Site liquefaction characteristics of offshore wind farms under earthquakes in the South Yellow Sea, China

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Abstract. Jiangsu offshore belongs to the South Yellow Sea, which is the most concentrated area of offshore wind farms in China. At present, it accounts for about 70 ~ 75% of the total installed capacity of China. The overburden layer thickness in this area is quite large; soils along the pile depth (generally 40 m ~ 60 m) are mainly silty sand, silt, and silty clay. Especially, there are 20 m thick liquefiable silty sand layer below the mud surface. The South Yellow Sea is also the earthquake activity area in China. Therefore, this paper researched the seismic liquefaction characteristics of offshore wind farms in this area. Firstly, the drilling borehole data of 50 offshore wind turbines in Dongtai, South Yellow Sea were analyzed, and a representative formation model was established. Secondly, the soil dynamic parameters for site dynamic analysis were calibrated based on dynamic triaxial test and resonance column test. Then, the site liquefaction was researched, the El Centro wave was put into the model, and the peak ground accelerations (PGAs) near the ground surface were adjusted to 0.05 g, 0.1 g, 0.2 g, and 0.4g, respectively. The characteristics of excess pore pressure ratio, total settlement, and layered settlement under earthquakes were analyzed. It was found that the strata in this area consisted of liquefiable soils. When input PGA = 0.05 g, the excess pore pressure ratio of each layer is less than 1.0, and the total settlement is about 1 cm; when PGAs = 0.1 g and 0.2 g, only the surface layer (within 12 m) is completely liquefied, and the total settlement is 10 cm and 17 cm, respectively; when PGA = 0.4 g, the 20 m strata below the mud surface are completely liquefied and the total settlement is about 30 cm. Under different earthquakes, the settlement of surface soil (12 m) accounts for the largest proportion (more than 95% when PGAs = 0.1 g and 0.2 g). Therefore, the site liquefaction characteristics under earthquakes should be considered for pile foundation design of offshore wind turbines.

Key words: Offshore wind turbine; South Yellow Sea; Earthquake; Liquefaction analysis; Ground settlement.
1. Introduction

Chinese offshore wind farms started in 2009. After over 10 years of rapid development (Figure 1), it has become the forefront of the international offshore wind power [1]. The installed capacity and under construction projects rank first and third in the world, respectively [2]. In particular, various offshore wind farms in China are located in the Jiangsu Sea area [3] where the stratus in this area is loose silt and silty sand (liquefiable soils).

The earthquake may cause catastrophic damage on offshore wind farms. During earthquakes, the excess pore pressure and settlement will occur in the liquefied layer. Although the seismic research on offshore wind turbine foundation has gradually increased currently [4–7], the response of offshore wind farms under earthquake is not very clear. Moreover, most of the offshore wind farms were built in the last five years with insufficient design specifications related to the earthquake action in the offshore wind turbine [8–9].

![Figure 1. Recent development trend of offshore wind power (data from CWEA and GWEC) Note: Chinese Wind Energy Association (CWEA); Global Wind Energy Council (GWEC)](image)

Therefore, this paper selects a wind farm in the southern Yellow Sea area of Jiangsu Province to research this problem. The dynamic response of the soil layer under seismic actions and the liquefaction settlement were researched by a one-dimensional seismic analysis program (LIQCA). The results can provide theoretical basis for the seismic optimization design of offshore wind turbine foundations.

2. Study area and stratum

2.1. Study area

Seismic activity is frequent in the southern Yellow Sea and there are many potential seismogenic faults around offshore wind farms. The North-Jiangsu Coastal Fault is located outside the coastal zone, extending intermittently in the northwest direction for more than 200 kilometers. The fault has apparent activity (badly faulting) in the quaternary, having significant impacts on the formation and development of the modern coastlines. There have been several destructive earthquakes with a maximum magnitude of 6 within the range of 5–25 km along the North-Jiangsu Coastal Fault.

