Available online at www.sciencedirect.com

ScienceDirect

Soils and Foundations xxx (2018) xxx–xxx

www.elsevier.com/locate/sandf

Robust design and optimization procedure for piled-raft foundation to support tall wind turbine in clay and sand

Nadarajah Ravichandran [a,*], Shweta Shrestha [b], Kalyan Piratla [c]

[a] Glenn Department of Civil Engineering, Clemson University, 202 Lowry Hall, Clemson, SC 29634, United States
[b] Glenn Department of Civil Engineering, Clemson University, 123 Lowry Hall, Clemson, SC 29634, United States
[c] Glenn Department of Civil Engineering, Clemson University, 310 Lowry Hall, Clemson, SC 29634, United States

Received 25 July 2017; received in revised form 15 January 2018; accepted 20 February 2018

Abstract

A geotechnical design and optimization procedure for piled-raft foundations to support tall wind turbines in clayey and sandy soil are presented in this paper. From the conventional geotechnical design, it was found that the differential settlement controlled the final design and was considered as the response of concern in the optimization procedure. A parametric study was subsequently conducted to examine the effect of the soil shear strength parameters and wind speed (random variables) on the design parameters (number and length of piles and radius of raft). Finally, a robust design optimization procedure was conducted using a Genetic Algorithm coupled with a Monte Carlo simulation considering the total cost of the foundation and the standard deviation of differential settlement as the objectives. This procedure resulted in a set of acceptable designs forming a Pareto front which can be readily used to select the best design for given performance requirements and cost limitations.

Keywords: Piled-raft foundation; Robust design; Optimization; Wind turbine; Renewable energy

1. Introduction

Wind energy, an alternative to conventional energy produced by burning fossil fuels, is a renewable and clean energy which produces no greenhouse gas emissions during operation, consumes no water, and uses only a little land. With the rapidly growing world population, it is essential to increase the production of energy using sustainable sources such as wind to meet the demand. One of the cost-effective ways to increase the production of wind energy is to build taller towers. Since a higher and steadier wind speed can be accessed at higher elevations, building taller towers can increase the wind energy production of a single turbine. The study of Lewin (2010) revealed that an increase in turbine elevation from 80 m to 100 m would result in a 4.6% higher wind speed which translates to a significant 14% increase in power output. A further increase in tower height from 80 m to 120 m would result in an 8.5% higher wind speed and a 28% increase in power output. It should also be noted that the higher initial construction cost and the lower operational cost of wind turbines make it economical to build a few taller towers rather than several normally sized towers to maximize the wind energy production.

Increase in tower height, however, leads to significant geotechnical engineering challenges because the foundation design loads (vertical load, horizontal load, and bending moment) increase with the increasing tower height. Larger loads not only result in the larger foundations demanding

Peer review under responsibility of The Japanese Geotechnical Society.
* Corresponding author.
E-mail addresses: nravic@clemson.edu (N. Ravichandran), kpiratl@clemson.edu (K. Piratla).

https://doi.org/10.1016/j.sandf.2018.02.027

significant resources to be allocated for the design and construction of the foundations, but they also present challenges in choosing the appropriate type of foundation as well as the optimal design parameters. Among the many types of foundations used for supporting wind turbines, a piled-raft foundation is considered to be effective for supporting tall wind turbines, especially for improving serviceability requirements (Shrestha, 2015). The centrifuge model tests performed by Sawada and Takemura (2014) on three types of model foundations (piled-raft, pile group, and raft alone) subjected to vertical, lateral, and bending moment loads also show that the vertical bearing capacity of the piled-raft foundation is the largest among the three foundations considered. This may be due to the higher bearing capacity of the raft and the increase in pile capacity due to the increase in soil stiffness caused by the raft contact stress. The same study also concludes that the settlement due to various loads can be reduced by using a piled-raft foundation.

The geotechnical design of a piled-raft foundation is complicated, especially when the foundation is subjected to a larger horizontal load and bending moment. The complexity increases even further when uncertainties in the wind load and the soil parameters must be incorporated into the design process, to increase its robustness, while keeping the cost at the lowest possible value. The selection of suitable design variables, such as the number of piles, the length of the piles, and the radius of the raft, for given loading and soil conditions, is another challenge because of the existence of a large number of acceptable designs. Selecting the best design that suits the performance and cost limitations is not straightforward in the conventional design. In such situations, the robust design optimization technique can be used to produce a relationship between the measure of robustness and the total cost of the foundation enabling the easy selection of the best design for a given set of performance requirements and cost limitations.

It is well recognized that the uncertainties of the soil parameters and the loads are unavoidable in the design of foundations. In a deterministic design approach, engineers use a factor of safety (FS) to cope with the uncertainties in the entire solution process. Usually, a larger FS is used when the uncertainties of the soil parameters and the loads are higher. Although design optimization is performed in the day-to-day engineering profession, the traditional optimization procedure becomes inefficient for the design problem pursued in this study. This is because the pool of acceptable designs in the traditional optimization is small and the problem is simplified to reduce the number of random and design variables within a manageable range. To consider the uncertainties in a systematic and accurate manner, a reliability-based approach supported by automated computer algorithms must be considered. Researchers have proposed various methods that consider the uncertainties in the soil parameters explicitly for the design of geotechnical as well as other engineering systems (Duncan, 2000; Griffiths et al., 2002; Phoon et al., 2003a, b; Fenton and Griffiths, 2008; Schuster et al., 2008; Juang et al., 2009, 2011; Wang et al., 2011; Zhang et al., 2011). Recently, one of the authors and his colleagues developed a reliability-based robust design methodology for the design of an individual drilled shaft in sand considering the uncertainties of the soil parameters (Juang et al., 2013). Additional literature on the geotechnical design concept and the design optimization is presented in the optimization section.

This methodology is employed in the current study for the design of a piled-raft foundation considering not only the uncertainties of the soil parameters, but also of the wind speed which affects the horizontal load and the bending moment. The spatial variation in strength and stiffness properties is unavoidable especially when the foundation design is being made for the construction of a wind farm which covers a large area. Conducting a subsurface exploration to accurately determine the soil properties and to design a piled-raft foundation for each wind turbine would be expensive and is not recommended in practice. Therefore, it is necessary to develop a design procedure considering the possible variations in soil properties so that the design will be accurate. Similarly, the wind speed which affects the horizontal load and the bending moment at the base of each tower also varies with the location, height, and time. Therefore, the wind speed must also be considered as an uncertain parameter in the design. Both aforementioned uncertain parameters have a significant impact on the selection of an optimum design for given site conditions, performance requirements, and cost limitations. A systematic incorporation of multiple random variables in the design requires an advance optimization procedure with predefined objectives such as cost and performance limitations.

To demonstrate the procedure, a 130-m-tall onshore wind turbine in clayey and sandy soil is considered. In the design optimization, the wind speed, the undrained cohesion of the clayey soil, and the friction angle of the sandy soil are taken as the random variables, while the length of the piles, the number of piles, and the radius of the raft are taken as the design variables. The differential settlement of the piled-raft, which is an overall stability parameter critical to fulfilling the serviceability requirement, is considered as the response of concern. The outcome of the optimization is presented in graphical form as a Pareto front which can be used to select the best design for a given set of performance requirements and cost limitations. The design procedure presented in this study can also be directly applied to other structures which are supported by a piled-raft foundation and subjected to combined vertical, lateral, and bending moment loads.

2. Deterministic geotechnical design of piled-raft foundation

2.1. Deterministic loads and soil properties

The wind turbine foundation is subjected to vertical load due to the self-weight of the superstructure, horizontal load due to the wind force on the above-ground components, and bending moment due to the wind load. The calculation of each load for the design is discussed below.

The vertical load on the foundation is the dead load due to the weight of all the components above the ground. It is calculated by summing the weights of the tower and the other components of the wind turbine such as the nacelle and the rotor. The sample wind turbine tower considered in this study is a hybrid hollow cylindrical tower with the lower 93 m made of concrete and the upper 37 m made of steel. Its diameter gradually varies from 12.0 m at the base to 4.0 m at the top. The weight of the nacelle and that of the rotor were obtained from Malhotra (2011). The final dead load of the tower is calculated to be 51.71 MN.

The wind action on the structures above the ground induces horizontal load on them which results in a horizontal force and bending moment at the base of the tower. The wind load is calculated following the procedure described in ASCE 7-10 (2010) using the mean survival wind speed of 125 mph. This mean wind speed is considered to be appropriate because most wind turbines have a survival wind speed of 112 mph to 134 mph (Wagner and Mathur, 2013) and its range lies between 89 mph and 161 mph. It is general practice to design wind turbines for the survival wind speed; and hence, the foundation here is also designed for the survival wind speed. The cut-off wind speed, which is lower than the survival wind speed, is not considered in this study. The standard deviation of the wind speed used in this study is 18 mph and the above-mentioned range covers ±2 standard deviations above and below the mean value (used in the parametric study and design optimization sections). This range in wind speed covers hurricanes of category 1–5 (5 being the extreme). The total horizontal load and bending moment are calculated to be 2.26 MN and 144.89 MNm, respectively.

