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Reliability analysis of laterally loaded piles for an offshore wind turbine support structure using response surface methodology

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Abstract. With an increasing demand of a renewable energy, new offshore wind turbine farms are being planned in some parts of the world. Foundation installation asks a significant cost of the total budget of offshore wind turbine (OWT) projects. Hence, a cost reduction from foundation parts is a key element when a cost-efficient designing of OWT budget. Mono-piles have been largely used, accounting about 78% of existing OWT foundations, because they are considered as a most economical alternative with a relatively shallow-water, less than 30 m of seawater depth. OWT design standards such as IEC, GL, DNV, API, and Eurocode are being developed in a form of reliability based limit state design method. In this paper, reliability analysis using the response surface method (RSM) and numerical simulation technique for an OWT mono-pile foundation were performed to investigate the sensitivities of mono-pile design parameters, and to find practical implications of RSM reliability analysis.

Keywords: pile foundation; mono-pile; reliability analysis; response surface method; soil-pile interaction

1. Introduction

With an increasing demand of a renewable energy, several offshore wind farms are being planned around the world. It is well known that offshore wind turbine (OWT) foundations take account of a significant cost of the total budget. Hence, reducing of the foundation cost is the key points to more cost-efficient wind energy design projects. Many studies on OWT support structure have been conducted until recently (Adhikari and Bhattacharya 2011, Alati *et al*. 2014, Kim *et al*. 2014). Two aspects of interests when designing an OWT foundation are the selection of types of foundation and design methodology. Firstly, there have been developed various types of OWT foundations, including gravity-typed structures, mono-pile, jacket, tripods and suction bucket types.

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Most of those types of foundations include pile foundation except gravity-typed structure. Mono-pile is especially the most popular, accounting for 75~80% of existing OWT foundations, by reason that those have been the most economical alternative with the relatively shallow-water depths (Doherty and Gavin 2011). Hence, laterally loaded piles play an important role in OWT foundations. Secondly, design methods are categorized into two main approaches, in which one is deterministic approach and the other is probabilistic approach. Deterministic approach such as allowable stress design in general is based on factor of safety criteria believed to be conservative. International trend of design standards is gradually changing toward probabilistic approach. Actually, the design codes for OWT such as IEC, GL, DNV, API, ISO and Eurocode have been developed in the form of limit state design method based on semi-probabilistic and reliability approach. On the one hand, one of the well-known methods in practice to analyze pile behaviors under lateral loadings is to model the pile as a vertical beam supported by a set of springs. The p-y curves vary according to soil properties, pile dimension, soil profile, etc. However, due to the inherent uncertainties in nature, it is difficult to determine the load-displacement characteristics of the p-y curves with soil depth precisely in deterministic approach. The factor of safety obtained in a deterministic manner does not explicitly take account of the uncertainties of load and resistance such as wind, wave and soil properties. Because of such uncertainties, it is necessary to adopt a probabilistic approach which can consider implicit uncertainties in load and resistance when pile designing. This paper deals with the reliability analysis of mono-pile foundation for an OWT, which are going to install at the test-bed site in West-South coastal zone of Buan-Yeongkwang at the Yellow Sea of Korea. The 5 MW NREL wind converter was employed in the study, and reliability analyses using response surface method and Monte Carlo Simulation technique for the serviceability limit states of the OWT mono-pile were performed. Numerically comparison studies of the reliability analysis with a deterministic analysis are carried out and practical implications of the RSM based reliability method are discussed.

2. Reliability analysis methodlogy for a laterally loaded pile

2.1 Analysis of laterally loaded pile behaviors

For laterally loaded piles, the beam on elastic foundation approach using the p-y curve method is often adopted to obtain load responses and lateral displacements upon loading. This model characterizes soils as a series of discrete linear or non-linear elastic springs at interconnected nodal points of assumed pile segments (Winkler 1867).

$$E_p I_p \frac{d^4 y}{dz^4} + p(y) = 0 \tag{1}$$

where E_p is the elastic modulus of pile; I_p is the moment of inertia of pile section; y is the horizontal displacement; z is the depth coordinate; $p(y)$ is a function which represents the nonlinear load-deflection relationship of the soil surrounding the pile.

