Review of offshore monopile design for wind turbine towers

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ABSTRACT

There are several options in determination of foundation types for offshore wind turbine towers. The foundation types are mainly determined based on the sea depth. For relatively shallow sea depth, jacket structure or monopile is one of the most preferred offshore wind turbine foundation types. Foundations of wind turbines are more critical against lateral loads or moments compared to their vertical load; therefore, reliable assessment on their lateral behavior is important. As the existing design specifications of laterally loaded pile is developed based on the database from the experiments of lateral load test results of small-diameter piles. Validation of suitability of using the existing design specifications to design monopiles against lateral load is important. Therefore, in this study, review on offshore foundation design for wind turbine towers especially focusing on lateral behavior of monopile was conducted.

Keywords: offshore wind turbine foundation, lateral behavior, monopile, large-diameter steel pile

1 INTRODUCTION

Recently, oil and gas prices have dropped significantly compared to those at the early 2010s. The development of wind turbines has marched in place these days; however, development on wind turbine technologies should be continued for the preparation of renewable energy regulations such as renewable portfolio standard (RPS) or renewable obligation in the near future.

In the past when oil and gas prices were high, renewable energy was considered as an alternative clean and sustainable energy. Energy from wind turbines was one of the most favourable clean energy resources. Serious efforts and investments was put on development of efficient wind turbine facilities (generators, gears, and blades) and successfully larger energy production capacity with larger size wind turbines were manufactured and installed both of onshore and offshore conditions.

Against the increased sizes of wind turbine superstructures, their foundations sizes were also required to have much higher capacities against vertical and lateral loads. For wind turbines, offshore wind turbines are preferred to onshore ones to resolve the inherent noise and vibration problems produced from generators and blades. There are still numerous ongoing or future offshore wind turbine projects in the Northern Europe, the United States, China, and Korea.

To support the increased size and loads of the offshore wind turbine superstructures, the diameters of offshore monopiles have increased up to 10 m. Full-scale lateral load tests in fields on these large monopiles practically not possible due to high cost and difficulty in setting high capacity loading facilities under severe offshore environment.

Offshore wind turbine monopiles are more critical to horizontal and moment loadings than to vertical loading, the appropriate use of p-y method is crucial. The most widely used p-y method is the one recommended by the American Petroleum Institute (API), and engineers use the API p-y method to design laterally loaded large-diameter piles without any serious criticism. The API p-y method (API, 1993) is developed after Reese et al. (1974) and O’Neill and Murchison (1983) based on series of lateral loading experiments on small diameter piles (pile diameter less than equal to 610 mm). The proposed API p-y method may not be suitable for its use on offshore wind turbine large-diameter monopiles. Therefore, diameter effect on the existing p-y method should be evaluated.

For clays, diameter effects on the p-y formulation were experimentally identified by Reese et al. (1975), Stevens and Audibert (1979); however, there is limited literature supporting the diameter effect on p-y method for sands.
In this study, as full-scale test on lateral behaviour of offshore wind turbine monopile is impractical, centrifuge tests were performed on large-diameter monopiles in sands.

2 CENTRIFUGE TEST PREPARATION

The 240 g-ton and 5-m radius geotechnical centrifuge facility at the Korea Advanced Institute of Science and Technology was used. Two levels of centrifuge accelerations, 60g and 75g, were considered to simulate approximately diameter of 6m of monopile. Pairs of strain gauges were attached at different levels on monopile models (Table 1) to assess moment, stress, and lateral movements.

For the formulation of sand layers, nonplastic find and clean sands [grouped SP based on Unified Soil Classification System; ASTM D2487 (ASTM, 2011)] with fine contents of 1% were used and their relative densities were controlled by adjusting target falling height of sand particles in the container, which will attached to centrifuge facility. Such preparation of sand layers is named “air pluviation method.” The particle size distribution is shown in Fig.1 with the comparison of other standard sands. The main reason of using smaller sizes of sand particles compared with the other standard sands is to minimize the scale effect between pile surface roughness and sand particles.

When assessing the lateral behavior of piles, interface friction between pile surface and surrounding soils is an important factor. The interface friction is a function of ratio of median particle size (d_{50}) to pile surface roughness (Uesugi et al. 1990; Subba et al. 1998; Porcino et al. 2003). Internal fiction angle of sands is another important factor of lateral pile response. The peak friction angle of the sands varied from 42.8° to 47.5° depending on the confining pressure based on the triaxial test results.

