Computers and Geotechnics 91 (2017) 131–145

Contents lists available at ScienceDirect

Computers and Geotechnics

journal homepage: www.elsevier.com/locate/compgeo

Research Paper

Upper bound solution of a laterally loaded rigid monopile in normally consolidated clay

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ARTICLE INFO

Article history:
Received 19 January 2017
Received in revised form 11 July 2017
Accepted 12 July 2017

Keywords:
Laterally loaded piles
Plasticity
Upper bound
Coupled Eulerian-Lagrangian
Centrifuge test

ABSTRACT

A centrifuge test is performed to investigate the ultimate lateral capacity of a free-head rigid monopile foundation in normally consolidated (NC) clay. Based on the test result, an upper-bound velocity field is constructed to illustrate the evolution of the soil resisting mechanisms with the progressive rotation and the effect on reaction forces during loading. Simultaneously finite element analyses are also employed to model the centrifuge test and verify the conclusion of the upper bound analysis. Finally, an empirical expression is presented for predicting the upper-bound collapse loads of monopiles with different eccentricity/length ratios and length/diameter ratios in NC clay.

1. Introduction

Monopiles comprising of single open-ended steel pipes are often employed as foundations for offshore wind farm structures in waters of up to 30 m deep. Fig. 1 illustrates a typical offshore wind turbine on a monopile foundation and the main loads, i.e., wind, waves and currents of varying eccentricities (measured as the distance between the loading points to the surface of the mud). Therefore, the lateral response of a monopile under such harsh environmental conditions have received considerable research attention, in which the limit load is one of the most important design parameters. For example, DNV [9] proposed the *p-y* curve as the design methodology for monopiles, which is strongly dependant on the accuracy of the lateral limit loads.

Hansen [13] first presented a slip-line plasticity solution for a two-dimensional lateral circular pile. Broms [4] thereby assumed a limiting soil pressure profile (force per unit length on the pile) to calculate the ultimate lateral force and moment on pile head by the limit equilibrium analysis. In order to obtain more accurate profiles or pile-head limit loads, significant efforts have been made from experimental measurements [26,15,20] to rigorous plasticity limit analyses, including the lateral limit resistance of a two-dimensional pile section [12,27,39] and the pile-head limit capacity of a three-dimensional laterally loaded pile [32,29,22,38].

Most of their studies were conducted based on clay with uniform strength, which are usually used to model the heavily over-consolidated (OC) clay, which is usually used to describe the soil condition of most onshore foundations. However, a great amount of marine clay is at the normally-consolidated (NC) or slightly over-consolidated state. From the centrifuge test reported by Zhang et al. [40], it can be observed that a gap at the back of the pile would form gradually with the deflection of the pile in OC clay. On the contrary, the centrifuge tests performed by both Jeanjean [20] and Khemakhem et al. [21] revealed that the gap would not occur for the laterally loaded pile in NC clay as the very "soft" clay near the surface deforms with the pile under the action of gravity. Senanayake et al. [33] has demonstrated that the formation of the gap has a strong impact on the lateral behaviour of a monopile such that the limit-load equations obtained from the uniform foundation may underestimate the actual resistance of a monopile in NC clay.

The works of Hamilton and Murff [15] and Jeanjean [20] are the studies to investigate the lateral limit loads of piles in NC clay. However, the model pile used in their studies were flexible. Both Leblanc et al. [24] and Lombardi et al. [25] indicated that long, slender and flexible piles are expected to fail by formation of plastic hinges, while rigid piles usually behave in an almost rigid fashion such that the formation of plastic hinges is not expected. It is evident that the bearing mechanisms between flexible and rigid piles are different, with most current monopile foundations for offshore wind turbines being rigid piles with large diameters ranging

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http://dx.doi.org/10.1016/j.compgeo.2017.07.009
0266-352X/

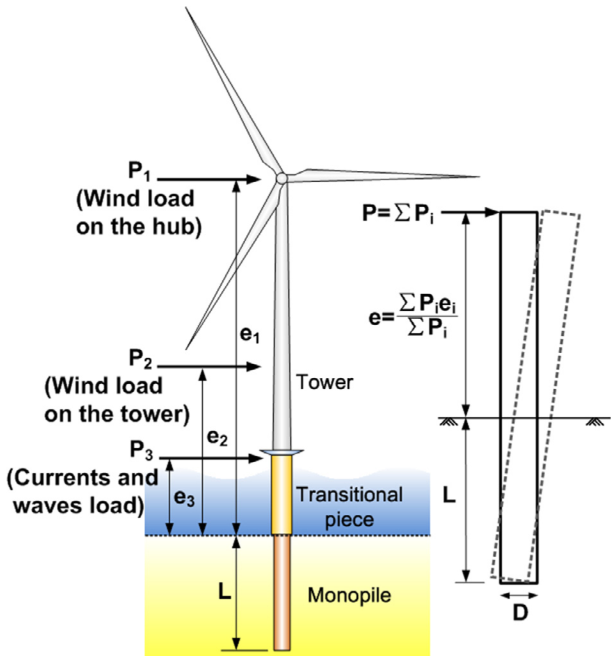

Fig. 1. Schematic diagram of a wind farm with a monopile foundation.

from 2 to 8 m and having a the length/diameter ratio that ranges from 4 to 12 [18,5]. Hence, it is necessary to investigate the lateral capacity of a rigid monopile in NC clay.