Different methods have been proposed in sands and clays, respectively. For piles embedded in clays, the methods proposed by Matlock (1970), Reese *et al*. (1975) and Dunnavant and O'Neill (1989) are the representative cases that are popular in practice as adopted in international design specifications (API 2005, DNV 2013).

2.2 Reliability analysis methods

Reliability analysis has been frequently applied to civil and offshore structures design and safety reassessment of the existing structures. Failure probability of the performance function of the structures can be estimated by carrying out reliability analysis using the Monte Carlo simulation (MCS) and the first/second-order reliability methods (FORM/SORM). MCS is a numerical process to evaluate the performance function throughout the repeated calculations based on a large number of realizations of the random variables defining the function. A MCS starts with a generation of random numbers with respective prescribed probability distributions. Methods for generating a set of random numbers with well-known distributions are widely available. The accuracy of the probability of failure obtained through MCS will improve with the sample size which is number of random numbers generated for each distribution. The ordinary Monte Carlo method can be prohibitively costly for cases with very small failure probabilities, and where the deterministic analysis for each simulation trial is computationally intensive.

Reliability index approach is one of the most reliable computational methods for structural reliability. Practical difficulty or unnecessary hardship in computing probability of failure directly has led to the development of various approximation methods, of which the first-order reliability method (FORM) is considered to one of the most reliable computational methods. FORM is an analytical approximation in which the reliability index is interpreted as the minimum distance from the origin to the limit state surface in standardized normal space and the most probable failure point (MPFP, design point) is searched using mathematical simulations.

2.3 Response surface method

The FORM generally demands the values and partial derivatives of the limit state function (LSF) with respect to the design random variables. Such calculations can be performed efficiently when the LSF $g(X')$ can be expressed in an explicit form or simple analytical form in terms of the design random variables X'. However, when the LSF is implicit, such calculations require additional efforts. A few approaches have been developed to cope with the problems with implicit LSF. One of the popular approaches is the response surface method (RSM). Response surface is the derived virtual surface which can be represented by the function of random variables. The surface is found by regression with limited responses from structural analysis and expressed in an explicit function of random variables. Then FORM is easily applied by using approximate response surface function. LSF in implicit form can be written as

$$g(X) = R(X_1, X_2, \ldots X_n) - S(X_1, X_2, \ldots X_n) \tag{2}$$

where R is the resistance; S is the loading function; X_i is the random variable.

First/Second order approximation of Eq. (2) can be expressed as

$$g'(X) = c_0 + c_1 X_1 + \cdots + c_n X_n \tag{3}$$

$$g''(X) = c_0 + \sum_{i=1}^{k} c_i X_i + \sum_{i=1}^{k} c_{ii} X_i^2 + \sum_{i<j} c_{ij} X_i X_j \tag{4}$$

where C_i is regression coefficient estimated by using structural responses.

The approximated function $g'(X)$ is a first-order model, when the response is a linear function of independent variables. When there is a curvature in the response surface, the first-order model is insufficient, a second-order model is useful in approximating a portion of the true response surface. The second-order model includes all the terms in the first-order model, plus all quadratic terms like $C_{ii}X_i^2$ and all cross product terms like $C_{ij}X_iX_j$.

It is important to select sampling points for the accuracy of approximation of response surface. There are many designs available for fitting a second-order model. The most popular one is the central composite design (CCD) and the other one is the Bucher-Bourgunnd (B-B) method. The CCD involves 2k the axial points, 2^k factorial points and 1 central point. While B-B method involves only the axial points and central point but not cross term of factorial points. In these methods, sampling points to evaluate the coefficients C_0, C_i, C_{ij} are possible combinations of X_i's. The sampling points are selected to be located at $\mu \pm f \cdot \sigma$, where μ and σ are the mean and the standard deviation, respectively, and f is the axis point distance, which is a parameter determining the upper and lower limits in selection ranges.