For the design of a tower in clayey soil, a unit weight of 18 kN/m^3 and mean undrained cohesion of 100 kPa are assumed. These values represent stiff to very stiff clay. Based on the literature survey (Phoon et al., 2003a,b, 2008), the standard deviation of undrained cohesion is assumed to be 20 kPa. For the parametric study and optimization procedure, the undrained cohesion is varied between 60 kPa and 140 kPa. This represents ±2 standard deviations. The modulus of elasticity of the soil is calculated by employing widely used empirical correlations (USACE, 1990; Duncan and Buchignani, 1976) between the undrained cohesion and the modulus of elasticity. For the above-mentioned range in undrained cohesion, the range in the modulus of elasticity is calculated to be between 21 MPa and 49 MPa. Similarly, for the design of a tower in sandy soil, a site with a single layer of sandy soil is considered with the unit weight and mean friction angle of 17.2 kN/m^3 and 34°, respectively. A standard deviation of 3.4° is assumed for the friction angle. For the parametric study and design optimization, the friction angle is varied between 27.2° and 40.8°. This represents ±2 standard deviations. The modulus of elasticity of the sandy soil is varied between 1.25 MPa and 62.5 MPa (Wolff, 1989; Kulhawy and Mayne, 1990). These variations in the strength and deformation parameters and loading indicate that a significant variation in performance (safety and serviceability) is possible. This requires a systematic approach to quantify the variation in performance and corresponding cost which is the focus of this study.

2.2. Geotechnical design procedure

The advantage of a hybrid foundation, such as a piled-raft for supporting a larger load, is that it utilizes the higher bearing resistance of the raft to overcome bearing capacity failure and the higher resistance from piles to overcome total and differential settlements. Although individual design procedures for rafts and piles have been well documented, the design of piled-rafts is complicated and only a limited amount of documentation is available in the literature. Determining the share of the load carried by the raft and the piles and calculating the mobilized strength for a given settlement are the most challenging tasks in the design. This is mainly due to the lack of understanding of the complex soil-raft-pile interaction. Hence, a reliable design guideline is not yet available in the literature, particularly when the piled-raft is subjected to vertical, horizontal, and bending moment loads.

This study includes a preliminary geotechnical design of a piled-raft foundation following the procedure outlined by Hemsley (2000), in which the design procedure proposed by Poulos and Davis (1980) and Randolph (1994) are incorporated. In this procedure, the design variables, i.e., the radius of the raft, the length of the piles, and the number of piles, are assumed and adjusted until all the design requirements are met. To reduce the complication in the design procedure, the type and the size of the piles are fixed to be pre-stressed concrete piles of size 0.457 m (18″). The design requirements include checks for the vertical load capacity, the bending moment capacity, the horizontal load capacity, the total and differential settlements, and the rotation of the tower. A minimum factor of safety of 2 is considered to be safe (Hemsley, 2000) for the vertical load, horizontal load, and bending moment capacity checks. The maximum total settlement of 45 mm is allowed. A vertical misalignment within 3 mm/m of the tower is considered to be safe against the rotation of the tower (Grunberg and Gohlmann, 2013). For this allowable vertical misalignment, the safe horizontal displacement due to the bending moment at the top of a 130-m-tall tower is calculated to be 390 mm. Hence, the safe rotation of the tower (θ) is determined to be 0.17° calculated using the safe horizontal displacement and the height of the tower.

2.2.1. Check for vertical capacity

To determine the ultimate vertical load capacity of the piled-raft foundation, first the ultimate capacity of the individual components (i.e., raft $P_{u\text{-}R}$ and pile $P_{u\text{-}P}$) are calculated for the assumed trial dimensions. The ultimate bearing capacity of the raft is calculated using the general bearing capacity equation (Meyerhof, 1963). Since the piled-raft foundation in this study is for a wind turbine tower, a circular raft is considered so that there will be an equal capacity in all directions when the wind turbine rotor rotates. The size of the raft is determined based on the tower base diameter. Since the radius of the base of the tower in this study is 6.0 m, the radius of the raft is considered to be 7.5 m which provides sufficient clearance and doesn't cover a large area. The ultimate vertical pile capacity of a single pre-stressed concrete pile of size 0.457 m is calculated as the sum of the skin and the toe resistances. The skin resistance is calculated using α and the basic friction theory for piles in clayey and sandy soil, respectively, and the toe resistance is calculated using Meyerhof's method for both clayey and sandy soil (Das, 2016). Then, the ultimate vertical capacity of a block ($P_{u\text{-}B}$) is calculated as the sum of the ultimate vertical capacity of the circular pile group block of the soil, the raft, and all the piles and the portion of the raft lying outside the periphery of the pile group. Finally, the ultimate vertical load capacity of the piled-raft foundation is considered to be the lesser of: (i) the sum of the ultimate capacities of the raft and all the piles i.e., $P_{u\text{-}PR} = P_{u\text{-}R} + N_p P_{u\text{-}P}$, where N_p is the number of piles and (ii) the ultimate capacity of the block i.e., $P_{u\text{-}B}$. It should be noted that the determination of the number and the length of the piles is an iterative procedure. The number and the length of the piles are adjusted until all the design requirements are met. Finally, the factor of safety for the vertical load capacity is calculated using Eq. (1).

$$FS_P = \frac{\min(P_{u-PR}, P_{u-B})}{P} \tag{1}$$

where P is the design vertical load.

2.2.2. Check for moment capacity

The ultimate bending moment capacity of the piled-raft foundation is calculated following a similar procedure to that used for calculating the ultimate vertical load capacity, i.e., the lesser of: (i) the sum of the ultimate moment capacity of the raft ($M_{\text{u-R}}$) and all the individual piles in the group ($M_{u\text{-}P}$), i.e., $M_{u\text{-}PR} = M_{u\text{-}R} + M_{u\text{-}P}$, and (ii) the ultimate moment capacity of a block ($M_{\text{u-B}}$). The ultimate moment capacity of the raft, $M_{\text{u-R}}$, for the assumed dimension is calculated using Eq. (2) (Hemsley, 2000).

$$\frac{M_{u-R}}{M_m} = \frac{27}{4}\frac{P}{P_u}\left[1 - \left(\frac{P}{P_u}\right)^{1/2}\right] \tag{2}$$

where M_m is the maximum possible moment that the soil can support, P is the applied vertical load, and P_u is the ultimate centric load on the raft when no moment is applied. In this study, M_m for a circular raft is calculated by modifying the equation used to calculate M_m for a rectangular raft given in Hemsley (2000). The modified equation for M_m for the circular raft used in this study is given in Eq. (3).

$$M_m = \frac{q_u D^3}{4}\left(\frac{\pi}{4} - \frac{1}{3}\right) \tag{3}$$

where q_u is the ultimate bearing capacity of the raft and D is the diameter of the circular raft. The ultimate moment of all the piles, $M_{u\text{-}P}$, for the assumed length and number of piles is calculated using Eq. (4) (Hemsley, 2000).

$$M_{u-P} = \sum_{i=1}^{N_p} P_{uui}|x_i| \tag{4}$$

where P_{uui} is the ultimate uplift capacity of the i^{th} pile, $|x_i|$ is the absolute distance of the i^{th} pile from the center of the group, and N_p is the number of piles. Similarly, the ultimate moment capacity of the block, $M_{u\text{-}B}$, is calculated using Eq. (5) given below (Hemsley, 2000).

$$M_{u-B} = \alpha_B \bar{p}_u B_B D_B^2 \tag{5}$$

where B_B and D_B are the width and the depth of the block, respectively, $\bar{p}_u$ is the average lateral resistance of the soil along the block, and α_B is the factor depending upon the distribution of ultimate lateral pressure with depth (0.25 for a constant distribution of $\bar{p}_u$ and 0.2 for linearly increasing $\bar{p}_u$ with depth from zero at the surface). Hemsley (2000) proposed Eq. (5) for the design of rectangular rafts and pile arrangements. Since, in this study, the raft and the pile arrangements are circular, the raft section is converted to an equivalent rectangular section in order to use Eq. (5). Finally, the factor of safety is calculated using Eq. (6).

$$FS_M = \frac{\min(M_{u-PR}, M_{u-B})}{M} \tag{6}$$

where M is the design moment.