This study selected a 200 MW offshore wind farm in Dongtai, Jiangsu Province as the research object, the drilling and cone penetration test (CPT) data of 50 wind farm stations were analyzed. The offshore wind farm project is located in the Jiangjiawu area between Dongsha and Beitiaozini, about 36 km from the shore with the sea area of about 29.8 km². The mud surface in the engineering site has an elevation of 0.05 ~ 8.9 m. Fifty wind turbines with a single capacity of 4.0 MW are deployed in this
project with the total installed capacity of about 200 MW. The wind turbines adopt a single pile foundation with a steel pile (diameter 5.5 ~ 6.5 m); the average pile length in the shallow and deep-water areas are 63 and 72 m, respectively, and the average depth under the mudline is 58 ~ 60 m.

2.2. Site stratigraphic model

The free-rotating drilling of the XY-II drilling rig was used for offshore drilling operations and the double-bridge static CPT was used for in-situ testing. For the soil alone the exploration depth is quaternary sedimentary strata, covering many soil types, such as sandy, silt, and cohesive soil. The drilling depth of this survey is 65.60 ~ 79.75 m, all-penetrating the silt sand layer of ⑥-1 or ⑧ more than 10 m; the depth of CPT is 34.10 ~ 56.70 m. For the convenience of analysis, the strata were combined during dynamic analysis (six strata for the site), as shown in Table 1 and Figure 2.

| Simplified analysis of strata | Actual strata | Soil property | Thickness (m) | Gravity density (g/cm³) | 0.5 mm | 0.25 mm-0.25 mm | 0.25 mm-0.075 mm | 0.075 mm-0.005 mm | <0.005 mm |
|-----------------------------|---------------|---------------|---------------|--------------------------|--------|----------------|-----------------|-----------------|----------|
| 1 (liquefiable)             | ①            | Silty sand    | 4             | 1.97                     | 0.4    | 42.0           | 53.5            | 4.1             |          |
| 2 (liquefiable)             | ②            | Silt          | 8             | 1.96                     | 0.3    | 28.3           | 66.6            | 4.8             |          |
| 3 (non-liquefiable)         | ③-1          | Silty sand    | 6             | 1.98                     | 4.1    | 44.8           | 64.6            | 4.6             |          |
| 4 (liquefiable)             | ①-1          | Silty clay    | 2             | 1.89                     | /      | 18.0           | 73.0            | 9.0             |          |
| 5 (non-liquefiable)         | ②-1          | Silty clay    | 7             | 1.96                     | 0.1    | 8.3            | 69.2            | 22.4            |          |
|                            | ②-2          | Silty clay    | 6             | 1.9                      | /      | 5.7            | 81.0            | 13.3            |          |
| 6                            | ①-1          | Silty         | 32            | 1.98                     | 26.7   | 44.5           | 25.0            | 3.7             |          |

![Figure 2. Typical strata and the mesh of one-dimensional simulation](image-url)
3. Site dynamic analysis model

3.1. Dynamic constitutive model and parameters

In the one-dimensional seismic analysis program for the liquefiable sites, the liquefiable soil is described by a dynamic cyclic elastoplastic constitutive model (E-P model) and the non-liquefiable soil is characterized by the Ramberg-Osgood constitutive model (R-O model) [10–12].

The 1st, 2nd, 4th, and 6th layers of the simplified stratum in Table 1 are liquefiable layers. The numerical simulation adopts the E-P constitutive model with the main parameters, such as over-consolidation surface, yield surface, and plastic potential function. Among them, equation (1) defines the over-consolidation surface, and the parameters are shown in equations (2) ~ (4).