Table 1. Pile configurations for different g levels.

| g-level (g) | Material     | E (GPa) | Model | Prototype |
|------------|--------------|---------|-------|-----------|
|            |              | D_{m} (mm) | t_{m} (mm) | D_{p} (m) | I_{p} (m⁴) |
| 60 Stainless steel | 199         | 101.6 | 2.0 | 6.1 | 10.06 |
| 75 Copper   | 117          | 79.4  | 1.2 | 6.0 | 7.13 |

*E is the elastic modulus of monopile; D_{m} and t_{m} are the model-scale diameter and wall-thickness of monopile; D_{p} and t_{p} are the prototype-scale diameter and wall-thickness of monopile.

Fig. 1. Particle size distribution of the sands used in the experiment.

Fig. 2 shows schematics of completion of pile and soil (or rock) layer setup. The weathered rock was made by mixing sand and cement and cured to have unconfined compressive strength of 5 MPa. The soft rock layer is manufactured by carving a huge natural granite rock having unconfined compressive strength range 117~147 MPa.

The five tests cases in Table 2 are planed varying the pile material, the acceleration level, the relative densities of sands, the end bearing conditions, and the embedded depth.
### Table 2. Detailed test conditions.

| Test ID | g-level (g) | D (m) | L (m) | L/D | Sand Thickness (m) | Dr (%) | Note |
|---------|-------------|-------|-------|-----|--------------------|--------|------|
| C1      | 75          | 6.0   | 31.0  | 5.2 | 38                 | 86     |      |
| C2      | 75          | 6.0   | 42.8  | 7.2 | 50                 | 84     |      |
| C3      | 60          | 6.1   | 31.4  | 5.2 | 37                 | 82     |      |
| C4      | 60          | 6.1   | 31.4  | 5.2 | 20                 | 87     | Rock Bearing Layer |
| C5      | 60          | 6.1   | 31.4  | 5.2 | 20                 | 58     |      |

*D is the diameter of monopile; L is the embedment length of monopile; Dr is the relative density of sands.

3. **LOADING AND UNLOADING TESTS**

After preparation of centrifuge container setup, the container is attached to main centrifuge testing apparatus and is rotated at 70g for an hour for the full soil saturation and stabilization of foundation.

Tests were performed by controlling displacement rate to capture any softening behavior of monopile. The small displacement rate of 0.05 mm/s is implemented to prevent excessive pore pressure within the foundation.

#### 3.1 Load-displacement curves

In the experiment, considering the loading characteristics of offshore wind turbine foundations, relatively high lateral force and moment are applied (by controlling displacement) at the loading point of 33 m above the seabed in prototype scale. However, the lateral displacement \( y_{5.5} \) at 5.5 m above the seabed in the prototype scale is measured to evaluate lateral monopile behavior.

For the five cases in Table 2, the relationships of lateral load \( H \) and lateral displacement \( y_{5.5} \) are presented in Fig. 3.

![Fig. 3. Relationships of lateral load and lateral displacement of offshore monopiles presented in Table 2.](image)

#### 3.2 Soil reaction \( p \) and lateral displacement \( y \) calculation based on strain measurements

Based on the measured moment profiles obtained from pairs of strain measurements, the moment \( M \) profile equation should be determined using the following form to mathematically integrate or differentiate its equation.

\[
M = a_0 + a_1 z + a_2 z^2 + a_3 z^3 + a_4 z^4 + a_5 z^5 \quad (1)
\]

The moment equation [Eq. (1)] provides both soil reaction \( p \) and lateral displacement \( y \) using the following equations:

\[
p = -\frac{d^3 M}{dz^3} \quad (2)
\]

\[
y = \int \left[ \phi dx \right] dz = \int \left[ \frac{M}{EI} dz \right] dx \quad (3)
\]

where \( z \) is the depth from the seabed surface; \( \phi \) is the curvature of the monopile; and \( EI \) is the flexural modulus of the monopile.

The \( p-y \) relationship is obtained for each depth of interest within sand layers, and the \( p-y \) relationships from the measurements presented much lower initial stiffness \( k_i \) and ultimate soil reaction \( p_u \) compared to those from the corresponding API \( p-y \) relationships. This implies that the offshore large-diameter monopiles exhibit much softer lateral behavior that that of small diameter conventional piles.

To quantify both initial stiffness \( k_i \) and ultimate soil reaction \( p_u \), from the derived \( p-y \) measured points, \( k_i \) and \( p_u \) were calculated using the following equation based on the regression analysis.

\[
p = \frac{y}{k_i + \frac{y}{p_u}} \quad (4)
\]

#### 3.3 Initial stiffness \( k_i \) of experimental \( p-y \) relationship

Change of initial stiffness \( k_i \) of \( p-y \) relationship within sands with increasing depth is summarized in Fig. 4.
It is clear shown that initial stiffness $k_i$ values of all the five cases are much lower than that proposed by API method (API, 1993). Even the two rock-socketed monopiles had lower $k_i$ values compared to that of API (1993).

