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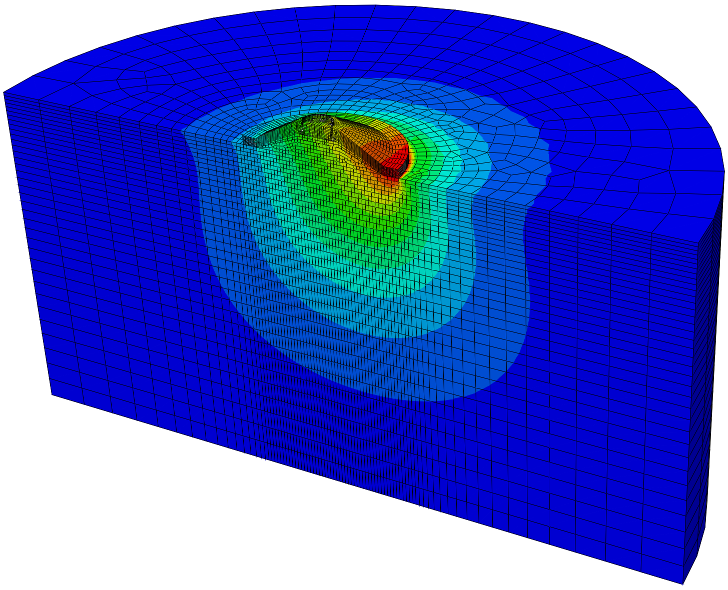

# WINDMILL FOUNDATIONS ON WEAK SOILS
## Evaluation of New Proposals for Cost-Efficient Solutions

WAEL MOHAMED

Structural Mechanics

*Licentiate Dissertation*

DEPARTMENT OF CONSTRUCTION SCIENCES

DIVISION OF STRUCTURAL MECHANICS

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LICENTIATE DISSERTATION

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Lund, November 2016

Wael Mohamed

## Abstract

Much scientific evidence indicates that the Earth's climate is changing and the world has become more concerned about the global climate. This has encouraged many countries to consider future clean power production with wind power taking an increasing part. In Sweden, the government declared the intention of generating 30 TWh/year from the wind by 2020 instead of 7.1 TWh/year achieved in 2012. Egypt, a developing country with a population of more than 90 million, has also specified that the Egyptian wind power share will reach 20% by 2020 instead of 1% in 2014. To achieve these goals, hundreds of new wind turbines will definitely have to be built, requiring prompt and extensive studies on how this can be achieved.

On weak soil, supporting structures constitute the larger share of the cost of modern wind turbines after the turbine itself. Therefore, the cost can be decreased considerably with an improvement in the design methods of the supporting structures. The objective of this work is to conduct a comprehensive analysis of the geotechnical behaviour and the cost of three new foundation solutions and to examine their performance in comparison to the conventional foundation solutions. To achieve this objective, settlement and tilting response of the new and the classical foundations are investigated by using finite element analysis together with a cost comparison between the new and the traditional foundations.

The new foundation solutions designed are a conical raft, a flat circular raft surrounded by a water tank, and a conical raft surrounded by a water tank to decrease the foundation cost and to improve the geotechnical behaviour of the foundation. A comparative study of the mentioned foundation solutions has been performed numerically using finite element simulations. The results of the current study show that the geotechnical behaviour of a conical raft is improved compared to the geotechnical behaviour of a flat circular raft if the same diameter is used. Also, the conical raft requires less concrete and rebar volume to pass requirements on settlement and tilting. Concerning the active foundation solution using water tanks, the idea is to use a movable load to balance the overturning moment in order to decrease the tilting. It is shown that there is an economic benefit of this active system compared to the use of piling. The results show that using the active foundation solution can reduce the foundation tilting compared to a piled raft with long friction piles on deep soft clay layer. Also, the initial cost of the water tank foundation is less than the initial cost of the piled raft.

# Contents

## II Appended publications

**Paper A**
*A comparative study of two onshore wind turbine foundations*
W. M. Mohamed and P.E. Austrell
Submitted for publication

**Paper B**
*Optimization of wind turbine foundations for poor soil*
W. M. Mohamed and P.E. Austrell
Based on a paper published in the Proceedings of COMPDYN 2015, Crete; Greece; 25 - 27 May 2015

**Paper C**
*New onshore wind turbine foundation system for poor soil*
W. M. Mohamed and P.E. Austrell
Based on a paper published in the Proceedings of EWEA 2015, Paris; France; 17 - 20 November 2015

**Paper D**
*Evaluation of a new foundation solution for onshore wind turbines on soft clay*
W. M. Mohamed and P.E. Austrell
Submitted for publication

# Part I

Introduction and overview of the work

# 1. Introduction

One of the biggest challenges facing the world is the change from non-renewable energy sources to renewable sources such as wind power. Today, almost 90% of Egypt’s energy originates from thermal power plants and less than 1% comes from wind power [1]. Moreover, in Sweden about 78% of the electricity comes from hydroelectric power and nuclear power and 4% of the total Swedish electricity production comes from wind power [2].

Global climate change forces many countries to invest in renewable resources. In general, most onshore wind turbines in many countries are built on soils with good properties. However, there are a lot of good wind spots in regions with weak soils. Therefore, probably in the near future wind turbines may have to be built on weak soils. The soil geotechnical conditions and the foundation method for resisting the overturning forces are the main factors when selecting wind turbine foundations. On regions with weak soils, deep foundations such as piled raft and tensionless pier, are used to support wind turbines. Deep foundations have several disadvantages when compared to shallow foundations, being mainly high cost and long construction time.

The most significant constraint on construction projects is the financial feasibility. To make sure that the construction is reasonably priced, a cost analysis of alternative designs and materials has to be performed in order to choose the best design. On deep soft soil, foundations make up a high percentage of the total cost. Therefore, the foundation costs can be reduced by using new cost-efficient solutions to resist the overturning moment.

## 1.1 Conventional wind turbine foundation types

There are several types of onshore wind turbine foundation solutions. The suitable foundation type is chosen based on some aspects such as soil geotechnical properties and the turbine loads. The foundation shape and dimensions are then designed to minimise the cost. There are two main foundation categories described by being shallow or deep. Shallow foundations are often found in regions with strong soil, and they transmit the loads to the nearest soil layer. Deep foundations, on the other hand consequently transmit the applied loads to deeper soil layers.

Three existing onshore foundation solutions will be discussed in this section. The first and the most popular one is a gravity foundation. This solution is considered as a traditional shallow foundation solution for onshore wind turbines on strong soils. In this solution mainly the foundation weight and bearing pressure on the foundation base are utilized to resist the vertical loads and the overturning moment. Concerning the horizontal loads, they are resisted by the friction between the foundation and the soil [33]. The second solution is Patrik & Henderson (P&H) tensionless pier. This solution, which is shown in Figure 1-a, is considered as a deep foundation solution. It mainly uses the horizontal resistance of the surrounding soil to resist the horizontal forces and the overturning moment. Here, the vertical load is resisted by bearing on the pier base in additional to friction between the soil and the pier [33]. The third solution is a piled foundation. This solution, which is shown in Figure 1-b, is also considered as a deep foundation solution. Friction piles are used in this solution if there is a very deep soft soil layer. However, end bearing piles are used if the height of the soft soil layer is smaller.

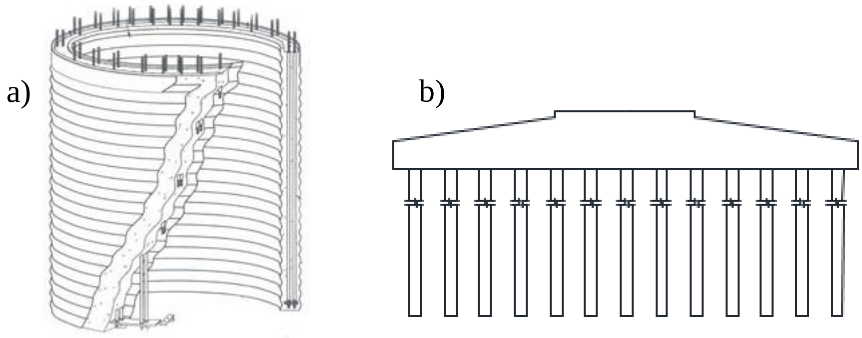


**Figure 1** Onshore wind turbine deep foundations a) A P&H Tensionless Pier [33], and b) a piled raft.

Wind turbine foundation costs vary significantly from country to country due to the variation of construction material costs. Usually, the foundation cost range depends on many factors such as the soil properties, the foundation type, and the foundation dimensions. According to Tinjum and Christensen (2010) [4] the octagonal raft foundation cost range is from \$100,000 to \$250,000 depending on the foundation dimensions for diameters ranging from 12 m to 18 m. A cost-benefit study of wind energy showed that onshore wind turbine foundations without piling on good soil make up about 3%~7% of the total costs [5, 37]. Deep foundation systems, on the other hand, will provide excellent performance for settlement and tilting. However, this comes at a high cost, both financially and environmentally, and takes longer time to construct. Manwell et al. [6] evaluated cases where piling was required and found that foundation costs accounted for 28% of the total costs. Engineers are continuously searching for the most cost-effective and trustworthy support solutions for wind turbines. In the decision process, the chosen foundation solution should provide a balance of cost, speed of construction, quality, and performance.

## 1.2 Shell foundations

Shell footings are used in some designs considered in this work. New onshore wind turbine foundation solutions are proposed using a conical raft with and without an active stabilisation system comprised of water tanks. A conical raft has not yet to the author's knowledge been used as a foundation supporting a wind turbine. However, it has been used as a footing for a TV tower in Germany, some telecommunication towers in China and Russia, factories in India, a stadium in U.S.A, chimneys in Poland and small houses in Japan [10]. The main concept of using shell footings was developed to increase the footing load capacity and decrease the total settlement. Many researchers have studied the geotechnical behaviour of shell footings. Some are discussed below.

Nicolls and Lzadi (1968) [7] conducted an experimental study on a conical and a hypar shell footing and compared them with circular and square flat footings. The main conclusion of the study was that the ultimate bearing capacity, as well as the settlement, were remarkably improved as compared to the flat footings. Jain et al. (1977) [8] studied the effect of several parameters on the behaviour of conical shell footings under vertical load using linear solid axisymmetrical FE analysis. The results showed that the soil pressure was reduced. Moreover, the bearing capacity was increased by using an edge beam. Kurian (1994) [9] studied the behaviour of different types of shell foundations using FE analysis and compared them with flat plates under partial contacts. The main results of the study illustrated that the shell footings were more stable than the flat footings. Also, using shell foundations is favourable for economic reasons.

Abdel-Rahman (1996) [10] investigated the geotechnical behaviour of shell footings experimentally and compared with flat footings. The main results were; 1) the ultimate bearing capacity of shell foundations are significantly higher (8%-15%) than the conventional flat counterparts with the same plane dimensions, and 2) the shell foundations have better settlement behaviour than the traditional flat ones. Huat (2006) [11] studied a shell footing of conical shape numerically and showed that; the shell footing had a better load capacity compared to the flat footing for a similar plane dimensions. Also, adding edge beams at the bottom of the conical footings increase the load capacity of the footing. Shaligram (2011) [12] illustrated that the ultimate bearing capacity increases with a decrease in the shell angle of intersection.

## 1.3 Aim, significance and limitations

A major contribution of this work is to present new cost-efficient wind turbine foundation solutions. The findings of this research give a base for using new foundation solutions to support onshore wind turbines on weak soils. Active foundation systems using a movable load and rafts with conical shape have never been used as foundations supporting onshore wind turbines. This work is intended to encourage engineers to consider new foundation solutions and achieve the most stable, safe, and economical option in locations with weak soil conditions. Concerning the limitations at this stage, this study will not deal with offshore foundation solution. Also, fatigue is not considered in this study.