2.2.3. Check for horizontal capacity

Broms' solution for lateral pile analyses in cohesive and cohesionless soil, outlined in Gudmundsdottir (1981), was used to determine the lateral capacity of a single pile. Although it is for single pile analyses, it is assumed that all the piles in the group will have similar behavior. The horizontal coefficient of subgrade reaction is used to determine the horizontal load capacity ($V_{u\text{-}P}$) and horizontal deflection (y_H) of a single pile using the procedure described in Gudmundsdottir (1981) for sandy and clayey soil. The horizontal capacity of all the piles in the foundation system is estimated as the sum of the horizontal capacities of all the piles, i.e., $V_{u\text{-}PR} = N_p V_{u\text{-}P}$, assuming that all the piles in the group will behave in the same way. Finally, the factor of safety is calculated using Eq. (7).

$$FS_V = \frac{V_{u-PR}}{V} \tag{7}$$

where V is the design horizontal load.

2.2.4. Pile-raft-soil interaction and resultant vertical load-settlement behavior

The vertical load-settlement behavior of the piled-raft was estimated by the approach proposed by Poulos (2001) in conjunction with the method used for estimating the load sharing between the raft and the piles presented in Randolph (1994). The values of stiffness for the piles, the raft, and the pile-raft as a block are used to estimate the load sharing between the raft and the piles. The stiffness of the piled-raft, K_{pr}, is estimated using the following equation proposed by Randolph (1994):

$$K_{pr} = XK_p; \quad X = \frac{1 + (1 - 2\alpha_{rp})K_r/K_p}{1 - \alpha_{rp}^2(K_r/K_p)} \tag{8}$$

where K_r is the stiffness of the raft, K_p is the stiffness of the pile group, and α_{rp} is the pile-raft interaction factor. In this method, the interaction between the pile and the raft is incorporated by using the pile-raft interaction factor. However, the interactions between the raft and the soil and between the pile and the soil, which depend on the settlement, are not considered. The pile-raft interaction factor is assumed to be 0.8 considering the fact that as the number of piles increases the value of α_{rp} increases, and it reaches the maximum value of 0.8 as reported by Randolph (1994). The stiffness of the raft is estimated using the method outlined by Randolph (1994), while the stiffness of the pile group is estimated using the method proposed by Poulos (2001). In this method, the target stiffness of the piled-raft is determined by dividing the total vertical load by the assumed allowable settlement, and then Eq. (8) is solved to determine the stiffness of the pile group. When the foundation is subjected to the vertical load, the stiffness of the piled-raft will remain operative until the load-bearing capacity of the pile is fully mobilized at load P_A, as shown in Eq. (9) (also in Fig. 2). After calculating the values for K_{pr}, K_r, and P_A, the load-settlement curve (P vs. S) for the piled-raft foundation is developed using Eq. (9). Then, the settlement of the foundation due to the design vertical load is determined through the use of the load-settlement curve.

$$\left.\text{For } P \leqslant P_A; S = \frac{P}{K_{pr}} \quad \text{For } P > P_A;\ S = \frac{P_A}{K_{pr}} + \frac{P - P_A}{K_r}\right\} \tag{9}$$

2.2.5. Pile-raft-soil interaction, differential settlement, and tower rotation

When the piled-raft foundation is subjected to combined loading, piles on one side of the neutral axis will be in tension and those on the other side in compression. The vertical displacement of the piled-raft foundation due to horizontal and bending moment loads affects the vertical resistance of the piles on the tension and compression sides resulting in a difference in mobilized resistance (Sawada and Takemura, 2014). The difference in mobilized resistance results in the difference in the vertical displacement of the piles in tension and compression which, in turn, results in differential settlement (S_{diff}). During the vertical displacement, there will be interactions among the soil, the piles, and the raft which may have an impact on the capacity of the foundation. The calculation of the differential settlement of the combined piled-raft foundation system, due to the bending moment and considering the interactions among the various components, is a challenging task in the design of piled-raft foundations. The accurate procedure for estimating the differential settlement of a piled-raft foundation subjected to a coupled load (vertical load, lateral load, and bending moment) is not yet available in the literature. The present paper proposes a new method for calculating the differential settlement of a piled-raft foundation. In this method, the total applied bending moment is divided between the raft and the piles such that the differential settlements of the individual components are equal, which is considered to be the differential settlement of the piled-raft foundation. The assumption made here is that the pile head is connected rigidly to the bottom of the raft; and therefore, both the piles and the raft will rotate by the same amount when the foundation is subjected to a bending moment load. The estimation of the percentage of the moment shared by the raft and the piles, to induce an equal amount of differential settlement, is calculated using an iterative procedure in this study. A schematic of the proposed concept is presented in Fig. 1. The calculation of the differential settlement of the individual components (raft and piles) is discussed in the following section.

2.2.5.1. Differential settlement of raft. The differential settlement of the raft is estimated based on the rotation (θ) due to the wind load. The rotation is calculated using Eq. (10) given by Grunberg and Gohlmann (2013).

$$\theta = \frac{M_{found}}{c_s I_{found}}; \ c_s = \frac{E_s}{f' \sqrt{A_{found}}} \tag{10}$$

where M_{found} is the fixed-end moment at the soil-structure interface (percentage of the moment shared by the raft to result in a differential settlement equal to that of the piles in this study), c_s is the foundation modulus, I_{found} is the second moment of inertia for the area of the foundation, E_s is the modulus of elasticity of the soil, f' is the shape factor for overturning (0.25), and A_{found} is the area of the foundation. After calculating θ, the differential settlement of the raft is determined using a simple trigonometric relationship.

2.2.5.2. Differential settlement of piles. The differential settlement profile of the piles as a group is estimated considering the equivalent vertical loads due to the dead load and bending moment shared by the piles. First, the vertical load on each pile is estimated and then the settlement of each pile head is calculated following the procedure outlined by Fellenius (1999). As discussed above, the pile resistance will be different on the tension and the compression sides which will result in the difference in vertical settlement depending on the location of the pile with respect to the neutral axis. Hence, the settlements of the piles in a vertical

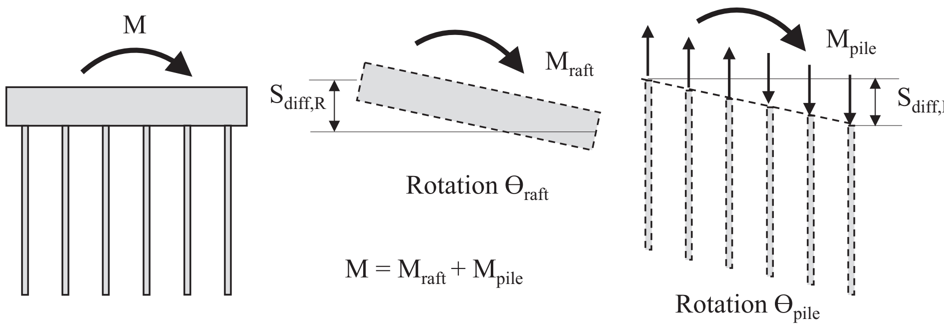


Fig. 1. Schematic of proposed differential settlement concept for piled-raft foundation.

section (2-dimensional elevation) are approximated by a straight line to produce the settlement profile for the piles. The above-mentioned procedure is repeated by adjusting the load shared by the piles and the raft until the settlement profiles for the piles and the raft match, which is considered to be the settlement profile of the piled-raft system.

The allowable differential settlement of the piled-raft foundation addressed in this study is 45 mm which is calculated using the allowable rotation (0.17°) and the diameter of the raft (15 m). The allowable horizontal displacement (ΔH) at the top of the tower is 390 mm.

2.2.6. Final design of piled-raft foundation

The final design results of the piled-raft foundation for the mean wind speed and the soil properties, obtained by following the above-mentioned procedure, are given in Table 1. Based on the checks of the vertical capacity and the moment capacity, presented earlier, it is found that the final design of the piled-raft foundation is controlled by the individual failure (either the raft or the piles fail, i.e., case 'i' in ultimate vertical and moment capacity determination) of both clayey and sandy soil. In both soils, the thickness of the raft is 1.2 m at the depth of 1.5 m. The total settlement (S_{tot}) listed in Table 1 is determined using the load-settlement curve for the piled-raft foundation shown in Fig. 2 for the design vertical load (51.71 MN). It can be observed in Table 1 that the final designs of the piled-raft foundation in both types of soil have satisfied the safety and settlement requirements. The total piles are divided equally and arranged circumferentially at radial distances of 5.3 m and 6.7 m at equal spacing maintaining the pile-to-pile spacing of at least three times the pile size.

It should also be noted in Fig. 2 that the design vertical load is smaller than P_A (=184.04 MN for clay and = 203.41 MN for sand) which indicates that both the raft and the piles are contributing to the support of the load and that the pile capacity is not fully mobilized at this vertical load. A sample pile configuration for the piled-raft foundation in clayey soil is illustrated in Fig. 3.

