Bearing Capacity of the High-Rise Pile Cap Foundation for Offshore Wind Turbines

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Abstract: A proper foundation design is crucial for the structural stability of offshore wind turbines (OWT) in the severe ocean environments. As one of the typical foundation type for OWT, the high-rise pile cap foundation has been utilized in wind farm of China. In this study, a 3-dimensional finite element model for simulating the pile-soil interaction subjected to multi-axis complex loads is proposed and verified with existing experimental data. The bearing capacity of the high-rise pile cap foundation under various ultimate limit state (ULS) conditions is investigated. Local scour usually occurs around the pile and significantly reduces the bearing capacity of the foundation. Numerical simulation shows that scour could amplify the lateral displacement and tilt of the foundation.

Key words: high-rise pile cap foundation; pile-soil interaction; bearing capacity; scour effect

Introduction

In recent decades, quite a few pile foundations have been utilized in the prosperously developing OWT. Large-diameter monopile foundation is a common choice for most offshore wind farms in Europe, while the high-rise pile cap foundation is currently employed in East China Sea area, due to different soil properties and pile driving ability between Europe and China. The height of the wind turbine hub can reach approximately 100 meters for a 3 MW turbine, producing huge lateral force in the order of 5 MN and bending moment in the order of 1×10^8 N.m. These huge loads pose significant challenge in the design and construction of the foundation.

Many different calculation methods have been developed and employed by various researchers to assess the bearing capacity of monopile and pile groups. Poulus (1980) put forward a state-of-art review on the elasticity method of pile foundation design. Poulos and Hull (1989) found that the selection of the soil parameters and their distribution with depth have significant influence on the prediction accuracy. However, the method is only valid for small strains as soil is more likely to behave elasto-plastically while the soil is assumed to be elastic in this method.

In current design of laterally loaded offshore monopiles, the *p*-*y* curves are typically used to characterize the interaction between pile and soil. The *p*-*y* approach is adopted in the offshore design codes of American Petroleum Institute (API, 2007) and Det Norske Veritas (DNV, 2010). This method assumes that the pile acts as a beam supported by a series of uncoupled springs, which represent the soil reaction. The non-linear behavior of the springs is specified by the assi[1]gned *p*-*y* curves. At a given depth, the *p*-*y* curve relates the soil reaction *p* at a given depth to the corresponding pile lateral deflection *y*. Details on the *p*-*y* methodology for analysis of laterally loaded piles can be found in Reese et al. (1974) and Reese and Van Impe (2001). The

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diameters of the monopiles utilized in offshore wind farms could be up to 5~7.5 m (Achmus et al., 2009) and outside the scope of existing experience, which is particularly evident when comparing the database used by Murchinson and O'Neill (1984) to develop the existing API approach (D<1.5 m) with the monopile diameters being installed today. Paul and Kenneth (2011) presented main limitations and differences between the API (2007) design code and industry practice, including the different failure modes, diameter effects, ignoring pile properties, et al. According to the results of the numerical calculations carried out for monopiles of large diameter, Abdel-Rahman and Achmus (2005) concluded that the *p-y* curve method given in API (2007) underestimates pile deformations for large-diameter monopiles.

Much effort has been devoted to the 3-dimensional numerical simulation of large-diameter monopiles under either monotonic or cyclic loading, e.g. Abdel-Rahman and Achmus (2005), Leblanc et al. (2010), Achmus et al. (2009), and Bourgeois et al. (2010). These fully 3-dimensional numerical simulations often propose empirical modifications of the *p-y* curves to account for the diameter effect, group effect and interactions with neighbouring structures, providing a way to overcome the aforementioned limitations of the classical *p-y* curve method.

The existing studies focus on the bearing characteristic of large-diameter monopile foundation rather than high-rise pile cap foundation, which is at present the most popular foundation type for OWT in East China Sea area. In this study, a 3-dimensional finite element model of high-rise pile cap foundation is established to shed some light on its bearing behavior.