The primary goal of this work is to find new foundation solutions being able to reduce tilting, settlement, and costs compared to deep foundations. To accomplish this, new cost-efficient foundation solutions are investigated using a raft with a conical shape, a flat circular raft and a conical raft both surrounded by an active system using water tanks to balance the overturning moment. The main aims of this study are: 1) to show the geotechnical behaviour of the new foundation solutions on weak soils, 2) to make a comparison between the new solutions and the most common foundation systems considering different soil conditions, and finally 3) to present the main risks of using an active foundation system.

## 1.4 Overview of thesis

The thesis is divided into two parts. The first being an overview and summary and the second part is the four appended papers. In Part I, a summary of the dissertation is presented. In addition to this introduction (this chapter), the thesis summary is arranged in other seven chapters. The second chapter reviews the main steps of the geotechnical design of the shallow foundations and how to check the stability of the foundations. The third chapter introduces studied soil cases, soil properties, geotechnical material models, and the loads. The fourth chapter presents the foundation description. The sixth chapter shows the FE models. The seventh chapter show the summary of the appended papers. The eighth chapter includes the summary of the results in this research.

Part II consists of four appended papers. Paper A is an investigation concerning the geotechnical behaviour of the conical shell footing. Also, a geotechnical comparison between the traditional flat raft and the conical shell footing is made. In Paper B, the geotechnical behaviour of the circular raft surrounded by water tanks is investigated using Gothenburg soil properties, and a cost comparison between this new foundation solution and a piled raft is made according to Swedish construction costs. In Paper C, geotechnical and economic comparisons between the circular raft surrounded by water tanks are done using layered soil near Port-Said city. In Paper D, the geotechnical behaviour of the conical raft using an active system is investigated using Gothenburg soil, and the effect of using EPS foam, under the conical raft, is studied. Also, a cost comparison between this new foundation solution and a piled raft is done according to Swedish construction costs.

# 2. Geotechnical design of shallow foundations

The design procedure for shallow foundations consists of four parts. The first involves determining the applied loads on the foundation. The second being a soil investigation to provide all needed data of the ground properties. This investigation consists of laboratory testing of soil samples and also in-situ testing. The third is a geotechnical design, and the final part involves a structural design [15].

The geotechnical design considers many aspects such as determination of the required foundation dimensions and the required foundation weight to remove the failure probability of the foundation considering soil bearing capacity, sliding, overturning, settlement, and tilting. Figure 2 shows a real overturning failure of a shallow wind turbine foundation. A calculation can be done to check soil bearing capacity, sliding, and overturning aspects using simple equations. However, checking settlement and tilting aspects need special geotechnical programs which incorporate nonlinear soil models. In this study, soil bearing capacity, sliding, and overturning aspects are done first by using hand calculation formulas to check the required diameter of the foundation. Then an FE program -Abaqus- is used to analyse the settlement and tilting of the foundations. The geotechnical design aspects of wind turbine foundations are discussed in the following sections.

**Figure 2** Stability considerations: overturning failure of a wind turbine foundation

## 2.1 Check for bearing capacity

Shallow foundations have to be designed to keep the maximum imposed load in relation to the allowable bearing capacity of the soil. There are two expressions related to the bearing capacity of the soil: ultimate bearing capacity $q_u$ and allowable bearing capacity $q_{all}$. The ultimate bearing capacity $q_u$ is the theoretical maximum pressure that can be supported without causing shear failure [41, 42, 44]. The allowable bearing capacity $q_{all}$ is equal to the ultimate bearing capacity divided by a factor of safety $f_s$ and the allowable bearing capacity is used in the geotechnical design. The factor of safety $f_s$ ranges from 2 to 3 [13]. In this study, the factor of safety is assumed to be 2.26 [34]. For fully drained conditions the ultimate bearing capacity of the soil can be calculated from Meyerhof general bearing capacity formula [14, 19, 35]. However, for undrained conditions, Skempton's equation is used to calculate the ultimate bearing capacity of the soil [53].

The soil under the foundation is exposed to an approximate triangular pressure distribution according to Figure 3b, due to the overturning moment $M = H \cdot h$ with $H$ being the resulting horizontal force from the wind and $h$ is the vertical distance between the line of action of the resulting horizontal force and the tower base as shown in Figure 3a.

It is well known that soil cannot carry any tension stresses at the interface between the foundation and the soil. Therefore, the effective contact area between the foundation and the soil is expected to be decreased as the overturning moment from the wind load increases. The effective foundation area is the centrally loaded area as shown in Figure 4. The effective foundation area depends on the value of the load eccentricity and the foundation radius. The load eccentricity $e$, which is shown in Figure 3c, can be calculated from

$$\mathrm{e} = M_t / V \tag{1}$$

where $M_t$ is the total bending moment at the foundation base level and $V$ is the sum of all the vertical loads. The total bending moment and total vertical loads can be calculated from the following equations [15]

$$M_t = M + H\,y \quad ; \quad V = N + W_f \tag{2}$$

where $H$ is the horizontal force, $y$ is the vertical distance between the foundation level and the horizontal force at the tower base, $N$ is the vertical load which is given by the tower, nacelle, and blades and $W_f$ is the self-weight of the foundation.

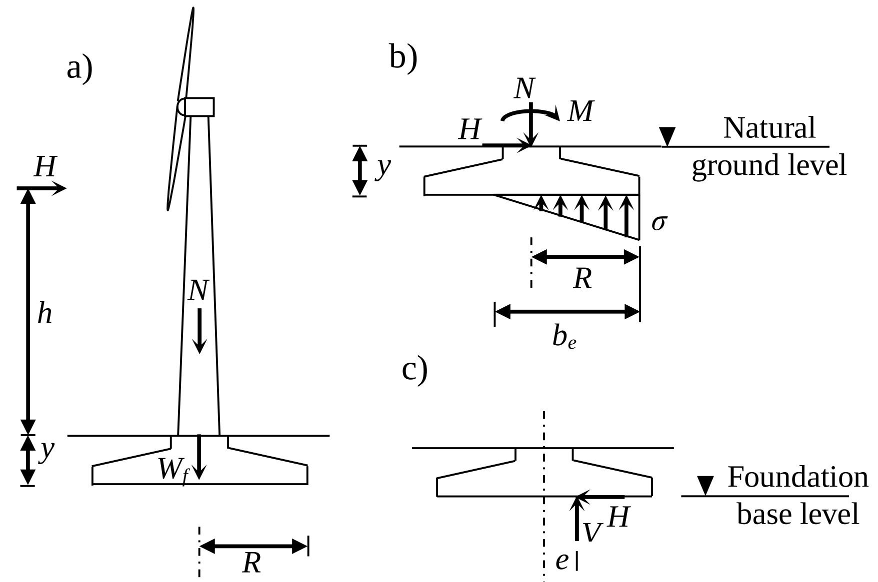


**Figure 3** a) windmill loads, b) loads on the foundation and stress distribution under the foundation, and c) reaction forces from the soil.

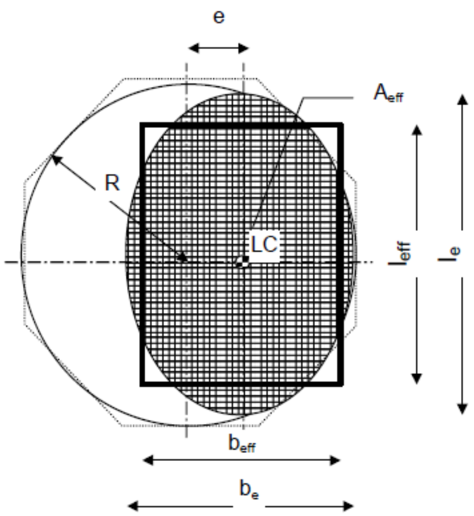

**Figure 4** the effective area for a circular foundation [15].

In this study, the effective area for a circular foundation can be expressed as a rectangular area ($b_{eff}$ . $l_{eff}$) that origins from an elliptical area as shown in Figure 4 [15]. The effective area can be calculated by using the following formula [15]:

$$A_{eff} = 2[R^2 \arccos\left(\frac{e}{R}\right) - e\sqrt{R^2 - e^2}] \tag{3}$$

The shape of the effective area for circular foundations is elliptic. The major axis values can be calculated from [15]

$$b_e = 2(R - e) \tag{4}$$

$$l_e = 2R\sqrt{1 - \left(1 - \frac{b_e}{2R}\right)^2} \tag{5}$$

If the effective area shape is not rectangular, it should be transformed into an equivalent rectangular shape to use it in the bearing capacity equations. The equivalent rectangular dimensions can be calculated from [15]

$$l_{eff} = \sqrt{A_{eff}\frac{l_e}{b_e}} \tag{6}$$

$$b_{eff} = \frac{L_{eff}}{l_e} b_e \tag{7}$$

For saturated soft soils, the undrained analysis is necessary to evaluate the short-term bearing capacity which can be more critical than the long-term bearing capacity. Skempton's equation is used to calculate the ultimate bearing capacity for undrained clay soils [53].

$$q_u = c_u N_c s_c d_c + q \tag{8}$$

where $c_u$ is the undrained cohesion, $N_c$ is the bearing capacity factor in the case of zero friction angle, $s_c$ is a shape factor, $d_c$ is a depth factor, and $q$ is the surrounding stress at the foundation base level due to the pressure from the earth. For drained conditions Meyerhof general bearing capacity formula is used as mentioned.

After calculating the ultimate bearing capacity in the drained and undrained conditions, the allowable bearing capacity should be calculated. The allowable bearing capacity is equal to the minimum value of the ultimate bearing capacity in the drained and undrained conditions divided by a factor of safety $f_s$. As mentioned the factor of safety is assumed to be 2.26 [34].

$$q_{all} = q_u / f_s \tag{9}$$

The required diameters of the foundations are calculated by putting the maximum compressive stress under the foundation Eq. (10) equal to the allowable bearing capacity [54]. By using iteration, the required diameter can then be calculated.

$$\sigma = \frac{V}{b_{eff} l_{eff}} + \frac{6M_t}{b_{eff} l_{eff}^2} \tag{10}$$

$$\sigma = q_{all} \tag{11}$$

## 2.2 Check for overturning and sliding

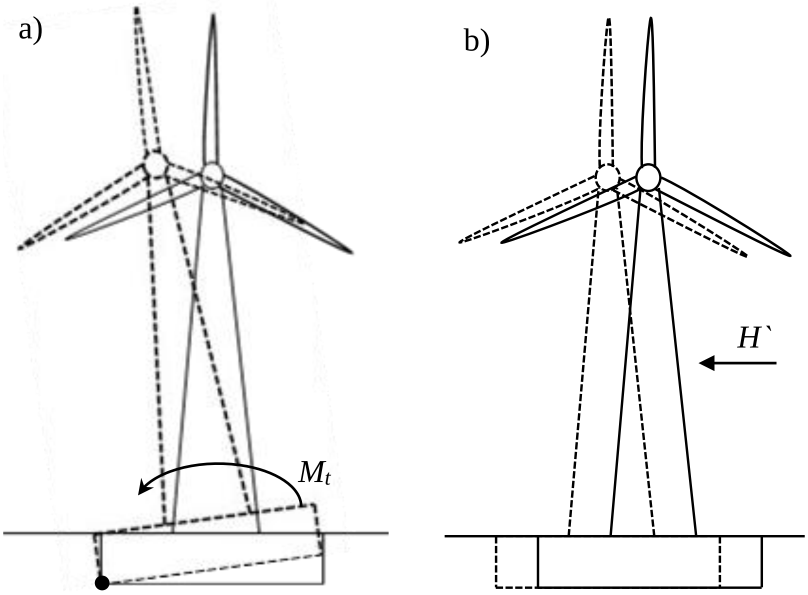


**Figure 5** Failure of wind turbine foundation a) by overturning, and b) by sliding.

In order to prevent overturning, the loads given in Figure 6 must be balanced by loads from the ground acting on the bottom surface of the foundation.