Although the structural design of a piled-raft foundation is important in terms of ensuring the structural safety of the foundation components, it is not considered in this study. This study focusses on performing the geotechnical design of the piled-raft foundation and addressing one of the design issues (calculation of the differential settlement). Other than the structural design, extreme events, such as hurricanes and earthquakes, and long-term events, such as consolidation, which the wind turbine may face during its lifetime, have not been explored in this study. Nevertheless, the authors' insights on the performance of the wind turbine during the occurrence of these events are briefly discussed here. Although the effect of hurricanes is not explicitly considered in this study, the wind speed range taken for the parametric study (next section) covers hurricanes of category 1 to 5 fairly well. However, the sustainability of wind turbine towers during such events is not investigated. Similarly, the authors believe that giving consideration to earthquakes in the design will add horizontal force to the wind turbine tower which will induce additional bending moment at the bottom of the tower demanding a larger foundation. In addition, when the wind turbine tower tilts, its center of gravity changes. This induces additional bending moment at the base of the tower. However, these components are not considered in this study. Likewise, the long-term consolidation settlement is not addressed here. The authors believe that if the consolidation settlement is considered, then the total and differential settlements will increase.

3. Design and random variables and conventional parametric study

A parametric study is conducted to determine the effect of the variation in the loading and the soil properties on the

Table 1
Design results of piled-raft foundation for mean case.

Soil	L_p (m)	N_p	R_r (m)	FS_P	FS_M	FS_V	S_{tot} (mm)	S_{diff} (mm)	ΔH (mm)	y_H (mm)
Clay	20	40	7.5	3.55	3.35	12.94	42.38	44.30	384.71	9.97
Sand	35	36	7.5	7.91	4.32	7.40	42.17	44.90	389.11	27.89

Please cite this article in press as: Ravichandran, N. et al., Robust design and optimization procedure for piled-raft foundation to support tall wind turbine in clay and sand, Soils Found. (2018), https://doi.org/10.1016/j.sandf.2018.02.027

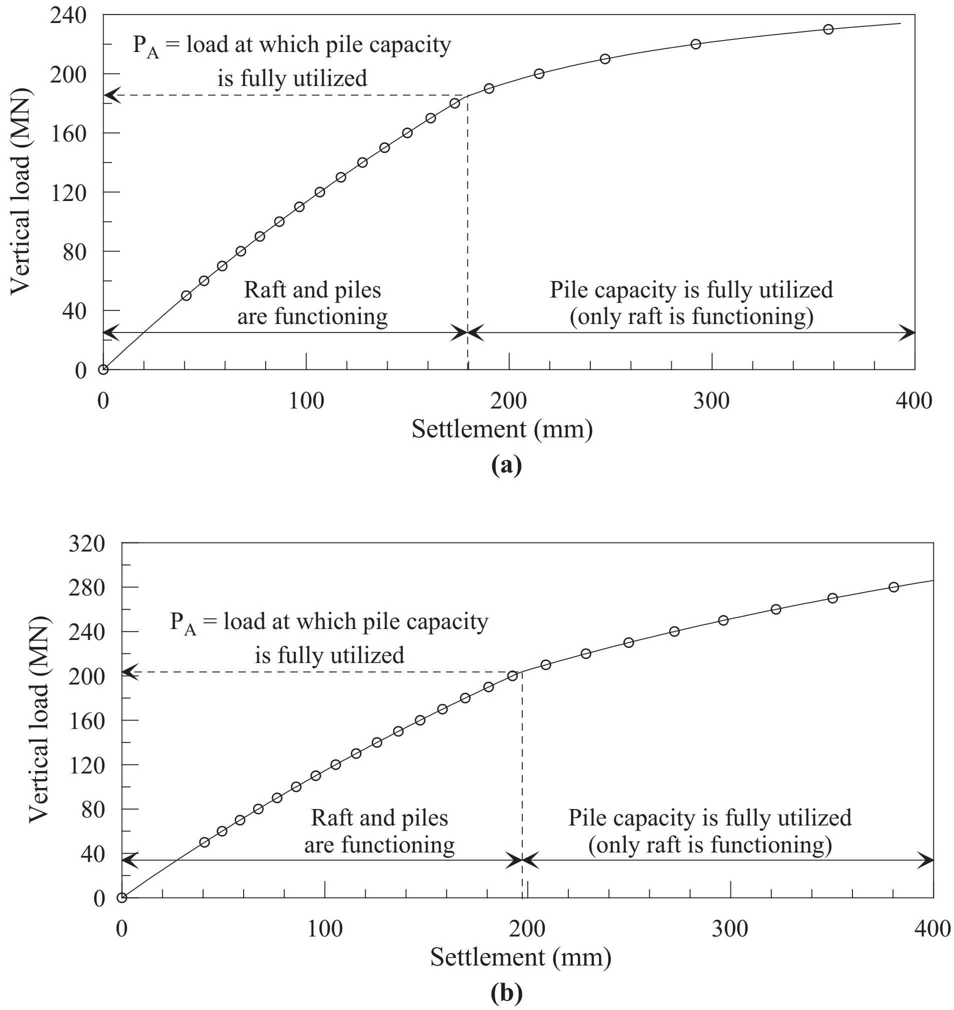


Fig. 2. Calculated load-settlement curves for piled-raft foundation in (a) clayey soil and (b) sandy soil.

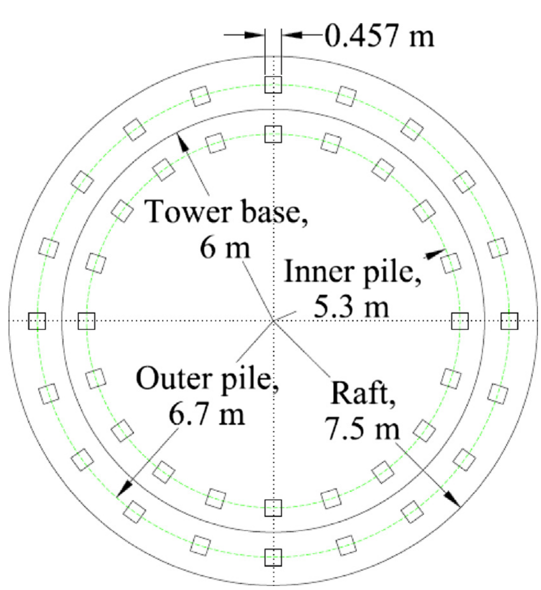


Fig. 3. Sample plan view of final design outcome for piled-raft in clay.

design outcomes. The random variables taken here are the undrained cohesion and the wind speed in the clayey soil and the friction angle and the wind speed in the sandy soil, and the design variables taken here are the number of piles, N_p, the length of the piles, L_p, and the radius of the raft, R_r, for both soils. For each case of the parametric study, the design requirements are met by adjusting only one of the design variables at a time and keeping the rest of them constant at their mean values. Details of the results of the parametric study for both soils are discussed below.

3.1. Variation in undrained cohesion

In this study, the variation in undrained cohesion (c_u) in the clayey soil is estimated by considering the low site variability. According to the SCDOT Geotechnical Design Manual (2010), the coefficient of variation (COV) for low site variability is less than 25%. Hence, for this study, 20% is selected as a reasonable COV (Phoon, 2008). Using the mean c_u value of 100 kPa and the COV of 20%, the standard deviation is determined to be 20 kPa. Hence,

c_u is varied between 60 kPa and 140 kPa, i.e., ±2 standard deviations considering a uniform probability distribution, and the designs are performed for each c_u value keeping the wind speed constant at its mean value. The effect of varying c_u on the design variables is shown in Fig. 4. The results indicate that N_p, L_p, and R_r decrease with increasing c_u because a higher c_u provides a higher bearing capacity of the piled-raft foundation. In Fig. 4(a), it can be noticed that N_p for the lowest c_u is 66. Dividing these piles equally along the two circumferences of radius 6.7 m and 5.4 m will not satisfy the pile-to-pile spacing requirement of at least three pile sizes. The maximum N_p that the circumferences of radius of 6.7 m and 5.3 m can accommodate, while maintaining the required pile spacing, are 30 and 24, respectively. Therefore, whenever N_p exceeds 54 (= 30 + 24), the extra piles, i.e., (N_p − 54), are arranged along the third circumference of radius 3.9 m. The radius of the third circumference is determined based on the spacing between the previous two circumferences. Moreover, it must be borne in mind that the maximum N_p allowed with the circumference of radius of 3.9 m is 18 to maintain the required pile-to-pile spacing. Hence, whenever N_p exceeds 72 (= 54 + 18), additional circumference is required. It is suggested that the additional circumference be added to the inner area, as much as the dimensions allow, because adding piles to the inside will not increase the surface area of the foundation. Nevertheless, when the additional circumference cannot be added to the inner area, due to size and/or space constraints, the size of the raft should be increased to add piles along the outer circumference if necessary. In Fig. 4(c), it can be seen that R_r remained the same even with the increase in c_u beyond 100 kPa because it cannot be lower than the bottom diameter of the tower.

