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Research Article

Geotechnical Preliminary Design of Onshore Wind Turbine Foundations

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Abstract
A simplified geotechnical design procedure for foundations of onshore wind turbines is often beneficial because it can provide the types and sizes of foundations required to perform financial feasibility analysis of a project, and can also be used for preliminary design. This study presents a simplified method for designing onshore gravity-based foundations based on limited required data, namely, loads acting on foundations, data provided by manufacturers, and soil properties. The detailed design issues that may be essential for the final design are also discussed. A flowchart of the design process is also presented to visualize the geotechnical design of such foundations. The required calculation procedures can be easily carried out either through a series of spreadsheets or simple hand calculations. An example problem explaining the design of onshore wind turbine foundations was presented to demonstrate the proposed calculation procedure. The data used for the calculations were obtained from the literature. The proposed flowchart provides a quick way for the preliminary design of foundations of onshore wind turbine foundations. The foundation system obtained from the preliminary geotechnical design was numerically modeled using Plaxis 3D software. The global stability of the foundation system under Ultimate Limit State (ULS) loads and soil pressure developed under the foundation were calculated. The soil pressure beneath the foundation under Serviceability Limit States (SLS) loads and the total settlement and rotation of the foundation were calculated. The safety calculation performed in the Plaxis 3D software verified the safety of the foundation dimensions calculated using the analytical method. Consistent with the analytical approach, the numerical calculation under SLS loading revealed that no gapping develops between the foundation and underlying soil. Furthermore, the values of settlement and rotation of the foundation through the application of the Strain Influence Method and Plaxis 3D were found to be compatible with each other.
Keywords: Onshore wind turbines, preliminary design, foundations, numerical analysis

Karadaki Rüzgar Türbin Temellerinin Geoteknik Ön Tasarımı

Öz

Karadaki rüzgâr türbinlerinin temelleri için basitleştirilmiş bir geoteknik tasarım prosedürü, bir projenin finansal fizibilite analizini yapmak için gerekli olan temel tiplerini ve boyutlarını sağlayabildiği ve ön tasarım için de kullanılabileceği için genellikle faydalıdır. Bu çalışma, temellere etki eden yükler, üreticiler tarafından sağlanan veriler ve zemin özellikleri gibi sınırlı verilere dayalı olarak karadaki ağırlık tipi temellerin tasarımı için basitleştirilmiş bir yöntem sunmaktadır. Nihai tasarım için gerekli olacak ayrıntılı tasarım konuları da tartışılmaktadır. Bu tür temellerin geoteknik tasarımını görselleştirmek için tasarım sürecinin bir akış şeması da sunulmuştur. Gerekli hesaplama prosedürleri, bir dizi elektronik tablo veya basit el hesaplamaları yoluyla kolayca gerçekleştirilebilir. Önerilen hesaplama prosedürünü göstermek için temellerin tasarımını açıklayan örnek bir problem sunulmuştur. Hesaplamalarda kullanılan veriler literatürden elde edilmiştir. Önerilen akış şeması, karadaki rüzgâr türbini temellerinin ön tasarımı için hızlı bir yol sağlar. Ön geoteknik tasarımdan elde edilen temel sistemi, Plaxis 3D yazılımı kullanılarak sayısal olarak modellenmiştir. Temel sisteminin Nihai Limit Durumu (ULS) yükleri altındaki genel stabilitesi ve temel altında gelişen zemin basıncı hesaplanmıştır. Servis Sınır Durumları (SLS) yükleri altında temelin altındaki zemin basıncı ve temelin toplam oturması ve dönmesi hesaplanmıştır. Plaxis 3D yazılımından elde edilen sonuçlar ile ön tasarımdan elde edilen sonuçların uyum içinde olduğu görülmüştür. Plaxis 3D yazılımında yapılan güvenlik hesaplaması, analitik yöntemle hesaplanan temel boyutlarının güvenliğini doğrulamıştır. Analitik yaklaşımla uyumlu olarak, SLS yüklemesi altındaki sayısal hesaplama, temel ile altındaki zemin arasında temas kaybı olmadığını ortaya çıkarmıştır. Ayrıca deformasyon etki yöntemi ve Plaxis 3D'nin uygulanmasıyla elde edilen temel oturma ve dönme değerleri birbirleriyle uyumlu bulunmuştur.
Anahtar Kelimeler: Kara rüzgar türbinleri, ön tasarım, temeller, sayısal analiz

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1. INTRODUCTION

The need for energy is increasing day by day due to the rapid increase in the human population and the rapid growth of the world economy (Figure 1). Fossil fuels are still mostly used in energy production (Figure 2a). For this reason, the problem of global warming with carbon dioxide emissions released into the environment has been revealed with all its nakedness. Developed countries have started to emphasize the use of renewable energy sources by turning to more environmentally friendly solutions to reduce carbon emissions to the environment. Among renewable energy sources, wind energy has significantly increased its share of global energy production in recent years (Figure 2b).

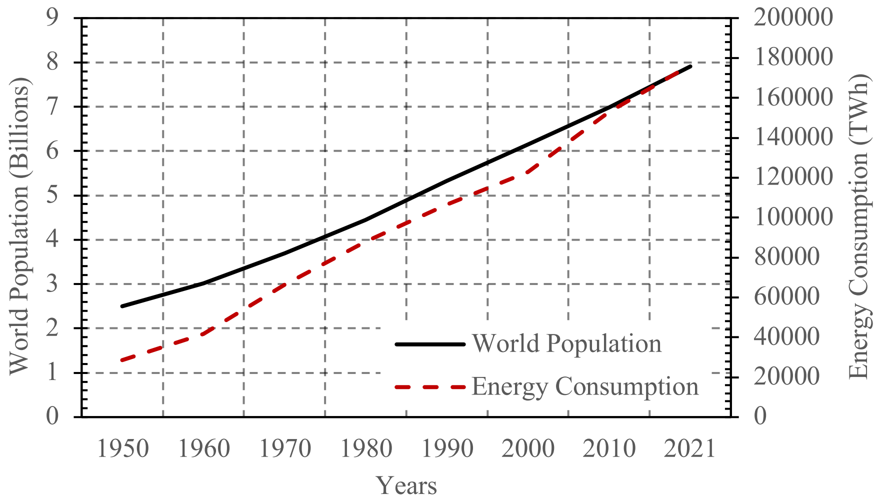
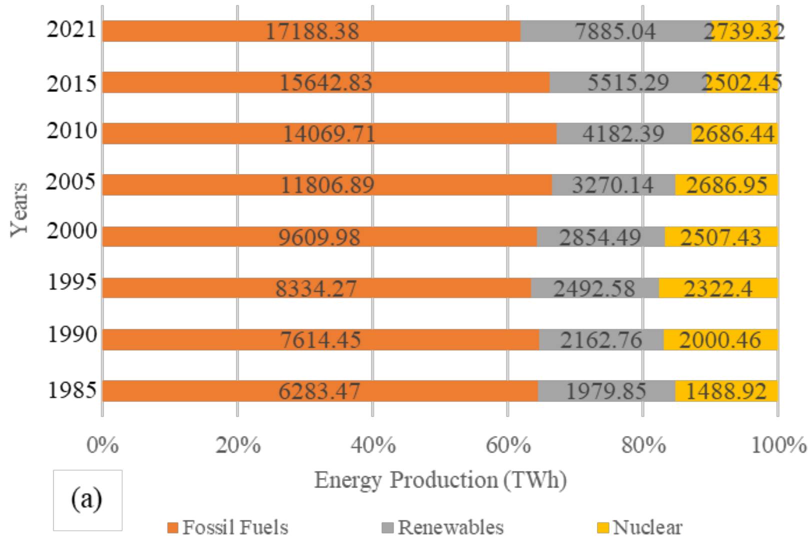


Figure 1. Variation of the population of the world (billions) and energy consumption (TWh) with years [1]

Wind turbines can be called onshore or offshore depending on the location where they are built. Wind turbines built in the sea or ocean can be called offshore, whereas those built on land can be called onshore. Onshore wind turbine foundations are generally designed as octagonal or circular gravity foundations (Figure 3). Onshore wind turbine foundations are subject to the horizontal load (H) caused by wind loads and the overturning moment (M) that is produced by the horizontal load. The foundation is also subject to the vertical loads (V) brought on by the self-weight of the foundation and wind turbine. Additionally, the foundation is affected by the twisting moment (M_z). The loads on the foundation of onshore wind turbines are depicted in Figure 4.



