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Research Paper

Numerical modeling of the seismic performance of monopile supported wind turbines in sandy soils susceptible to liquefaction.

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Abstract

Since 1980, as wind farms have moved from coastal to offshore areas, the wind energy industry has been completely transformed which in turn has led to the increase in the construction of wind turbines. On the other hand, harsher offshore environmental conditions have led to larger lateral loads and anchorages applied to the wind turbines and specifically to their piles than other coastal and offshore structures. Thus, more solid piles are required to ensure proper rigidity and bearing capacity. Liquefaction is one of the most important seismic hazards through which various damages caused to different parts of wind turbines. In order to develop coastal and offshore structures in Iran, a study of liquefaction is of great importance due in part to the high risk of seismicity. In this study, the effect of liquefaction on seismic response of offshore wind turbines is examined taking advantage of a finite element model. To this end, all analyzes have been carried out in both occurrence and non-occurrence of the liquefaction, so that by comparing these two modes, the mechanisms affecting the seismic behavior of wind turbines are understood. As depth increases, the possibility of liquefaction is reduced due to higher pressure. Liquefaction is considered to a depth of 20 m and structural behavior is evaluated based on the level of seismic hazard, the thickness of the susceptible layers, soil compaction, the non-fluidizing top layer, the gradient of the earth, the thickness of the monopole, the dimensions of the wind turbine and different soil layering conditions. According to the mentioned factors, a comprehensive and parametric study of the behavior of wind turbines in seismic zones, and in different loading conditions, pile diameters and soil layering is carried out in soils prone to liquefaction. Since analyzes are performed in both occurrence and non-occurrence of the liquefaction, the number of analyzes and computational cost in this research becomes enormous. Therefore, there is a need for a highly effective software and a practical modeling method that will allow for this comprehensive study. Open Sees software and beam on nonlinear Winkler foundation approach are used to model the soil-pile-structure interaction. The minor differences observed in the laboratory values compared to the numerically calculated ones may refer to the fact that the chamber is not modeled. In the bottom layer, as the depth decreases, the elastic response spectra record larger values which are due to the resonance in the structure.

Keywords: *Use about five key words or phrases in alphabetical order; Separated by Semicolon.*

1. Introduction

The remarkable harvest of offshore wind energy as a major renewable energy source to replace fossil fuels has grown dramatically in recent decades. The wind turbines are relatively new structures built to produce electricity from winds for a lifespan of 25 to 30 years [1-3]. The interactions between seabed soil and offshore wind turbine foundations make the nonlinear modeling of these structures a complex process and it should be carried out with realistic and accurate assumptions. The high cost of designing and implementing these structures, as well as their important role in the energy industry and subsequently the development of a country, has led researchers to always seek to improve the performance of these structures in different conditions. In seismic hazard zones, a wide range of geotechnical hazards such as faulting, liquefaction, seafloor landslides, tsunami, mud volcanoes, etc. can affect the performance of these structures. One of the most important seismic hazards is liquefaction and is known by two 1964 earthquakes in Alaska and in Niigata, Japan. Liquefaction is one of the major causes of damage to soils and foundations during earthquakes. It's one of the most important aspects of seismic research and the design of foundations[4]. In coastal and offshore areas, deep foundations are used to reduce the likelihood of destructive effects of liquefaction such as unacceptable settlements and displacements. However, the use of deep foundations does not necessarily prevent the damages caused by excessive lateral displacements.

By moving wind turbines to offshore areas, the demand for the safety and sustainability of their foundation is intensified. Considering that the cost of a wind turbine foundation accounts for an average of 25% to 34% of its total capital, a study on the suitable foundation in the offshore wind energy industry is important [5, 6]. More than 75% of wind turbines in Europe have been built on monopolies. This popularity is achieved due in part to simple design, relatively low production costs, no need to prepare the seabed during installation, high energy absorption and the independence of their maintenance costs. The use of monopiles as a foundation for wind turbines that are to be installed at depths of less than 30 meters is very common for sandy and clay soils, but there are a lot of technical and economic limitations for their use at depths greater than 30 meters [7], [8].

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The accurate and actual assessment of loads applied to wind turbines seems to be difficult in coastal areas. This difficulty is due to the nonlinear kinematic wave, the variation of wave curves, the wave flow disturbance, the effect of the structure on the wave field, the probability of the vortex flow generation, and the resonance in the wind turbine. In order to optimize the design of the wind turbine, adequate information is needed on the actual conditions of the sea including irregular waves and directional distribution [7]. In seismic hazard zones such as the east coast of China and the west coast of the United States, the seismic load is decisive while designing. In such areas, the earthquake center may be located in the coast or remote areas offshore and threatens the safety of wind turbines [9]. Offshore monopiles are distinct from other piles due to their relatively large geometric dimensions and enduring more dynamic environment [10]. Another aspect in the design of offshore wind turbine foundations is that the weight of the structure itself is relatively low compared to other offshore structures. Hence, lateral loads which are somewhat similar to the gravity loads are of particular importance in the design. For offshore wind turbines, the ratio of lateral load to vertical load is 1.4 to 2.6, while this ratio is 0.25 for the oil platforms [7]. Despite the vast investigations conducted on the behavior of piles in static loading, there are many uncertainties about how piles actually behave due to the complexity of their seismic behavior [11].

