Deformation and Stress Analysis of Bucket Foundation under Multiple Loads

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Abstract. Single-pile composite tubular foundation is a new type of offshore wind turbine foundation. Offshore wind turbine single-pile composite tubular foundation (PBCF) can have the bearing advantages of pile foundation and tubular foundation at the same time. The transfer of load between pile foundation and foundation tube is the key of pile-tube cooperative bearing. The wide and shallow tubular foundation is a new type of offshore fan foundation developed for the load characteristics of China offshore. The classical formula for calculating the bearing capacity of foundation cannot accurately calculate the ultimate bearing capacity of wide-shallow cylindrical foundation under composite loading mode. By establishing a large-scale three-dimensional simulation model of single-pile composite tubular foundation-foundation, calculating its stress and deformation under the combined action of various loads, and evaluating the stability of single-pile composite tubular foundation, we think it is an important means of tubular foundation design. The research results can provide references for the selection and optimal design of infrastructure.

1. Introduction

Wind energy, as a clean and pollution-free energy, has gradually attracted people's attention. Due to the limited land-based wind power resources, countries around the world have turned their eyes to offshore wind power resources in recent years. As a new type of offshore wind power foundation, bucket foundation has the characteristics of fast offshore installation and low construction cost [1], and has broad application prospects. In 1994, Norway Institute of Geotechnical Engineering successfully developed Europipe16/11-E large jacket platform, which marked the emergence of suction bucket foundation [2]. In 2010, China's first offshore wind turbine prototype project with composite tubular foundation was completed in a sea area in Qidong, Jiangsu Province, China, as shown in figure 1.

Suction bucket foundation is a kind of steel offshore engineering foundation structure with closed upper end and open bottom end. Because of its advantages such as simple construction, fast construction speed, low cost and recyclability, it has been used for mooring offshore floating structures and offshore platform foundations [3,4]. In recent years, it has also been gradually applied to offshore wind power tower foundations [5]. With the rapid growth of wind power generation platforms in China [6], it is more and more urgent to study the ultimate bearing capacity of bucket foundation. Bucket foundation often bears huge loads from the upper wind turbine and tower, and its bearing capacity is closely related to its diameter and height. Many scholars at home and abroad have done a lot of research on the vertical bearing capacity through model test centrifuge test numerical software analysis and theoretical derivation. Bransby [7], Sharma [8] and Gourvenec [9] have studied...
the bearing capacity of suction foundation under horizontal and vertical loads and bending moment composite loads by finite element method. The ultimate envelope diagram of bearing capacity of suction foundation and the influence of skirt length of suction foundation on the envelope diagram are obtained. Many scholars have studied the characteristics of vertical bearing capacity of bucket foundation based on finite element software, and compared with the model test results, the results are ideal.

![Composite bucket foundation](image)

**Figure 1.** Composite bucket foundation

2. **Geographical and Geological Conditions**

The proposed wind farm is located in the sea area south of Shapa Town, Yangxi County, Guangdong Province, China, and belongs to the transitional zone between south subtropical zone and middle subtropical zone, with abundant sunshine and rainfall and subtropical monsoon climate. The monsoon activity in the area where the project is located is obvious. It is controlled by the cold high pressure in the mainland in winter, prevailing in the northeast wind, and in summer it is in front of the southwest trough of low pressure. Under the influence of the subtropical high, the southeast wind blows more. The annual average temperature is 22.5°C, the annual average rainfall is 2310.7 mm, and the annual average wind speed is 3.0 m/s. The tides in the sea area where this project is located are irregular semidiurnal tides, with an average sea level elevation of 0.67 m and an average high tide level of 1.43 m.

The Caledonian granite gneiss ($\gamma_3$) and the third invasion of biotite granite in the early Yanshan period ($\gamma_5^{3(3)}$) are the main areas, and Cretaceous sandstone (K) is the local area. Quaternary strata are mainly alluvial silt, clay, sand and residual sandy cohesive soil. According to the survey results and the geological data of the adjacent sites, the overburden of the wind farm site mainly includes Quaternary Holocene marine strata ($Q_{4m}$), Quaternary Upper Pleistocene marine-terrestrial interbedded strata ($Q_{3mc}$) and Quaternary residual soil layer ($Q_{el}$) according to its genetic types. The underlying bedrock is Mesozoic Late Cretaceous sandstone and Paleozoic Cambrian granite gneiss. The thickness of the overburden increases gradually from northwest to southeast.