\[ f_s = \bar{\eta}_0 + M_m \ln \frac{\sigma'_m}{\sigma_{vo}} = 0 \] (1)

\[ \bar{\eta}_0 = \left( \eta_{ij} - \eta_{ij(0)} \right) \left( \eta_{ij} - \eta_{ij(0)} \right)^{\frac{1}{2}} \] (2)

\[ \eta = (\eta_{ij} \cdot \eta_{ij})^\frac{1}{2} \] (3)

\[ \sigma'_m = \sigma'_{vo} \exp\left( \frac{1 + e_0}{\lambda - \kappa} \nu^\gamma \right) = OCR' \sigma'_{vo} \exp\left( \frac{1 + e_0}{\lambda - \kappa} \nu^\gamma \right) \] (4)

where \( f_s \) is over-consolidation surface. When \( f_s < 0 \), the soil is in the over-consolidation zone and when \( f_s > 0 \), the soil is in the normal consolidation zone. \( \eta_{ij} \) denotes the stress ratio and \( \eta_{ij(0)} \) is the initial value of \( \eta_{ij} \); \( \sigma'_m \) represents the average effective stress; \( M_m \) is the ratio of phase transition stress; \( \sigma'_{mb} \) is the intersection of \( \eta_{ij(0)} \) and \( f_s \); \( \sigma'_{mb} \) is the initial value of \( \sigma'_{mb} \); \( e_0 \) denotes the initial void ratio; \( \lambda \) is the compression index; \( \kappa \) is the expansion index; \( \nu^\gamma \) is the plastic volume strain; OCR is the over-consolidation ratio.

The model yield surface model adopts the non-linear kinematic hardening criterion. The yield surface function was composed of \( f_y1 \) and \( f_y2 \). More description of the yield surface function and non-linear transport to hardening parameters are explained in the references [10–12].

3.2. Soil dynamic test

The dynamic triaxial tests and resonance column tests were performed on the simplified stratum, as described in Table 1. Since the soil is mainly silt and silty sand, the samples have been slightly disturbed and the remolding soil samples are used in this research. Sine wave excitation frequency of 1 Hz, consolidation ratio \( k_c \) of 1.0, and pore pressure ratio \( r_u = 1 \) were used as the criteria for liquefaction. The vertical axis of liquefaction strength curve is dynamic shear stress ratio (CSR), and the horizontal axis is the vibration number. Figure 3 shows the numerical simulation parameters of the first layer of liquefiable soil calibrated by dynamic triaxial tests. Table 2 summarizes the parameters of the dynamic analysis of all soil layers.
3.3. Input ground motion

The ground motion El Centro wave was selected, commonly used to analyze seismic liquefaction. The specific earthquake event and seismic station record belong to El Centro Array #4 (1979). Figure 4 shows the ground motion's time history and frequency spectrum, where the peak value of ground motion was adjusted to 0.1 g (about 1 m/s²). In the analysis, the horizontal ground motion was inserted from the bottom of the model. For site liquefaction analysis, the peak ground accelerations (PGA) near the ground surface were multi-iterated as 0.05 g, 0.1 g, 0.2 g, and 0.4g, respectively.

Monitoring data for numerical analysis include acceleration, settlement, and excess pore pressure ratio, where the information of acceleration and settlement were stored at the grid node; the information of excess pore pressure ratio is stored in the grid unit. To accurately calculate the excess pore pressure of the 2nd layer of the liquefiable layer, the excess pore pressure ratio monitoring at the position of −20 m is added.