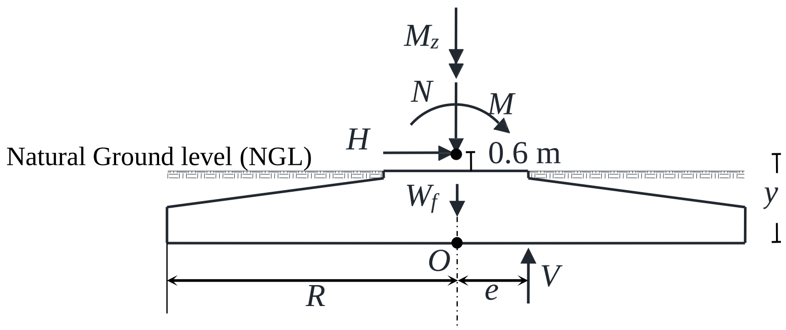


**Figure 6** Resistance against overturning.

The resultant force from the ground $V$ is acting with an eccentricity $e$ due to the overturning moment. The foundation weight $W_f$ is now included and force equilibrium yields

$$V = N + W_f \tag{12}$$

Moment equilibrium around point O yields

$$H \cdot y + M = V \cdot e \tag{13}$$

The total overturning moment is defined as

$$M_t = H \cdot y + M \tag{14}$$

The limiting case occurs theoretically when the eccentricity $e = R$ the foundation radius giving the stability moment $M_s$ as

$$M_s = V \cdot R \tag{15}$$

When there is no more resistance to overturning. One way to prevent overturning failure, as shown in Figure 5a, is according to Das (2007) [52] to define a factor of safty $2<f_s<3$ with respect to the stability moment in the ultimate limit state giving

$$M_t = M_s/f_s \tag{16}$$

Another suggestion is given by Szerzo (2012) [16] foucusing on the eccentricity instead. The eccentricity $e$ should fulfil

$$e = M_t/V \quad \begin{cases} \le 0.25R & \text{serviceability limit state} \\ \le 0.58R & \text{ultimate limit state} \end{cases} \tag{17}$$

Wind turbines supporting structure systems are subjected to high horizontal loads. These horizontal loads can cause a sliding failure. The sliding failure is shown in Figure 5b. The following equations must be fulfilled to check the sliding resistance according to DNV Risø [15]

$$(A_{eff}\, c + V \tan Ø) \,/\, H' > 1 \tag{18}$$

$$H' / V < 0.4 \tag{19}$$

Where $A_{eff}$ is the effective foundation area, $c$ is the cohesion of the soil, $Ø$ is the friction angle of the soil, $V$ is the sum of the vertical loads and $H'$ is the equivalent horizontal force defied below. When a twisting moment $M_z$ is applied to the foundation in addition to the horizontal force $H$, the interaction between the twisting moment and the horizontal force can be considered by replacing $H$ and $M_z$ with an equivalent horizontal force $H'$ [17]. The equivalent horizontal force can be calculated from

$$H' = 2M_z/l_{eff} + \sqrt{H^2 + \left(2M_z/l_{eff}\right)^2} \tag{20}$$

where $M_z$ is the twisting moment, $H$ is the horizontal force and $l_{eff}$ is the effective foundation length.

## 2.3 Check for settlement and tilting

Provided that shear failure, overturning, and sliding are prevented, settlement and tilting are considered serious problems for shallow foundations that rest on weak soils. As mentioned the FE program-Abaqus-is used to calculate the maximum settlement value and the tilting of the foundations. The maximum settlement for any shallow foundation system should not exceed the allowable settlement. This allowable value depends on the foundation type and the soil type, for example, the allowable settlement is 12 cm for a raft foundation on clay according to Euro

Code [18]. The allowable settlement for the same foundation is 10 cm on the sand and 15 cm on clay according to the Egyptian Code [19]. Tilting is equal to the differential settlement divided by the foundation effective width $b_{eff}$ given in Eq.(7). The maximum tilting should not exceed 1mm/m [16, 20].

The check for bearing capacity, sliding resistance and overturning resistance are used in this study to calculate the required raft diameter. However, settlement and tilting are used to compare between the foundation solutions, check the properties of all the foundation options and find the appropriate choice.

# 3. General information about soil composition and properties

A brief description of the main soil expressions that are used in this work is given in this section. A soil mass is a system consisting of solid particles and voids as shown in Figure 7. The voids may contain water or air or both. The soil can be classified into three types on the basis of moisture content: saturated, dry, and partially saturated. In the saturated case the voids are full of water, in the dry case the voids are full of air, and in the partially saturated case the voids are containing both water and air.

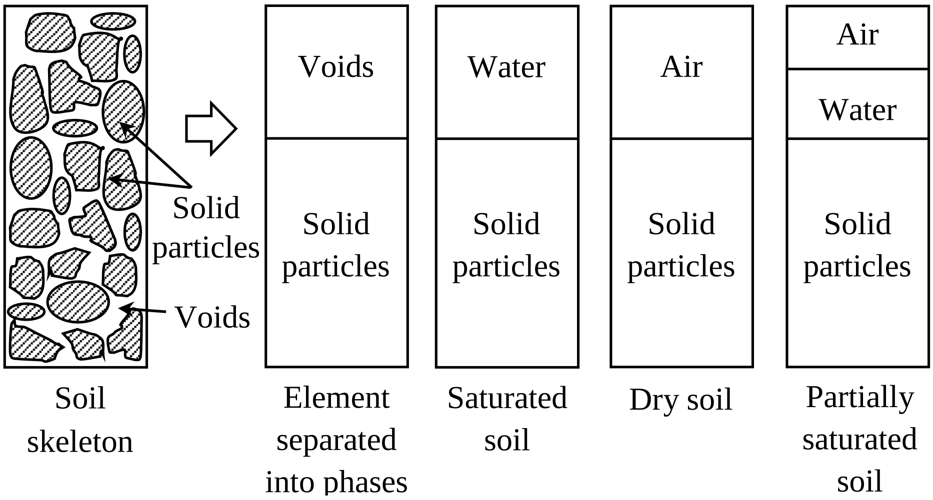


**Figure 7** Soil classification on the basis of moisture content.

Some definitions are presented here to clarify the meaning of some soil properties used in the following sections. Void ratio, water content, friction angle, cohesion, and dilatancy angle are used in the analysis of the foundations placed on the mentioned soil profiles. The void ratio $e$ is defined as the ratio of the volume of voids to the volume of solid particles in a given volume of soil [55]. Water content $w$, also called moisture content, is the ratio of the weight of water to the weight of solids in a given mass of soil [55]. Soil friction angle $\phi$ and cohesion $c$ are considered as shear strength parameters of soils. The shear strength of a soil is defined as the maximum shear resistance that the soil is capable of developing. The shear strength of the soil consists of the friction between the soil particles and the bonding or attraction at particle contacts which is called cohesion. Dilatancy in the soil during shear occurs because the grains in a compacted state are interlocking (expand in volume) as shown in Figure 8. The angle of dilation $\psi$ controls an amount of plastic volumetric strain developed during plastic shearing.

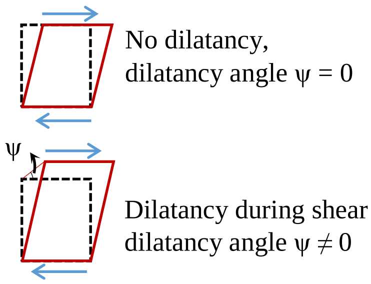


**Figure 8** Dilatancy during shear.

Soil has pores that provide a passage for water. The amount of water and the water movement in the soil have a significant effect on the behaviour of the soil. When the soil layer is subjected to an external compressive pressure, a settlement may take place through rearrangement of the soil grains due to a change of the volume of the voids. In the case of saturated soil, the settlement can take place if water is pushed out of the voids. The permeability of the soil and the location of free draining boundary surfaces control the required time for the settlement to take place. The permeability of the soil can be defined as the capacity of the soil to permit water to pass through its void spaces [55]. In sand, settlement occurs immediately due to high permeability. However, In clay, settlement occurs after a long time due to low permeability.

An idealised system shown in Figure 9 can be used to describe the process of consolidation in a simple way. The spring in the idealised system represents the soil and the water which fills the container represents the water in the soil.When the system is subjected to an external compressive pressure $p$ and the drainage is prohibited, the total pressure is initially taken by the water as shown in Figure 9a. In this stage, the pore water pressure $u$ is equal the total pressure $p$ and the spring is not compressed (no stress on the solid particles). If a valve is opened, the drainage of water will occur, and a part of the pressure is transferred to the spring, and it compresses as shown in Figure 9b. In the second stage, the solid particles will take a part of the total pressure. After some time, the drainage of water will stop, and the spring alone will resist the applied pressure as shown in Figure 9c. In the final stage the effective pressure $p'$, which means the stress carried by the solid particles of the soil, is equal to the total pressure $p$.

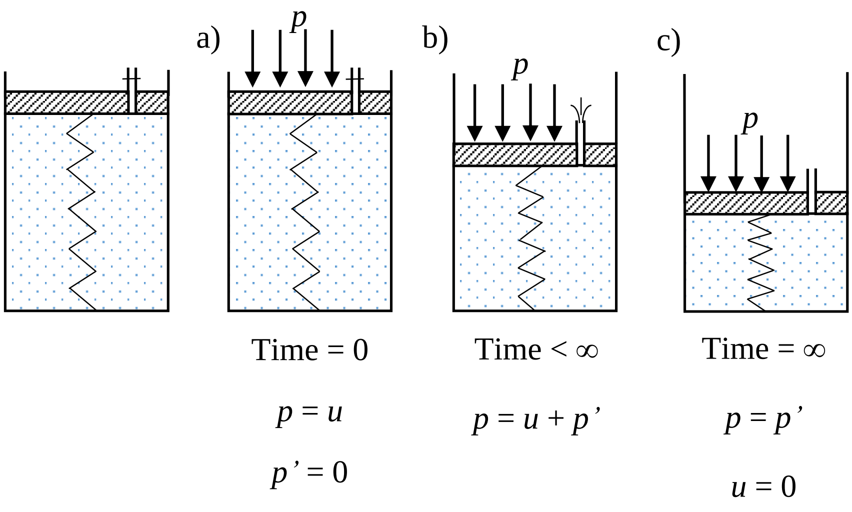


**Figure 9** An idealised system to describe the process of consolidation.

Consolidation is the process involving a settlement occurring at the same time with a flow of water out of the soil mass and with a slow transfer of the applied pressure from the pore water to the solid particles [56]. However, swelling can also occur, it is a process opposite to consolidation, which involves an increase in the water content due to an increase in the volume of the voids [56]. The drainage conditions, the thickness of the clay layer, and the excess load at the top of the clay are the main factors which decide the time taken for full consolidation [56]. Clay is called normally consolidated if the present effective pressure $p_0'$ is the maximum pressure to which the layer has ever been subjected to at any time in its history [56]. However, clay is called overconsolidated if the soil was subjected, at one time in its history, to a larger effective pressure, $p_c'$, than the present effective pressure $p_0'$ [56]. The larger effective pressure, $p_c'$ in the case of overconsolidation clay is called preconsolidation pressure. The overconsolidation ratio (OCR) is the ratio between the preconsolidation pressure, $p_c'$, and the present effective pressure $p_0'$ [56].