Meymand has presented four approaches including beam on elastic foundation, elastic continuum, finite element and boundary element methods, and beam on nonlinear Winkler foundation to find the effects of soil-pile-structure interaction on the seismic response of structures [11]. Maheshwari and Truman, by presenting a full 3D model of soil-pile-structure examined the effect of taking into account the nonlinear behavior of soil in the seismic response of the structure. In their Study, an advanced plastic model was developed using HISS element to express the soil behavior. Their findings suggest that consideration of nonlinear behavior will have a significant effect on the structure response [12].

For the first time, Casagrande introduced the liquefaction of sand and its peripheral effects. He used the critical porosity principle to show the conditions that trigger liquefaction. He also reported that the sand having a porosity greater than the critical porosity tends to reduce its size when it's under shear. This condition may increase the pore pressure in the absence of drainage in which there is no possibility of changing the volume. This increase may be large enough to account for the occurrence of liquefaction. On the contrary, the sand having a porosity less than the critical porosity tends to expand while exposed to shear. This condition may decline pore water pressure in the absence of drainage. As a result, the effective stress is increased which in turn may lead to the enhancement of resistance and stability.

Battachary introduced the buckling failure mechanism for piles installed in soils susceptible to liquefaction. If a pile is placed in both susceptible and non-susceptible to liquefaction layers, there is a possibility of buckling [13]. Abdoun et al. have investigated the load-deformation behavior of laterally loaded pile groups and individual piles by conducting a centrifuge test [14]. In their study, they have also considered the effect of the non-fluidizing top layer on the stress distribution of the piles, the effect of the anchor force induced into the piles as well as the coincidence of kinematic and inertial loads. They have reported that in the presence of non-fluidizing top layer, its reaction force has a remarkable effect on the bending moment. They have also announced that in all cases, the maximum bending moment occurs at the boundary between the susceptible and non-susceptible to liquefaction layers.

In order to understand the nonlinear dynamic behavior of the soil-pile-structure system in soils susceptible to liquefaction, Kamijo [15] et al., taking advantage of earthquakes triggered by explosion, examined the seismic behavior of a 2×2 pile group. Their reports suggest that the maximum bending moment occurs at the pile-to-cap connection. They have also reported that under complete liquefaction, accelerations transmitted to the ground surface are decreased. They have further stated that accelerations in the structure are reduced during liquefaction, and therefore the inertia forces applied to the piles are declined. Since the soil compaction is decreased, the natural frequency of the soil-pile-structure system is subsequently reduced with the development of liquefaction.

Motamed et al. conducted a small scale shaking table test to evaluate the behavior of pile groups 3×3, 6×6 and 11×11 under a 1g gravity field. They have examined parameters such as amplitude and input excitation frequency, relative compaction of soil and non-fluidizing top layer thickness. They observed that the greater the non-fluidizing top layer thickness is, the higher the moment is [16].

Ghosh et al. [17] studied seabed liquefaction and suggested that in the occurrence of liquefaction, the soil column filters the spectral response in lower periods. Sottile et al. investigated the liquefaction of a tripod platform offshore Egypt. In their research, the beam on nonlinear Winkler foundation approach as well as Winkler spring model were used. They have reported that the reduction in the axial bearing does not usually have much effect on the design of the pile. The horizontal motion of the non-fluidizing top layer and the lateral expansion cause significant forces applied to the pile.

Ku et al. [18] studied a jacket foundation wind turbine load-bearing capacity offshore Taiwan. An analysis of the effective stress in soil along with considering the increase in water pressure was performed. In their research, a numerical method was developed in order to evaluate the design of offshore turbines in sandy and clay soils. Katsanos et al. [19] have compared the published researches related to seismic analysis, design, and evaluation of wind turbines. They have reported that foundations installed in seismic zones could be vulnerable to damage. Sam Austin et al. Have studied the effect of soil-pile-structure interaction on the seismicity response of wind turbines. Four types of foundation including 1) mat foundation; 2) monopile; 3) pile group; and 4) mat foundation anchored to the seabed are discussed. They have reported that the natural frequencies obtained from the finite element model are well suited to the frequencies recommended for a 65 kW turbine structure.

Abhinav et al. [20] have investigated the dynamic response of a jacket foundation for offshore wind turbines under different soil conditions, and aerodynamic and hydrodynamic loads. Their results have shown that the dynamic response of the turbine is significantly depended on the soil parameters.

2. The proposed model and the assumptions applied

2.1. Materials Behavior Models

In this study, diverse behavior models have been used to model steel, far-field sandy and clay soils, and near-field sandy and clay soils (interaction area).

2.1.1. Steel behavior model

Structural members and piles are considered to be made of steel. Steel 02 material in the Opensees library is used to define steel. The stress-strain diagram of steel is schematically illustrated in figure 3.1.

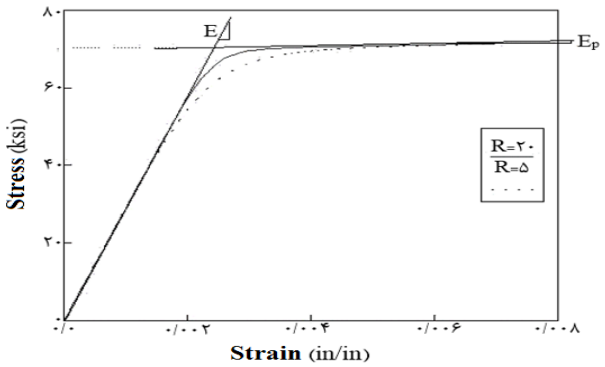


Fig. 2.1: Stress-Strain Diagram of Steel 02 [21].

