the Lysmer-Kuhlemeyer (1969) dashpots by using the method of Joyner and Chen (1975) [27]. The displacement time history of the recorded ground motion by using the multi-excitation method defined in the program is applied to the system [2].

After applying the sea-wave and seismic loads and FE analysis of the system, the results for the deformation of the monopile foundation and the surrounding soil are obtained in specified locations. Later, the performance of the system during the sea-wave load and the coupled sea-wave and the seismic loads are compared and presented in graphs. In the first case, only the sea-wave load is applied to the OWT system for 302 seconds, and in the second case, the sea-wave load is exerted on the system for 270 seconds (30 cycles) (the time that the steady state situation is reached), and then the sea-wave and seismic loads are applied for 32 seconds (total 302 seconds load application). The variation in the effective vertical stress in the soil medium is a significant sign of excess pore water pressure generation, which can cause liquefaction in the soil domain.

Pile lateral displacement and rotation at the seabed surface are presented in Figure 4. The results for pile deformation develop by reaching the seabed surface, and the highest rates are achieved at the seabed surface. The monopile deformation is more affected during the sea-wave load application, and seismic motion causes a reduction in the deformation values. The values are smaller than the limits defined in DNVGL-ST-0437 [9], and the results obtained for seabed are presented.

Soil effective vertical stress and shear strain are good representatives for the evaluation of pore water pressure generation and liquefaction possibility in the soil domain. Increasing soil shear strain and decreasing effective vertical stress (to reach zero), lead to the increase in excess pore water pressure which makes liquefaction possible (r_u = 1.0). Figures 5 and 6 illustrate the soil response in time as, soil shear strain, effective vertical stress, and excess pore water pressure ratio. The response is presented at two specific locations around the pile foundation. The results are evaluated with a 5 m distance from the pile location to avoid interface effects on the outcomes.

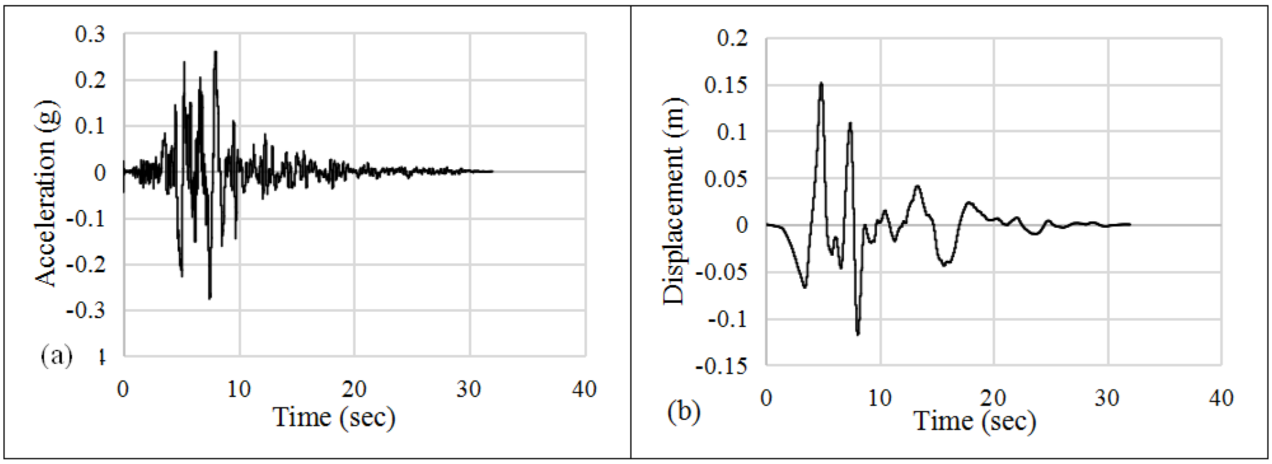


Figure 3. (**a**) Acceleration, and (**b**) displacement time histories of the recorded motion.