3.2. Variation in friction angle

The variation in friction angle (ϕ) in the sandy soil is estimated by considering a COV of 10%, which is a suitable value for the friction angle variability (Phoon, 2008). The standard deviation of the friction angle is calculated to be 3.4° using a mean value of 34° and a COV of 10%. This resulted in a variation in ϕ between 27.2° and 40.8°, i.e., ±2 standard deviations considering a uniform probability distribution. The effect of this possible variation in design variables is shown in Fig. 5. The results indicate that N_p, L_p, and R_r decrease with an increasing ϕ because a higher ϕ increases the bearing capacity of both the raft and the piles resulting in the increase in bearing capacity of the piled-raft foundation. It can be seen in Fig. 5(a) that the N_p required for the lowest ϕ is 68. Hence, for this case, the piles are arranged along the three circumferences of radius 6.7 m, 5.3 m, and 3.9 m, as discussed in the previous section. It can be observed in Fig. 5(b) that the decrease in length that occurs while increasing the friction angle from 37.4° to 40.8° is insignificant compared to the other increments in the friction angle. This is because, for a friction angle of 40.8°, the bending moment requirement is not satisfied for the smaller-length pile even though it is overdesigned for the other design requirements. Hence, the bending moment capacity controls the design for the highest friction angle limiting the length of the pile to its minimum requirement (while the other design variables are kept constant). Similarly, the rate of decrease in R_r is not consistent for all friction angles, as can be seen in Fig. 5(c). The R_r required to fulfill all the design requirements for the lowest friction angle is found to be extremely large because the modulus of elasticity calculated using the correlation (Wolff, 1989; Kulhawy and Mayne, 1990) for the lowest friction angle is very low; and hence, it requires a very large raft to meet the differential settlement requirement. However, it is not practical to construct such a huge raft. In such a case, therefore, it is suggested that the length of the pile and the number of piles be increased. Similar to the clayey site, the minimum R_r requirement based on the bottom diameter of the tower is maintained for higher friction angles.

3.3. Variation in wind speed

The wind speed (V) is varied between the range of survival wind speed, i.e., between 89 mph and 161 mph (±2 standard deviations) with the mean value of 125 mph and the standard deviation of 18 mph considering a uniform probability distribution. The designs are performed for each wind speed keeping the undrained cohesion and friction angle constant at their mean values for clayey and

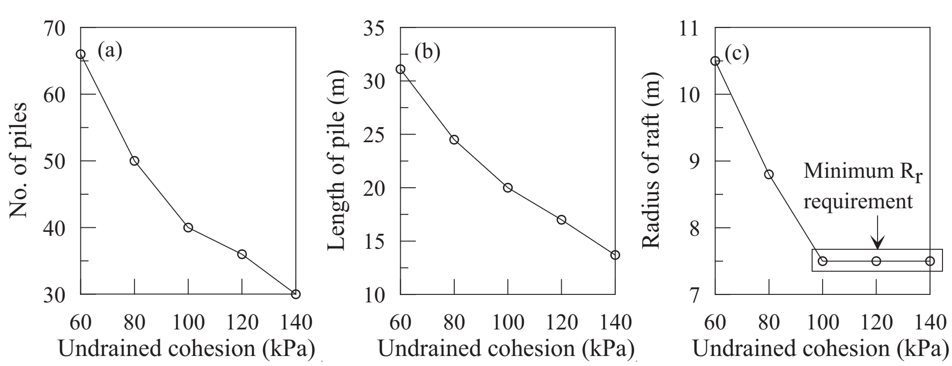


Fig. 4. Effect of variation in undrained cohesion on (a) number of piles, (b) length of piles, and (c) radius of raft in clayey soil.

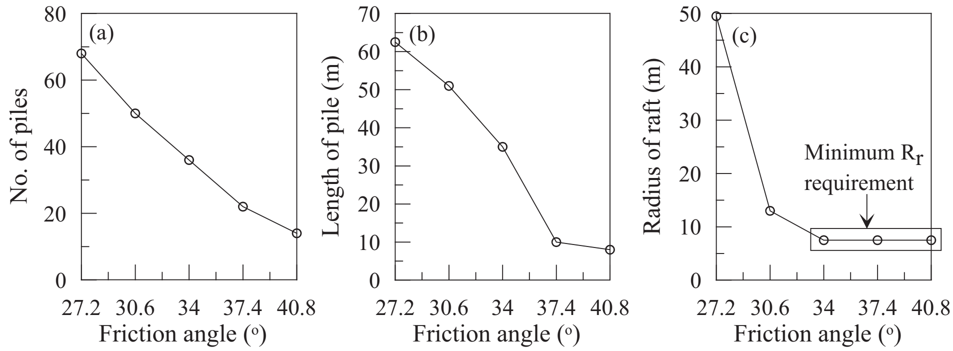


Fig. 5. Effect of variation in friction angle on (a) number of piles, (b) length of piles, and (c) radius of raft in sandy soil.

sandy soil, respectively, and varying only one design variable at a time to meet the design requirement. The adjustments required for N_p, L_p, and R_r, with the variation in V, are shown in Figs. 6 and 7 for the clayey soil and the sandy soil, respectively. For both soil conditions, it is observed that N_p, L_p, and R_r increase with an increasing wind speed. It can be seen in Fig. 6(a) that the N_p required for the highest V is 66. Hence, the piles are arranged along the three circumferences following the same rule as that in the previous section. It is noticed in Fig. 7(b) that the rate of increase in the length of the pile, when the wind speed is increased from 89 mph to 107 mph in case of sandy soil, is smaller compared to the other increments in wind speed (107–125, 125–143, and 143–161 mph). This is because it is found that, for the lowest wind speed, decreasing the length of the pile to below 22 m would result in a higher design load compared to the capacity. Thus, the minimum length of the piles required to carry the design axial load due to the moment and the self-weight are increased, although this results in the overdesign of the other design checks. Finally, for lower wind speeds, the radius of the raft is maintained at the minimum requirement for both soil conditions based on the bottom diameter of the tower, as shown in Figs. 6(c) and 7(c).

Although the above parametric studies show the changes in design variables for the range of possible variations in loads and properties of the soil, they only show the effect of a single variable at a time. Moreover, there is no

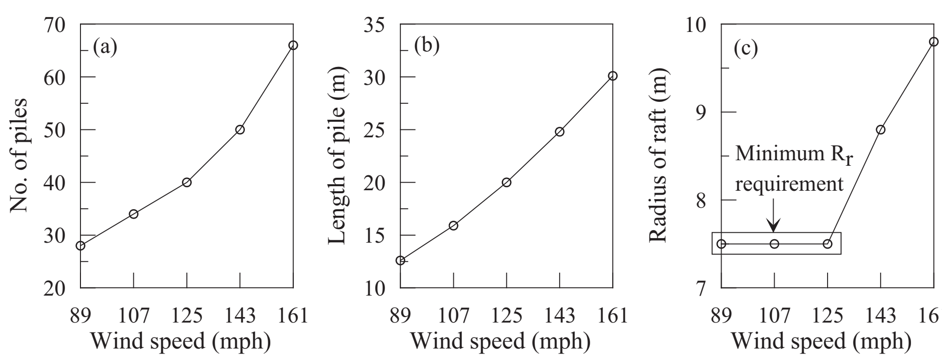


Fig. 6. Effect of variation in wind speed on (a) number of piles, (b) length of piles, and (c) radius of raft in clayey soil.

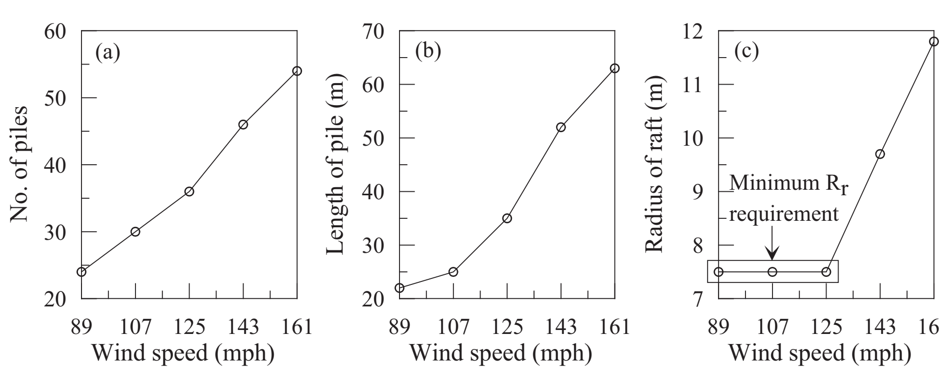


Fig. 7. Effect of variation in wind speed on (a) number of piles, (b) length of piles, and (c) radius of raft in sandy soil.

quantitative measure of the variation in the responses (differential settlement in this case) for the selected variation in loads and soil properties. Therefore, a procedure is needed that can systematically consider the randomness in the loads and soil properties and can provide a quantitative measure of the performance. Such a procedure is presented in the next section.