The probabilistic characteristics of an original limit state may not be properly represented by the response surface function evaluated from information at the sampling points in the vicinity of the mean values of basic random variables. To improve the accuracy of the response surface method, Bucher and Bourgunnd (1990) suggested an alternative process of selecting the sampling points. In the first step of this algorithm, the mean vector is selected as the center point. Then the response surface is used to find an estimate of the design point, X_D, on an interpolated limit state. In next step, the new center point is chosen on a straight line from the mean vector μ_X to X_D so that $g(x) = 0$ at the new center point, X_M, from linear interpolation, i.e.

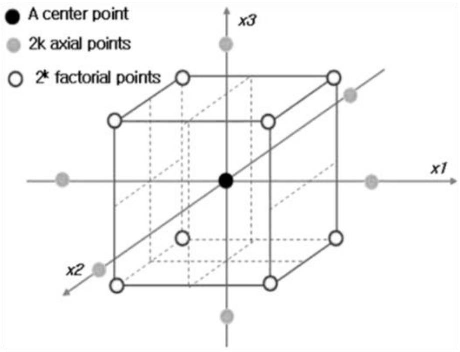
$$X_M = \mu_X + (X_D - \mu_X)\frac{g(\mu_X)}{g(\mu_X) - g(X_D)} \tag{5}$$

This process is assumed to guarantee that the sampling points chosen according to the new center point include information on an original failure surface sufficiently. This method is also called the adaptive response surface method.

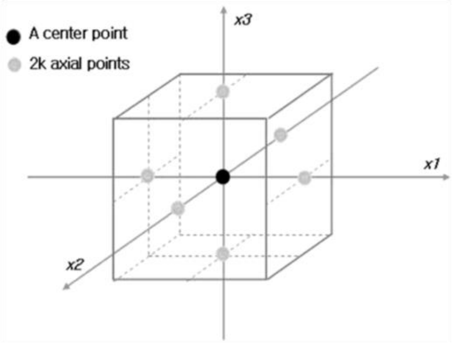
3. Reliability analysis

3.1 Analysis case

This paper focuses on preliminary design of OWT foundation at the test-bed site of Buan-Yeongkwang in the Yellow Sea of Korea. NREL 5.0MW OWT mono-pile type is referred for a comparison as shown in Fig. 2, which has a hub height of 87.6 m and water depth of 15.0 m (see Table 1). The combined load calculations at a seabed are shown in Table 2, which is based on DLCs 1.3, 1.4 and 6.2 of IEC 61400-3 standard. Ground conditions are also shown in Table 3, which were estimated from geotechnical report including SPT, CPT and unconfined & triaxial compression tests at the test-bed site (KEPRI, 2013). Table 3 shows that material properties of seabed soils are for a total stress analysis, considering that seabed soils consist of low permeable clay layer.



(a) Central Composite Design(CCD)



(b) Bucher-Bourgunnd(B-B) method

Fig. 1 Experimental designs for fitting response surfaces

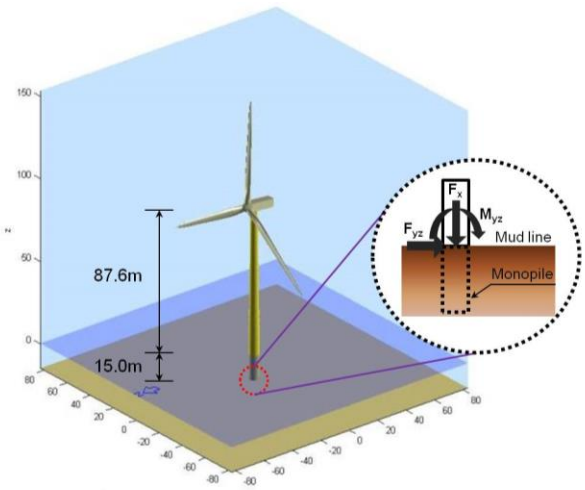


Fig. 2 Reference offshore wind turbine (OWT) model

Table 1 Dimensions of the reference OWT

	Turbine	Hub height (m)	Water depth (m)
Dimensions	NREL 5.0 MW	87.6	15.0

Table 2 Combined loads at seabed (KEPRI, 2013)