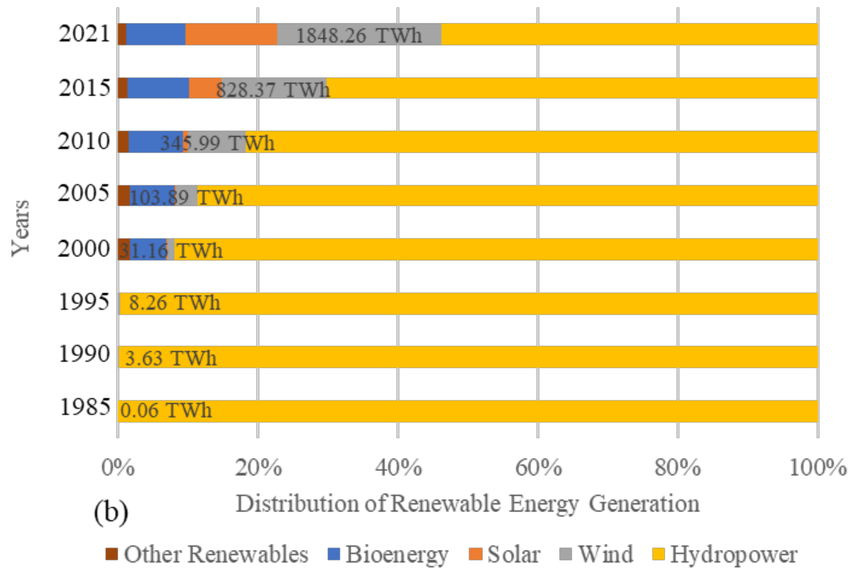


Figure 2. (a) Electricity generation in the world from fossil fuels, nuclear and renewables [1], (b) renewable energy generation in the world between 1990 and 2020 (numbers written on the figure represent the amount of energy generation by wind) [1]

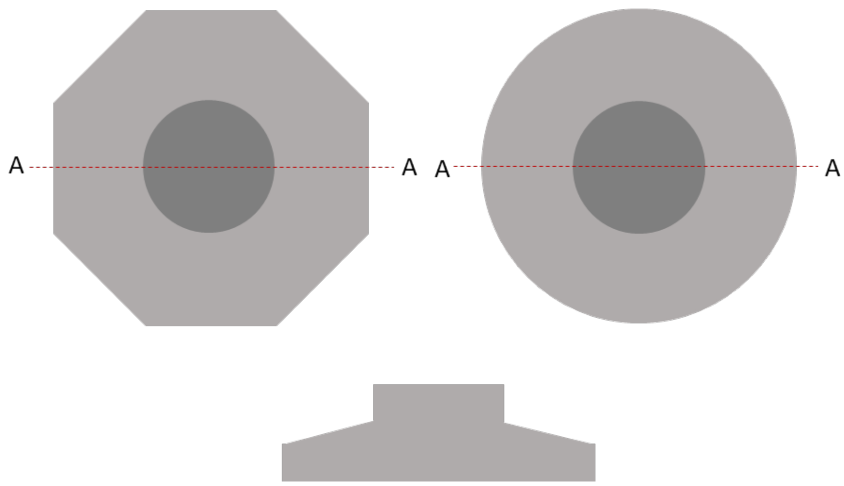


Figure 3. Shallow gravity based foundations for onshore wind turbines: (a) octagonal foundation, (b) circular foundation and (c) cross-section view of the foundations (A-A)

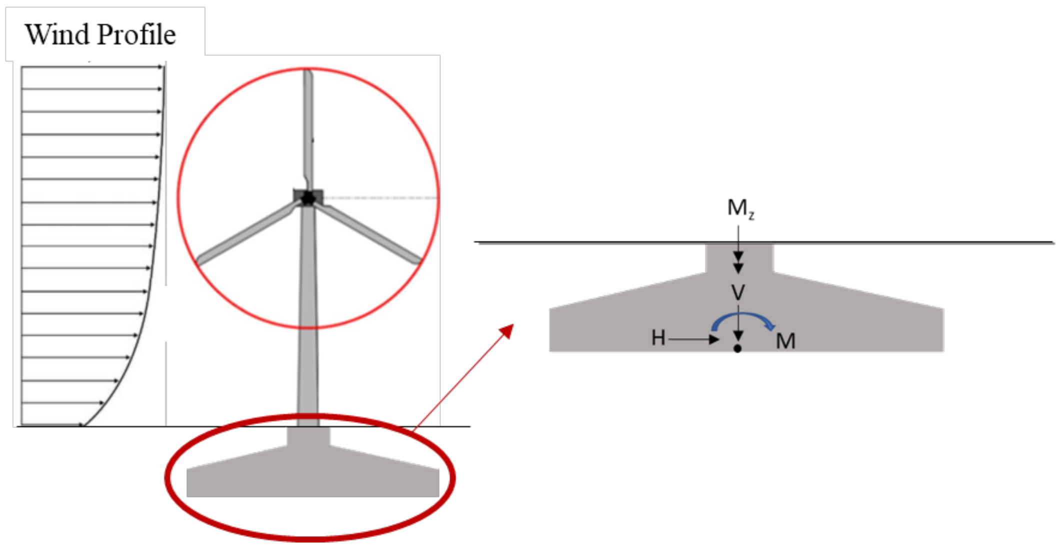


Figure 4. Loads acting on the onshore wind turbine foundations

The preliminary geotechnical design and sizing of these foundations constitute the first stage of detailed geotechnical design. When literature studies were examined, it was observed that a detailed approach for the preliminary geotechnical design of onshore wind turbine foundations was not presented. Therefore, in this study, a methodology was presented to demonstrate the geotechnical preliminary design stages of onshore wind turbine foundations. A flowchart outlining the steps to be followed for the geotechnical design of such foundations was proposed. All the steps required for the presizing of the foundation were explained in detail, and the proposed methodology was explained with the help of a worked example. The methodology followed in this paper does not provide a detailed design and optimisation of the foundation system but targets to provide a quick tool for initial design. The foundation system, which was obtained as a result of the preliminary design, was also numerically modelled using Plaxis 3D software. The numerical results and those obtained as a result of the preliminary design were compared.

1.1. A Brief of Literature Review

Field data from two wind turbine locations were collected and examined by Yilmaz [2]. Prior to construction, strain and pressure sensors were installed on the foundations of the wind turbines. In addition, strain gauges were used to monitor the developing strains on the towers. For the chosen site, the author focused on soil deformations, bearing pressure distributions, and moments at the base of the tower.

Mawer [3] discussed the mechanics of a wind turbine and its supporting systems in addition to the geotechnical design of gravity-based wind turbine foundations. The author selected three exemplary wind turbine sites and used as actual design examples to address each stage of the geotechnical design phase that was covered.

A study on the geotechnical design and construction of wind turbine gravity based foundations, including geotechnical investigation methodologies, geotechnical hazard evaluation, stability, stiffness, bearing capacity, and differential settlement, was presented by Ntambakwa et al. [4] in 2016.

Mohamed and Austrell [5] studied the geotechnical behaviour of three various foundations for onshore wind turbines. Traditional flat rafts are the first design, followed by deep flat rafts and conical rafts. It was done a comparative analysis of the conventional and new approaches. To increase the resistance of the soil-structure interface, a conical raft is used. It was demonstrated that a conical raft significantly reduced tilting when compared to a flat circular raft. Thus, it was concluded that using a conical raft could make the foundation smaller, perhaps resulting in cost savings.

A wind turbine with a 1.5 MW capacity on stiff clay was used as a case study in the study of Yaşar [6], and analyses of the foundation were performed using analytical methods, probabilistic methods using Monte Carlo simulation, and 3D FEM. In conclusion, three-dimensional (3D) finite element (FE) analysis was suggested as an alternative to analytical methods for precisely calculating the rotation and settlement of wind turbine foundations.

2. Materials and Methods

2.1. Methodology

A flowchart summarizing the steps for the preliminary geotechnical design of onshore wind turbine foundations is presented in Figure 5. Foundation dimensions and loads acting on the foundation are first estimated. Soil data is then obtained to start the preliminary geotechnical design of the onshore wind turbine foundations. Two limit states are considered during geotechnical design: (1) Ultimate Limit State (ULS) and (2) Serviceability Limit State (SLS). These geotechnical limit states are summarized in the IEC TC88-61400-6 [7].

Ultimate Limit State (ULS) includes several design situations: (1) overturning failure, (2) bearing capacity or sliding failure, and (3) structural failure.