2.1.2. Far-field clay soil behavior model

Pressure independent multi-yield material was used to define clay soil. This material is applied for the simulation of the one-way and periodic response of materials whose shear behavior is not susceptible to change in confinement. The behavior of the material is linear elastic while the gravity load is applied and during the dynamic analysis, by updating the behavior, the strain-strain response of the material becomes elastoplastic. For this material, Plasticity is defined based on a multi-level concept. In order to make use of the material, it is only necessary to properly define the specific gravity, shear modulus, Poisson ratio and bulk modulus, and no specific calibration is required.

2.1.3. Far-field sandy soil behavior model

Choosing a proper behavior model for sandy soil is of high importance. Models in which more sand characteristics are incorporated are usually able to predict more accurate results. The applied behavior model should be able to model the main characteristics of the saturated sand during earthquakes and under conditions of initial shear stress and all-around stresses in a wide range of relative densities.
In this research, the considered behavior model for sand is based on studies conducted by Prevost et al. [22] in which a multi-surface approach is suggested to simulate the cyclic reciprocating behavior of soils. This behavior model has been modified by Elgamal et al. [27] to consider the effects of liquefaction and in the OpenSees library is defined as Pressure- Independent Multi Yield Material. This is an elasto-plastic material to simulate pressure-sensitive materials under general loading modes. The material is known by volume expansion and contraction due to the shear stress and liquefaction which typically occurs in sands and silts during one-dimensional or periodic loading. The behavior of the material is linear elastic while the gravity load is applied and during the dynamic analysis, by updating the behavior, the strain-strain response of the material becomes elastoplastic
These behavior models are calibrated for 4 types of sandy soils with different standard penetration resistances (N-values) and 3 types of clay soils with different undrained shear strengths. Since analyzes are performed in both occurrence and non-occurrence of the liquefaction, some changes must be made in the behavior model. In the first step, the coefficient of permeability for the sandy soil is assumed to be a high value (1 m/s). In the next step, all the contraction and dilatation parameters used in the behavior model are considered to be zero. In this way, the rising trend in the pore water pressure is prevented while the shear modulus and the hysteresis damping constant are not affected.

2.1.4. The near-field sandy and clay soils behavior models

Two models of lateral and axial springs have been used to model the soil-pile-structure interaction area.

2.1.4.1. Lateral springs

In order to model the behavior of soil subjected to lateral loads, a combination of one-dimensional materials and zero-length elements have been used. In this research, two types of one-dimensional materials are considered for the lateral springs as follows:
Pysimple1: this kind of spring is used to define the behavior of clay and sandy soils. In this study, it is used to define the clay soil around the piles.
PyLiq1: this kind of spring is used to define the behavior of sandy soils (occurrence of liquefaction) subjected to lateral loads.

2.1.4.2. Axial springs

In order to model the behavior of soil subjected to axial loads, a combination of one-dimensional materials and zero-length elements are used as well. In this study, three types of one-dimensional materials are considered for the axial springs as follows:
Tzsimple1: this material is used to model the frictional resistance of the pile and behavior of soil subjected to axial loads.
TzLiq1: this material is used to model the frictional resistance of the pile and soil behavior against the axial loads in the case of liquefaction.
Qzsimple1: this material is used to model the ultimate resistance of the pile.
It should be noted that TzLiq1 is a one-dimensional material of the conventional curves of t-z, which considers liquefaction effects similar to those of the lateral spring PyLiq1. The API code has been used to assign specifications to the P-y, T-z, and Q-z springs.

2.2. Element used in modeling

The elements used for modeling the soil-pile-structure system include: 1) nonlinear beam-column elements for jacket structures and piles; 2) elastic beam-column elements for deck structural members; 3) zero-length elements to define Winkler springs in the interaction area and (4) nine-node quadrilateral elements for the plane strain.

2.3. Applied formulation for the sandy soil

Nine-node quadrilateral elements for plane strain are used to model two-dimensional soil network. These elements are used to simulate the simultaneous solid-fluid reactions based on the Biot's theory. Each of the four corners of the element has three degrees of freedom.
Soil-pile-structure interaction seems to be very difficult to understand. Therefore, simplified models attempting to capture the main aspects of it have been developed. Among these models, those based on the Winkler nonlinear springs are known as the P-y method and are often used to investigate the soil-pile-structure interaction. A careful study of the interaction area requires a comprehensive finite element model which would be complex and time consuming for large structures such as offshore platforms. It is also necessary to have precise contact elements that consider the soil-pile-structure interaction effects. As an alternative method, the beam on nonlinear Winkler foundation approach will be helpful in such cases.

2.4. The beam on nonlinear winkler foundation approach assumptions and validating soil-pile-structure interaction model

The results given in the Wilson study have been used to validate the soil-pile-structure interaction model. These experiments are divided into five groups based on the type of soil and the input record as follows:

1) In the first group, namely CSP1, the soil consists of two sand layers. The top layer is 6.1 meters thick with a density of 55% and the bottom layer is 11.4 meters thick made from the sand with a density of 80%.
2) The second group which is called CSP2 consists of a 6.1 meters thick top sand layer with a density of 35% to 40% and a 6.1 meters thick bottom sand layer with a density of 80%. In this case, there is a high possibility of liquefaction due to the low density of surface sandy soil.
3) The third category, namely CSP3, includes models in which the top and bottom layers are considered to be sand with a density of 55% and 80%, respectively.

4, and 5) for these groups, CSP4 and CSP5, their top layers are made of a 6.1 meters thick consolidated soil, and their bottom layers are made of sand with a density of 80%. It is worth noting that the results of these experiments are scaled to the actual dimensions; hence, the actual dimensions of the soil-pile-structure system have been used in the finite element model.