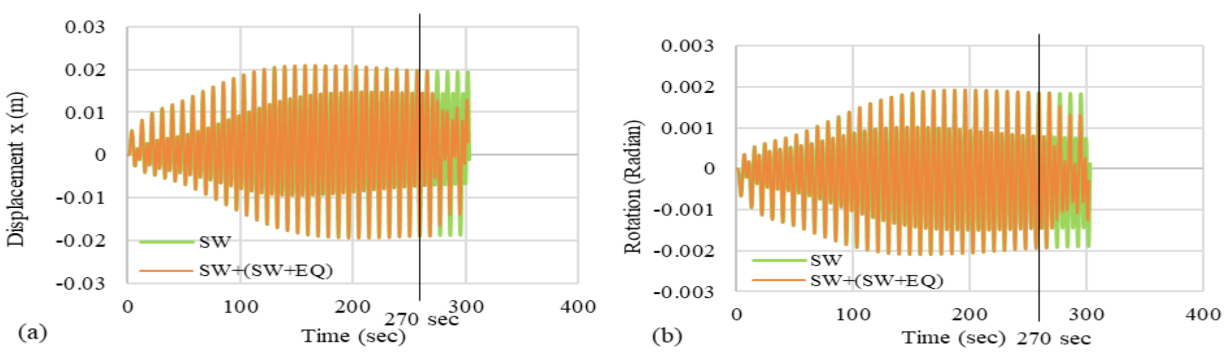


Figure 4. Response at the seabed: (**a**) pile lateral displacement, and (**b**) pile rotation.