Serviceability Limit State (SLS) comprises several design situations: (1) lateral and rotational stiffness of the foundation under dynamic and static conditions, (2) excessive long term tilt and ultimate settlement of the foundation and (3) long term degradation of geotechnical capacity which leads to failure of other limit states.

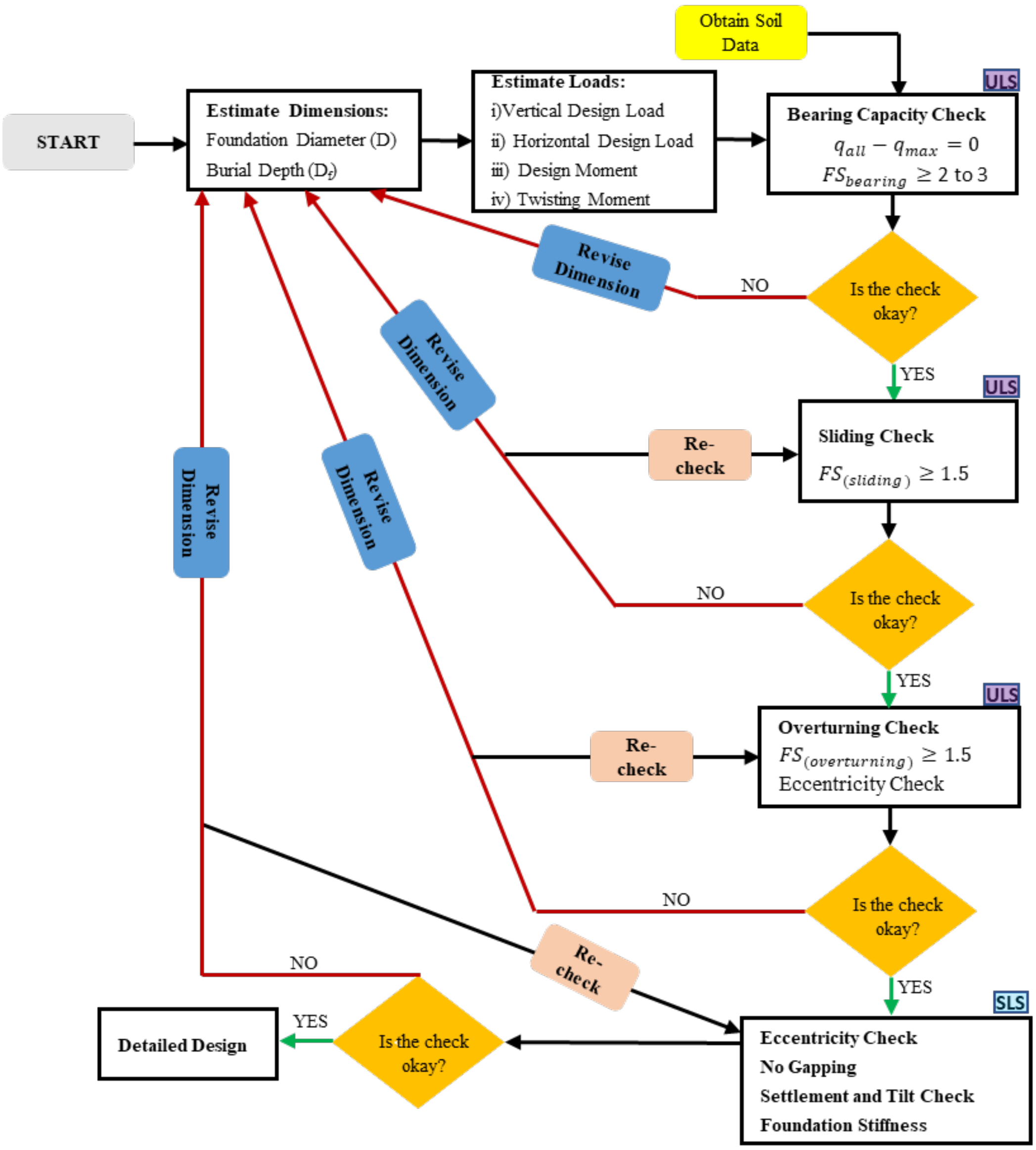


Figure 5. Flowchart showing preliminary geotechnical design of onshore wind turbine foundations

Under ULS conditions, safety against bearing capacity failure, sliding failure and overturning failure must be assessed. After ULS check is complete, SLS check must be done considering (1) foundations stiffnesses under dynamic and static conditions specified by the turbine manufacturer, (2) limit values for ultimate settlement and long term tilt suggested by the turbine manufacturers and (3) prevention of geotechnical failure due to long term degradation under cyclic loads.

Preliminary geotechnical design of onshore wind turbine foundations includes bearing capacity, sliding and overturning checks under ULS loads. It also includes eccentricity, no gapping, foundation stiffness, and settlement and rotation checks under SLS loads. Long term degradation of geotechnical strength and stiffness due to cyclic loads, and displacements and rotations caused by cyclic loads should be calculated by using advanced finite element models in the detailed design stage. This study focuses on the steps for preliminary geotechnical design of onshore wind turbine foundations. The following sections explain the details of each steps required for dimensioning the foundation considering ULS and SLS checks.

2.1.1. Bearing Capacity Check

At the foundation level, wind loads acting laterally on the wind turbines produce an overturning moment (M). The pressure distribution of the foundation on the soil is not uniform because of the overturning moment acting on the foundation. As soil cannot sustain any tensile strain at the soil-foundation interface, an increase in the overturning moment causes a decrease in the contact area between the foundations. In the case of circular foundations, the effective foundation area is elliptical, as shown in Figure 6. This foundation area is known as the effective area (A_{eff}), because it is subjected to compressive stress.

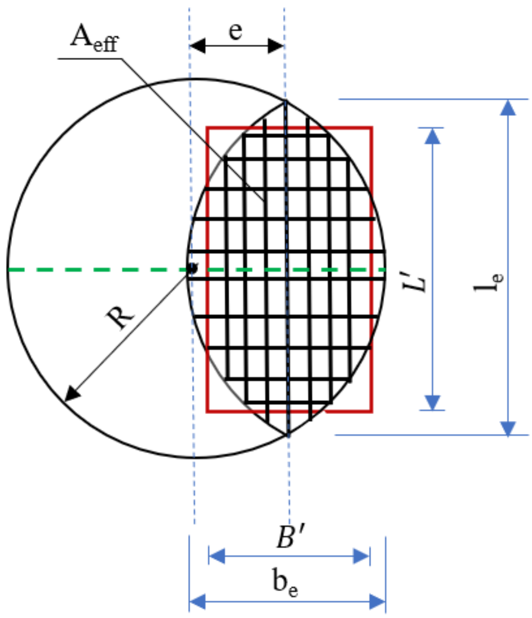


Figure 6. Calculation of the effective area (A_{eff}) for circular foundations

The effective area of the circular foundation with a diameter of D may be expressed as:

$$A_{eff} = 2\left[\frac{D^2}{4} arccos\left(\frac{2e}{D}\right) - e\sqrt{\frac{D^2}{4} - e^2}\right] \quad (1)$$

where D is the diameter of the foundation and e is eccentricity. The eccentricity (e) is equal to the ratio of design overturning moment ($M_{d,overturning}$) to the vertical design load (F_{zd}).

$$e = \frac{M_{d,overturning}}{F_{zd}} \quad (2)$$

There are several analytical solutions in the literature for calculating bearing capacity of shallow foundations [8-12]. In this study, the bearing capacity equations proposed by Meyerhoff [10] is used.

$$q_{ult} = cN_cS_cd_ci_c + qN_qS_qd_qi_q + 0.5\gamma B'N_\gamma S_\gamma d_\gamma i_\gamma \quad (3)$$

where N_c, N_q, N_γ are bearing factors, $S_c, S_{q,} S_\gamma$ are shape factors, $d_c, d_{q,} d_\gamma$ are depth factors $i_c, i_{q,} i_\gamma$ are load inclination factors and B' is the effective width.

If the groundwater table is close to the foundation, various adjustments to the ultimate bearing capacity formulae are nonetheless required. For the purpose of considering the effect of water table on the bearing capacity, three separate scenarios are taken into account: (1) the groundwater table located at $0 \leq D_1 \leq D_f$, (2) the groundwater table located at $0 \leq d \leq B$, and (3) the groundwater level placed at $d \geq B$.