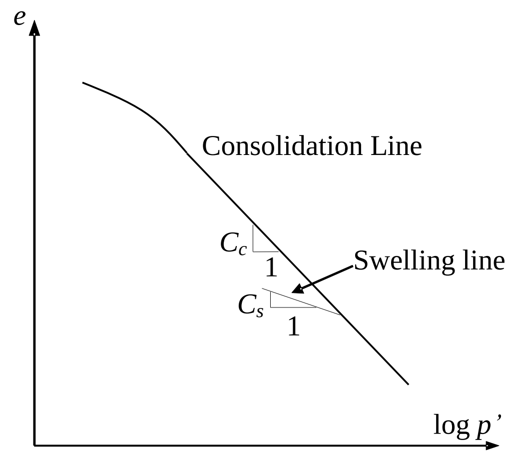


**Figure 10** Void ratio versus effective stress plotted in a logarithmic scale.

Compression index $C_c$ represents a deformation characteristic of soft soils and also it describes variation of the void ratio $e$ as a function of the change in effective pressure $p$' plotted in a logarithmic scale (the slope of the consolidation line in the linear part). However, swelling index $C_s$ is the slope of the swelling line as shown in Figure 10. The swelling line is entered upon unloading of the soil. Reloading also follows the swelling line until the pressure exceeds the preconsolidation pressure and then it begins to move down the consolidation line again. In the modified Cam-clay model, two of the required material parameters are related to the compression index and the swelling index. The first parameter is the modified compression index $\lambda$ which is related to the compression index $C_c$ through

$$\lambda = C_c/2.3 \tag{21}$$

and the second parameter is $\kappa$ which is related to the swelling index through

$$\kappa = C_s/2.3 \tag{22}$$

These parameters are material input data in the analysis of the Gothenburg clay using a modified Cam-clay model.

## 3.1 Soil properties, soil modelling and loads

Three soil profiles are used in this thesis; a typical stiff clay soil profile and two real soil profiles. The real soils are observed soil profiles found near Gothenburg city in Sweden and near Port-Said city in Egypt. These cities are good wind spots but the soil profiles there are considered as very weak soils. The main reasons for choosing Gothenburg and Port-Said soils are: 1) to show the geotechnical behaviour of the new foundations on a deep soft clay (Gothenburg soil) and to make a cost comparison according to Swedish construction costs, 2) to show the effect of having a real layered soil (a sand layer followed by a deep soft clay) in relation to the new foundations behaviour and to make another cost comparison according to Egyptian construction costs to check the economical difference in case of changing prices, and 3) finding out the best foundation solution for each case.

## 3.2 Port-Said soil properties

This section will focus on available Port-Said soil properties and the soil parameters that are required for analysing a circular raft with an active system and to compare with a piled raft. According to Golder Associates (1979) [21], Port-Said soil consists of five layers. The first layer is a thin layer of very soft surface clay with an average thickness of 0.2 m in the northern part to 2 m in the south. Below the surface clay, there is fine sand with an average thickness of about 6 m. The sand grades downward through a transition zone into the clay. The clay extends to an average depth of about 50 m below the ground surface; this clay layer rests in turn on a very hard clay. Groundwater in Port-Said lies 2 m below the natural ground level (NGL). A typical soil profile of Port-Said is shown in Figure 11.

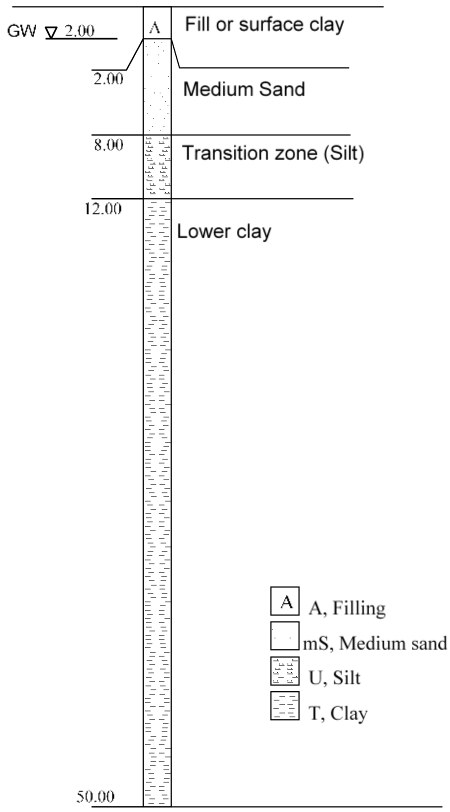


**Figure 11** Soil profile of Port-Said in Egypt.

Many geotechnical reports have described the properties of Port-Said soil. However, none of these reports show a specific value of the modulus of compressibility $E_s$ or the modulus of elasticity (Young's modulus) $E$ of the clay layer. FE programs require these parameters that are needed to analyse the foundation solutions on Port-Said soil. However, it is possible to calculate these parameters. Considering the available water content for the clay layer taken from Golder Associates report (1979), the modulus of compressibility $E_s$ can be calculated in equations related to the depth. Starting with the relationship between compression index $C_c$ and water content $w$ according to Nishida (1956) [38] expressed as

$$C_c = 0.0054(2.6w - 35) \tag{23}$$

where $w$ is the water content in (%).

The modulus of compressibility can be determined by equating settlement obtained from both compression index $C_c$ and initial void ratio $e_o$ with that obtained from the average vertical stress $\Delta\sigma$ and the modulus of compressibility $E_s$ [39, 40] as shown in the following equation:

$$\frac{1}{E_s}\Delta\sigma H = \frac{C_c}{(1+e_o)}\log\left[\frac{\sigma_o + \Delta\sigma}{\sigma_o}\right]H \tag{24}$$

where $E_s$ is the modulus of compressibility of Port-Said lower clay, $\Delta\sigma$ is the average vertical pressure increase in clay, $H$ is the layer thickness, and $\sigma_o$ is the initial overburden pressure in a layer. The overburden pressure is the pressure or stress imposed on a layer of soil by the weight of overlying soil.

Reda (2009) [22] determined the modulus of compressibility of Port-Said lower clay according to Eq.(24) and verified it by experiments. $E_s$ increases with depth and can be approximated by the following linear formula

$$E_s = E_{so}(1 + 0.06\,z) \tag{25}$$

where: $E_s$ is the modulus of compressibility, $E_{so}$ = 2 MN/m$^2$ is the initial modulus of compressibility, and $z$ is the depth measured from the upper surface of the lower clay layer in Figure 11.

The modulus of elasticity (Young's modulus) $E$ is also required as input to soil models in FE programs. The modulus of elasticity $E$ can be obtained from the following equation using Poisson's ratio $v$ [36]

$$E = E_S\frac{1-v-2v^2}{1-v} \tag{26}$$

## 3.3 Gothenburg soil properties

According to Olsson (2010) [23] the investigated soil profile in the Gothenburg area contains a few meters of fill followed by deep soft clay layer 40 m in thickness, and below the clay layer, there is a non-cohesive material a few meters in thickness on the rock. The over consolidation ratio (OCR) in the area is about 1.3 and the ground water level is about 1.5-2.0 m below the ground surface. The water content is about 80% in the top part of the clay, decreasing to around

50% at a depth of about 35 m. The soil profile can be seen in Figure 12. The unit weight is about 16 kN/m$^3$, the soil Young's modulus $E$ of the clay layer is about 5 MPa and drained Poisson's ratio is about 0.3 [23]. All the required soil parameters for the modified Cam-clay model and the Mohr-Coulomb model are presented in Olsson's work [23].

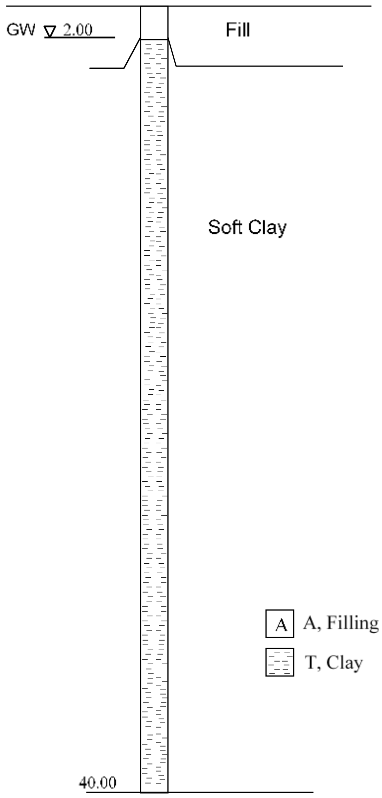


**Figure 12** Soil profile of Gothenburg in Sweden

## 3.4 Soil modeling

The geotechnical material models adopted in this work are the Mohr-Coulomb model (elastic perfectly plastic model) and the modified Cam-clay model (critical state model) [24, 25]. The Mohr-Coulomb model is used because it is common and its parameters are easy to obtain. However, the initial state of consolidation of the soil, such as the preconsolidation pressure $p_c'$ or the overconsolidation ratio *OCR*, cannot be specified in the Mohr-Coulomb model. In order to make high precision analysis of soft clay with high water content as in the Gothenburg soil, a coupled pore fluid diffusion/stress analysis using the modified Cam-clay model is needed.

The Mohr-Coulomb model is used to model three different soil profiles (stiff clay, Gothenburg soil and Port-Said soil). However, the modified Cam-clay is used only to model Gothenburg soil profile.

### 3.4.1 Mohr-Coulomb model

The Mohr-Coulomb model requires five input parameters. The Mohr-Coulomb model's stress-strain behaviour is linear in the elastic range with two defining parameters Young's modulus $E$ and Poisson's ratio $v$ [46]. However, the internal friction angle $\phi$ and the cohesion $c$ define the failure criteria. Also, the dilatancy angle $\psi$ is required to describe the flow rule (associated or non-associated flow rule) [25]. If the plastic potential function is equal to the yield function, the flow rule is called associated flow rule. However, if they are not equal, the flow rule is called non-associated flow rule. The behaviour of sand with both negative and positive dilatancy can be described by using a non-associated flow rule [46].

The failure criterion of the Mohr-Coulomb model regarding principal stresses is written [25] as:

$$\frac{\sigma_1' - \sigma_3'}{2} = c' \cos\phi' + \frac{\sigma_1' + \sigma_3'}{2} \sin\phi' \tag{27}$$

where $\sigma_1'$ and $\sigma_3'$ are the principal maximum and minimum effective stresses ($\sigma_1' > \sigma_3'$), $c'$ is the effective cohesion, and $\phi'$ is the effective internal friction angle [25]. The yield surface plot is shown in Figure 13.