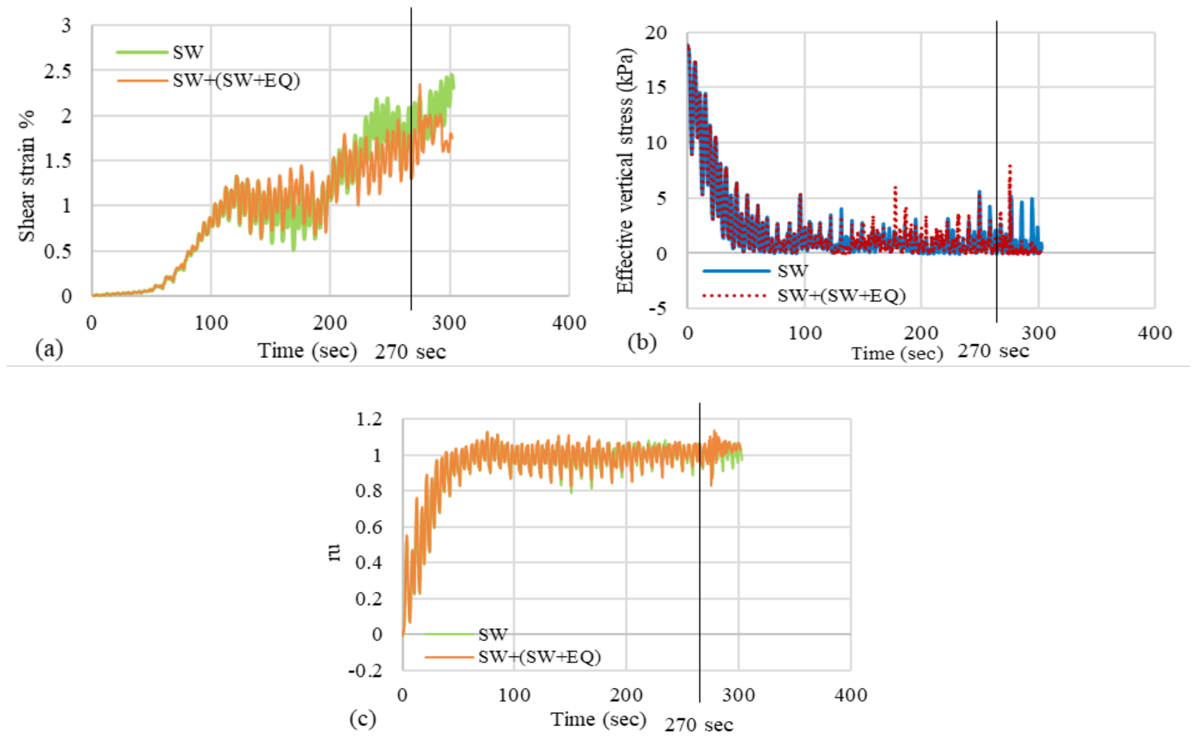


Figure 5. At y = –2 m, and x = +5 m, (**a**) soil shear strain, (**b**) soil effective vertical stress, and (**c**) soil excess pore water pressure ratio.

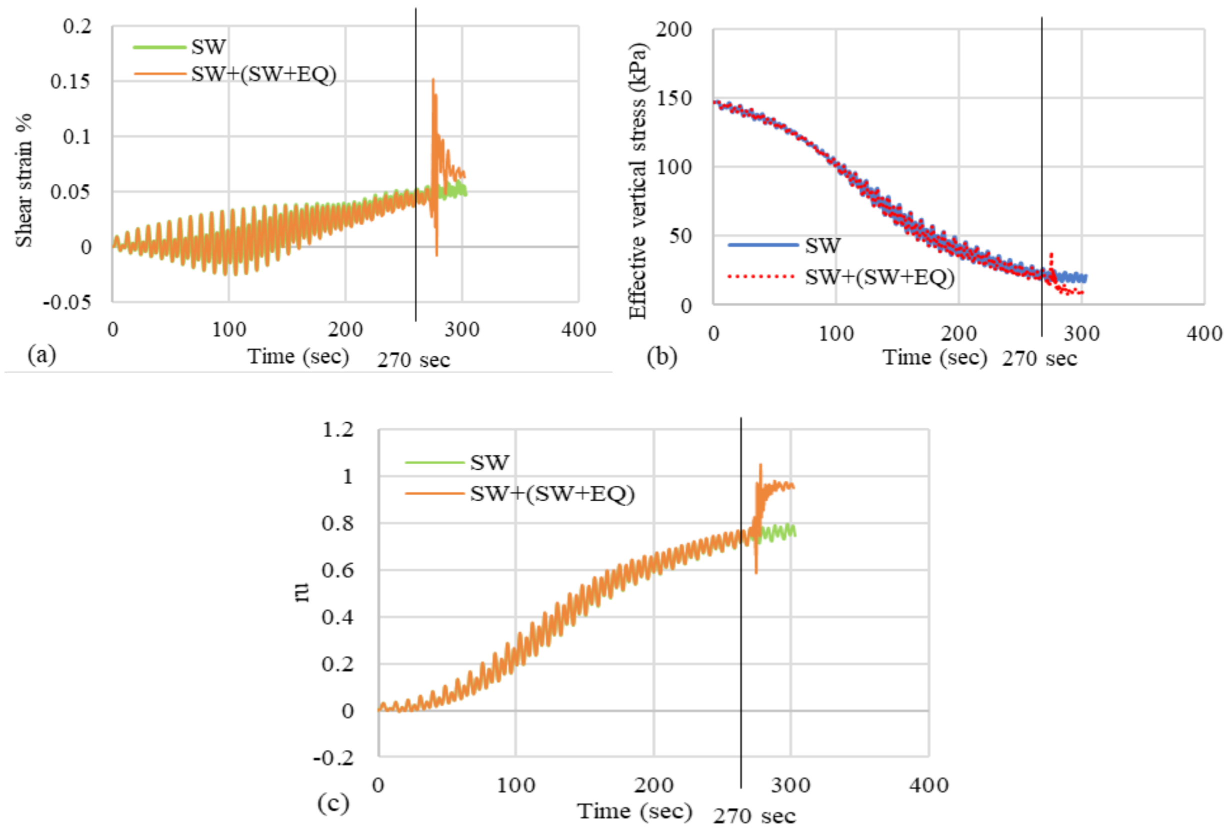


Figure 6. At y = –15 m, and x = +5 m, (**a**) soil shear strain, (**b**) soil effective vertical stress, and (**c**) soil excess pore water pressure ratio.