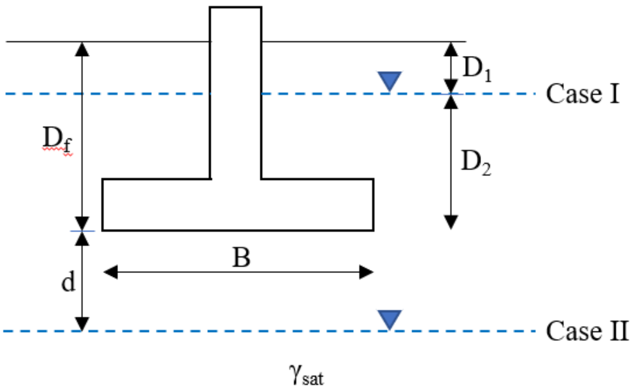


Figure 7. Effect of groundwater level on bearing capacity of foundations

For the first scenario ($0 \leq D_1 \leq D_f$), the effective surcharge term (q) in the bearing capacity equation takes the form $q = D_1\gamma + D_2(\gamma_{sat} - \gamma_w)$ where γ_{sat} is saturated unit weight of soil and γ_w is unit weight of water. In addition, the value of γ in the bearing capacity equation has to be replaced by $\gamma' = \gamma_{sat} - \gamma_w$.

For the second scenario ($0 \leq d \leq B$), the effective surcharge term (q) is equal to $\gamma \cdot D_f$. Also, the value of γ in the bearing capacity equation has to be replaced by $\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$ where γ is natural unit weight of the soil and γ' is effective unit weight of the soil.

For the third scenario ($d \geq B$), the water does not have any effect on the bearing capacity.

The allowable bearing capacity (q_{all}) can be calculated as follow:

$$q_{all} = \frac{q_{ult}}{FS_{bearing}} \tag{4}$$

The value of $FS_{bearing}$ ranges between 2 and 3 [13].

In order to calculate the diameter of the foundation that is required for the safety in terms of bearing capacity, the difference between maximum compressive pressure under the foundation (q_{max}) and q_{all} should be equal to zero.

$$q_{max} = \frac{F_{zd}}{(B')(L')} + \frac{6 \cdot M_{d,overturning}}{(B')(L'^2)} \tag{5}$$

$$q_{all} - q_{max} \geq 0 \tag{6}$$

where F_{zd} is vertical design load and $M_{d,overturning}$ is design overturning moment.

2.1.2. Sliding Check

Sliding check must also be done for foundations subjected to horizontal loads. The factor of safety against sliding under the foundation base can be calculated as:

$$FS_{(sliding)} = \frac{H_R}{H_d'} \tag{7}$$

where H_R is horizontal resisting forces and H_d' is the effective horizontal design force.

The effective horizontal design force acting at the interface of foundation-soil may be expressed as:

$$H_d' = \frac{2 \times M_z}{L'} + \sqrt{H_d^2 + \left(\frac{2 \times M_z}{L'}\right)^2} \tag{8}$$

where M_z is the twisting moment, L' is the effective length and H_d is the design horizontal load.

According to IEC TC88-61400-6 [7], a minimum factor of safety against sliding ($FS_{(sliding)}$) of 1.5 must be satisfied.

The total horizontal resisting forces acting at the interface between the foundation and the soil can be calculated as:

$$H_R = F_{zd} \cdot \tan(k_1 \cdot \phi) + D \cdot k_2 \cdot c \tag{9}$$

where F_{zd} is vertical design load, ϕ is the internal friction angle of the soil under the foundation, D is diameter of the foundation, c is the cohesion of the soil under the foundation, k_1 and k_2 range from $1/2$ to $2/3$. In case of eccentric loading, B' value should be used instead of D in the equation.

2.1.3. Overturning Check

The factor of safety against overturning ($FS_{overturning}$) can be calculated by dividing stabilizing moment ($M_{d,stabilizing}$) by design moment ($M_{d,overturning}$). The stabilizing moment can be expressed as:

$$FS_{overturning} = \frac{M_{d,stabilizing}}{M_{d,overturning}} \tag{10}$$

$$M_{d,stabilizing} = F_{zd} \cdot R \tag{11}$$

where R is radius of the foundation and F_{zd} is the vertical design load.

The value of $FS_{overturning}$ should be at least 1.5 ($FS_{overturning} \geq 1.5$) as suggested in the work of Morgan and Ntambakwa [14] in order to prevent overturning. According to IEC TC88-61400-6 [7], the value of $FS_{overturning}$ must have a minimum value of 1.1 when the foundation is placed on the soil material. By focusing on eccentricity, Szerzo [15] suggested a method for assessing safety against overturning. The value of e must satisfy the following requirements:

$$e = \begin{Bmatrix} \leq 0.25R & for\ serviceability\ limit\ state \\ \leq 0.58R & for\ ultimate\ limit\ state \end{Bmatrix} \tag{12}$$

The required foundation diameter (D) can be calculated by taking the value of $FS_{overturning}$ equal to 1.5 in Equation 10 and taking the value of e equal to $0.58R$ in Equation 12. The required foundation diameter can be found from the abovementioned equations iteratively by assuming an initial diameter and iterate the value until the difference between Equation 10 and Equation 12 is small enough.

2.1.4. Settlement and Rotation (Tilt)

The settlement of a foundation can be categorized into two major groups : (1) elastic (immediate settlement) and (2) consolidation settlement. The immediate or elastic settlement of a foundation occurs during or immediately following the construction of the structure. Consolidation settlement occurs over time. The sum of the elastic and consolidation settlements represents the overall settlement of the foundation. There are two phases of consolidation settlement: (1) primary and (2) secondary.

After the primary consolidation is complete, secondary consolidation settlement develops because of slippage and reorientation of soil particles under a prolonged load.

Maximum allowable foundation tilt should be specified by the turbine manufacturers. Unless this value is not specified by the manufacturers, a value of 3 mm/m for rotation of the tower base may be assumed according to the IEC TC88-61400-6 [7]. Settlement limits should also be determined by the superstructure design team. In the absence of specific criteria imposed by the design team, a value of 25 mm may be assumed for allowable total settlement according to the IEC TC88-61400-6 [7].

2.1.4.1. Elastic Settlement

Elastic settlement of flexible foundations based on the theory of elasticity may be expressed as (see Bowles [13])

$$\delta_{e(flexible)} = q_0 \cdot (\alpha D') \cdot \frac{1-\mu_s^2}{E_s} \cdot I_s \cdot I_f \tag{13}$$

where q_0 is the net applied pressure on the foundation, α is a factor that depends on the location below the foundation where settlement is being calculated, D' is D/2 for the center of foundation ($D' = D$ for the corner of foundation), μ_s is Poisson's ratio of the soil, E_s is the average modulus of elasticity of the soil under the foundation from z = 0 to z = 5D or z = rock level (whichever is smaller), I_s is the shape factor [16] and I_f is depth factor [17]. The elastic settlement of a rigid foundation can be calculated as:

$$S_{e(rigid)} \approx 0.93 \cdot S_{e(flexible)} \tag{14}$$

Elastic settlement of granular soils can also be evaluated by using strain influence factor suggested by Schmertmann et al. [18]. The settlement can be calculated as:

$$S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z \tag{15}$$

where I_z is strain influence factor, C_1 is a correction factor for the depth of foundation embedment, C_2 is a correction factor to account creep in soil, $\bar{q}$ is stress at the foundation level, q is the effective stress at the base of the foundation, E_s is the modulus of elasticity of soil and Δz is the thickness of soil layers underneath the foundation.

2.1.4.2. Primary Consolidation Settlement

Due to stress increase, elastic settlement of a saturated clay layer immediately occurs. The excess pore water pressure produced by loading dissipates over long period since the hydraulic conductivity of clays are small. Therefore, the volume change in the clay associated with the dissipation of the excess pore water pressure may continue long after the elastic settlement. This time-dependent settlement is called as consolidation.

The primary consolidation settlements of normally consolidated and overconsolidated clays can be calculated using the formulas recommended in geotechnical textbooks.

2.1.4.3. Secondary Consolidation Settlement

At the end of primary consolidation where complete dissipation of excess pore water pressure occurred, some settlement may be observed due to the plastic adjustment of soil fabrics. This settlement is named as secondary consolidation. The secondary consolidation settlements of soils can be calculated using the formulas recommended in geotechnical textbooks.

2.2. Foundation Stiffness

The foundation stiffnesses for various conditions are summarised in DNVGL-RP-C212 [19]. The considered conditions are (1) foundation on surface of soil stratum with finite thickness over bedrock, (2) foundation on surface of soil stratum with finite thickness over elastic half space, contingent on the upper layer being less stiff than the underlying elastic half space, (3) foundation buried in soil stratum with finite thickness over bedrock. The expressions for foundation stiffnesses for the abovementioned conditions can be found in DNVGL-RP-C212 [19].