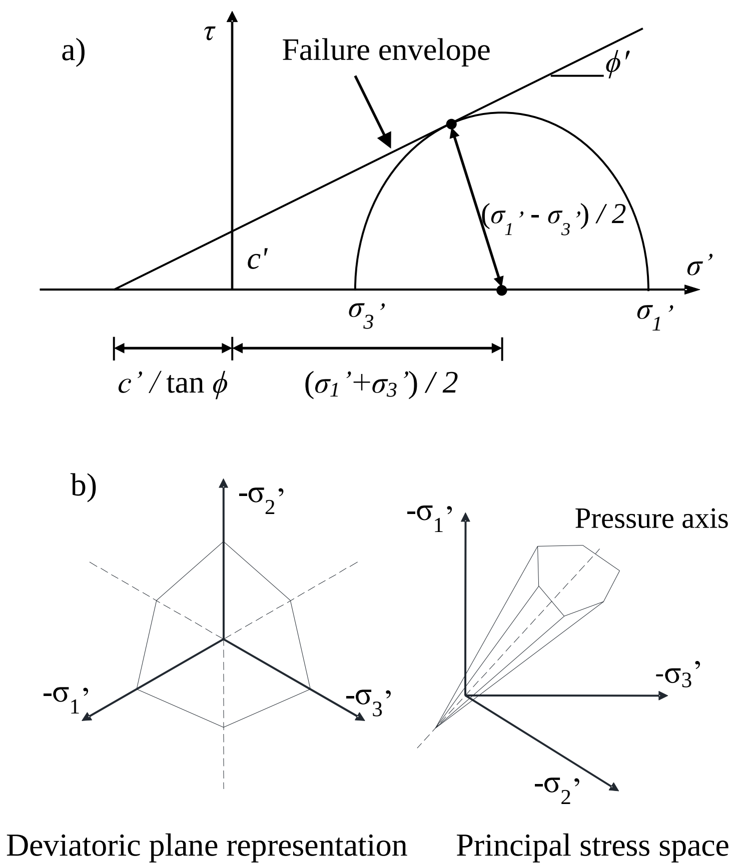


**Figure 13** a) The Mohr-Coulomb failure criterion in the space of shear and normal stresses, and b) yield surface in the deviatoric plane [45].

Failure occures when $\frac{\sigma_1' - \sigma_3'}{2}$ is reaching the failure envelop. The advantages of this model are its mathematical simplicity, parameters easy to obtain, and the general level of acceptance. The limitations are the numerical implementation of a failure criterion containing corners in the deviatoric plane that complicates the convergence in the plastic regime. Moreover, this model neglects any effect from the intermediate principal stress $\sigma_2$ which leads to an underestimation of the yield strength of the material [47, 48].

### 3.4.2 Modified Cam-clay model

The Cam-clay and modified Cam-clay models are the first critical state models developed for describing the behaviour of clay. The main advantage of these models is the ability to model the effects of changes in the soil volume due to the changing water content, giving a potential for more accurate result. In the modified Cam-clay model, the state of a soil sample is characterized by three parameters: mean effective stress *p'*, deviatoric shear stress *q'*, and specific volume *N* [58] that can be calculated as

$$p' = \frac{1}{3}(\sigma_1' + \sigma_2' + \sigma_3') \tag{28}$$

$$q' = \frac{1}{\sqrt{2}}\sqrt{(\sigma_1' - \sigma_2')^2 + (\sigma_2' - \sigma_3')^2 + (\sigma_3' - \sigma_1')^2} \tag{29}$$

$$N = 1 + e \tag{30}$$

The logarithmic relationship between the void ratio *e* and the mean effective stress *p',* which shown in Figure 14, is the basic assumption in this model. To understand this relationship, it can be assumed that a soil sample is subjected to a drained isotropic compression ($\sigma_1'=\sigma_2'=\sigma_3'$). On first loading, the soil moves down from point *A* along the normal consolidation line as shown in Figure 14. If the soil sample unloaded from point *B*, it moves along the swelling line towards point *C*. However, if the soil sample reloaded again, it moves back down the same swelling line towards *B* until point *B* is reached and then it begins to move down the normal consolidation line again [25].

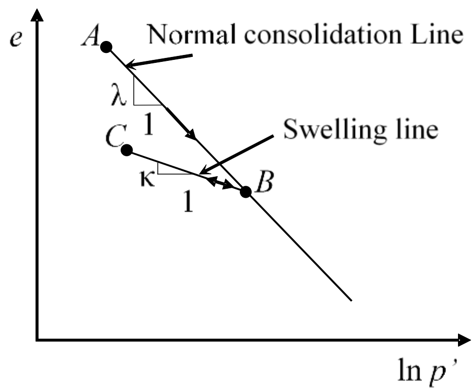


**Figure 14** The logarithmic relationship between the void ratio *e* and the mean effective stress *p'* [46].

Concerning shearing, sustained shearing of a soil sample finally leads to a state in which further shearing can occur at a constant stress with no volume change [58]. This state is called the critical state and characterized by the critical state line (CSL) as shown in Figure 15. In the *p'*-*q'* plane the *CSL* is a straight line passing through the origin with the slope equal to *M* being one of the characteristic of the material [58]. The consolidated drained triaxial test is used to obtain the critical state line. Moreover, the slope of the critical state line $M$ is related to the critical state friction angle $\phi'_{cs}$ [61] by:

$$M = \frac{6 \sin \phi'_{cs}}{3 - \sin \phi'_{cs}} \tag{31}$$

A simple laboratory test can be used to determine the critical state friction angle $\phi'_{cs}$. For normally consolidated clay this model has better suitability to describe deformation than failure [46]. The yield function of the modified Cam-clay model is determined from the following equation:

$$\frac{q'^2}{p'^2} + M^2 \left(1 - \frac{p'_c}{p'}\right) = 0 \tag{32}$$

where $q'$ is the deviatoric shear stress, $p'$ is the mean effective stress, $p_c'$ is the pre-consolidation pressure which means the larger effective pressure in the soil history, and $M$ is the slope of the critical state line in $p'$- $q'$ space. The modified Cam-clay yield surface plots as an elliptical curve as shown in Figure 15. The specification of the modified Cam-clay model requires six material parameters. These parameters are the slope of the normal compression line $\lambda$, the slope of a swelling line $\kappa$, the slope of the critical state line $M$, the specific volume at unit pressure, the Poisson's ratio $v$, and the preconsolidation pressure $p_c'$. The preconsolidation pressure $p_c'$ controls the size of the yield surface.

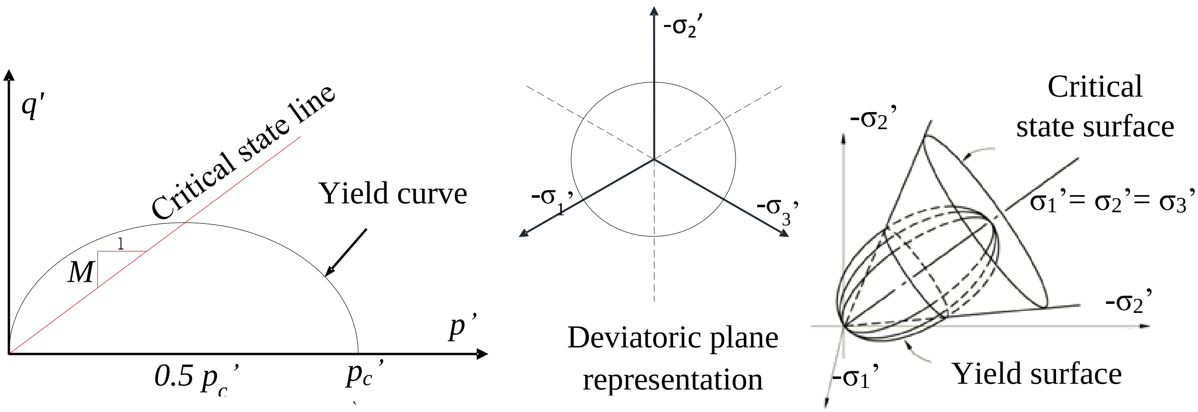


**Figure 15** Modified Cam-Clay yield surface in p\`- q\` space and in the deviatoric plane [45].

In soil modelling, the more fundamental elastic parameters of shear modulus $G$, and bulk modulus $K$, are preferred because they allow the effects of volume change and distortion to be decoupled [58]. In the modified Cam-clay model, the elastic volumetric strain increment [25] is given by

$$d\varepsilon_v^e = \frac{\kappa}{N} \frac{dp'}{p'} \tag{33}$$

This make the bulk modulus $K$ proportional to the mean effective stress $p'$, and the specific volume $N$ [25] and is calculated as

$$K = \frac{dp'}{d\varepsilon_v^e} = \frac{N\,p'}{\kappa} \tag{34}$$

This assumption results in a nonlinear elastic model in which the bulk modulus *K* varies according to Eq.(34). The modified Cam-clay formulations require specification of either shear modulus *G* or Poisson's ratio *v*. When *v* is given as a constant then *G* is determined using the following relationship

$$G = \frac{3(1-2v)}{2(1+v)}K \tag{35}$$

## 3.5 Loads

A limit state design concerns a set of performance criteria that must be met when a construction is subjected to loads [49]. There are two important conditions to be considered: the ultimate limit state (ULS) at failure and the serviceability limit state (SLS) under working loads [50]. The purpose of the serviceability limit state (SLS) requirements are to ensure that a construction is not affected by large settlements, tilting, cracks in concrete, etc [51]. These performance criteria must remain within pre-specified acceptable levels. The serviceability limit state (SLS) loads are used in the FE analysis in this study to calculate the settlement and the tilting of the foundations. The main purpose of the ultimate limit state (ULS) requirements are to ensure that the structure must not collapse when subjected to the highest load for which it was designed. The foundation needs to be verified in terms of bearing capacity, sliding resistance, and overturning resistance in the ultimate limit state (ULS). The load definitions are shown in Figure 16. Each set contains a centric vertical load *N* from the tower, blades, and nacelle being the same in both cases, a horizontal load *H*, a torsional moment $M_z$, and a bending moment *M*. Tower loads indicated in Table 3 are given at 0.6 m above the slab upper surface. Moreover, load conditions corresponding to a 2 MW wind turbine with 80 m tower height at the west coast of Sweden, are assumed [20].

**Table 3** Tower loads at 0.6 m above the slab upper surface.

| Type of limit state | Loads | | | |
|---|---|---|---|---|
| | N (kN) | H (kN) | M (kNm) | $M_z$ (kNm) |
| SLS | 3510 | 482 | 35108 | 303 |
| ULS | 3510 | 797 | 63825 | 1640 |

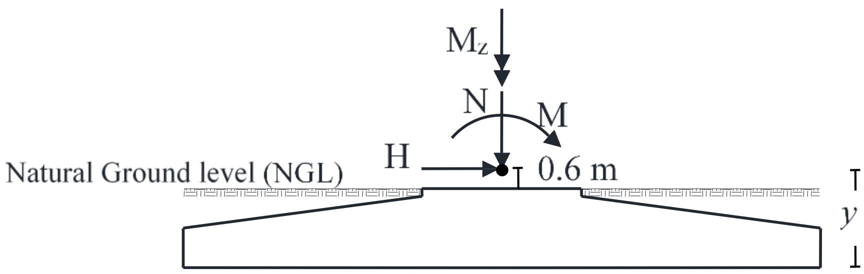


**Figure 16** Definition of load denotations.

# 4. Foundation descriptions

The considered foundations in this work are a flat circular raft, a conical raft, a piled raft, a flat circular raft surrounded by an active stabilisation system using a water tank, and a conical raft also surrounded by an active stabilisation system using a water tank. In the following section a simple description of the considered foundation systems is given.

## 4.1 Flat circular raft

A flat reinforced concrete raft is the most common foundation system for onshore windmills because it is suitable for many soil types. This kind of foundation system uses the foundation weight to resist the vertical load and the overturning moment. Details of the geometry and dimensions of a flat circular raft are shown in Figure 17-a.

## 4.2 Conical raft

The first shell footing was built in 1953 for a warehouse in Mexico City [26]. Several other projects have benefited from shell foundation systems in many countries like India, France, Russia, Germany and China. According to the studies mentioned in the introduction, the conical footings are capable of supporting higher vertical loads, have better settlement characteristics and are economical in terms of material volume need compared to the conventional flat footings. However, a shell footing has never been used as a foundation system for onshore wind turbines. As for the flat footing it uses the gravity foundation method to resist the extremely high overturning moment. The idea of using a conical raft involves increasing the stability loads due to the weight of the deeper soil placed over the foundation and also increasing the contact area between the soil and the foundation. Figure 17-b shows the geometry and dimensions of the proposed conical raft being the subject of Paper A.