4. Robust design optimization of piled-raft foundation

4.1. Concept of robust design optimization

The conventional design of foundations is typically based on trial and error procedures considering cost and safety criteria. The lowest-cost design that satisfies the safety requirements is then identified and selected as the final design. In order to select the final design out of the pool of acceptable candidate designs, optimization tools can be employed for a desired performance criterion. Valliappan et al. (1999) performed a design optimization of a piled-raft foundation on c-ϕ soil. The objective functions in their study were the cost of the foundation and the design variables, including the thickness of the square raft, the cross-sectional area, and the length and the number of piles. The optimization was conducted by constraining the settlement and the differential settlement within the allowable limits. Kim et al. (2001) reported the optimal pile arrangements of a piled-raft foundation on clayey soil for different loading conditions. The optimization was performed to minimize the differential settlement. To this end, an implicit function of the locations of the maximum and minimum settlements of the square raft was taken as the objective function, while the locations of the piles were taken as the design variables. Chan et al. (2009) performed an optimization of pile groups in different multi-layered soils using a Genetic Algorithm (GA). Their objective was to minimize the material volume of the foundation subjected to several constraints, including the maximum differential settlement, while the design variables considered the location, the cross-sectional area, and the number of piles as well as the thickness of the square pile cap. In another study, by Leung et al. (2009), the piled-raft foundation was optimized with two objectives, namely, to maximize the overall stiffness and to minimize the differential settlement considering the length of the pile as a design variable. Although several of these previous studies presented efficient optimization approaches for piled-raft designs, they ignored to a large extent the uncertainties associated with the soil properties as well as the loads. These uncertainties in the input parameters cause uncertainties in the predicted system response, and a high variability in the response may lead to economically inefficient designs. Therefore, along with cost (or material usage) optimization, the concept of a robust design to identify the design least sensitive to uncertainties is employed in this paper. As shown in Fig. 8, introducing robustness into the design reduces the variation in system response and prevents the designed system from experiencing an unsatisfactory performance.

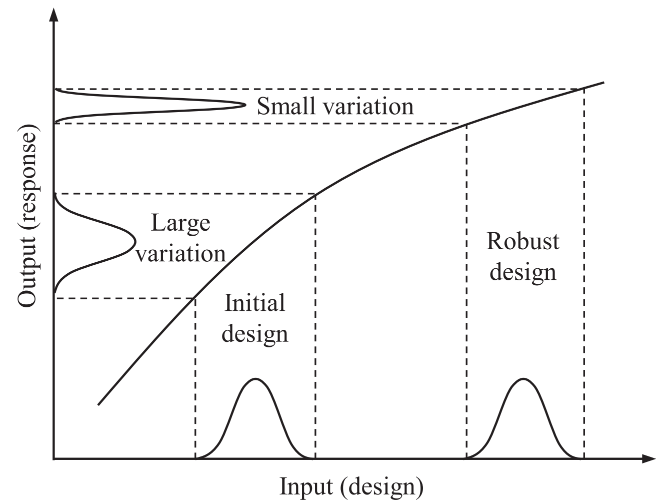


Fig. 8. Robustness concept (modified after Phadke, 1989).

In recent years, reliability-based robust designs for foundations and geotechnical systems have been employed frequently since the concept of uncertainties in soil and robust designs was introduced in geotechnical engineering. Juang and Wang (2013) presented a reliability-based robust design optimization method for shallow foundations using the Non-dominated Sorting Genetic Algorithm (NSGA-II). They assumed uncertainties in the soil parameters, such as the effective friction angle and the undrained shear strength, and considered the dimensions of the foundation as the design variables. The objectives of the optimization were to minimize the cost of construction and to maximize the robustness, considering the standard deviation of the failure probability as a measure of robustness. Juang et al. (2013) presented the robust geotechnical design methodology for drilled shafts in sand using NSGA-II. Sandy soil uncertainties, such as the friction angle, were included in that study and the design variables were the diameter and the length of the shaft. The shaft was optimized for cost, while the standard deviation of the failure probability was constrained to the target failure probability. Based on these studies, NSGA-II was found to be an effective and efficient tool for conducting a multi-objective optimization that would result in a set of preferred designs known as the Pareto-optimal front. In their study, the robust geotechnical design methodology was reported as a complementary design approach for conventional trial-and-error design procedures.

4.2. Proposed optimization procedure for piled-raft foundation using response surface

In this study, the design optimization of the piled raft-foundation to support a tall wind turbine subjected to vertical load, horizontal load, and bending moment at the foundation level is performed considering V (wind speed) and c_u (undrained shear strength) for clayey soil and V (wind speed) and ϕ (friction angle) for sandy soil as the uncertainty parameters (or random variables), while N_p, L_p, and R_r are considered as the design variables for both types of soil. The range and probability distribution of the random variables considered for the optimization are the same as those presented in the parametric study section of this paper, i.e., the uniform probability distribution. The uniform probability distribution is thought to be the simplest distribution among the ones commonly used in the robust design optimization. Nevertheless, the uniform distributions for the undrained shear strength and the friction angle are considered to be appropriate for this study because they have low coefficients of variations (20% for undrained cohesion and 10% for the friction angle) and cover a good range of stiff to very stiff clay and loose to dense sand, respectively, fairly well. In contrast, the wind speed is better represented by the Rayleigh, Weibull, Lognormal, Gamma, and Beta distributions (Morgan et al., 2010). Still, the uniform distribution is used for the wind speed in this study because the aforementioned distribution models are more complicated than the uniform distribution and some of them require more than two parameters. A parametric study may be conducted to investigate the effect of the probability distribution on the robust design optimization.

A bi-objective robust optimization is performed in this study using NSGA-II, a toolbox in MATLAB, to minimize the effect of the uncertainties on the response and to capture the set of optimal designs in terms of cost efficiency and robustness. The first objective considered here is the total cost of the piled-raft foundation calculated with the unit price data from the RS Means cost database. The unit prices of pre-stressed concrete piles and rafts are considered to be \$193.19/m and \$342.13/m^3, respectively (RS Means, 2013). It should be noted that these unit prices include the estimated costs for material, labor, and equipment, but exclude the overhead and profit. Since the design of piled-raft foundations is controlled by the differential settlement (S_{diff}), it was considered as the response of concern. As reported by Wang et al. (2014), the standard deviation of the response can be considered as an appropriate measure of the robustness resulting in a smaller variation in the response results corresponding to a more robust design. Thus, in the current study, the standard deviation of the differential settlement is taken as the second objective of the optimization. The standard deviation of the differential settlement for numerous design candidates is computed by coupling the optimization program with a Monte Carlo simulation using a code developed in MATLAB. The flowchart for the design and optimization procedure is presented in Fig. 9 with the details below.

To predict the approximate behavior of the piled-raft foundation in a simplified manner and to avoid thousands of cumbersome calculations, a response surface is developed based on the response and the variables (the random and design variables). For this purpose, several design sets (L_p, N_p, and R_r) are selected and the corresponding differential settlements are determined for the variation in random variables. A regression analysis of the results of the differential settlement on both soil types is subsequently performed to establish a response surface. For the clayey soil, the response surface is established in terms of S_{diff}, V, c_u, L_p, N_p, and R_r, as presented in Eq. (11). Similarly, for the sandy soil, the response surface is established in terms of S_{diff}, V, ϕ, L_p, N_p, and R_r, as presented in Eq. (12). The coefficients for the determination (or R^2) value obtained from the regression analysis are 0.91 and 0.90 for the clayey soil and the sandy soil, respectively. This indicates that the proposed response function fits the data reasonably well.

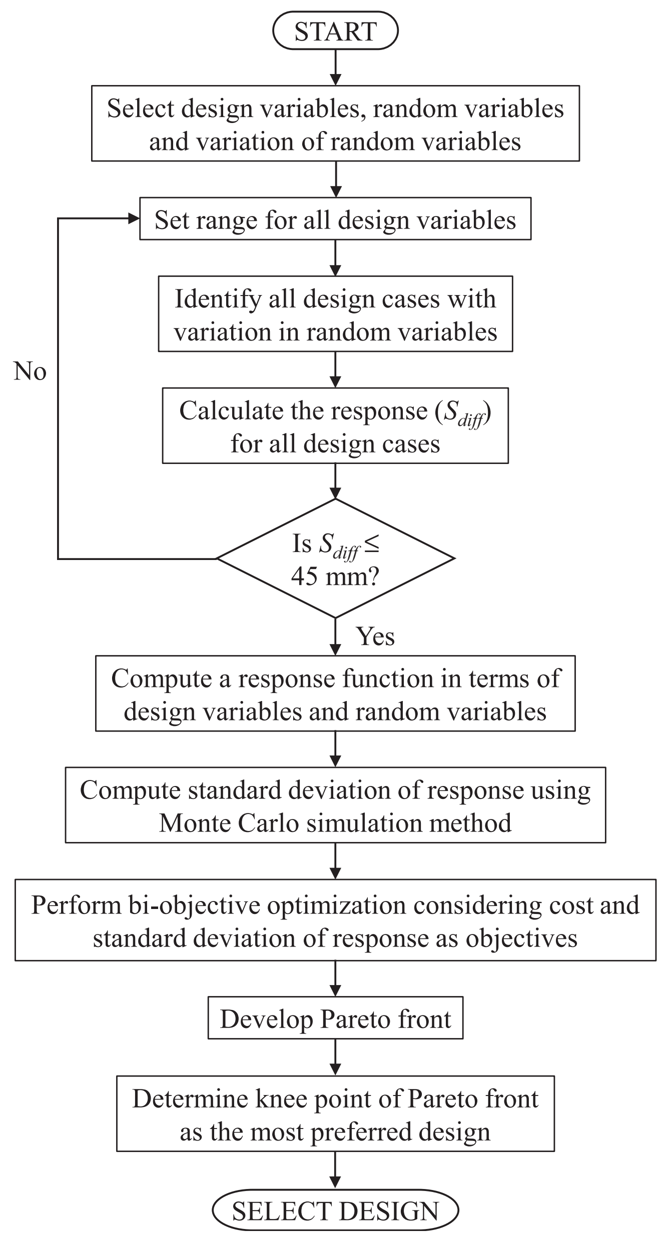


Fig. 9. Flowchart illustrating the geotechnical design optimization procedure.