Local scour will occur around the pile foundations due to the action of waves and current. The scour depth could significantly reduce the bearing capacity of the foundation. This scour effect is also investigated in the present study.

Numerical model

Finite element mesh and constitutive models of materials

The bearing process of high-rise pile cap foundation under an axial or vertical load can be considered as a plane-symmetric problem. A three-dimensional plane-symmetric finite element model is thereby proposed for simulating the pile-soil interaction using the finite element code ABAQUS.

Fig. 1 illustrates the geometry of the finite element model, which is mainly consisted of the foundation and the surrounding soil. The foundation is composed of four vertical open-ended steel piles and a circular concrete cap fixed upon their upper surfaces. The cap and piles are connected with a "tie-type" constraint provided by ABAQUS software. There exist soil-plugs inside the pile interior surfaces. Both the foundation and soil are composed with three-dimensional 8-node reduced-integration solid elements (C3D8R). The computational grids get denser in the closer proximity to the pile for high computation efficiency. A total of approximately 20000 elements are utilized. The interfacial behavior is a key issue to efficiently simulate the complex pile-soil interaction. A contact-pair algorithm was adopted to characterize the interfacial constitutive relationships between pile exterior surfaces and surrounding soil, pile interior surfaces and soil-plugs, respectively.

Since the failure of the pile structure usually does not occur during the loading process in the field, the piles and the cap are assumed to be linear elastic with different mechanical parameters (see Table 1). The Mohr-Coulomb plasticity constitutive model (M-C model) is used to simulate the elasto-plastic behavior of the soil.

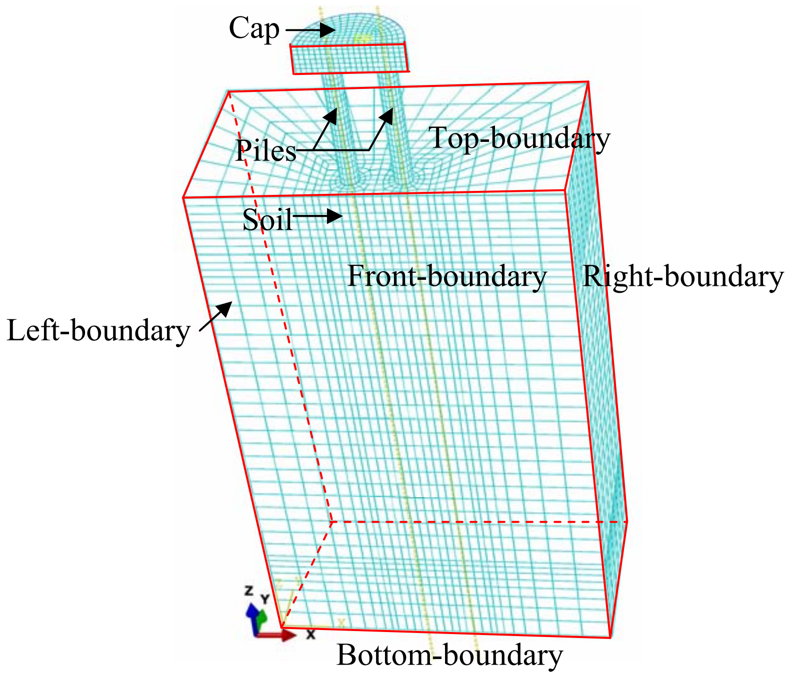


Fig.1 Three-dimensional finite element meshes of foundation and surrounding soil

Boundary conditions and loading conditions

As illustrated in Fig. 1, the top-side of the soil is treated as a free boundary. The front-side of the model is a plane-symmetric boundary, i.e. both the rotational degrees of freedom in x and z directions and translational degree of freedom in y direction are constrained to zero. At the left, right and back-sides of the soil, the translational degree of freedom in x and y directions are constrained. At the bottom of the model, the translational degrees of freedom in both x, y and z directions are fixed.