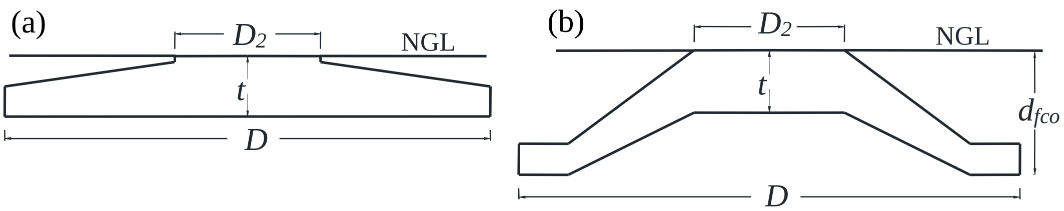


**Figure 17** Foundation types being the subject of Paper A: (a) flat circular raft, (b) conical raft

## 4.3 Piled raft

A piled raft is a foundation having piles to stabilise the foundation. The raft foundation and the piles are designed to cooperate to ensure that the settlement does not exceed the allowable settlement value. In this study, friction piles are used to reduce settlement and tilting. The geometry of the piled raft is shown in Figure 18.

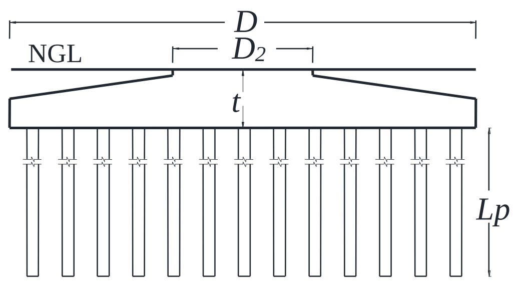


**Figure 18** Piled raft geometry.

## 4.4 Raft using an active stabilisation system

The concept of an active stabilization system is a novel idea using the weight of water to stabilise against the overturning moment. Two possible solutions are proposed and discussed in this thesis. The cross sections of these foundations are shown in Figure 19.

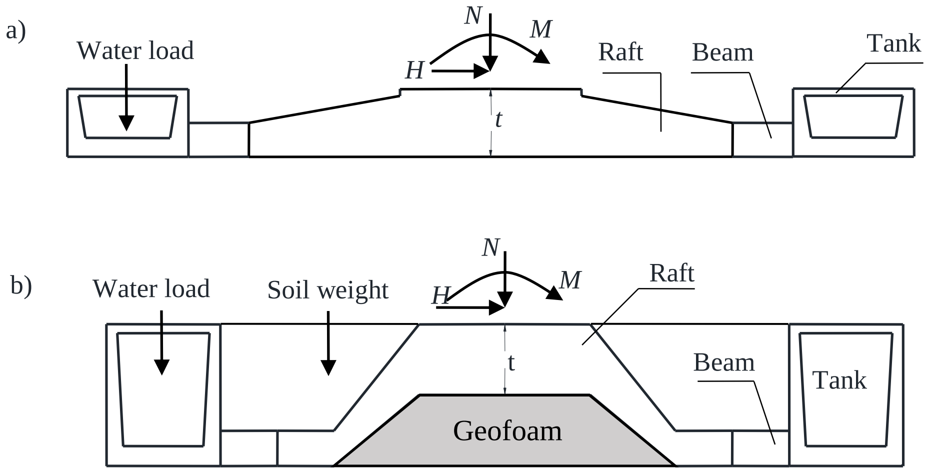


**Figure 19** The cross section of a) a flat circular raft using an active system, and b) a conical raft using an active system.

The water tank is divided into a number of compartments, and only one or two compartments will contain water, and all compartments are connected to an active system to move water between the compartments according to the wind direction. The reasons for using water as a movable load are: 1) the ease of moving water with wind turbine movement, 2) the ease of moving a spicific volume of water (not all the water volume) in order to give stabilising moment almost equal to the overturning moment in case of normal wind speeds, and 3) in case of very low wind speed, the water can be distributed equally using gravity alone in all the compartments.

There are two options to get a big resisting moment from the water load; the first is to use a big water volume and the second is to use long beams connecting the raft with the water tank. The design according to Figure 19 is evaluated in Paper B and C. The effect of using extended polystyrene (EPS) geofoam on the geotechnical behaviour of the conical raft with a water tank studied in Paper D. The main reason to use the EPS geofoam under the conical raft is to reduce job-site challenges. Geofoam has been used in the field of ground improvement during the last few decades. EPS geofoam is formed in blocks that can easily be shaped at the site. This reduces the work compared to compacting the soil with a conical shape under the raft. EPS geofoam is ultra-lightweight fill material with very good engineering properties compared to other fills. This lightweight material can reduce the settlement values and can also improve the stability against shear failure. EPS geofoam is manufactured in a range of compressive resistances [31]. The water movement system depends on a number of electric water pumps and pipes with electric valves. This number depends on the water volume and the movement time required. In general, water movement depends on the wind speed and the turbine rotor hub position, which consequently, depends on the wind direction. The control system for the water movement system uses data from the wind direction and wind speed sensors being used in the yaw system of the wind turbines. The following section will present the main parts of the water movement system and how to deal with risks.

# 5. Water movement system

Active system with water is used in Paper B and C for Gothenburg and Port-Said soils respectively. A brief discussion of this system is given here to show the idea and also discuss costs. In Paper D the system was improved to make it less complex and also cheaper to build and maintain.
During the last two years of this work, water movement systems were designed and developed in order to improve the performance and decrease the cost. In the first version (Paper B and C), described here the water tank is divided into four compartments, as shown in Figure 20, where only one or two compartments will contain water. In the last version (Paper D), the water tank is divided into eight compartments to improve the foundations geotechnical behaviour. The water movement systems are designed with a number of pipes with valves to move half the water volume of the filled tank using gravity alone and with a number of electric motors to move the other half. The water system described here uses a 5 m by 5 m tank in cross section. The total cost will include the units' costs, the construction cost, the power cost and the maintenance cost after 20 years lifetime. The annual maintenance cost for the motors in this study will be 5% of its original cost.

Designing the water movement systems are carried out for some constant values such as tank width $B_{tank}$ = 5.0 m, tank hieght $H_{tank}$ = 5.0 m and water volume = 275 $m^3$. Between all compartments, there are two pipes to move air from the compartments, which are intended to be filled to the other compartments that are intended to be emptied. The position of the water is depending on the wind direction. The distribution of the water according to the wind direction is presented in the appended articles. In case of a very low wind speed, all the electric valves will be opened to distribute the water volume equally in all compartments.

The system depends on seven electric motors between every two compartments, and one extra motor will be added for safety as shown in Figure 20. The solution to make the motors move water in two directions between every two compartments is shown in Figure 20. This system needs seven motors with a flow rate equal to 0.33 $m^3/s$ to move the whole water volume in two minutes from one compartment to the nearest compartments. The active yaw system speed for modern wind turbines is less than 0.5 degree per second [28]. This means that modern wind turbines need more than six minutes to turn 180˚, and this water movement system needs just four minutes to reach the required position (two minutes to reach the nearest compartment and another two minutes to reach the required compartment).

The comparison between all the versions shows that using pipes to move half of the water volume is very effective concerning costs. Also, the number of motors has the biggest effect on the cost. Using pipe systems connecting all the compartments are saving money and time. The most important advantage of the last version of the water movement system mentioned in Paper D is that the system can move the water from one compartment to any compartment (not just to the nearest compartment). Also, using eight compartments improve the overturning resistance in all the load cases.

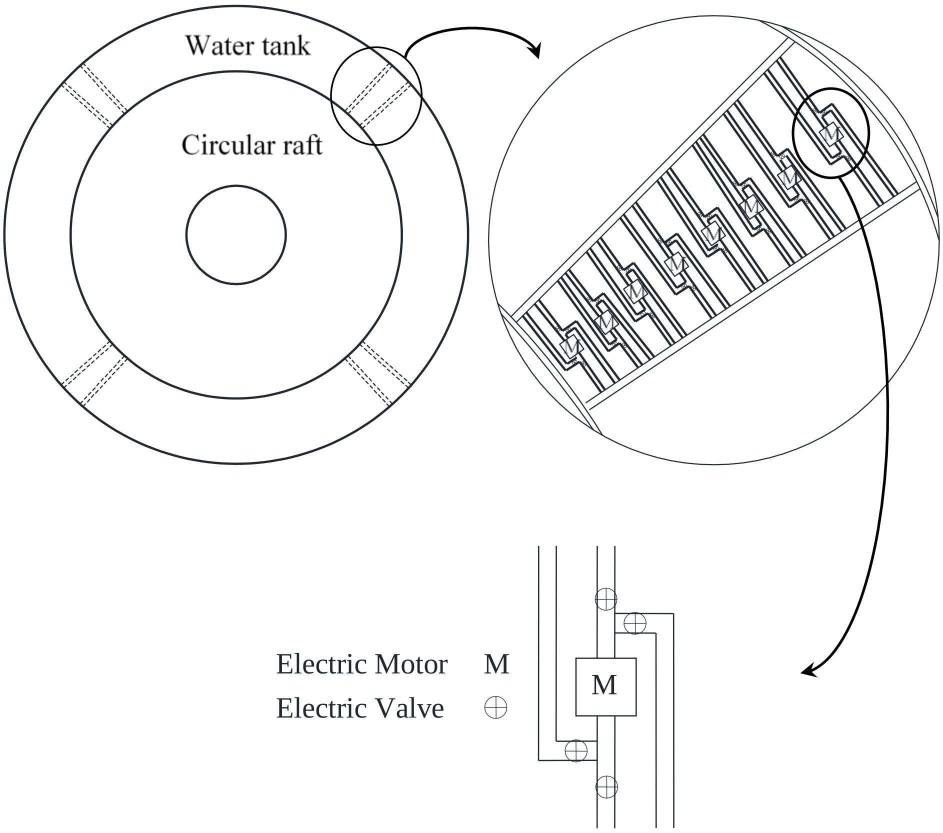


**Figure 20** The first version of the water movement system, and the solution to make the motor move water in two directions between connecting compartments.

All active systems face some risks, the risks have to be evaluated, and suggestions for reducing these risks have to be studied. Motor damage, water movement time, water freezing, water leaks, and power cut are the main risks of the mentioned active system. Lubrication of the motors shafts every few months, changing some parts every year, and using extra motors will minimise the motor damage risk. The water movement speed should be faster than the yaw system speed to decrease the water movement time risk. To minimize the water freezing risk, a closed tank with thick concrete sections is used. The minimum thickness of the tank section is 25 cm. The joints in the concrete tanks may cause water leaks and this problem can be prevented by using waterstop in these joints. Wind turbines already have Uninterruptible Power Supply (UPS) that supplies power to critical wind turbine components. The actual backup time for the UPS system is proportional to the power consumption. Actual backup time is 35 minutes for safety systems and the re-charging time is approximately 2.5 hours [28]. The water systems need few minutes to move the whole water volume from one compartment to another compartment.

# 6. FE Analysis of soil-foundation interaction

Three-dimensional finite element models were created using Abaqus for analysis of the soil-structure interaction. In some cases due to symmetry, half of the soil-structure system is modelled. The computational region for the studied soil is a half cylinder with radius 50 m and the depth in the z-direction is 50 m as shown in Figure 21. In other cases, full soil-structure system is modelled with a computational region of 100 × 100 × 50 m where the depth in the z-direction is 50 m. The continuum C3D20R element was selected in the cases of using Mohr-Coulomb soil model. It is a 20 node brick element of second order interpolation using reduced integration. However, a coupled pore fluid diffusion/stress analysis was performed to investigate the time-dependent behaviour of the soil in the cases of using the modified Cam-Clay model. Pore fluid/stress elements (C3D20RP) were selected to model the time-dependent behaviour of Gothenburg soil in Paper D.