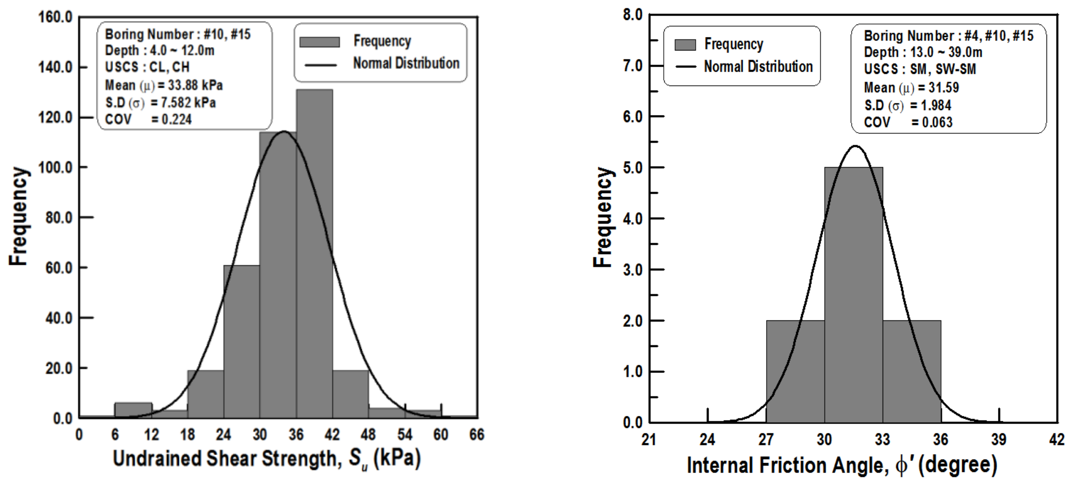
	F_x (kN)	F_{yz} (kN)	M_{yz} (kN·m)
Load	11,525.0	1,676.9	168,507.0

Table 3 Ground conditions and material properties of seabed soils

Soil layer		Depth (m)	Thickness (m)	Unit weight, $_{sat}$ (kN/m^3)	Cohesion, c (kPa)	Internal friction angle, (°)
Clay	CH	0~5.0	5.0	17.0	20.00	-
	CL(1)	5.0~12.3	7.3	18.0	33.54	-
Sand	SM	12.3~23.0	10.7	19.0	16.63	31.59
Clay	CL(2)	23.0~30.0	17.0	18.0	60.00	-

Table 4 Mono-pile foundation dimensions

	Case 1	Case 2	Remarks
Pile diameter (m)	6.0	7.0	steel pile
Pile wall thickness (mm)	60.0	30.0	
Embeded pile length (m)	21.6	22.5	embedded in sand



(a) Undrained shear strength (b) Internal friction angle

Fig. 3 Histogram showing soil properties of the target area

It is important to quantify the uncertainties of loads such as gravity, wind, wave, current, and material properties in a probabilistic approach. Reliability analysis requires probability distribution function and variability of design parameters. In this paper, undrained shear strength including cohesion and internal friction angle, which are main design parameters of seabed soils, are defined as random variables. As shown in Fig. 3, they are normally distributed and coefficients of variance (COV) are estimated 0.26 (26%) and 0.063 (6.3%) as cohesion and internal friction angle, respectively, based on statistical analysis from site investigation results (Yoon *et al*. 2014).

Mono-pile foundation dimensions such as pile diameters, thickness and embeded pile length are assumed reasonably through a preliminary analysis based on the above described ground conditions. Table 4 presents dimensions for two types of steel mono-pile in terms of pile diameter 6.0 m and 7.0 m. The pile is modeled by beam elements with Young's modulus of 2.1×10^8 kPa and unit weight 77 kN/m^3. And surrounding soils are discretized by nonlinear springs to model soil-pile interaction. In this paper, t-z and q-z curves are based on API (2005), and p-y curves are modelled by API (2005) and Evans and Duncan (1992) to clay and sand layers, respectively. Element size was specified to be 0.1 m.