The small-strain soil modulus shall be used to verify the dynamic foundation stiffness. The SLS load level must be used to calculate the foundation stiffness, which is a function of contact area. Any reduction from complete contact must be taken into account in the stiffness calculation. The verification of static foundation stiffness, if prescribed by the turbine manufacturer, is based on a soil modulus that accounts for the decrease in small-strain shear stiffness as a function of real soil strain at the SLS load level.

3. Results and Discussions

3.1. Worked Example

3.1.1. Problem Description

An onshore wind turbine was assumed to be placed on a sandy soil layer for this worked example, as shown in Figure 8. The groundwater level was assumed to be at the ground surface. The thickness of the soil underlying the onshore wind-turbine foundation was assumed to be 50 m. The considered soil properties, including the internal friction angle (ϕ), cohesion (c), elastic modulus (E_s), Poisson's ratio (μ), saturated unit weight (γ_{sat}), and moist unit weight of the soil (γ_{moist}), are summarized in Table 1.

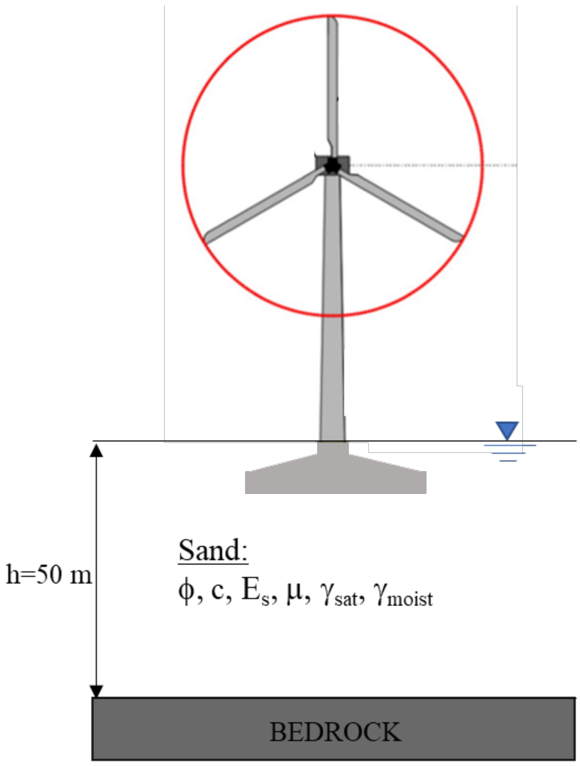


Figure 8. The schematic illustration of an onshore foundation placed on the sandy soil

Table 1. Properties of the soil considered in the worked example

Internal Friction Angle (ϕ)	Cohesion (c)	Young's Modulus (E_s)	Poisson's Ratio (μ)	Saturated Unit Weight (γ_{sat})	Moist Unit Weight (γ_{moist})
(°)	(kPa)	(kN/m^2)	(-)	(kN/m^3)	(kN/m^3)
33	0	40000	0.3	18	17

Two different sets of loads were used in this study: (1) ultimate limit state (ULS) loads and (2) serviceability limit state (SLS) loads for a 2 MW wind turbine located on the west coast of Sweden [20]. The loads acting on the foundation were the horizontal load (H), vertical load (V), bending moment (M), and twisting moment (M_z), as shown in Figure 9. The loads listed in Table 2 are characteristic loads for a wind turbine located on the west coast of Sweden [20]. Partial load factors for the ULS and SLS conditions were included. All loads acts 0.6 m above the ground surface (t_4=0.6 m).

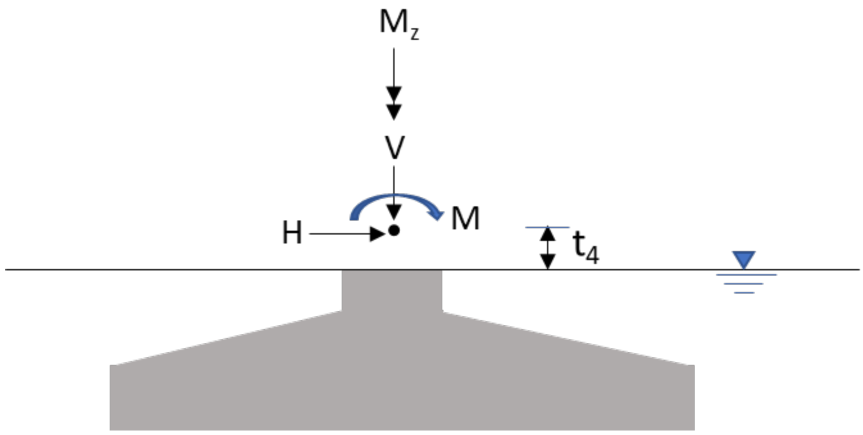


Figure 9. Tower loads acting on the foundation

Table 2. Tower Loads, characteristic values (partial load factors are included) [20]

Limit State	V (kN)	H (kN)	M (kNm)	M_z (kNm)
SLS	3510	482	35108	303
ULS	3510	797	63825	1642

The geometry of the foundation is shown in Figure 10 along with the relevant dimensions. The selected dimensions of the foundation are summarized in Table 3. The value of the foundation diameter (D) was calculated by following the geotechnical design steps described in Figure 5 (flowchart).

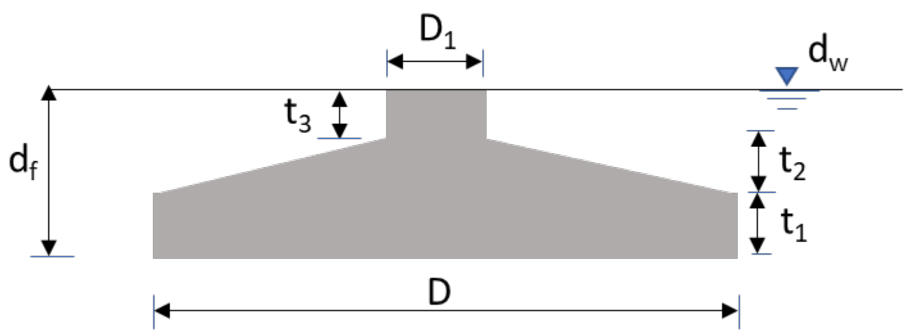


Figure 10. Dimensions of the foundation

Table 3. Dimensions of the foundation

D_1	d_f	t_1	t_2	t_3
(m)	(m)	(m)	(m)	(m)
6	3	1	1.25	0.75

3.1.2. Geotechnical Design

3.1.2.1. Bearing Capacity (ULS)

The calculations began with an estimation of the foundation diameter (D). For the initial calculations, the foundation diameter is assumed to be 15 m. The allowable bearing capacity (q_{all}) and maximum foundation pressure at the foundation (q_{max}) base were compared. The foundation diameter increased until the allowable foundation bearing capacity (q_{all}) exceeded the maximum foundation pressure (q_{max}). It was calculated that a foundation diameter of 19 m was sufficient in terms of the bearing capacity ($q_{all} - q_{max} > 0$) (Table 4). In the calculations, the safety factor against bearing capacity ($FS_{(bearing)}$) was considered to be 3.

Table 4. Bearing capacity calculation (ULS)

D	A_{eff}	B'	L'	q_{ult}	q_{all}	q_{max}
(m)	(m^2)	(m)	(m)	(kN/m^2)	(kN/m^2)	(kN/m^2)
15	5.27	1.06	4.99	902.36	342.88	17050.6
16	22.39	2.77	8.09	1087	362.33	2679.13
17	45.32	4.42	10.26	1236.99	412.33	1112.33
18	72.27	6.01	12.03	1377.53	459.18	631.02
19	102.33	7.54	13.56	1512.61	504.20	418.32

3.1.2.2. Sliding Check (ULS)

Sliding check was also carried out on the 19 m foundation diameter, which was found to be sufficient in the bearing capacity calculations (Table 5). As a result of the calculations, it was seen that the foundation diameter of 19 m was safe for sliding ($FS_{(sliding)}$>1.5).