2.4.1. Finite element model description

The finite element model presented in this study includes soil elements for recording the response of far-field soil, non-linear beam-column elements, and the springs. Nonlinear time history analysis of soil-pile-structure system was conducted taking advantage of OpenSees and two-dimensional plane strain model.

2.4.1.1. Damping

Rayleigh damping coefficients have been used to define material damping properties. For this purpose, the solution domain is divided into structural and soil domains and the coefficients are applied separately to each of them. It is worth noting that in the soil domain, a large share of energy is dissipated through the hysteresis damping based on the Elgamal [23] elastoplastic behavior model. Hence, in order to retain the stability of the numerical solution in very small strains, a small amount of Rayleigh damping is applied to the soil network. The radiation damping is obtained using the coefficients proposed by Burger from the following equation:

Eq.3.1 C_L=4Bρv_s

Where:
c_L: Radiation damping of soil
B: Pile diameter
ρ: Soil density and
v_s: Shear wave velocity of soil

2.4.1.2. Boundary conditions in the two-dimensional mesh of the soil

The soil element points are tied at the bottom of the soil column for both transition state degrees of freedom and the analysis is performed with the use of a uniform excitation pattern. This assumption reflects the fact that all seismic wave energy is dissipated in the soil environment. Due to the high rigidity of the chamber compared to the soil inside, this assumption seems to be logical [24]. The thickness of the soil layer is considered to be very high. In such a way the soil elements are extremely heavy so that the pile doesn't have any kinematic effect on the two-dimensional mesh of the soil which is intended to model the behavior of far-field soil.
The thickness of the soil members is proportional to the shear wavelength of the softest layer. In order to record the shear wave propagation through the soil network, at least four elements should be located along the shortest wavelength. The maximum thickness of the soil elements is calculated with the use of following equation [33]:
Eq. 3.2.
Where:

$$h_{max} = \frac{v_s}{8f_{max}}$$

h_{max}: Maximum height (m)
f_{max}: The maximum frequency content of ground motions (Hz)
v_s: shear wave velocity in the soil layer (m/s)

2.4.1.3. Modeling soil-pile-structure interaction through beam on nonlinear Winkler foundation approach

In case of direct connection of the points of the two-dimensional network of the soil into the points of the nonlinear beam-column elements of the pile by soil springs, fictitious forces may appear in the pile [25], [26]. This is happened due to the settlement of the soil resulting from the gravity load, but in reality, it is assumed that the soil settlement is a time-consuming process and the pile enters the soil after the settlement. A relative distance is created between the points of the pile and the nearest point of the two-dimensional mesh of the soil with respect to gridding of the soil and the coordinates of the points of the pile elements. This item is more likely to appear in oblique piles and irregular two-dimensional square grids. Therefore, in these cases, zero-length elements cannot be used to connect these points with different coordinates. Hence, based on the reasons stated above, the method of defining virtual points in the same coordinates with the nearest point of the two-dimensional soil network is used.
The virtual points are defined in two dimensions and three transitional degrees of freedom with the same coordinates compared to the nearest points of the soil network to the points of the pile. These points are tied in their rotational degrees of freedom. Since these points are expected to apply vertical and horizontal displacement of the two-dimensional mesh of the soil through the Winkler springs to the pile elements, there should be no rotation applied to the points during the analysis. If the rotational degrees of freedom are not tied for points, the finite element model will be unstable. Soil springs are defined through zero-length elements and connecting points of the two-dimensional network of the soil to the virtual points in the same coordinates. While the gravity load is applied, these virtual points are tied at the transitional degrees of freedom in both horizontal and vertical directions to the points of beam-column elements of the pile. Consequently, the gravity displacements of the soil are not applied to the interaction zone and the structure which in turn may prevent the creation of fictitious forces. The same virtual points are used to define other soil springs.

2.4.1.4. The solving method

Large numerical damping values and time intervals are taken into account when the gravity loading is applied. As large numerical damping values are assigned, the nonlinear transitional analysis simulates the quasi-static loading conditions [27]. Therefore, with the assignment of values of 1.5 and 1 for the Newmark integration parameters (i.e. β and γ), the analysis process is greatly dampened. Dynamic analyzes are carried out taking advantage of the average acceleration method. Transitional analysis with a variable time interval automatically reduces the time step in case of non-convergence of the solution. It should be noted that transitional analyzes have been used for both gravity and dynamic loading in order to prevent numerical problems caused by simultaneous static and transitional analyzes. The analysis parameters used in this study are listed in Table 3.1.

Table 5.1: Analysis Engine Parameters

Dynamic analysis	Static analysis	Analysis options
Plain	Plain	Constraints
RCM	RCM	Counter
ProfileSPD	ProfileSPD	Analysis system
NormDispIncr	NormDispIncr	Test
KrylovNewton	KrylovNewton	Algorithm
Newmark (γ=0.5,β=0.25)		
Newmark (γ=1.5,β=1)		

2.4.1.5. The analysis process

The analysis was performed according to the following steps
Step one: in this step, the geometry of the finite element model is defined. This includes the geometry of the soil column, pile, and the virtual points. Several fiber elements are assigned while non-linear beam-column elements are defined for the piles. Nonlinear springs of soil are created and defined in the points of the two-dimensional network of the soil and virtual points described in the same coordinates by zero-length elements. The virtual points are not tied to the pile points.
Step two: An elastic analysis is used to simulate underlying conditions. This step is performed as a transitional analysis with very large time intervals to obtain the real hydrostatic pressure.
Step three: Soil behavior is altered and it is assumed as a nonlinear material. Several transitional dynamic analyzes are conducted to adjust the initial values. For a better convergence process, these steps are performed with smaller intervals.
Step four: The virtual points of the soil and the points of the pile elements are tied together at this step. Since the structural system

would not participate in the gravity load analysis of the two-dimensional network of soil, and the soil displacement resulted from the soil settlement are unrealistically applied to the soil springs and the structure, the fictitious forces are prevented from being created. Several transitional analyzes are performed apply the dead load of the structure.