Following design codes of DNV (2010), characteristic combined load effect can be obtained by combining the individual characteristic load effects due to the respective individual environmental load types (see Table 1). Each load case is defined as the combination of two or more environmental load types. For each load type in the load combination of a particular load case, the table specifies the characteristic value of the corresponding, separately determined load effect. The characteristic value is specified in terms of the return period. The final results of the loads for 5 different ULS cases are given in Table 1. The sketch of static loads exerted on the foundation is illustrated in Fig 2.

Table 1. Load combinations and corresponding results of load calculations

		Environmental load type and return period to define characteristic value of corresponding load effect					Loads			
Limit state	NO.	Wind	Waves	Current	Ice	Water level	F_x (kN)	F_y (kN)	F_z (kN)	M_y (kN.m)
Ultimate limit state (ULS)	1	50 years	5 years	5 years		50 years	2402	0	3140	93458
	2	5 years	50 years	5 years		50 years	2057	0	3941	59631
	3	5 years	5 years	50 years		50 years	2003	0	3140	60070
	4	5 years		5 years	50 years	Mean water level	3225	0	0	57245
	5	50 years		5 years	50 years	Mean water level	3673	0	0	89986

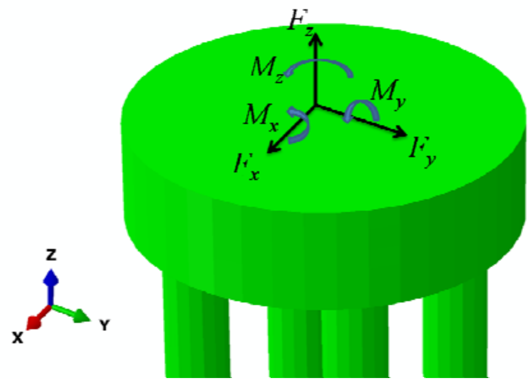


Fig.2 Sketch of static loads

Model parameters

As a case study, the parameters for the examined foundation are listed in Table 2, considering the practical condition of the water depth and the design parameters of similar preceding projects. According to the geotechnical investigation result, the soil can be approximately divided into three layers, as listed in Table 3. The pile-soil frictional coefficient (μ) is calculated with following formula proposed by Randolph and Wroth (1981)

$$\mu = \tan\left[\frac{\sin\phi \times \cos\phi}{(1+\sin^2\phi)}\right] \tag{1}$$

where ϕ is the angle of soil internal friction.

Table 2. The parameters for piles and cap of the foundation

	Wall thickness (m)	Embedded pile length (m)	Length/Height L/D (m)	Pile/Cap diameter D(m)	Mass density ρ (kg/m^3)	Young's modulus E (GPa)	Poisson's ratio υ
Piles	0.05	65	83.5	2.5	6900	210	0.3
Cap	/	/	4.0	15.0	2300	28	0.17

Table 3. Properties of the layered soil

Stratum No.	Stratum Name	Stratum Thickness	Mass density ρ (kg/m^3)	Cohesion strength c(kPa)	Angle of internal friction ϕ (°)	Young's modulus E (MPa)	Poisson's ratio υ
1	Silt	10.55	1700	5.0	20	5.0	0.32
2	Silt	4.8	1960	15.0	20	9.0	0.30
3	Silty clay	56.45	1900	15.0	25	10.0	0.30

Verification with experimental results

To verify the proposed finite element model, comparisons are made between the present numerical results and two series of existing experimental results, i.e. the laterally-loaded pile tests by Rao & Prasad (1993) (see Fig.3 (a)); and the tests of vertically-loaded pile groups with a cap by McCabe & Lehane (2006) (see Fig.3 (b)) The calculated result is in good agreement with the experimental data, which indicates that the present finite element model is capable of simulating the bearing process of monopile and pile groups.