3. **Parameter Calculation and Loading Conditions**

The mechanical parameters determined comprehensively according to the site survey and indoor test are shown in table 1.
Table 1. Physical and mechanical parameters

| Stratum                  | Severe (kN/m) | Elastic modulus (MPa) | Poisson's ratio | Cohesion (kPa) | internal friction angle (°) |
|--------------------------|---------------|-----------------------|-----------------|----------------|---------------------------|
| Flowing mud ~ silt       | 15            | 0.15                  | 0.45            | 0.3            | 0.1                       |
| Flowing mud ~ silt       | 15            | 3.81                  | 0.42            | 4.5            | 0.8                       |
| Flowing mud ~ silt       | 15            | 3.81                  | 0.4             | 9              | 1.4                       |
| silt                     | 15            | 5.7                   | 0.4             | 12             | 1.4                       |
| Muddy soil               | 16            | 7.5                   | 0.4             | 29.0           | 2.0                       |
| Silty clay               | 17.5          | 12.9                  | 0.35            | 42.0           | 3.0                       |
| coarse sand              | 19.5          | 90                    | 0.25            | 0.1            | 35                        |
| Silty clay               | 17.5          | 36                    | 0.35            | 75.0           | 3.0                       |
| Completely weathered granite | 21        | 120                   | 0.2             | 50.0           | 50.0                      |

Considering the main loads on the suction bucket and the combined action of gravity, horizontal load, wave force, bending moment and torque of the upper fan, the applied external loads are shown in table 2.

Table 2. External loads

| Loads          | Dead load (kN) | Horizontal force (kN) | Wave load (kN) | Bending moment (kN·m) | Torque (kN·m) |
|----------------|----------------|-----------------------|----------------|-----------------------|---------------|
| values        | 9056           | 2584.7                | 18526.83       | 258179                | 12980         |

In the finite difference calculation program, the volume modulus and shear modulus of rock and soil are calculated by the following formulas:

\[ K = \frac{E}{3(1-2\nu)} \]  \hspace{1cm} (1)
\[ G = \frac{E}{2(1+\nu)} \]  \hspace{1cm} (2)

Where, \( K \) and \( G \) are bulk modulus and shear modulus respectively, \( E \) is the elastic modulus, \( \nu \) is Poisson's ratio.

For the parameters of the contact surface, the normal stiffness \( k_n \) and shear stiffness \( k_s \) are taken as 10 times the equivalent stiffness of the ‘hardest’ soil layer in the adjacent area of the contact surface \([5]\), namely

\[ k_n = k_s = 10 \max \left( \frac{K + \frac{4}{3}G}{\Delta z_{\min}} \right) \]  \hspace{1cm} (3)

Where, \( \Delta z_{\min} \) is the minimum dimension of the connecting area in the normal direction of the contact surface.

The calculation results of contact parameters are shown as table 3.
Table 3. Contact parameters

| Stratum                  | $K$ (MPa) | $G$ (MPa) | $k_n=k_s$ (MPa/m) | $\Delta z_{\text{min}}$ |
|--------------------------|-----------|-----------|-------------------|-------------------------|
| Flowing mud ~ silt       | 0.5       | 0.051724  | 5.69              |                         |
| Flowing mud ~ silt       | 7.9375    | 1.341549  | 97.26             |                         |
| Flowing mud ~ silt       | 6.35      | 1.360714  | 81.64             |                         |
| Silt                     | 1.9       | 2.85      | 57                |                         |
| Muddy soil               | 12.5      | 2.678571  | 160.71            |                         |
| Silty clay               | 14.33333  | 4.777778  | 207.04            |                         |
| Coarse sand              | 60        | 36        | 1080              |                         |
| Silty clay               | 40        | 13.33333  | 577.78            |                         |
| Completely weathered granite | 66.66667 | 50        | 1333.33           |                         |

4. Deformation and Stress Analysis

The $x$ and $z$ displacement nephograms of the soil layer around the barrel are shown in figure 2 and figure 3. The maximum horizontal displacement of the soil is 70 mm, which appears in the shallow surface layer near the barrel, and the direction is right; The maximum vertical displacement of the soil is 106 mm, which also appears in the shallow surface of the stratum, and the direction is downward.