Generally in the serviceability limit state design for the stability of the whole OWT structure under lateral loads such as wind, wave and current, etc., it shall be ensured that lateral deflection and rotational angle tolerances should not be exceeded. Accordingly, the major failure modes of mono-pile are considered as the lateral pile head displacement and rotational angle, and limit state functions (LSFs) can be expressed as follows

$$g_1 = \delta_a - \delta_{\max}(c_{u1}, c_{u2}, c_3, \phi_3) \tag{6}$$

$$g_2 = \theta_a - \theta_{\max}(c_{u1}, c_{u2}, c_3, \phi_3) \tag{7}$$

where δ_a and θ_a are the allowable lateral displacement and rotations of the pile head; δ_{max} and θ_{max} are the lateral pile head displacement and rotational angle; c_{u1} is the undrained shear strength of clay_CH layer; c_{u2} is the undrained shear strength of clay_CL(1) layer; c_3 and ϕ_3 are the cohesion and internal friction angle of sand_SM layer.

δ_{max} and θ_{max} are performance functions of random variables such as c_{u1}, c_{u2}, c_3 and ϕ_3 by numerical investigation results. δ_a and θ_a are considered as 1% of pile diameter and 0.3 degrees respectively (DNV 2013 and Kuo *et al*. 2008). To convert Eqs. (6) and (7) to explicit one, g_1 and g_2 are expressed as a function of design random variables as follows

$$\begin{aligned} g_1' = {} & C_0 + C_1 c_{u1} + C_2 c_{u2} + C_3 c_3 + C_4 \phi_3 + C_5 c_{u1}^2 + C_6 c_{u2}^2 + C_7 c_3^2 + C_8 \phi_3^2 \\ & + C_9 c_{u1} c_{u2} + C_{10} c_{u1} c_3 + C_{11} c_{u1} \phi_3 + C_{12} c_{u2} c_3 + C_{13} c_{u2} \phi_3 + C_{14} c_3 \phi_3 \\ & + C_{15} c_{u1} c_{u2} c_3 + C_{16} c_{u1} c_{u2} \phi_3 + C_{17} c_{u1} \phi_3 + C_{18} c_{u2} c_3 \phi_3 + C_{19} c_{u1} c_{u2} c_3 \phi_3 \end{aligned} \tag{8}$$

$$\begin{aligned} g_2' = {} & C_0' + C_1' c_{u1} + C_2' c_{u2} + C_3' c_3 + C_4' \phi_3 + C_5' c_{u1}^2 + C_6' c_{u2}^2 + C_7' c_3^2 + C_8' \phi_3^2 \\ & + C_9' c_{u1} c_{u2} + C_{10}' c_{u1} c_3 + C_{11}' c_{u1} \phi_3 + C_{12}' c_{u2} c_3 + C_{13}' c_{u2} \phi_3 + C_{14}' c_3 \phi_3 \\ & + C_{15}' c_{u1} c_{u2} c_3 + C_{16}' c_{u1} c_{u2} \phi_3 + C_{17}' c_{u1} \phi_3 + C_{18}' c_{u2} c_3 \phi_3 + C_{19}' c_{u1} c_{u2} c_3 \phi_3 \end{aligned} \tag{9}$$

where $\mathrm{g}_1'$and g_2' are approximated functions of LSF; C_i and C_i' are the regression coefficients of response surface to be estimated from structural analysis.

Reliability analyses were conducted using in-house software "FROW" developed in KIOST (2012). FROW is a special purpose program for the offshore wind turbine foundation, which can analyze a mono-pile and gravity-typed structure under lateral and vertical loading. It computes not only foundation behavior such as settlement, deflection, axial stress, bending moment and soil response with respect to depth in nonlinear soils, but also its reliability index and failure probability based on Level-II and Level-III reliability analysis method. CCD and B-B method are used to formulate LSFs to reliability analysis. Subsequently, adaptive response surface method is used to obtain optimized approximated functions. And FORM is used to calculate a reliability index. Finally reliability indices computed by the RSM-FORM are compared with MCS results for verification. MCS with 100,000 trials were carried out for each LSF, in which output sample variance of LSF gives within 0.01%.