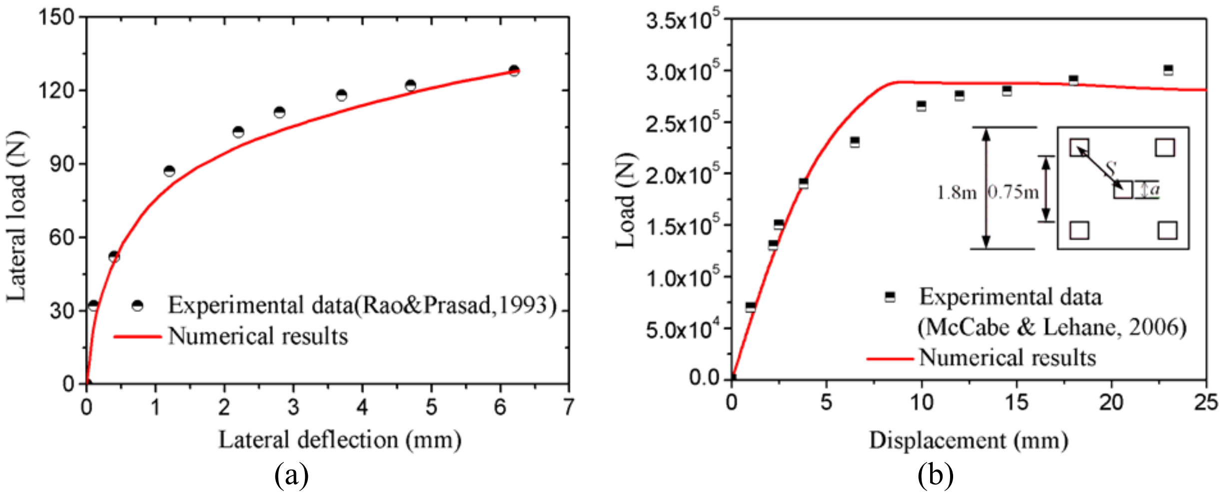


Fig. 3. Comparisons between the present numerical results and the existing experimental results of (a) tests of monopile by Rao & Prasad(1993) (D=18.0 mm, L=270.0 mm, γ=18620N/m3, c_s=10.0 kPa, φ=25°); and (b) tests of pile groups with a cap by McCabe & Lehane (2006) (a =0.25 m, L=6.00 m, γ=18130 N/m3, c_s=22 kPa, ϕ=35°)

Numerical results and discussions

Displacements under ULS conditions

For an onshore wind turbine, the maximum allowable tilt at pile head after installation is typically between 0.003 to 0.008 radian (0.2 degrees to 0.45 degrees). A somewhat larger tilt 0.009 (0.5 degrees) may be allowed for offshore wind turbines (Malhotra, 2011).

The displacement contour of foundation and soil under different ULS conditions are given in Fig 4. It can be seen that for ULS-5 (Fig. 4(e)), the pile experiences the most serious deformation. For ULS-1 (Fig. 4(a)) and ULS-4 (Fig. 4(d)), the deformation of the model is still great, but not so much compared with ULS-5. For ULS-2 (Fig. 4(b)) and ULS-3 (Fig. 4(c)), deformation is obviously more limited than other cases. Analyzing Fig. 4 along with Table 3, it can be concluded that the wind load component and ice load component play a key role in the deformation of the pile. This could be owed to the great arm of wind force exerted on the blades and the huge value of ice force due to large contact area between the cap and surrounding ice. Taking the aforementioned allowable tilt at pile head as tan(α)=0.008 (α is the rotating angle of the cap top-side), the current design of the foundation can satisfy the bearing requirements for all the ULS conditions.

The tilt of the cap top-side (e.g. in Fig.4 (e), tan(α)=0.0062) is approximately the same as that of the piles section at the mudline (tan(β)=0.0056, β is the rotating angle of piles section at the mudline), which implies that the upper part of the foundation outside the soil generally rotates as a whole rigid body. Comparing the soil deformation inside the pile (soil-plug) and outside the pile, it can be seen that the deformation of the soil-plug is consistent with the soil outside the pile. And the soil-plug acts as a deformation core along with the piles. The soil-plug is favorable for piles to resist the vertical force and bending moment.