Step five: PyLiq1 and TzLiq1 are updated. These materials consider the average effective stress of the two dedicated elements of soil as a pre-consolidation stress prior to undrained loading. From this step onwards and throughout the time history analysis, the behavior of these materials will depend on the effective stresses of the soil and the pore water pressure.

Step six: In this step, transitional time history analysis with variable time intervals is used for seismic analysis of the finite element model for the soil-pile-structure system.

3. Results and discussion

In order to evaluate the finite element model for the soil-pile-structure system, the calculated dynamic responses are compared with the in the centrifuge test results reported by Abdoun et al [14]. Responses are compared in two general groups. In the first group, the accuracy of the far-field soil model including the elastic response of the soil at different depths (Figures 4.4 to 4.8) and the increasing trend in the pore water pressure in the two-dimensional soil network (Figures 4.9 and 4.10) is evaluated. In the second group, the structure acceleration time history response (Fig. 4.11), the elastic response of the structure (Fig. 4.12) and the time history of the horizontal displacement of the structure at different depths (Fig. 4.13) is investigated.

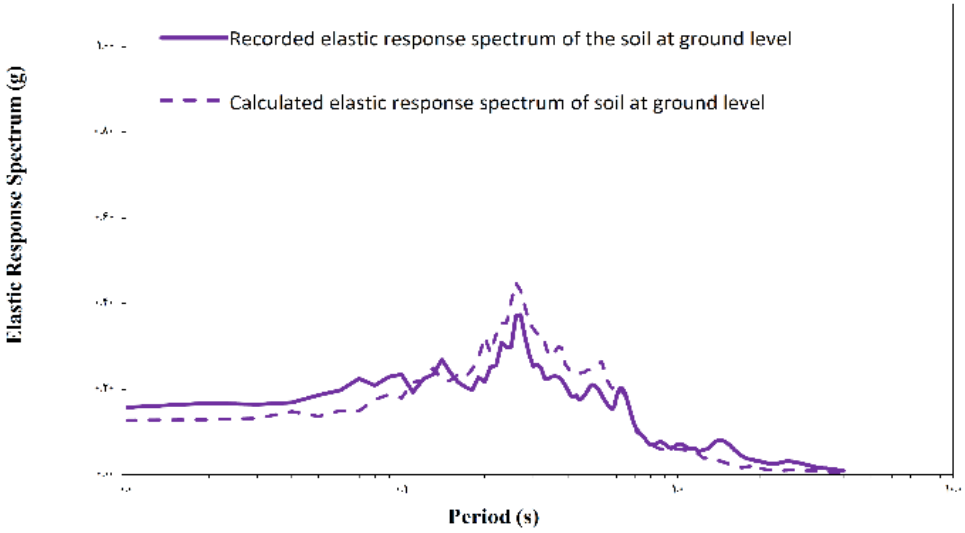


Fig. 3.1: Elastic Response of the Soil at the Ground Level.

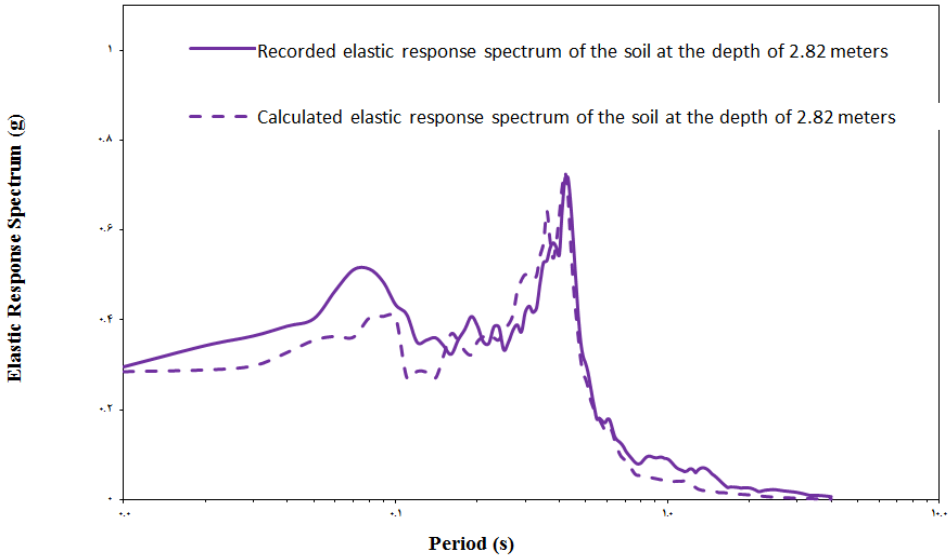


Fig. 3.2: Elastic Response of the Soil at the Depth of 2.82 M.