Table 5. Calculations for sliding check (ULS)

D	H	M_Z	H'_d	H_R	$FS_{(sliding)}$
(m)	(kN)	(kNm)	(kN)	(kN)	-
19	797	1642	1075.06	5406.55	5.03>1.5

3.1.2.3. Overturning Check (ULS)

The safety of the foundation against overturning was also evaluated (Table 6). As a result of these calculations, the safety factor against overturning was calculated as 1.9. Because this value was greater than 1.5, it was observed that the foundation was safe against overturning.

The suggestion proposed by Szerzo [15] was used to evaluate the safety of the foundation against overturning. The eccentricity (e) calculated under ULS loads is smaller than $0.58 \times R$ ($e \leq 0.58 \times R$), where R is the radius of the foundation. Therefore, the foundation is considered safe against overturning based on the suggestion given by Szerzo [15] (see Table 7).

Table 6. Overturning check based on the stabilizing moment (ULS)

$M_{d,overturning}$	$M_{d,stabilizing}$	FS
(kNm)	(kNm)	-
66694.2	126426.51	1.9>1.5

Table 7. Overturning check based on the eccentricity (ULS)

e	$0.58 \times R$
(m)	(m)
5.01	5.51

3.1.2.4. Gap Condition Check (SLS)

No tension should exist in the soil beneath the foundation. The minimum pressure under the foundation (q_{min}) must be greater than zero ($q_{min} > 0$) under the SLS loads. The calculations show that q_{min} is equal to 3.33 kN/m^2, which confirms that there is no tension in the soil underlying the foundation (see Table 8).

Table 8. Gap Condition Check (SLS)

D	A_{eff}	B'	L'	q_{max}	q_{min}
(m)	(m^2)	(m)	(m)	(kN/m^2)	(kN/m^2)
19	230.15	13.06	17.63	112.31	3.33

3.1.2.5. Foundation Stiffness (SLS)

The values of dynamic foundation stiffness were calculated based on the small-strain soil modulus. The equations for a rigid circular footing embedded in stratum over bedrock were used to calculate the values listed in Table 9. These values should be greater than those specified by the turbine manufacturer.

Table 9. Dynamic foundation stiffness values

K_V	K_H	K_R	K_T
(MN/m)	(MN/m)	(MNm/rad)	(MNm/rad)
2363.7	1837.6	165251.2	242980.8

The values of static foundation stiffness are calculated by considering soil modulus under the operational strains caused by SLS loads. The reduction in soil modulus as a function of actual soil strain under SLS loads should be taken into account to calculate static foundation stiffnesses. The calculated values for the static foundation stiffness should be greater than those specified by the turbine manufacturer.

3.1.2.6. Settlement and Rotation (SLS)

The total settlement and tilt of the foundation were calculated using strain-influence method proposed by Schmertmann et al. [18]. The serviceability limit state (SLS) loads are considered for the calculations. The variations in the strain influence factor (I_z) and elastic modulus of the soil with depth are shown in Figures 11a-b. The total settlement and rotation of the foundation were calculated to be 0.0253 m and 1.33 mm/m, respectively. A value of 3 mm/m for rotation of the tower and a value of 25 mm for allowable total settlement may be assumed according to the IEC TC88-61400-6 [7]. The calculated rotation of the foundation is smaller than the rotation limit value suggested in IEC TC88-61400-6 [7]. However, the calculated total settlement value is slightly greater than the suggested limit value in IEC TC88-61400-6 [7].

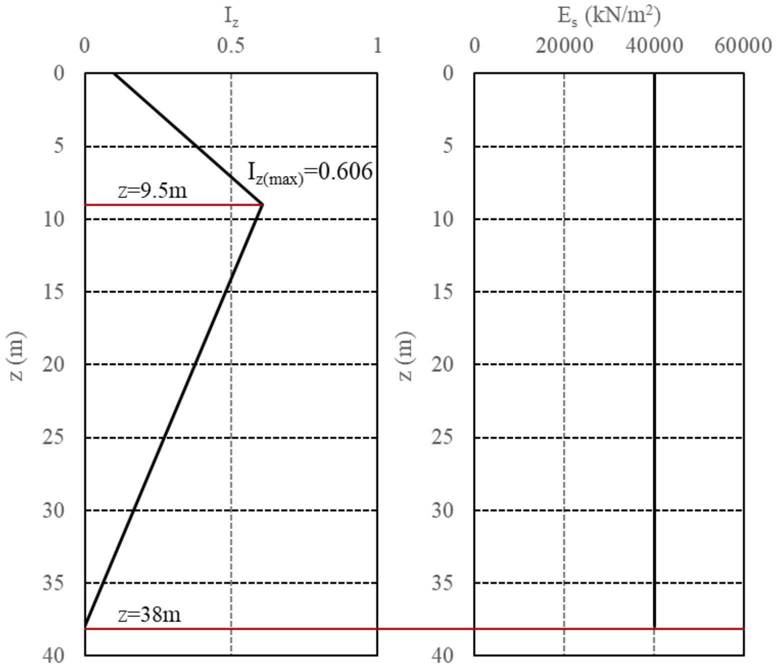


Figure 11. (a) Variation of strain influence factor (I_z) and (b) elastic modulus (E_s) of the soil with depth (z)

3.1.2.7. Detailed Analysis

Earlier steps are followed for the preliminary geotechnical design of the circular foundations for onshore wind turbines. The effects of cyclic loading on the degradation of soil stiffness and bearing capacity should be taken into account in the detailed analysis. The potential risk of sudden or progressive degradation of the soil stiffness and capacity should be evaluated as a part of foundation design. The maximum settlement and rotation of the foundation over the service life of the foundation should be controlled. In the preliminary design stage, elastic and consolidation settlements were considered. However, cyclic displacements and rotations of foundations occur due to repeated loads. The contribution of cyclic deformations to the total deformations of foundations should be taken into account.

In seismically active regions, the degrading effects of cyclic loading on the soil properties should be assessed for the site-specific conditions and taken into consideration in the design. Response spectrum analysis or time history analysis are needed to be performed to obtain seismic loads acting on the foundation, rotation and settlement of the foundation. The details of response spectrum analysis or time history analysis can be found in the relevant codes such as Turkish Earthquake Code [21] or Eurocode 8 [22].

3.2. Numerical Analysis

3.2.1. Ultimate Limit State (ULS)

The safety calculation was performed by using Plaxis 3D to obtain safety factor for the foundation under combined loading. For the safety calculation, ULS loads were considered. Figure 12 shows the variation of $\sum M_{sf}$ with total displacement |u| for three points (A, B and C) beneath the foundation. The point C is likely to be in the failure zone and the value of $\sum M_{sf}$ becomes stable for the point C. The factor of safety ($\sum M_{sf}$) of the foundation under combined loading was found to be 3.45 as seen in Figure 12.

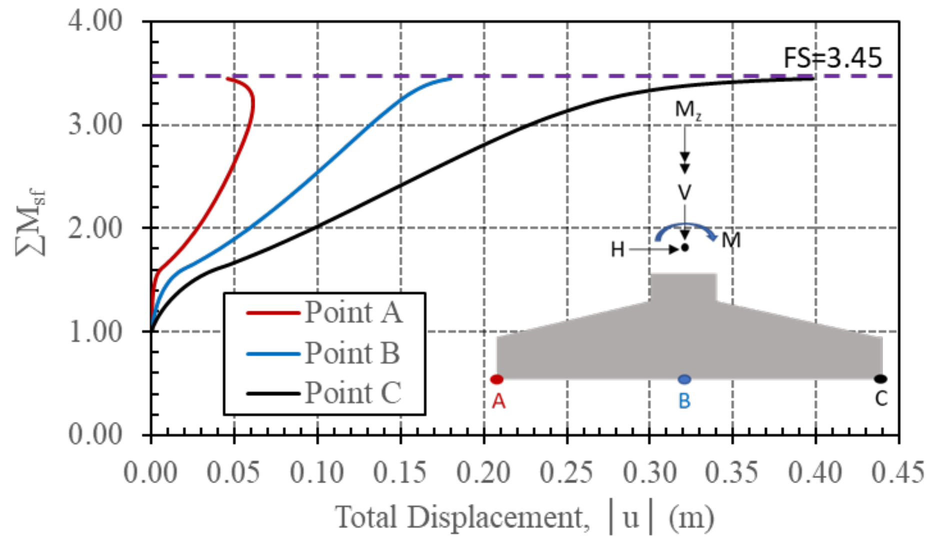


Figure 12. Factor of safety vs. Total displacement

The contours of effective normal stresses (σ'_N) developing beneath the foundation under ULS loads are shown in Figure 13. As observed in Figure 13, maximum base pressure developing underneath the foundation was calculated as 235.9 kN/m^2 which is smaller than the allowable bearing capacity of the foundation (504.20 kN/m^2).