Please cite this article in press as: Ravichandran, N. et al., Robust design and optimization procedure for piled-raft foundation to support tall wind turbine in clay and sand, Soils Found. (2018), https://doi.org/10.1016/j.sandf.2018.02.027

Foundation in clayey soil:

$$S_{diff} = \exp(19.74 + 3.74\ln(V) - 1.87\ln(c_u) - 3.04 \times \ln(L_p) - 3.66\ln(N_p) - 1.28\ln(R_r)) \quad (11)$$

Foundation in sandy soil:

$$S_{diff} = \exp(15.72 + 2.86\ln(V) - 2.19\ln(\phi) - 2.03 \times \ln(L_p) - 2.61\ln(N_p) - 0.54\ln(R_r)) \quad (12)$$

In this study, 10,000 simulations are performed to compute the standard deviation of the response for each design set considering the variation in random variables. From a parametric study, which is not presented in this paper, it was observed that 10,000 simulations produced a reasonably smoother Pareto front compared to 1000 simulations; and therefore, 10,000 simulations are considered adequate in this study. The robust design optimization procedure was also subjected to safety constraints of the allowable differential settlement ($S_{diff,all}$ = 45 mm) and the target reliability (β_t = 3), as the latter has been recommended by Kulhawy and Phoon (1996), to ensure the reliability of the foundation system. The reliability index of the system can be computed using the performance function of the system (g) defined as follows:

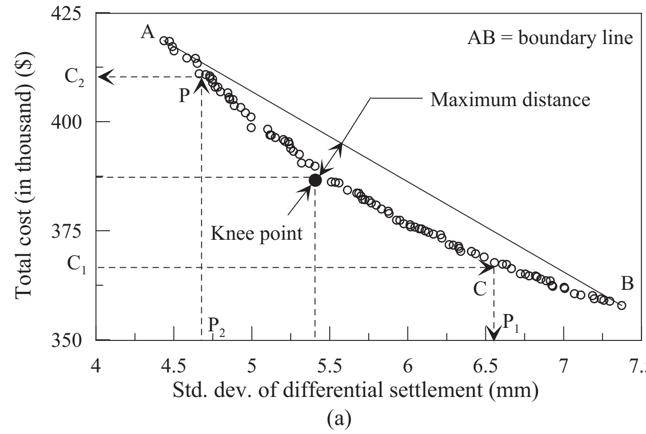
$$g(\theta, X) = S_{diff,all} - S_{diff}(\theta, X) \quad (13)$$

where θ and X indicate random variables and design variables, respectively. As seen in Eq. (14), the mean value of the performance function (μ_g) is calculated using the mean value of the response (differential settlement) which is computed via the MC simulation. It should be noted that the standard deviation of the performance function (σ_g) is equal to the standard deviation of the response also determined by the MC calculation ($\sigma_g = \sigma_{S_{diff}}$). The reliability index of the system (β) was then computed as expressed in Eq. (15), and the values less than β_t were considered unacceptable for the optimization.

$$\mu_g = S_{diff,all} - \mu_{S_{diff}} \quad (14)$$

$$\beta = \frac{\mu_g}{\sigma_g} \quad (15)$$

The preferred designs resulting from the optimization procedure are illustrated graphically in Fig. 10 in the form of Pareto fronts. The figure shows that the design with a lower cost may have higher vulnerability and higher response variability. It can be observed for the clayey soil in Fig. 10(a) that the standard deviation for the differential settlement increased from about 4.5 mm to 7.5 mm when the total cost of the foundation decreased from about \$420,000 to \$360,000. Similarly, for sandy soil, as shown in Fig. 10(b), the standard deviation for the differential settlement increased from about 5.0 mm to 8.1 mm when the total cost of the foundation decreased from about \$670,000 to \$540,000. It should be noted that the Pareto front changes with changes in the mean values of the random variables and the range in the design and random variables.



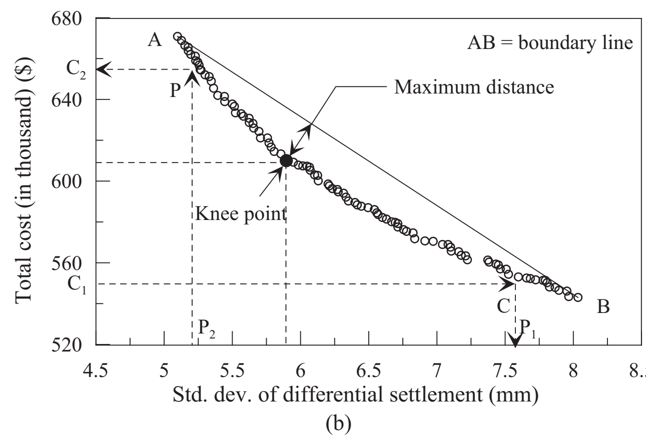


Fig. 10. Pareto fronts optimized for both total cost and standard deviation of (a) piled-raft in clayey soil and (b) piled-raft in sandy soil.

The most optimal design (i.e., balancing both objectives) can be obtained from the Pareto front using the knee point concept. Among the various methods available for determining the knee point, the normal boundary intersection (NBI) approach, illustrated in Fig. 10 and discussed in Juang et al. (2014) and Deb and Gupta (2011), is used in this study. In this method, the boundary line (AB) is created by connecting two extreme points in the Pareto front; then the distance of each point in the Pareto front from the boundary line is calculated. The point on the Pareto front with the maximum distance from the boundary line is identified and referred to as the knee point, as marked in Fig. 10. The optimal cost and the standard deviation of the response corresponding to the knee point are used to finalize the design solution.

The optimal length of the piles, the number of piles, and the radius of the raft for the wind tower designed in this study with clayey soil are found to be 30.4 m, 52, and 8.01 m, respectively, while the cost of that design is estimated to be \$386,580. Similarly, the results for the wind tower designed with sandy soil are 50.9 m, 54, and 7.96 m, respectively, with an estimated cost of \$610,024. A comparison between the conventional geotechnical design results, considering the mean design parameters, and the design optimization results, considering the variation in

Table 2
Comparison of conventional design and optimized design.

Soil	Conventional design				Optimized design				
	L_p (m)	N_p	R_r (m)	Total cost ($)	L_p (m)	N_p	R_r (m)	Std. dev of response (mm)	Total cost ($)
Clay	20	40	7.5	227,103	30.4	52	8.01	5.41	386,580
Sand	35	36	7.5	315,971	50.9	54	7.96	5.90	610,024

random variables for clayey and sandy soil, is given in Table 2. The standard deviation of the response (differential settlement) obtained from the design optimization is also presented in this table. For both types of soil, the introduction of the variation in random variables or noise factors in the design results in a more costly foundation compared to the conventional design for the mean design parameters. However, the variations in the responses (differential settlement) are reduced significantly for the foundation in both soils. It may be thought that the same response would have been achieved by using a larger factor of safety, but the robust design procedure presented in this paper considers multiple factors (random and design variables), reduces the variation in the responses systematically, and provides a numerical value for the variation in responses.

The use of Pareto fronts can be extended to determine cost-based and/or performance-based designs. For instance, as shown in Fig. 10, a client willing to spend C_1 for the construction of a foundation can select the design corresponding to point *C* on the Pareto front with the performance level of P_1. At the same time, a client who demands a certain level of performance, P_2 in Fig. 10, can select the design corresponding to point *P* on the Pareto front which will cost C_2 for the construction.