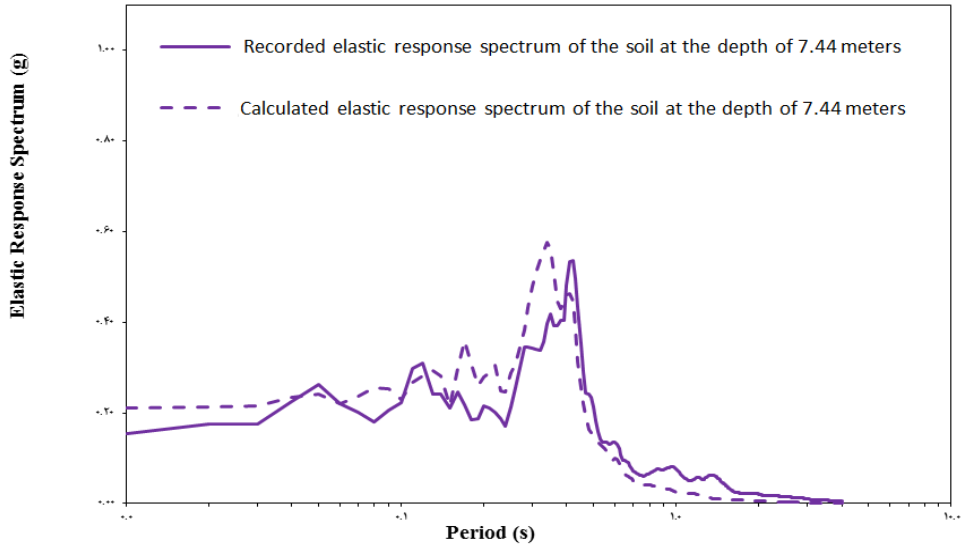


Fig. 3.3: Elastic Response of the Soil at the Depth of 7.44 m.

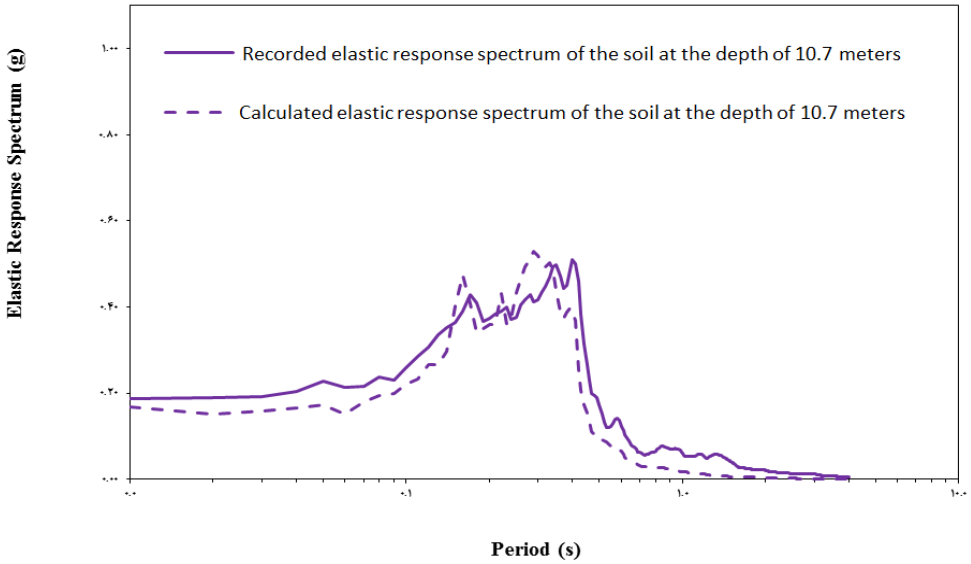


Fig. 3.4: Elastic Response of the Soil at the Depth of 10.7 m.

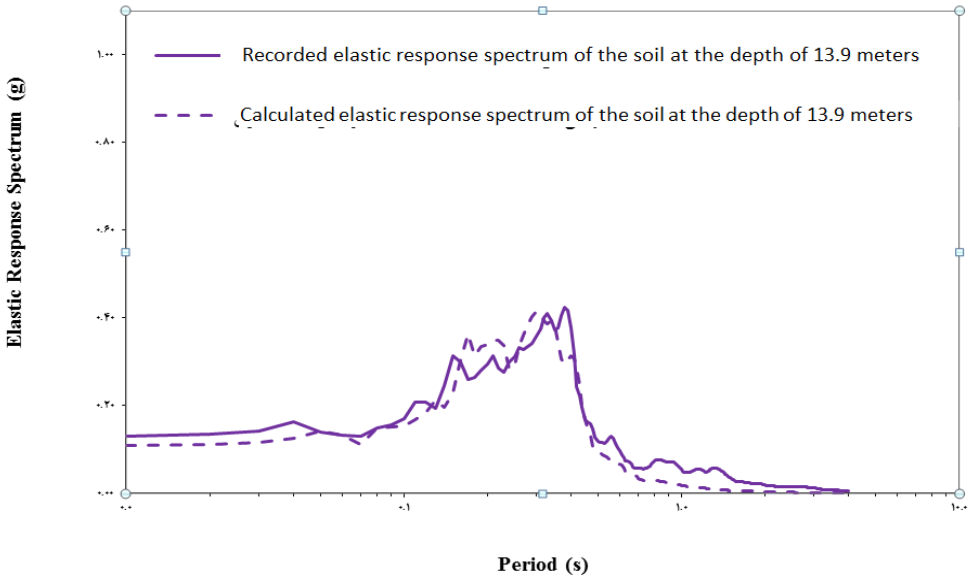


Fig. 3 5: Elastic Response of the Soil at the Depth of 13.9 m.