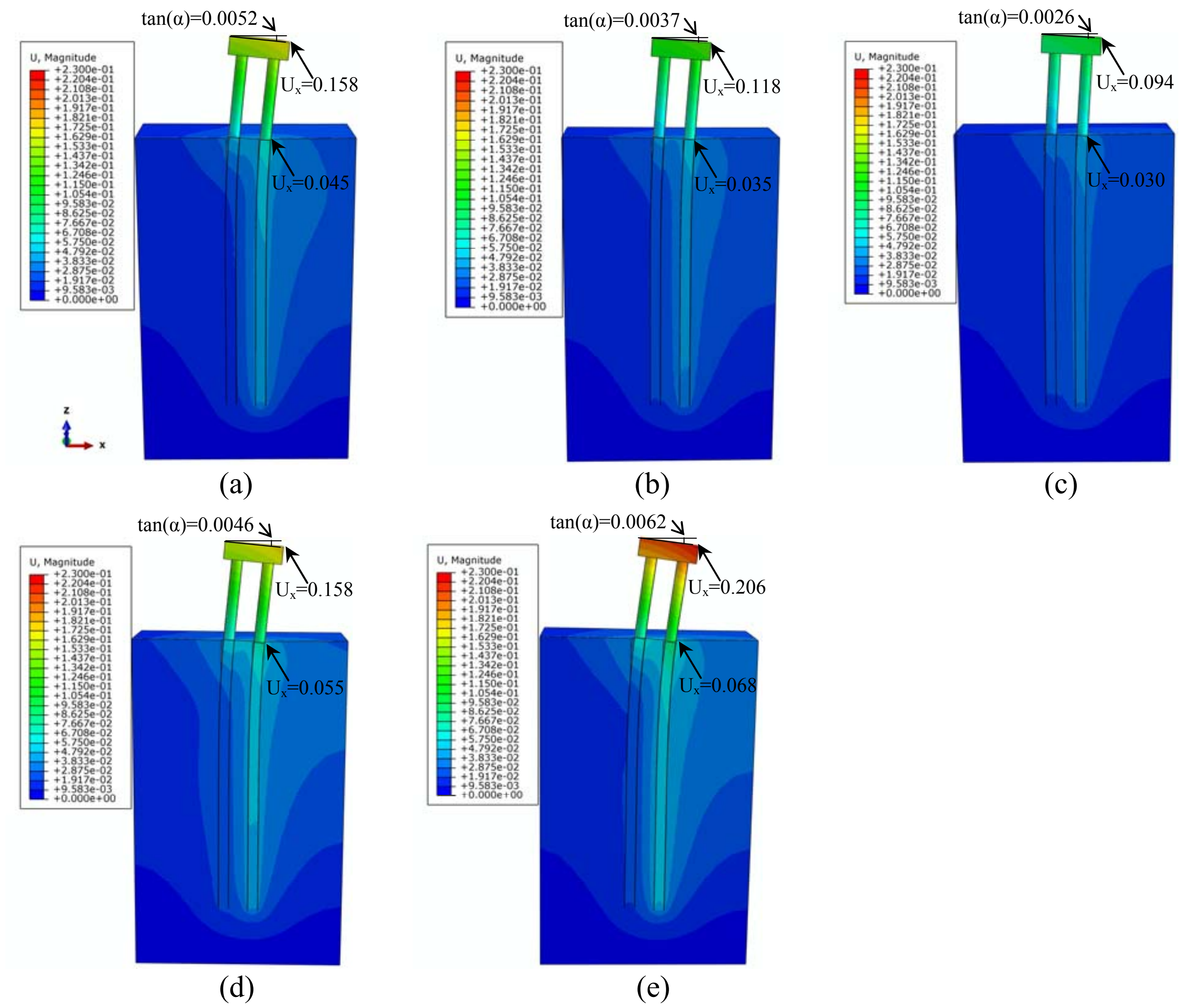


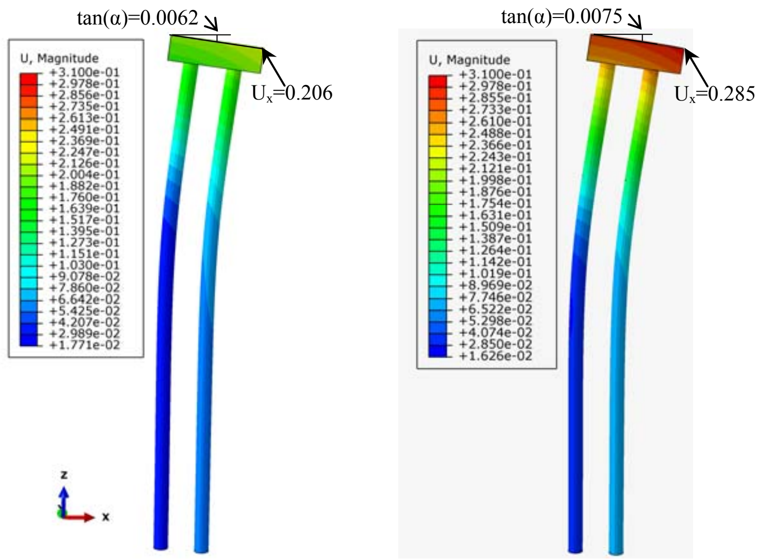
Fig.4 Displacement contour of foundation and soil under different ULS conditions: (a) ULS-1; (b) ULS-2; (c) ULS-3; (d) ULS-4; (e) ULS-5; (Deformation scale factor = 20; cut on x-z plane; unit: m)

Scour effect

Sumer et al. (1992) determined the mean value (S/D=1.3) and the standard deviation ($\sigma_{S/D}$=0.7) of the equilibrium scour depth in waves based on experiment result. In the severe shallow water environments where OWT locate, ocean waves and current are usually coexisting. Qi and Gao (2014) investigated scour around piles in combined waves and current through experimental modeling. They concluded that the superimposition of the waves on a current has much effect on the time-development of local scour and the resulting equilibrium scour depth. To ensure the safety of the OWT, a relatively conservative scour depth value of 5.0 m (2D) is assumed.

Li et al. (2013) demonstrated that assuming the removal of the whole soil layer above the scour depth is a conservative method considering the scour effect. Besides, judging from the load-deflection curves under various slope angles of the scour hole, the influence of the slope angle is not so much obvious. Therefore, in the present study, the removal of whole soil layer above the scour depth is adopted as a conservative approach to investigate the scour effect.