![X displacement](image1)

Figure 2. X displacement nephogram of soil layer

![Z displacement](image2)

Figure 3. Z displacement nephogram of soil layer

![Maximum principal stress](image3)

Figure 4. Maximum principal stress nephogram of silo-dividing plate

![Minimum principal stress](image4)

Figure 5. Minimum principal stress nephogram of silo-dividing plate
The displacement values of the left and right ends of the composite bucket at the mud surface are -105.8 mm and +28.6 mm, respectively. According to the vertical displacement values of the left and right sides of the bucket cover at the mud surface, the structural inclination rate of the composite bucket at the mud surface, i.e., the mud surface rotation angle, is calculated to be 3.73‰. The calculation results are within the allowable range of the code, and the overall deformation of bucket foundation is normal and controllable.

The maximum and minimum principal stresses of the connection between the bin separating plate inside the barrel body and the rib plate on the upper part of the barrel cover are shown in figure 4 and figure 5 respectively. The maximum tensile stress of the member is 49.5 MPa, which is concentrated on the left rib plate. The maximum compressive stress of the member is 106 MPa, which occurs in the minimum area at the lowest end of the right side compartment plate. It can be seen that the compressive stress of the member is obviously greater than the tensile stress, that is, the connecting member connected with the barrel bears greater compressive stress. However, the structural failure may be determined by the maximum tensile stress. On the whole, the stress values of members are all within 200 MPa, and within the allowable range of structural strength, they will not fail.

Because of the basic compressive stress of the stratum, the nephogram of the minimum principal stress of the soil alone is shown in figure 6. The compressive stress of the stratum from top to bottom is layered evenly and clearly, and gradually increases. The soil around the bucket is disturbed by the deflection of the suction bucket, with different compressive stress, no obvious stratification, and the phenomenon of partial compression concentration. It can be seen that the suction bucket has a great influence on the soil around the bucket, but has a limited influence on the horizontal stratum, and the far stratum is basically unaffected. Longitudinal influence is more profound than horizontal influence.

The cloud picture of the maximum shear strain increment of the barrel is shown in figure 7. The maximum shear strain is concentrated at the lower bottom of the inner barrel, with a maximum value of $4.3 \times 10^{-4}$, which is far less than the yield order of $10^{-1}$ of the soil layer. The barrel itself will not reach the yield failure level.

**Figure 6.** Minimum principal stress of soil layer (unit: Pa)  
**Figure 7.** Maximum shear strain increment of bucket  
**Figure 8.** Evolution curve of maximum shear strain increment of soil layer
The nephogram of the maximum shear strain increment of soil elements on both sides of the upper surface in contact with the bucket is shown in figure 8. With the bucket shifting under loading, the shear strain increment of soil elements gradually increases, and the plastic failure of the right soil is greater than that of the left soil, which is due to the right soil layer yielding easily due to compression.

5. Conclusion

Single-column composite tube foundation is an important technological innovation in the design of wind turbine foundation, a demonstration of our technology accumulation in the field of offshore wind power for many years, and a concrete embodiment of taking the initiative to undertake the historical mission of realizing the "parity" of offshore wind power. Its successful implementation will provide strong technical support for the large-scale application of tubular foundation in offshore wind power field, and also provide new ideas and solutions for offshore wind power project construction under complex geological conditions in offshore and deep water in China. In this paper, according to the actual sea conditions, the whole deformation and stress analysis of single pile bucket foundation are carried out through three-dimensional simulation under the combined action of multiple loads. The calculation results provide reference for the overall safety performance evaluation of composite bucket and the design of suction bucket.

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7. References

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