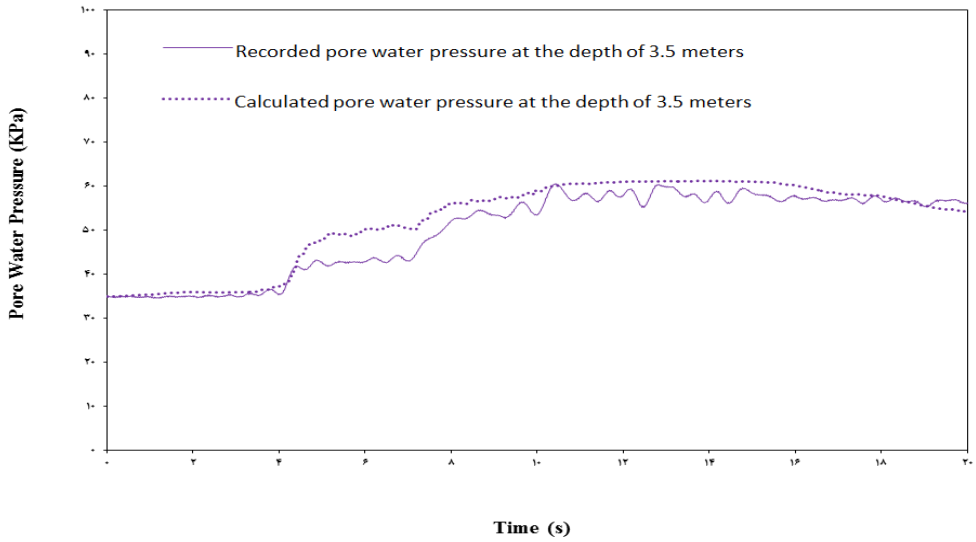


Fig. 3.6: Pore Water Pressure at the Depth of 3.5 m.

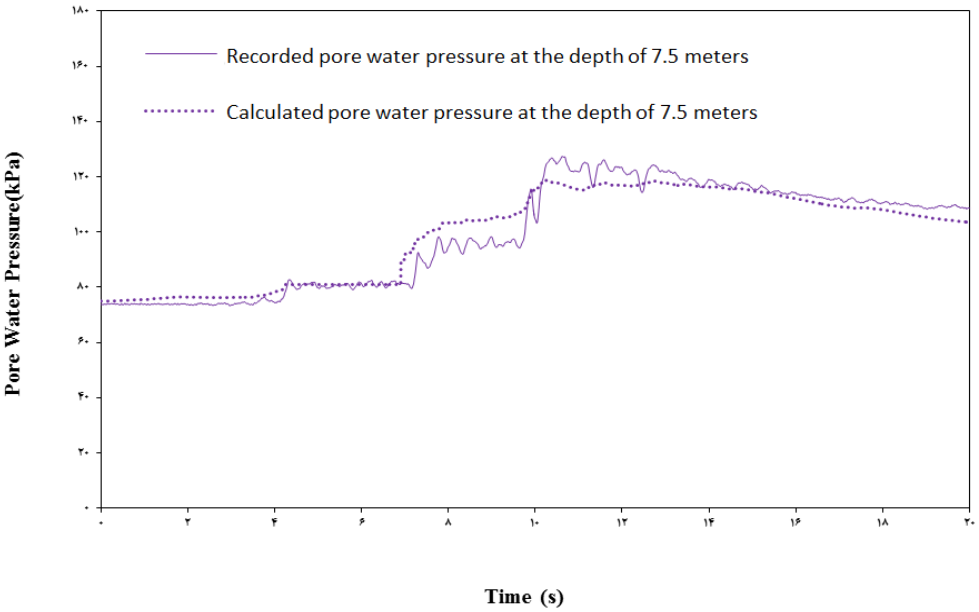


Fig. 3.7: Pore Water Pressure at the Depth of 7.5 m.

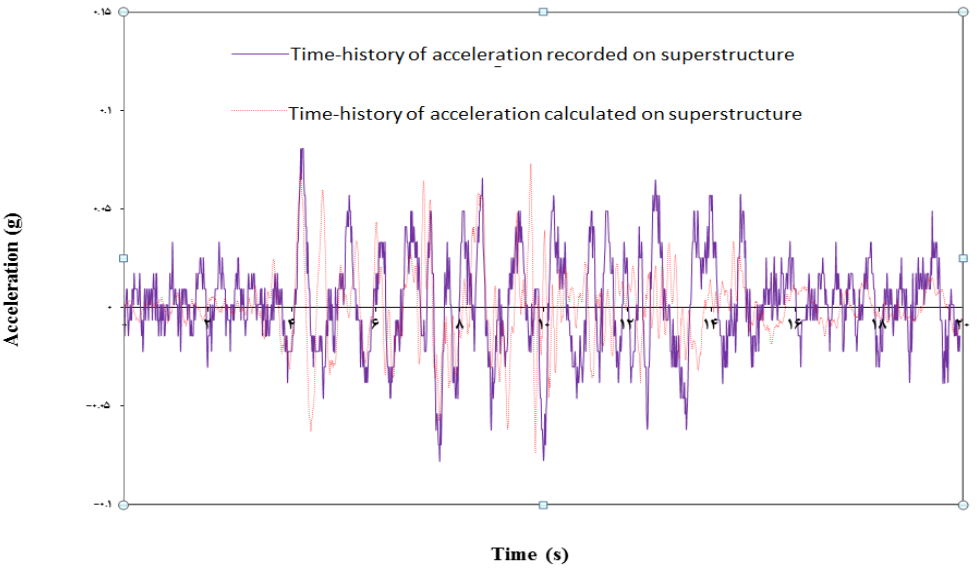


Fig. 3.8: Acceleration Time-History of the Structure.