In the centrifuge test performed by Zhang et al. [40] to study the lateral response of a fixed-head rigid pile, the pile reached the limit load at the pile-head displacement of around 0.25 times the pile diameter, which is close to the prediction of the p-y curve recommended by API [3]. However, in the centrifuge test of investigating the group-pile effect, Ilyas et al. [19] calibrated the lateral response of a free-head rigid single pile as reference. The results reveal that although the pile-head deflection exceeded one pile width, the ultimate capacity was still unable to be reached. A similar phenomena can also be observed in the tests of Khemakhem et al. [21] and Zhang et al. [41]. A possible reason is that the free-head pile would be inclined due to lateral loading, which results in the possibility that the measured reaction force might not be fully representative of the response of the pile at its initial vertical position. In contrast, for the fixed-head rigid pile, the pile shaft translates horizontally due to the lateral force and no inclination occurs.

Therefore, to further understand the actual ultimate capacity of a free-head monopile foundation, a centrifuge test is performed in NC clay. Based on the test results, an upper-bound velocity field is constructed to illustrate the effect of the progressive rotation on the limit load. Simultaneously finite element methods are also employed to model the centrifuge test and verify the conclusion derived from the upper bound analysis. Finally, an empirical expression is presented to predict the upper-bound limit loads of monopiles with different eccentricity/length ratios and length/diameter ratios in NC clay.

2. Centrifuge test

2.1. Normal consolidated clay sample

The test was carried out at 50*g* at the National University of Singapore geotechnical centrifuge lab. Clay samples were prepared in a cylindrical container of 550 mm in diameter and 400 mm in depth. To accelerate the consolidation of the clay sample, a 20 mm thick sand layer was placed at the bottom of the container. Kaolin clay slurry with a water content of 120% was then placed above the sand. The slurry was first consolidated under a pressure of 4 kPa at 1g for one week, and then underwent 16 h self-weight consolidation at 50*g* under double drainage to achieve at least 95% degree of consolidation.

2.2. Model pile

The model pile was fabricated from a hollow circular stainless steel tube with an external diameter of 38 mm and a wall thickness of 1.2 mm. To protect the gauges and wires, the model pile was then coated with a thin layer of epoxy (1 mm). To apply the epoxy layer, an extra steel tube with the inner diameter of 40 mm was fabricated. The model pile was then put inside the larger tube, and the liquid epoxy was filled into the gap between them. A thin coat of wax was painted on the inner wall to facilitate the ease of removal of the model pile to be taken out from the larger tube after solidification. In addition, wax was also coated on the strain gauges to prevent the epoxy from restricting its movement with the pile. The final diameter of the model pile is 40 mm (2 m prototype scale). The total length of the model pile was 280 mm (14 m in prototype scale), and the pile was embedded to a depth of 240 mm (12 m in prototype scale).

The flexural rigidity of the pile was calibrated according to the formula of material mechanics, $EI = M \cdot y_\varepsilon / \Delta\varepsilon_l$, where M is the bending moment on the pile shaft; $\Delta\varepsilon_l$ is the increment of the longitudinal strain at a distance of y_ε from the neutral axis due to M. The bending moments were generated by hanging the dead weights on the model pile with a simply-supported boundary condition. Simultaneously, the caused longitudinal strains $\Delta\varepsilon_l$ were logged by the strain gauges. The average flexural rigidity EI was calibrated to be 30 GNm^2 in prototype scale.

The weight of the model pile was measured to be 9.14 N. However, a soil plug would form after the pile installation. To measure the total weight, both the soil plug and pile were taken out together after the test. Further considering the soil plug and most of the pile shaft being under water, the resultant buoyancy force has to be deducted from their total weight. Based on the principle of the Archimedes drainage method, their final effective weight was measured to be 10.04 N (G_{pile} = 1255 kN in prototype dimension). In addition, by the suspension method, the center of gravity was measured to be almost at the mid-length of pile.

2.3. Model set-up

After the in-flight consolidation, the centrifuge was then spun down to assemble the load test frame and to penetrate the pile vertically into the soil at a constant rate of 1 mm/s using a closed-loop hydraulic servo-valve control system at 1*g*. Craig [7] and Ilyas et al. [19] reported that for piles in clay, no significant difference in the lateral pile capacity was noted for piles installed at 1*g* or at high *g*.

Numerous centrifuge tests have been reported to model the lateral behaviour of a free-head pile. A popular way of designing the loading systems is to connect one end of an actuator (a device for providing horizontal loads) to the pile head by a hinge to model the free-head condition, and to fix the other end on a stiff loading frame, which helps to provide the counter force and maintain the height (eccentricity) of the actuator during loading [15,19,20,21,41,43]. To investigate the reason of the continuous increase in the reaction force, a consistent loading method is adopted in the current test.

The T-bar penetrometer (see Fig. 2) has been widely used to measure the undrained shear strength profile of clay in-flight [11,14,41]. Hence, a T-bar penetration test was adopted in this study. The T-bar used in the experiment has a model length of