### Table 2. Simplified stratum dynamic analysis parameters of an offshore wind farm

| Layer | Initial void ratio (e0) | Permeability coefficient (m/s) | Natural density (g/cm³) | Compression Index λ | Rebound index κ | Shear modulus ratio G0/a₀ | Transformation stress ratio Mm | Hardening parameters B0 | B1 | Hardening parameters n | Anisotropy parameter Cd | Reference strain parameter γ_ref^e | Poisson's ratio ν | Cohesion c(KPa) | Internal friction angle (deg) | Non-linear coefficient α | Non-linear coefficient β | Shear modulus coefficient a | Shear modulus coefficient b |
|-------|-------------------------|-------------------------------|------------------------|---------------------|------------------|-----------------------------|-----------------------------|--------------------------|----|----------------------|------------------------|-------------------------|----------------|----------------|-----------------------------|----------------------|----------------------|-----------------------------|-----------------------------|
| 1st   | 0.728                   | 3.00E-06                      | 1.96                   | 0.011               | 0.001           | 1100                        | 0.95                        | 2000                     | 30  | 3                    | 2000                   | 0.005                   | 0.1             | /              | /                           | /                    | /                    | /                           | /                           |
| 2nd   | 0.67                    | 6.00E-06                      | 1.97                   | 0.01                | 0.001           | 800                         | 0.95                        | 3500                     | 30  | 6                    | 2000                   | 0.005                   | 0.1             | /              | /                           | /                    | /                    | /                           | /                           |
| 3rd   | 0.98                    | 3.80E-10                      | 1.9                    | 0.01                | /                | 600                         | 1.25                        | 3000                     | /   | 4                    | 1                      | /                      | 0.005           | 0.38            | /                           | 40                   | /                    | /                           | /                           |
| 4th   | 0.75                    | 5.00E-06                      | 1.95                   | 0.01                | /                | /                           | 1.25                        | 2800                     | /   | 3                    | /                      | /                      | 0.35             | 15              | /                           | /                    | /                    | /                           | /                           |
| 5th   | 0.94                    | 1.80E-09                      | 1.9                    | 0.01                | /                | /                           | /                           | 3000                     | /   | 1                    | /                      | /                      | 0.005           | 3               | /                           | /                    | /                    | /                           | /                           |
| 6th   | 0.64                    | 1.00E-05                      | 1.98                   | 0.01                | /                | /                           | /                           | /                        | /   | 6                    | /                      | /                      | 0.1             | 3               | /                           | /                    | /                    | /                           | /                           |

Figure 3. Parameter fitting of the numerical simulation
4. Site liquefaction analysis

The El wave is input and analyzed and the PGAs near the mud surface were adjusted to 0.05 g, 0.1 g, 0.2 g, and 0.4 g, respectively. The acceleration, settlement, and excess pore pressure ratio were monitored. Since the excess pore pressure of the non-liquefied layer dissipates slowly, the calculation analysis time is set to 24 h after the earthquake. This article only shows the situation 13.89 h after the earthquake (5 × 10^4 s).

4.1. Magnification effect of the site under earthquake

Figure 5 shows the magnification effect of the site under earthquake, illustrating that the non-linear magnification effect of the site is mainly reflected at about 20 m below the ground surface. Specifically, the ground motion gradually increased along the elevation. At the position of −40 m, the ground motion is about 1.2 to 1.4 times larger than that at −80 m. Soil at −21 ~ −36 m is a non-liquefied layer with an inhibitory effect on the seismic amplification. After that, the amplification effect near the surface may reach 2.0 ~ 3.0. The response spectrum from the acceleration records near the ground surface shows that due to the liquefaction near the ground surface, the ground motion frequency persists for an extended period (2 ~ 3 s).

4.2. Liquefaction effect of the site under the earthquake

The liquefaction effect of the site under the earthquake when the PGA reaches 0.2 g is shown in Figure 6. The acceleration response near the surface is the largest and the acceleration is gradually increased at the bottom. The development of excess pore pressure shown in Figure 5 implies that the 1st layer of soil has reached the fully liquefied state, while the other liquefiable layers have a ratio of excess pore pressure between 0.5 and 0.6, and the dissipation of excess pore pressure in the deep layers will cause the pore pressure to rise in the non-liquefied layer. The pore pressure of the surface liquefiable soil dissipates after about 2.8 h (0.2 g). After the dissipation of excess pore pressure, the stratum settlement is complete with a settlement value of 17.5 cm (0.2 g). The settlement of the first layer accounts for over 95% of the total settlement.

4.3. Final settlement under different PGA
The El waves of different PGAs were analyzed. The maximum excess pore pressure ratio of the site and the final settlement value of the ground were monitored at the same time.