4. Discussion

Investigating the results during the sole effect of sea-wave and the simultaneous effect of seismic and sea-wave loads shows that the response of the system goes on smoothly during the sea-wave load application while the seismic motion has a sudden effect on the response of the OWT system. The effect of seismic motion is mainly obvious on the soil shear strain values and they increase significantly during the application of seismic motion. By increasing depth from the seabed, the time required for effective vertical stress to reach zero increases, and this delays liquefaction. At 15 m below the seabed surface, the increase in soil shear strain over time is more obvious specifically during seismic motion application to the system. There is a sudden increase in the excess pore water pressure ratio and it reaches 1.0. This shows the effect of seismic motion on possible liquefaction for deeper parts of the seabed surface. Based on the presented graphs, at –2 m below the seabed surface, in the first 40 seconds of sea-wave load application, the effective vertical stress reaches zero, which makes the excess pore water pressure ratio proceed to 1.0, and soil liquefies. The duration required for liquefaction increases to 270 seconds while the depth increases to –15 m from the seabed surface. However, the maximum rate for the excess pore water pressure ratio is 0.75 during individual sea-wave load application to the OWT system.

By comparing the results for the response of soil and monopile, it is apparent that the seismic motion is more effective on the behavior of the soil surrounding the monopile. This results in a rise in the soil response and makes deeper locations of the soil medium liquefy.

5. Conclusions

In this research, the performance of OWT is studied through the 2D fully coupled *u-p* dynamic analysis by considering soil-monopile-structure interaction. In this regard, the effects of the coupled application of sea-wave and seismic loads on the performance of the monopile and the soil surrounding the foundation are investigated. The results are obtained in some specified locations based on the earlier analyses for this research and presented in the selected figures. In this analysis, the sea-wave load is applied for 270 seconds (30 cycles, to reach a steady state) to the OWT system based on a 5-m diameter monopile with 20 m embedment depth. Later the specified earthquake motion is exerted on the system while still, the sea-wave load remains in the system.

In reality, the combination of seismic motion and cyclic sea-wave load is not rational and the load combination performed in the current study is for simplification, and a closer to reality load combination is advised. According to the presented figures, the effective vertical stress approaches zero at about –15 m from the seabed surface. This behavior confirms the loss of strength in the soil medium, which is a primary sign of the appearance of liquefaction. The rise in the soil shear strain values and the drop in the rates for the effective vertical stress during the coupled effect of the sea-wave and seismic motion makes an increase in the excess pore water pressure ratio. This response makes the excess pore water pressure ratio approaches 1.0 over time, and liquefaction happens as a result. By comparing the results obtained through the sole effect of the sea-wave load and the coupled application of sea-wave and seismic loads to the system, it is derived that the seismic motion is more effective on the response of the soil surrounding the monopile. Applying seismic motion to the OWT system leads to the possibility of liquefaction for deeper locations of the soil medium where liquefaction is not the case during the sole effect of sea-wave loads. This is an ongoing study and the obtained results confirm the significant role of the seismic motion on the dynamic response of the soil surrounding the OWT. When it comes to sandy soil and seismic regions, liquefaction is an aspect that plays a significant role. It may cause excessive tilt and permanent rotation which affects the serviceability and stability of the OWTs and this effect should not be ignored in the design of OWTs in seismic-prone areas.

Author Contributions

Conceptualization, M.M.F., A.E., A.A., Methodology, M.M.F., A.E., A.A., Software and Analysis, M.M.F., Investigation, M.M.F., Validation, M.M.F., A.E., A.A., Writing-original draft preparation, M.M.F., Writing Review and Editing, A.A., A.E., M.M.F.

Conflict of Interest

The authors declare no conflicts of interest.

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