3.2 Reliability analysis results

Table 5 presents lateral displacements and rotational angles at pile head with the limit state failure modes, which were calculated at the mean values of random variables in a deterministic manner. Table 6 and Fig. 4 show reliability indices (β) and probabilities of failure (P_f) computed by the RSM-FORM and MCS for each critical failure mode. P_f herein means the probability exceeding allowable values of design criteria. These values were also estimated explicitly to account for the uncertainties of soil properties, which make it possible to draw more reasonable decisions in pile design, compared to the deterministic approach.

It can be observed, in Case 1, lateral deflections are dominant failure mode, whereas for Case 2, rotational angles are dominant failure mode at pile head. The βs by RSM-FORM and MCS in terms of each case critical failure mode show that the results of the adaptive RSM in combination with CCD and B-B method have the least relative errors, in the range of 0.3~1.93%, compared to MCS results, whereas the relative errors by only CCD or B-B method having not response surface modification procedure are relatively large in the range of 2.10~12.15% as shown Table 7. Hence, one can notice that the adaptive RSM is more capable of producing approximate performance functions and the RSM-FORM using the adaptive RSM could be useful tool for performing the reliability analysis in terms of computing time and accuracy.

Table 5 Structural response at the mean values of random variables

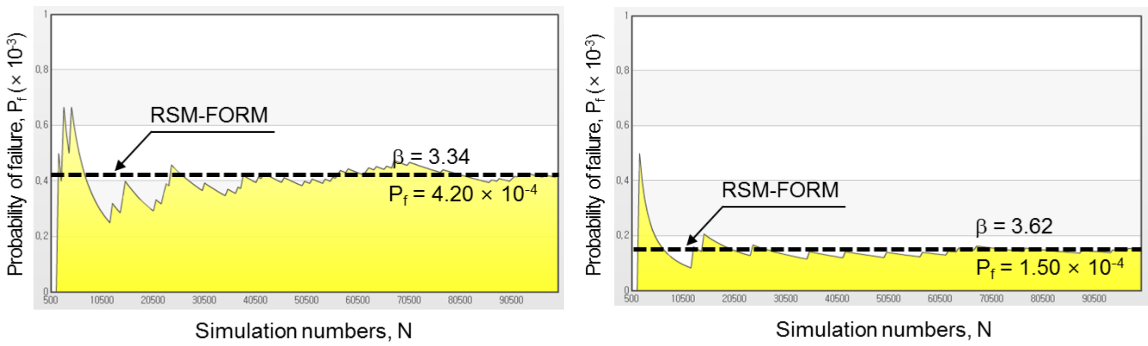
	Failure mode	Lateral Disp. (mm)		Rotation angle (°)	
		Computed	Allowable	Computed	Allowable
Case 1	Lateral Disp.	50.5	60.0	-	-
	Rotation angle	-	-	0.252	0.3
Case 2	Lateral Disp.	51.7	70.0	-	-
	Rotation angle	-	-	0.273	0.3

Table 6 Reliability index & probability of failure

	Failure mode	RSM-FORM			MCS	
		RSM	β	P_f	β	P_f
Case 1	Lateral Disp.	CCD	3.18	7.312×10^{-4}	3.34	4.200×10^{-4}
		B-B	3.41	3.261×10^{-4}		
		CCD & B-B	3.35	4.005×10^{-4}		
	Rotation angle	-	N/A	N/A	4.53	2.904×10^{-6}
Case 2	Lateral Disp.	-	N/A	N/A	5.91	1.739×10^{-9}
	Rotation angle	CCD	3.46	2.687×10^{-4}	3.62	1.500×10^{-4}
		B-B	4.06	2.439×10^{-5}		
		CCD & B-B	3.69	1.113×10^{-4}		