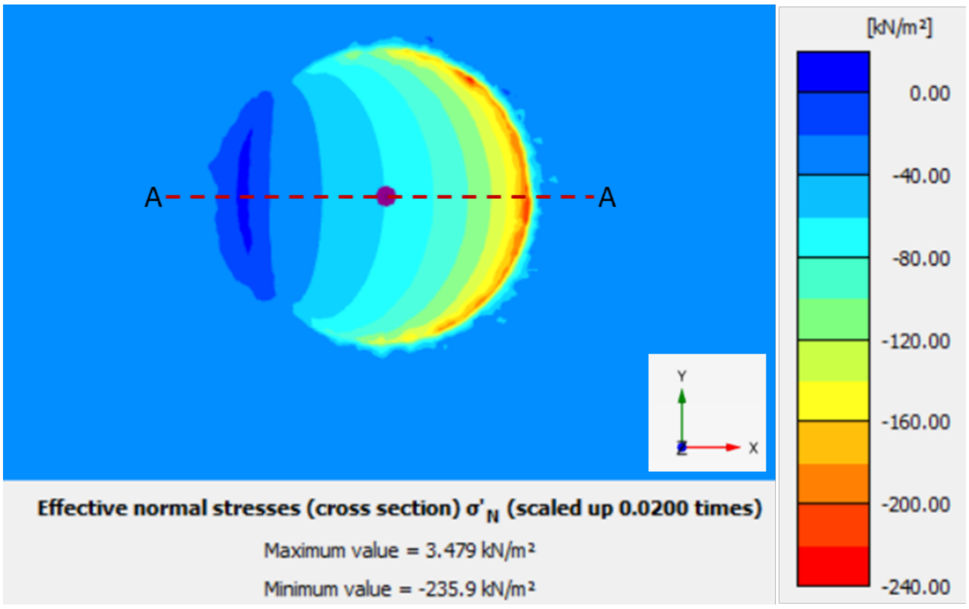
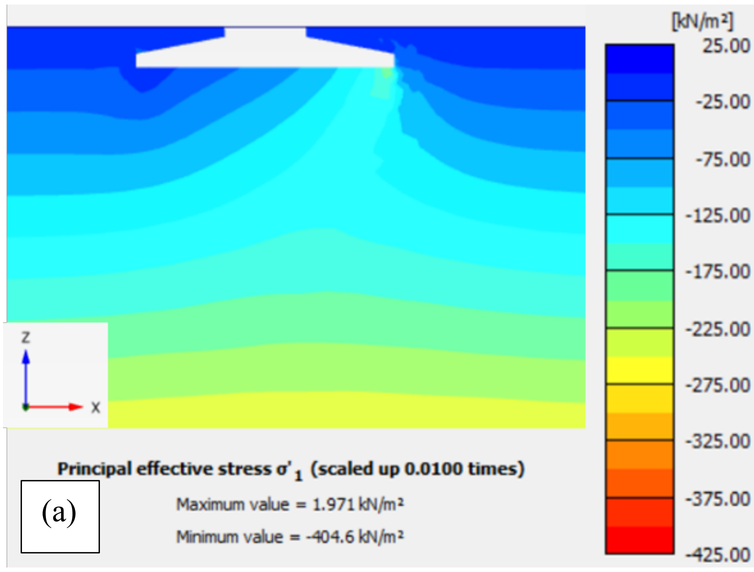


Figure 13. Effective normal stresses beneath the foundation under ULS loads

The contours of principal effective stress (σ_1') under the foundation is shown in Figure 14a. The variation of σ_1' for A-A cross-section is also shown in Figure 14b. As seen in the figures, triangular pressure distribution developed beneath the foundation under ULS loads. Gapping occurred between the foundation and underlying soil due to that tension stresses developed beneath the foundation under the ULS load condition. It was also expected that there would be a gap between the foundation and the underlying soil under ULS loads in the calculations performed using the analytical technique.



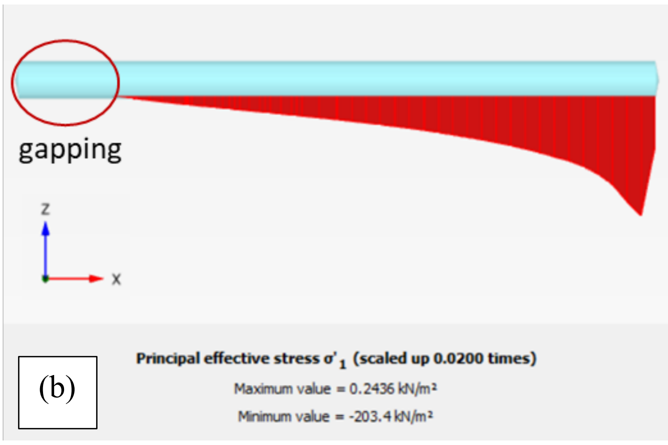
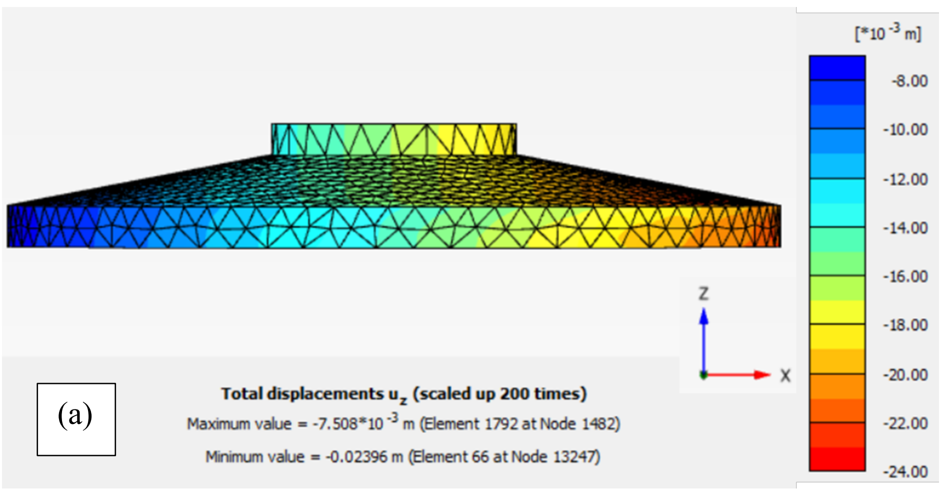


Figure 14. (a) Principal effective stress (σ_1') underneath the foundation (A-A cross-section) (b) the cross-section showing variation of principal effective stress (σ_1')

3.2.2. Serviceability Limit State (SLS)

The total settlement and rotation of the foundation under SLS loads were also calculated using Plaxis 3D. The Mohr-Coulomb constitutive model was used to model the soil behavior in the analysis. The soil parameters listed in Table 1 were used in the finite element (FE) model. The vertical deformation of the foundation is shown in Figure 15a-b. The total settlement and rotation of the foundation were calculated to be 0.02396 m and 1.26 mm/m. The calculated values of the total settlement and rotation of the foundation from the strain influence method and Plaxis 3D are summarized in Table 10.

The values calculated using the strain influence method and those calculated using Plaxis 3D were very close to each other. The total settlement and rotation of the foundation were estimated using the strain influence method, with a difference of 6%. The calculated total settlement values are very close to the limit value (0.025 m) suggested in IEC TC88-61400-6 [7]. Therefore, it is better to increase the foundation diameter to decrease the total settlement of the foundation.