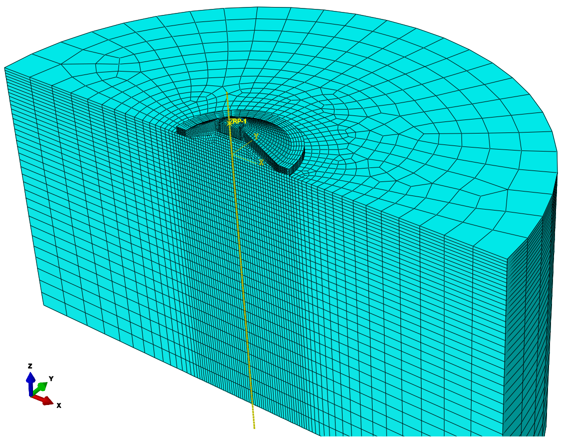

**Figure 21** Finite element mesh for the soil-foundation model

Structured meshes were used in all the cases. A fine grid was applied around the footing and a coarse grid for the far field. The minimum mesh size under the footing in some cases was 0.25 m and in other cases was 0.5 m. However, the maximum mesh size of the far field was four meters in some cases and six in other cases. The aspect ratio is equal to one in the soil meshes directly under the footings, and it is increasing for the far field. The footing slabs are modelled as linear elastic three-dimensional structures. A full interaction was assumed between the soil and the compression side of the foundation, while a surface contact interaction was used on the tension side of the footing and the soil. This interaction is used to prevent the soil from carrying any tension at the interface. The coefficient of friction between a clay soil and the footings was assumed to be 0.3. However, the coefficient of friction between a fine sand soil and the footings was assumed to be 0.4. In the case of using the Mohr-Coulomb model, two steps were used in the analysis; the first one was the geostatic step to make certain that the equilibrium is satisfied and the second step was the general static step to apply the tower loads. In the case of using the modified Cam-clay model, three steps were used in the analysis; the first one was the geostatic step also to make certain that the equilibrium is satisfied, the second step was the soil step to start the consolidation process in the soil (the drainage was prohibited), and the third was a soil step to investigate the time-dependent behaviour of the soil (the drainage was permissible).

For all the studied cases in this study, a convergence check was performed by using different element size meshes for the soil under the foundation. The results were shown to be convergent. Also, mesh independence studies were performed by using different dimensions for the soil-structure models to check that the meshes are independent of the boundary conditions.

# 7. Summary of appended papers

## Paper A

**A comparative study of two onshore wind turbine foundations**
W. M. Mohamed and P.E. Austrell
Submitted for publication

**Summary**

The geotechnical behaviour of a new foundation solution using a conical raft is investigated on a stiff clay soil profile. A geotechnical comparison between a flat raft and a conical raft is carried out. Also, the effect of various ways of apply the tower loads on the foundation is investigated. Moreover, a cost comparison between the mentioned foundation solutions is done.

**Contributions by W.M. Mohamed**

W. M. Mohamed contributed to the work by being the main author of the paper and planning the research tasks. He created FE models, performed calculations, and came to the conclusions.

## Paper B

**Optimization of wind turbine foundations for poor soil**
W. M. Mohamed and P.E. Austrell
Based on a paper published in the Proceedings of COMPDYN 2015, Crete; Greece; 25 - 27 May 2015

**Summary**

A geotechnical comparison between a new foundation solution using a flat circular raft with an active system with a surrounding water tank and a piled raft is made. A Gothenburg clay soil profile was used in this study. The material cost and the construction cost were according to Swedish prices. A first version of the water movement system is used in this study.

**Contributions by W.M. Mohamed**

W. M. Mohamed contributed to the work by being the main author of the paper and planning the research by creating FE models, performing calculations and drawing conclusions.

## Paper C

**New onshore wind turbine foundation system for poor soil**
W. M. Mohamed and P.E. Austrell
Based on a paper published in the Proceedings of EWEA 2015, Paris; France; 17 - 20 November 2015

**Summary**

The geotechnical behaviour of an innovative flat circular raft with an active system using a surrounding water tank and two types of piled rafts on Port-Said soil are investigated. A cost comparison is made between the foundation solutions according to Egyptian construction costs. Two water movement systems were compared in this study to show the effect of the motors flow rate and using pipes to move half of the water volume on the system cost.

**Contributions by W.M. Mohamed**

W. M. Mohamed contributed to the work by being the main author of the paper and planning the research tasks. He created FE models, performed calculations, and came to the conclusions.

## Paper D

**Evaluation of a new foundation solution for onshore wind turbines on soft clay**
W. M. Mohamed and P.E. Austrell
Submitted for publication

**Summary**

A geotechnical comparison between a new foundation solution using a conical raft with an active system with a surrounding water tank and a piled raft are investigated. A Gothenburg clay soil profile is used in this study. The effect of using EPS geofoam under the conical raft on settlement and tilting is also investigated. A detailed cost comparison was done according to Swedish prices. The water movement system is improved to make it cheaper and less complex. Moreover, in this study, the water tank is divided into eight compartments instead of four compartments to improve the overturning resistance.

**Contributions by W.M. Mohamed**

W. M. Mohamed contributed to the work by being the main author of the paper and planning the research by creating FE models, performing calculations and drawing conclusions.

# 8. Summary and future work

A summary of the work in the licentiate dissertation together with ideas for future work are presented here.

## 8.1 Summary

The main aim of this study was to establish new foundation solutions being more cost efficient than the most common foundation systems. This work discusses some important issues like settlement, tilting and cost. A number of case studies were presented to illustrate the behaviour of three foundation solutions: a conical raft, a circular raft with a water tank, and a conical raft with a water tank on soils with weak properties. The main aim is to reduce the costs and get a good geotechnical behaviour. In this regard, it was shown that a conical raft gives better results than a circular raft for soils having uniform weak properties, but it yields more excavation volumes. The total concrete volume and the steel weight were reduced by 16% and 31%, respectively. The total excavation volume, however, was increased with approximately 20% compared to the plane circular raft. The conical shape needs more construction time on the site compared to the flat shape. However, the total material cost for the conical raft is almost 12% less than for the circular flat raft according to construction costs in Egypt. The geotechnical performance of the conical shape is better than the geotechnical performance of the flat shape.

For soils containing deep soft clay layers, the active foundations overcome the tilting problem giving a tilting lower than a piled raft in many soil profiles. Using a piled raft gives the lowest magnitude of settlement. However, a water tank surrounding a conical raft gives the lowest value of tilting. The figures used in cost comparison were for particular load case, typical 2 MW load case, to clarify the difference in value between the new proposed solutions and the classical foundation solutions. For the cost issue, the circular raft surrounded by a water tank decreases the initial foundation cost by 32% compared to a piled raft with friction piles of 28 m length. Also, the conical raft surrounded by a water tank decreases the initial foundation cost by 16~35% compared to a piled raft with friction piles of 28 m length in Gothenburg (Sweden). Using geofoam under the conical raft decrease the maximum settlement of the foundation by almost 9% compared to the original case without geofoam. Also, there is a small difference in the tilting values.

Wind turbines already have Uninterruptible Power Supply (UPS) that supplies power to critical wind turbine components in case of power cut. Actual backup power time is thirty-five minutes for safety systems. To show the effect of using different moving times on the water movement system costs, two and fifteen minutes are used to move the whole water volume between the compartments. It was shown that using a long time decreases the motors flow rate which is considered as the main factor that affects the water movement system cost. The cost comparison shows that using fifteen minutes to move the whole water volume decreases the initial cost by almost 62% compared to using two minutes.

## 8.2 Future work

Wind energy projects repowering is an important topic nowadays. The definition of wind farms repowering adopted in this study is the complete dismantling of the existing turbine, tower and foundation at an existing project site and replaces these units with taller and larger units [59]. Wind farms repowering first appeared in the Danish wind power market and was followed by the Dutch and German markets. Nowadays, many other countries are also interested in repowering their existing wind farms [59]. In Germany, for example, a wind farm containing 116 wind turbines producing about 56 MW were dismantled in 2010 and replaced by 80 new wind turbines which can generate 183 MW [60]. The repowering concept shows a need for new solutions allowing wind power sites to be updated with taller and larger units in order to increase the energy production.  The investigation proposed in the near future intends to study the benefits - economical an environmental - of reusing the existing foundations.

Modern wind turbines are designed to withstand approximately for a 20~30 year lifetime. However, for high-quality concrete foundations, a very long design life of 100 years or more can be achieved, and no maintenance is required. Thus there is a big difference between wind turbines lifetime and foundations lifetime. Once the turbines exceed their design life, there are two options. The first option is to replace the out-of-date wind turbine with a new one and the second option is to have the whole windmill inculding the foundation removed entirely. There is a big economic benefit of replacing the out-of-date wind turbine with a new turbine of better power generation capacity and reuse the foundation. However, the blades evolve to be longer, lighter and more efficient due to the rapid evolution of blade design. This will change the design loads on the foundations (increase the dragging force and decrease the vertical force). This means more overturning loads.

In the near future a detailed investigation will be done to: 1) analyse some existing windmill foundations in Sweden and give some recommendations about the acceptable load changes (this will also involve methods to reinforce old foundations), 2) figure out the new design requirements for establishing new windmill foundations to be reusable, and 3) figure out how to prepare for reuse of the new foundation solutions.

# Bibliography

[1] Annual Report from Egyptian Electricity Holding Company (2011/2012). The Egyptian Ministry of Electricity and Energy (http://www.moee.gov.eg/english_new/EEHC_Rep/2011-2012.pdf)

[2] The Swedish Institute (2013) available at:

https://sweden.se/wp-content/uploads/2013/11/Energy-Low-Res.pdf

[3] Svensson H. (2010). *Design of foundations for wind turbines.* Report TVSM-5173, Master's dissertation, structural mechanics. Lund University, Sweden. Available at: www.byggmek.lth.se/fileadmin/byggnadsmekanik/publications/tvsm5000/web5173.pdf

[4] Tinjum, J.M., and Christensen, R.W. (2010). "Site investigation, characterization and assessment for wind turbine design and construction." *In Wind Energy Systems, edited by John D. Sorensen and Jens N. Sorensen, 26-45. Woodhead Publishing*.

[5] Snyder B. and Kaiser M.J. (2009). Ecological and economic cost-benefit analysis of offshore wind energy. *Renewable Energy,* 34**,** 1567-1578.

[6] Manwell JF, Macleod J, Wright S, Ditullio L and Mcgowan JG (2006). Hull Wind II: A Case Study of the Development of a Second Large Wind Turbine Installation in the Town of Hull, MA. *2006 American wind energy wind association windpower conference*. Available from:
http://www.ceere.org/rerl/publications/published/2006/AWEA%202006%20Hull%20II.pdf

[7] Nicholls, R.L., and Izadi, M.V. (1968) Design and testing of cone and hypar footing. Journal of soil and mechanics and foundation Div. ASCE, 94(SM1), 47-72.

[8] Jain, V. K., Nayak, G.C., and Jain, O.P. (1977) General behaviour of conical shell foundation. Proc. 3rd Int. Symp. Soil structure interaction, University of Roorkee, India, 2, 53-61.

[9] Kurian N. P. (1994) Behaviour of shell foundations ubder subsidence of core soil. Proc. 13th Int. Conf. Soil Mechanics and foundation eng. New Delhi, Indea, 2, 591-594.

[10] Abdel-Rahman M. (1996) Geotechnical behaviour of shell foundations. Doctoral thesis, Concordia University, Canada.