Figure 6. Excess pore pressure and displacement during and after the earthquake (Peak ground acceleration (PGA) = 0.2 g)

Figure 7 shows the maximum excess pore pressure ratio of each soil layer triggered by the earthquake and the maximum settlement value after the earthquake when El waves are input under different PGAs. The 1st and 2nd layers of the soil are completely liquefied when PGA = 0.1 and 0.4 g, respectively. The total surface settlement under 0.05 g, 0.1 g, 0.2 g, and 0.4 g are 1.01 cm, 12.17 cm, 17.5 cm, and 26.85 cm, respectively.

Figure 7. Maximum excess pore pressure ratio and permanent settlement under El wave with different PGAs

4.4 Layered settlement

From the previous analysis, it can be found that the 1st layer of soil is the easiest to liquefy and is the most significant proportion of the total settlement. To understand the specific settlement value and proportion of the 1st layer of soil, Table 3 lists specific calculated values, among which the layered settlement value of the 1st layer of soil is the total settlement value minus the calculation of the top
surface of the 2nd layer of soil value. Besides, no soil liquefied under PGA = 0.05 g, and the 1st layer of soil settlement accounts for 55% to 60%. When PGAs = 0.1 g and 0.2 g, only the 1st layer is completely liquefied and the settlement accounts for more than 95%. When PGA = 0.4 g, the 2nd layer reaches complete liquefaction and the sedimentation proportion of the 1st layer drops slightly, still accounting for 87%.

Table 3. Total ground and first layer settlement under different ground motions

| PGA(g) | Total ground settlement (m) | First layer settlement (m) | Settlement proportion of the 1st layer (%) |
|--------|---------------------------|---------------------------|------------------------------------------|
| 0.05   | 0.0101                    | 0.0056                    | 56.03                                     |
| 0.1    | 0.1217                    | 0.1167                    | 95.922                                    |
| 0.2    | 0.175                     | 0.1609                    | 96.347                                    |
| 0.4    | 0.2685                    | 0.2338                    | 87.073                                    |

4.5. Influence of liquefaction on offshore wind turbine pile foundation

According to the geological investigation, the ultimate vertical bearing capacity of a single pile is 60000 ~ 77000 kN when the average pile diameter is 6 m and the average value is about 68000 kN (without considering the liquefaction). In the calculation, the side resistance is 15 kPa within 6 m below the mud surface (one time of pile diameter), and 32 kPa within the depth range 6–12 m. However, according to the calculation in this paper, under the earthquake of PGA = 0.2 g, the surface layer is completely liquefied, the total settlement of the site is 16 ~ 17 cm, and the main settlement occurs within the depth of 12 m (96 ~ 97%). Thus, the lateral resistance of the surface layer should be deducted and negative friction caused by liquefaction settlement should be considered where the ultimate vertical bearing capacity of a single pile is about 59180 kN, about 85–90% of that without considering the liquefaction effect.

5. Conclusions

This paper researched the seismic liquefaction characteristics of an offshore wind farm site in the southern South Yellow Sea, China. Firstly, a generalized stratum model was established based on the statistical characteristics of drilling data from 50 wind farm stations in this area. Then, the dynamic analysis parameters of the soil were calibrated through dynamic triaxial tests and resonance column tests. Subsequently, an earthquake wave with PGAs of 0.05 g, 0.1 g, 0.2 g, and 0.4 g was used for the site liquefaction analysis. The conclusions are as follows:

1. The stratum in this area is liquefiable. When the PGA = 0.05 g around the surface, no liquefaction occurs and the total settlement is about 1cm;
2. When the PGAs are 0.1 g and 0.2g around the surface, only the stratum of the surface layer (within 12 m) is completely liquefied, and the total settlement is 10cm and 17 cm, respectively.
3. When PGA is 0.4 g around the surface, the stratum within 20 m is completely liquefied, and the total stratum subsidence is about 27 cm.
4. Under different earthquakes, the soil settlement of the surface layer (within 12 m) makes the most significant proportion, accounting for more than 95% when PGAs are 0.1 g and 0.2 g.
5. The liquefaction characteristics of the site should be considered for the design of offshore wind turbine pile foundations. For example, the ultimate vertical bearing capacity of the pile maybe reduced due to the soil liquefaction.

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