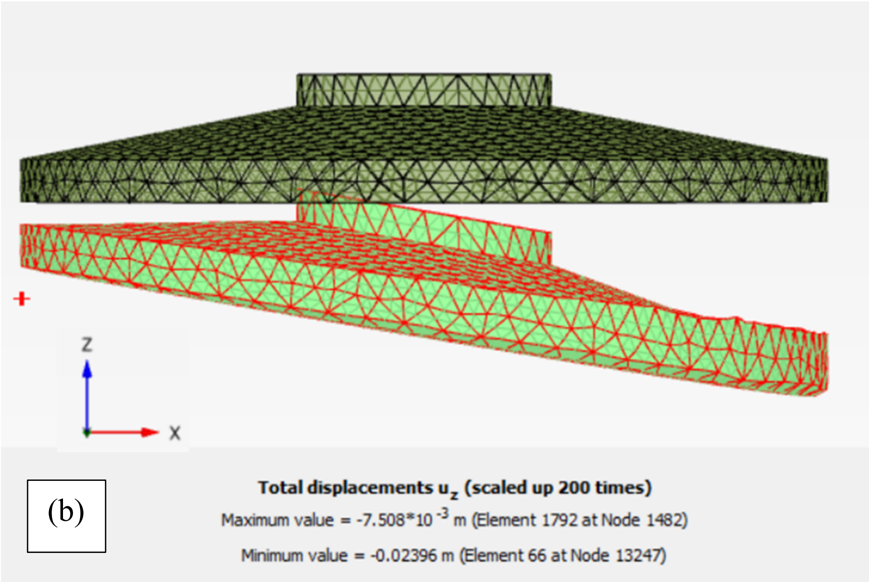


Figure 15. Vertical foundation displacement under SLS loads

Table 10. Comparison of the calculated values of total settlement and rotation

Strain Influence Method		**Finite Element Analysis (PLAXIS 3D)**	
Total Settlement (m)	Rotation (mm/m)	Total Settlement (m)	Rotation (mm/m)
0.0253	1.33	0.02396	1.26

The contours of effective normal stresses (σ'_N) developing beneath the foundation under SLS loads are shown in Figure 16. The contours of principal effective stress (σ'_1) under the foundation is shown in Figure 17a. The variation of σ'_1 for A-A cross-section is also shown in Figure 17b. As seen in the figures, trapezoidal pressure distribution developed beneath the foundation under SLS loads. No gapping occurred between the foundation and underlying soil since there is no tension stresses developed beneath the foundation under the SLS load condition. It was also expected that there would be no gap between the foundation and the underlying soil under SLS loads in the calculations performed using the analytical technique.

3.2.3. Discussion on the Numerical Results

The safety assessment carried out using the Plaxis 3D software confirmed the adequacy of the foundation dimensions determined by the analytical method. Likewise, the numerical analysis under SLS loading demonstrated that no gaps formed between the foundation and the underlying soil, in line with the analytical approach. Additionally, the settlement and rotation values of the foundation, obtained through both the Strain Influence Method and Plaxis 3D, were found to be consistent with each other. However, for stratified and complex soil conditions, numerical analysis should be performed to accurately calculate settlement and rotation of onshore wind turbine foundations.

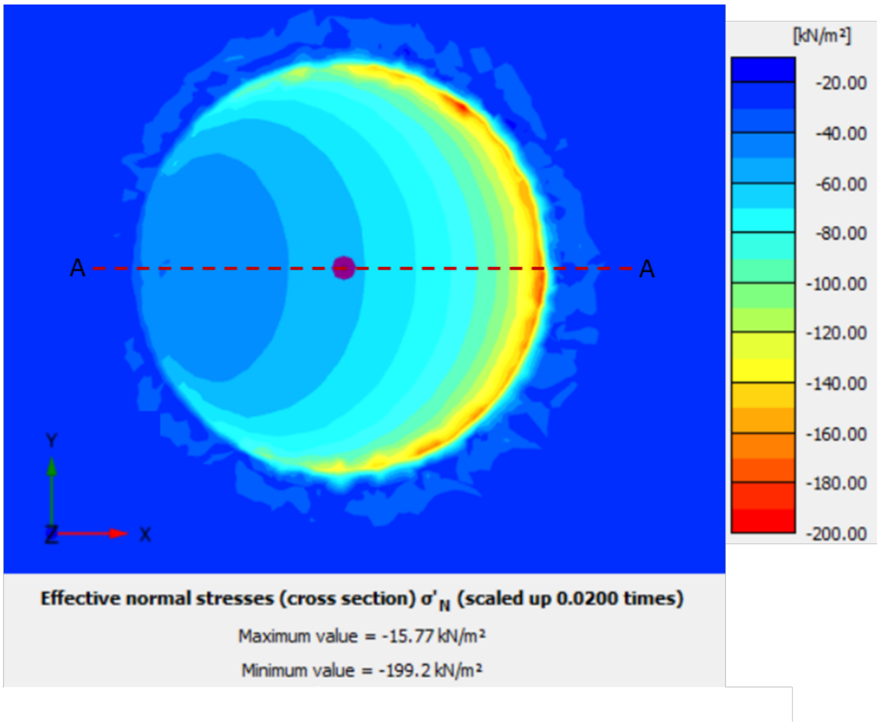


Figure 16. Effective normal stresses beneath the foundation under SLS loads

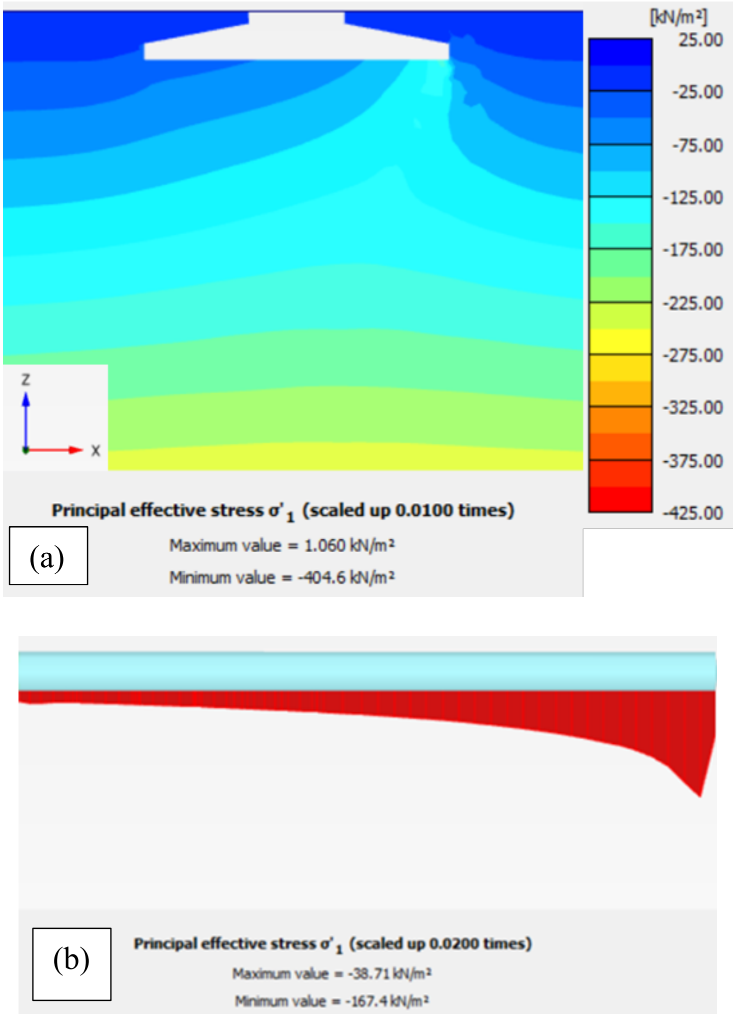


Figure 17. (a) Principal effective stress (σ_1') underneath the foundation (A-A cross-section) (b) the cross-section showing variation of principal effective stress (σ_1')

4. Conclusions

The geotechnical design of onshore wind turbine foundations requires iteration between Ultimate Limit State (ULS) and Serviceability Limit State (SLS) checks. Therefore, a flowchart was presented to demonstrate interdependency of ULS and SLS checks. All the steps explained in the flowchart can easily be implemented through a series of spreadsheets and a quick preliminary geotechnical design of onshore wind turbine foundations can be achieved. It should be noted that the methodology presented here should only be used for preliminary design. More advanced numerical analysis should be performed for the final design. A worked example was provided to explain each steps for the geotechnical preliminary design of onshore wind turbine foundations. Numerical analysis was also performed to confirm that the foundation dimensions obtained in the preliminary design were sufficient. It was observed that the results obtained from the numerical analysis and those obtained from the preliminary design were compatible.

During the preliminary design phase, elastic and consolidation settlements were taken into account. Nevertheless, cyclic displacements and rotations arising from cyclic loads occur in foundations. Thus, the influence of cyclic deformations on the overall foundation deformations must be considered in the detailed design. Moreover, in regions prone to seismic activity, it is crucial to assess the deterioration caused by cyclic loading on soil properties for the site-specific conditions and considered in the design. The methodology employed in this paper for the preliminary geoyechnical design of onshore wind turbine foundations does not encompass a detailed design and optimization of the foundation system. It does not incorporate considerations for cyclic load effects on the stiffness and strength of the soil. Instead, its primary objective is to suggest a quick tool for initial design purposes.

Ethics in Publishing

There are no ethical issues regarding the publication of this study.

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