5. Conclusion

A geotechnical design optimization procedure for a piled-raft foundation to support a tall wind turbine on clayey and sandy soil is presented in this paper. The procedure can be easily extended to the geotechnical design of piled-raft foundations to support other structures. The geotechnical design conducted in this study followed the analytical equations available in the literature and indicated that the final design is controlled by the differential settlement and the rotation of the foundation rather than the bearing capacity or the total settlement. The parametric study showed that for both types of soil, the design requirements can be met by increasing the number of piles, increasing the length of the piles, or increasing the radius of the raft when the wind speed is increased. For a higher undrained cohesion (in clayey soil) and a higher friction angle (in sandy soil), a smaller foundation was enough to meet the design requirements. The robust optimization procedure resulted in easy-to-use graphs, called Pareto fronts, which show a clear trade-off relationship between the cost and the standard deviation of the responses (differential settlement) for both soils. Although these graphs can be utilized to select the suitable design for a given set of performance requirements (variation in differential settlement) and cost limitations, the most suitable design solution is determined using the knee point concept.

References

ASCE (American Society of Civil Engineers), 2010. Minimum Design Loads for Buildings and Other Structures, Standard ASCE/SEI 7-10. ASCE, Reston, VA, USA, ISBN 978-0-7844-1085-1.

Chan, C.M., Zhang, L.M., Ng, J.T., 2009. Optimization of pile groups using hybrid genetic algorithms. J. Geotech. Geoenviron. Eng. 135 (4), 497–505.

Deb, K., Gupta, S., 2011. Understanding knee points in bicriteria problems and their implications as preferred solution principles. Eng. Optim. 43 (11), 1175–1204.

Das, B.M., 2016. Principles of Foundation Engineering, eighth ed. Cengage Learning.

Duncan, J.M., 2000. Factors of safety and reliability in geotechnical engineering. J. Geotech. Geoenviron. Eng. 126 (4), 307–316.

Duncan, J.M., Buchignani, A.N., 1976. An Engineering Manual for Settlement Studies. Department of Civil Engineering, University of California, Berkley.

Fellenius, B.H., 1999. Basics of Foundation Design, second ed. BiTech Publishers, Richmond, British Columbia.

Fenton, G.A., Griffiths, D.V., 2008. Risk Assessment in Geotechnical Engineering. John Wiley & Sons Inc, Hoboken, NJ, USA, ISBN 978-0-470-17820-1.

Griffiths, D.V., Fenton, G.A., Manoharan, N., 2002. Bearing capacity of a rough rigid strip footing on cohesive soil: A probabilistic study. J. Geotech. Geoenviron. Eng. 128 (9), 743–755.

Grunbeg, J., Gohlmann, J., 2013. Concrete Structures for Wind Turbines. Wilhelm Ernst & Sohn, Berlin, Germany, ISBN 978-3-433-03041-7.

Gudmundsdottir, B., 1981. Laterally loaded piles. M.S. thesis, University of Alberta, Edmonton, Alberta, Canada.

Hemsley, J.A., 2000. Design Applications of Raft Foundations. Thomas Telford Ltd., Heron Quay, London, ISBN 0727727656.

Juang, C.H., Fang, S.Y., Tang, W.H., Khor, E.H., Kung, G.T.C., Zhang, J., 2009. Evaluating model uncertainty of an SPT-based simplified method for reliability analysis for probability of liquefaction. Soils Found. 49 (12), 135–152.

Juang, C.H., Schuster, M., Ou, C.Y., Phoon, K.K., 2011. Fully-probabilistic framework for evaluating excavation-induced damage potential of adjacent buildings. J. Geotech. Geoenviron. Eng. 137 (2), 130–139.

Juang, C.H., Wang, L., 2013. Reliability-based robust geotechnical design of spread foundations using multi-objective genetic algorithm. Comput. Geotech. 48, 96–106.

Juang, C.H., Wang, L., Liu, Z., Ravichandran, N., Huang, H., Zhang, J., 2013. Robust design of drilled shafts in sand: New design perspective. J. Geotech. Geoenviron. Eng. 139 (12), 2007–2019.

Juang, C.H., Wang, L., Hsieh, H.S., Atamturktur, S., 2014. Robust geotechnical design of braced excavations in clays. Struct. Saf. 49, 37–44.

Kim, K.N., Lee, S.H., Kim, K.S., Chung, C.K., Kim, M.M., Lee, H.S., 2001. Optimal pile arrangement for minimizing differential settlements in piled raft foundations. Comput. Geotech. 28 (4), 235–253.

Kulhawy, F.H., Mayne, P.W., 1990. Manual on Estimating Soil Properties for Foundation Design. Electric Power Research Institute, Palo Alto, California.

Kulhawy, F.H., Phoon, K.K., 1996. Engineering judgment in the evolution from deterministic to reliability-based foundation design. Proceedings of Uncertainty, vol. 96, pp. 29–48.

Leung, Y.F., Klar, A., Soga, K., 2009. Theoretical study on pile length optimization of pile groups and piled rafts. J. Geotech. Geoenviron. Eng. 136 (2), 319–330.

Lewin, T.J., 2010. An investigation of design alternatives for 328-ft (100-m) tall wind turbine towers. M.S. Thesis, Iowa State University, Ames, IA, USA.

Meyerhof, G.G., 1963. Some recent research on the bearing capacity of foundations. Canadian J. 1 (1), 16–26.

Malhotra, S., 2011. Selection, Design and Construction of Offshore Wind Turbine Foundations, Wind Turbines. Dr. Ibrahim Al-Bahadly (Ed.), InTech, ISBN 978-953-307-221-0.

Morgan, E.C., Lackner, M., Vogel, R.M., Baise, L.G., 2010. Probability distributions for offshore wind speeds. Energy Convers. Manage. 52 (2011), 15–26.

Phadke, M.S., 1989. Quality Engineering Using Robust Design. Prentice Hall, NJ, US.

Phoon, K.K., Kulhawy, F.H., Grigoriu, M.D., 2003a. Development of a reliability based design framework for transmission line structure foundations. J. Geotech. Geoenviron. Eng. 129 (9), 798–806.

Phoon, K.K., Kulhawy, F.H., Grigoriu, M.D., 2003b. Multiple resistance factor design for shallow transmission line structure foundations. J. Geotech. Geoenviron. Eng. 129 (9), 807–818.

Phoon, K.K., 2008. Reliability-Based Design on Geotechnical Engineering: Computations and Applications. Taylor and Francis, CRC Press.

Poulos, H.G., Davis, E.H., 1980. Pile Foundation Analysis and Design. John Wiley, New York, USA.

Poulos, H.G., 2001. Piled raft foundation: design and applications. Geotechnique 51 (2), 95–113.

Randolph, M.F., 1994. Design methods for pile groups and piled rafts. State-of-the-Arts Report, 13th International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India, 5, pp. 61–82.

RS Means Building Construction Cost Data 2013, Construction Publishers & Consultants, Norwell, MA, ISBN 978-1-936335-56-5.

Sawada, K., Takemura, J., 2014. Centrifuge model tests on piled raft foundation in sand subjected to lateral and moment loads. Soils Found. 54 (2), 126–140.

SCDOT Geotechnical Design Manual Version 1.1, 2010. “Chapter 7 – Geomechanics”.

Schuster, M.J., Juang, C.H., Roth, M.J.S., Rosowsky, D.V., 2008. Reliability analysis of building serviceability problems caused by excavation. Geotechnique 58 (9), 743–749.

Shrestha, S., 2015. Design and Analysis of Foundation for Onshore Tall Wind Turbines. M.S. thesis, Clemson University, Clemson, SC, USA.

U.S. Army Corps of Engineers, 1990. Engineering and Design – Settlement Analysis. Engineering Manual, No. 1110-1-1904.

Valliappan, S., Tandjiria, V., Khalili, N., 1999. Design of raft–pile foundation using combined optimization and finite element approach. Int. J. Numer. Anal. Meth. Geomech. 23 (10), 1043–1065.

Wagner, H.J., Mathur, J., 2013. Introduction to Wind Energy Systems: Basics, Technology and Operation, second ed. Springer Verlag, Berlin Heidelberg, ISBN 9783642329753.

Wang, Y., Au, S.K., Kulhawy, F.H., 2011. Expanded reliability-based design approach for drilled shafts. J. Geotech. Geoenviron. Eng. 137 (2), 140–149.

Wang, L., Luo, Z., Khoshnevisan, S., Juang, C.H., 2014. Robust Design of Braced Excavations Using Multiobjective Optimization-Focusing on Prevention of Damage to Adjacent Buildings. Geotech. Spec. Publ., 3178–3187.

Wolff, T.F., 1989. Pile Capacity Prediction Using Parameter Functions, *Predicted and* Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium, sponsored by the Geotechnical Engineering Division, ASCE, Evanston, IL, June, 1989, ASCE Geotechnical Special Publication (23), 96–106.

Zhang, J., Zhang, L.M., Tang, W.H., 2011. Reliability-based optimization of geotechnical systems. J. Geotech. Geoenviron. Eng. 137 (12), 1211–1221.