From the comparison of C1 and C2 cases, embedment depth to diameter ratio influences the change of $k_i$ value; however, its effect on $k_i$ is not significant. Monopile with the higher embedment ratio (L/D=7.2) had a slightly higher $k_i$ compare with the case of L/D=5.2.

Comparison of $k_i$ values between C2 and C3 cases shows that pile stiffness is another influential factor in determination of $k_i$ because C2 uses lower flexural rigidity compared with that of C3. The stiffer stainless steel monopile in terms of flexural rigidity had the higher $k_i$ values.

Comparison between the group C1–C3 and the group C4–C5, end bearing soft rock layer and lower bottom weathered layer increased $k_i$ values within sand layers. However, $k_i$ comparison between C4 and C5 cases revealed that relative density of sands plays a significant role in $k_i$ values.

### 3.4 Ultimate soil reaction $p_u$ of experimental p-y relationship

Only a few ultimate soil reaction $p_u$ values were identifiable for shallow depth because lateral pile movements were not sufficiently high. For increasing depth from the seabed, lateral pile movement decreases. Therefore, in this study, ultimate soil reaction $p_u$ is assumed to be equal to that proposed by API (1993). This assumption does not have significant effect on lateral monopile behavior because the allowable lateral displacement is not significant.

### 4 RESULTS

The results of lateral load-lateral displacement curve obtained from direct measurements of experiments were compared with those from p-y analysis based on API (1993) and experimentally derived p-y relationship. The comparisons were made to verify our developed experimental p-y relationships. From Fig. 5(a) through 5(e), there found good agreements between measured load-displacement curve and curve obtained from the developed experimental p-y relationships. However, much stiffer lateral behavior curve was found for API cases compared with the other two curves.
5 CONCLUSIONS

Series of centrifuge tests were performed for different pile geometries, pile materials, sand relative densities, end bearing conditions. Based on the test measurements and analyses, experimental p-y relationships were developed for offshore large-diameter (diameter of approximately 6 m in prototype scale) monopiles embedded in sand layers. Significant lateral load and moment were applied for the assessment of lateral behavior of offshore monopiles.

From the measured pairs of strains for different level along the pile shaft, moment profiles were fitted to equation. Then, experimental p-y relationships were obtained by differentiating and integrating the moment equations. Assumptions are made for the unidentifiable ultimate soil resistances in this experiment to be equal to those proposed by API (1993).

The measured lateral load-displacement curve from experiments matched well with load-displacement curve produced based on the developed p-y relationships. These two load-displacement curves exhibit much smoother monopile lateral behavior compared with the curve predicted based on the p-y relationships of API (1993).

The authors concluded that significant diameter effect exists on the p-y relationships. Serious caution is required in designing large-diameter piles when API recommendations (API, 1993) is used. The lateral displacement from the API method could be significantly underestimated.

Significant soil or rock layer stiffness effect on the p-y relationships of adjacent layers was identified in this experiment. As other several previous researchers mentioned, engineers should consider layer stiffness effect on p-y relationship of adjacent soil or rock layers in their engineering decision makings in practices.

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REFERENCES

1) American Petroleum Institute, (1993): Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Working Stress Design, API Recommended Practice 2A-WSD, 20th ed., Washington, D.C.
2) ASTM D2487. (2011). Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), Annual Book of ASTM Standards, ASTM International, West Conshohocken, PA.
3) O’Neill, M.W. and Murchison, J.M. (1983): An evaluation of p-y relationships in sands, Research Report No. GT-DF02-83. University of Houston, Department of Civil Engineering.
4) Porcino, D., Fioravante, V., Ghionna, V.N., and Pedroni, S. (2003): Interface behavior of sands from constant normal stiffness direct shear tests, Geotech. Test. J., 26(3), 1–13.
5) Reese, L.C., Cox, W.R., and Koop, F.D. (1974): Analysis of laterally loaded piles in sand, Proceedings 7th Annual Offshore Technology Conference, Houston, TX, May 5-8, 473–483.
6) Reese, L.C., Cox, W.R., and Koop, F.D. (1975): Field testing and analysis of laterally loaded piles in stiff clay, Proceedings 7th Annual Offshore Technology Conference, Houston, OTC 2312, 671-690.
7) Uesugi, M., Kishida, H., and Uchikawa, Y. (1990): Friction between dry sand and concrete under monotonic and repeated loading, Soils Found., 30(1), 115–128.
8) Stevens, J.B. and Audibert, J.M.E. (1979): Re-examination of p-y curve formulations, Proceedings 11th Annual Offshore Technology Conference, Houston, TX. OTC 3402, 397-403.
9) Subba, R.K.S., Allam, M.M., and Robinson, R.G. (1998): Interfacial friction between sands and solid surfaces, Proceeding of the Institution of Civil Engineers—Geotechnical Engineering, 131(2), 75–82.