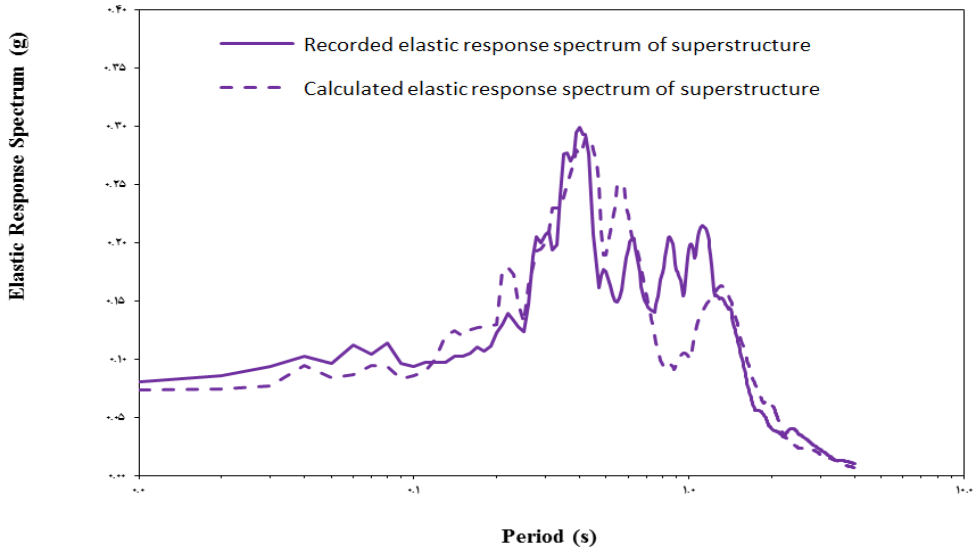


Fig. 3.9: Elastic Response Spectrum of the Structure.

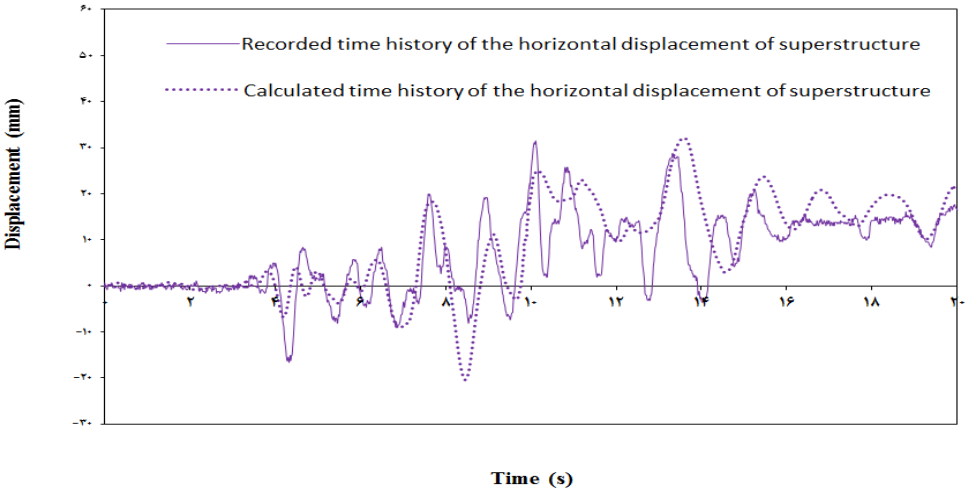


Fig. 3.10: Time History of the Horizontal Displacement of the Structure.

The general shape of the elastic response spectrum at different depths indicates the appropriate accuracy of the finite element model. The minor differences observed in the laboratory values and the numerically calculated ones may refer to the fact that the chamber is not modeled. It is noteworthy that in the bottom layer, as the depth decreases, the elastic response spectra records larger values which are due to the resonance in the wind turbine. But the elastic response spectrum of the soil is decreased at the ground level in comparison with the lower layers. The reason is that with increasing the pore water pressure and as the liquefaction happens, the top layer behaves like a very viscous fluid. Therefore shear waves are not propagated in the soil, and the elastic response spectra are declined.

Predicted values for pore water pressure are in a very good agreement with the experimental values. The accuracy of the numerical model is acceptable for the acceleration time histories and the elastic response spectra recorded in the structure. As shown in Fig. 4-13, the finite element model has carefully recorded the permanent and the maximum displacement in the structure, but there is a relative phase difference between the displacement obtained from the finite element model and the reported experimental results that can be caused by the numerical simulation errors.

- The minor differences observed in the laboratory values and the numerically calculated ones may refer to the fact that the chamber is not modeled.
- In the bottom layer, as the depth decreases, the elastic response spectra records larger values which are due to the resonance in the structure.
- The elastic response spectrum of the soil is decreased at the ground level in comparison with the lower layers. The reason is that with increasing the pore water pressure and as the liquefaction happens, the top layer behaves like a very viscous fluid. Therefore, shear waves are not propagated in the soil, and the elastic response spectra are declined.
- Predicted values for pore water pressure are in a very good agreement with the experimental values. The accuracy of the numerical model is acceptable for the acceleration time histories and the elastic response spectra recorded in the structure.
- The finite element model has carefully recorded the permanent and the maximum displacement in the structure, but there is a relative phase difference between the displacement obtained from the finite element model and the reported experimental results that can be caused by the numerical simulation errors.

4. Conclusions

The following conclusions are drawn from this Study:

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