The scour effect on the deformation of the foundation under ULS-5 condition is shown in Fig.5. The vertical displacement at the center of the upper cap surface increases from 0.204m to 0.285 m as scour occurs. And the tilt increases about 20% from 0.0062 to 0.0075. Coincidentally, the momentum exerted on the foundation at the mudline also increases 20% (see Table 1). Thus the increased momentum resulted from the lowering mudline should be responsible for the increasing of the tilt. As shown in Fig 5, the deformation of the upper part of the foundation is obviously amplified while the lower part of the foundation has hardly any difference. This indicates that the length of the exposed pile considerably affect the stiffness and deformation, and therefore the bearing capacity of the foundation.



(a) (b)

Fig.5 Displacement contour of foundation under ULS-5: (a) No scour; (b) Scour depth=5 m. (Deformation scale factor = 20)

Conclusions

A 3-D plane-symmetric finite element model to simulate the interaction between a high-rise cap foundation and the neighboring soil is proposed and verified with the existing experiments. As a case study, the total load values applied on the foundation under different ULS conditions are obtained. The deformation in various ULS conditions is examined with the proposed numerical model. The present designed foundation can satisfy the tilt criterion of OWT. Scour could obviously amplify the displacements of the foundation and decrease its bearing capacity. The increased momentum resulted from the lowering mudline should be responsible for the increasing of the tilt.

Acknowledgements

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