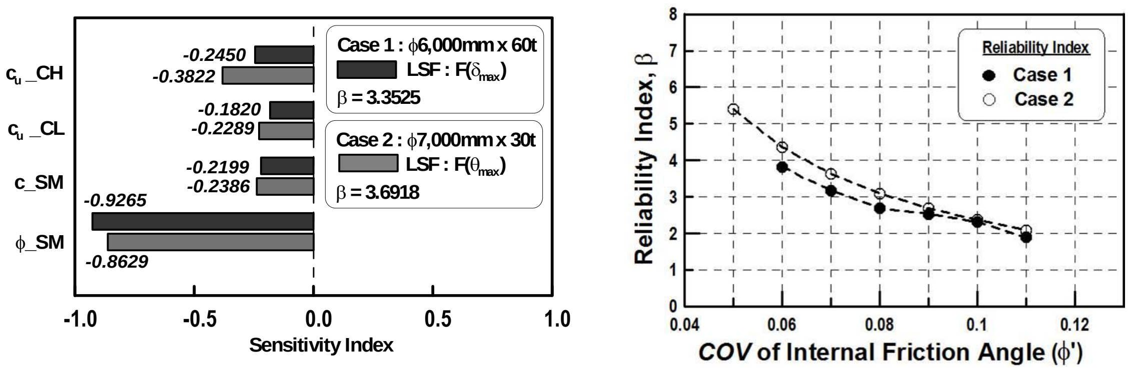
Table 7 Relative error compared to the MCS result

	Failure mode	RSM	β by RSM-FORM	Relative error (%)
Case 1	Lateral Disp.	CCD	3.18	4.79
		B-B	3.41	2.10
		CCD & B-B	3.35	0.30
Case 2	Rotation angle	CCD	3.46	4.42
		B-B	4.06	12.15
		CCD & B-B	3.69	1.93



(a) Case 1 (lateral displacement) (b) Case 2 (rotational angle)

Fig. 4 Comparison between MCS and RSM-FORM (CCD & B-B)



(a) Sensitivity indices of random variables (b) $COV_{random\ variables} - \beta$ relationships

Fig. 5 Sensitivity analysis results for uncertainty of random variables

Sensitivity indices of design parameters of random variables in a reliability analysis were derived by the RSM-FORM. As shown in Fig. 5(a), internal friction angle of sand layer (ϕ_SM) is a governing factor among random variables in mono-pile lateral behaviors. Accordingly, Fig. 5(b) shows that as COV of internal friction angle increase from COV=0.05 to COV=0.11, reliability index tend to decrease in the range of 50.41~61.21%. This means uncertainty of soil is greater, probability of failure exceeding the serviceability limit state is dramatically higher, and the accurate determination of the distribution of such parameters is very important in obtaining reliable probabilistic results.

4. Conclusions

Reliability analyses of laterally loaded pile for an NREL 5.0 MW mono-pile type offshore wind turbine support structure were performed to investigate the uncertainty and sensitivity of design parameters. Soil profiles and the uncertainties of soil were determined from a geotechnical site investigation at the test-bed in West-South coasts of Buan-Yeongkwang in Yellow Sea of Korea. First-order reliability method using the response surface method (RSM-FORM) and the Monte Carlo simulation were used in the analyses. The pile analyses were also carried out by modeling a laterally loaded pile as a vertical beam supported by a series of discrete springs, each of which has its own nonlinear load-displacement characteristics.

It is concluded that numerical results of the reliability analyses by the RSM-FORM agree well with those of the Monte Carlo simulations, with slightly differences in a probability of failure. It was also found that the adaptive RSM could also improve the accuracy of RSM-FORM. Finally, the sensitivity analysis of the design variables indicates that internal friction angle of seabed sandy soil is the most dominant design factor in mono-pile lateral behaviors, which shows that the variability of the internal friction angle increase, reliability index of mono-pile tends to decrease. Thus, the accurate determination of the uncertainty distribution of soil parameters is very important in obtaining reliable probabilistic results.

Acknowledgments

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