[11] Huat B.B.K. and Mohammed T.A. 2006. Finite Element Study Using FE Code (PLAXIS) on the Geotechnical Behaviour of Shell Footings. *Journal of Computer Science* 2 (1): 104-108.

[12] Shaligram P.S. (2011). Behaviour of triangular shell strip footing on georeinforced layered sand. *International Journal of Advanced Engineering Technology,* Vol.II, Issue II,192-196.

[13] Joseph E. Bowles. 1997. Foundation analysis and design. 5th ed. page 276

[14] Hansen, J. B. (1970), "A Revised and Extended Formula for Bearing Capacity," Danish Geotechnical Institute, Copenhagen, BuI. No. 28, 21 pp. (successor to BuI. No. 11).

[15] Det Norske Veritas, Copenhagen and Wind Energy Department, Risø National Laboratory (2002). Guidelines for Design of Wind Turbines, Second Edition, pp.187-221.

[16] Szerzo, Á. (2012). Optimization of foundation solutions for wind turbines. Mathematical Modeling in Civil Engineering 4: 215-224.

[17] Hansen, B., (1978). Geoteknik og Fundering, Del II Forelæsningsnotater til Kursus 5821 − Geoteknik 2, Notat nr. 16, Den private ingeniørfond ved Danmarks tekniske Højskole, Lyngby, Denmark.

[18] Bond, A.J., Schuppener, B., Scarpelli, G. and Orr, T. L. L. (2013) Eurocode 7: Geotechnical design worked examples. Publications Office of the European Union. Doi: 10.2788/3398. pp. 28.

[19] The Egyptian code of soil mechanics and foundation engineering. Egypt. 2005 (in Arabic).

[20] Svensson, H. (2010). Design of foundations for wind turbines. Master, Master's dissertation for structural mechanics. 161 p.
www. byggmek. lth. se/fileadmin/byggnadsmekanik/publications/tvsm5000/web5173. pdf.

[21] Golder Associates (1979) Geotechnical Report for Port-Said Area. Port-Said, Egypt.

[22] Reda, A. (2009) Optimization of piled raft in Port-Said. M.Sc. Thesis, Faculty of Engineering, Suez Canal University, Port-said, Egypt.

[23] Olsson, M. Calculating long term settlement in soft clays with special focus on the Gothenburg region. Thesis for the Degree of Licentiate of Eng., Department of Civil & Environmental Eng., Chalmers University of Technology, Gothenburg, Sweden. 2010.

[24] Ottosen N. S. and Ristinmaa M. 2005. *The Mechanics of Constitutive Modeling*. Elsevier Ltd. Chapter 8, pp 165-173.

[25] Potts D.M. and Zdravkovic L. 1999. Finite element analysis in geotechnical engineering theory. Thomas Telford, London, UK, Section 7, pp.151-155.

[26] Candela, F. (1955) Structural applications of hyperbolic paraboloidal shells. J. ACI, 26 (5), 397-415.

[27] Larock, B. E., Jeppson R. W. and Watters G. Z. (1999) *Hydraulics of pipeline systems*, CRC press LLC.

[28] Vestas Wind Systems (2010) General Specification V90–1.8/2.0 MW 50 Hz VCS. Document no.: 0004-6207 V05. Available from: http://ventderaison.eu/gembloux/eie_ABO-WIND/Annexes/Annexe_N_1_Courbe_acoustique_V90.pdf

[29] Sektionsfakta: kostnadsberäkningar av sektioner för byggbranschen. 13/14. (2014). Växjö: Wikells byggberäkningar. (in Swedish)

[30] The Iowa Environmental Mesonet (IEM) (2015) Available at:
http://mesonet.agron.iastate.edu/sites/windrose.phtml?network=SE__ASOS&station=ESGP

[31] Duškov, M., (1997) Materials Research on EPS-20 and EPS-15 Under Representative Conditions in Pavement Structures. Geotextiles and Geomembranes, Vol. 15: pp. 147-181.

[32] Horvath, J.S. (1995) Geofoam Geosynthetic. Horvath Engineering, P.C. Scarsdale, New York, USA, 217 p.

[33] Contech Engineered solutions [cited at 2016-01-15] available from: URL, http://www.conteches.com/Markets/Wind-Turbine-Foundations/Tensionless-Pier-Wind-Turbine-Foundation

[34] Morgan, K., Ntambakwa, E. (2008). Wind Turbine Foundation Behaviour and Design Considerations. AWEAWIND POWER Conference, Houston, June. 1-14.

[35] European Committee for Standardization (ECS), Eurocode 7 – Geotechnical design – Part 1: General rules, Swedish Edition SS-EN1997-1, Swedish Standards Institute (SIS), Stockholm 2008.

[36] Kany, M., El Gendy, M., and El Gendy A. Computer Analysis and Design of Foundations. Volume I: Theory used in the formulation of ELPLA. Chapter 8.

[37] Engström, S., Lyrner, T., Hassanzadeh, M., Stalin, T., and Johansson, J. (2010). Tall towers for large wind turbines. Report from Vindforsk project V-342 Höga torn för vindkraftverk.

[38] Nishida, Y. (1956). A brief note on compression index of soils. Journal of Soil Mechanics and Foundations Division, ASCE, 82, SM3, 1027-(1-14).

[39] DeBeer, E.E. 1965. Bearing capacity and settlement of shallow foundations on sand. Proceedings, Symposium on Bearing Capacity Settlement of Foundations, Duke University, Durham, N.C., 15-33.

[40] DeBeer, E. & Martens, A. 1957. Method of computation of an upper limit for the influence of heterogeneity of sand layers in the settlement of bridges. Proceedings, 4th International Conference on Soil Mechanics and Foundation Engineering, London, 1: 275-281.

[41] Terzaghi, K. (1943) Theoretical soil mechanics. Wiley. New York.

[42] Das, B.M. (2015) Principles of foundation Engineering, SI Edition. Chapter 4. pp 156-157. ISBN 1305446291, 9781305446298

[43] Vesic, A.S. (1973) Analysis of ultimate loads of shallow foundations. ASCE Journal of the Soil Mechanics and Foundation Division, Vol. 99, N°1, pp. 45-73.

[44] Three Primary types of bearing capacity failures of foundation. [Cited at 2016-04-04] Avilable at: http://civilblog.org/2015/07/25/3-primary-types-of-bearing-capacity-failures-of-foundation/

[45] ABAQUS Theory Manual, Version 6.12. Simulia.

[46] Ti KS, Huat BBK, Noorzaei J, Jaafar MS, Sew GS. A review of basic soil constitutive models for geotechnical application. Electronic Journal of Geotechnical Engineering 2009; 14:1–18.

[47] Labuz, J., Zang, A. (2012): Mohr-Coulomb Failure Criterion. - Rock Mechanics and Rock Engineering, 45, 6, 975-979

[48] Al-Ajmi, A.M., Zimmerman, R.W., 2005. Relation between the Mogi and the Coulomb failure criteria. Int. J. Rock Mech. Miner. Sci. 42 (3), 431–439.

[49] McCormac, Jack C. (2008). Structural Steel Design ($4^{th}$ ed.). Upper Saddle River, NJ: Pearson Prentice Hall. ISBN 978-0-13-221816-0.

[50] Nater, P. (2000). Ultimate (ULS) and serviceability (SLS) limit state design of rigid foundations on layered soils. Doctoral Thesis, Institute for Geochnical Engineering, Swiss Federal institute of Tecgnology, Switzerland.

[51] Limit state design [Cited at 2016-04-04] https://www.scribd.com/doc/51971809/ULS-and-SLS#

[52] Das, Braja M. 2007. Principles of Foundation Engineering. 6th ed. New Delhi:Cengage Learning.

[53] Skampton, A. W. 1951. The bearing capacity of clays. Proc. Build. Res. Congr. London.

[54] Wayne C. Teng (1962). Foundation Design. Published by Prentice Hall Professional Technical Reference, ISBN-13:978-013329805.

[55] P. Purushothama Raj (2008) Soil Mechanics & Foundation Engineering. Published by Dorling Kindersley. India. ISBN: 81-317-117-3

[56] Murthy V. N. S. (2003) Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering. ISBN 0-8247-0873-3.

[57] Helwany, S. (2007) Applied Soil Mechanics with ABAQUS Applications. ISBN 978-0-471-79107-2

[58] Description of Cam-Clay and modified-Cam-Clay critical state strength models [Cited at 2016-04-04]
https://www.rocscience.com/help/phase2/webhelp/pdf_files/theory/CamClay.pdf

[59] Eric Lantz, Michael Leventhal, and Ian Baring-Gould (2013) Wind Power Project Repowering: Financial Feasibility, Decision Drivers, and Supply Chain Effects. The National Renewable Energy Laboratory (NREL) technical report. Available from URL: http://www.nrel.gov/docs/fy14osti/60535.pdf

[60] Online page (Cited at 2016-05-16) Available from URL:
https://www3.stahleisen.de/Portals/stahleisen/E_140301214-Internet.pdf

[61] Santamarina, J. C. and Cho, G. C. (2001) "Determination of Critical State Parameters in Sandy Soils-Simple Procedure. Geotechnical Testing Journal, GTJODJ, Vol. 24, No. 2, pp. 185–192.

# Part II

# Appended publications

# Paper B

## Optimization of wind turbine foundations for poor soil

Based on a paper published in the proceeding of COMPDYN 2015

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**Optimization of wind turbine foundations for poor soil**

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**Keywords:** FE analysis, Wind turbine foundations, Circular raft with a water tank, Dynamic analysis.

**Abstract.** The main focus of this work is to investigate the geotechnical behaviour of a new windmill foundation solution on a soil with weak properties. The evaluated new solution is a flat circular raft surrounded by an active stabilisation system using a water tank. A comparative study of two foundation solutions, the new foundation solution and a piled raft with long friction piles, has been performed using finite element simulation. An existing soil profile in Gothenburg city in Sweden is used in this study. The results show that the new foundation solution decreases the differential settlement compared to a piled raft with 28 m pile length. Also, a cost comparison between the new foundation and a piled raft has been performed using Swedish construction costs. The cost comparison shows that the new foundation solution gives a significant decrease in the initial foundation costs compared to a piled raft with 28 m pile length. Also, the dynamic response of the whole structure using soil-foundation interaction was investigated. It is shown that the entire system of a 2MW wind turbine successfully avoids resonance through the rotor excitations in the case of using the new foundation solution.

## 1 INTRODUCTION

Currently, most of the windmills are built on soils with good properties to decrease the foundation costs. However, there are a lot of good wind spots in many regions with weak soils. Therefore, probably in the near future windmills may have to be built on weak soils. The main problems of soil containing a deep weak soil layer are settlement and differential settlement. Deep foundations such as piled raft and tensionless pier, are used to support windmills on regions with weak soils. Deep foundations have several disadvantages when compared to shallow foundations, being mainly high cost and long construction time.

Wind turbine foundation costs vary significantly from country to country due to the variation of construction material costs. Usually, the foundation cost range depends on some factors such as the soil properties, the foundation type, and the foundation dimensions. Flat raft foundation is considered as the traditional foundation solution for windmills on soils with good properties. According to Tinjum and Christensen (2010) [1] the octagonal raft foundation cost range is from $100,000 to $250,000 depending on the foundation dimensions for diameters ranges from 12 m to 18 m. A cost-benefit study of wind energy showed that onshore wind turbine foundations without piling on good soil make up about 3%~7% of the total costs [2, 3]. Deep foundation systems, on the other hand, will provide excellent performance for settlement and tilting. However, this comes at a high cost, both financially and environmentally, and needs longer time to construct. Manwell et al. [4] evaluated cases where piling was required and found that foundation costs accounted for 28% of the total costs. Therefore, windmills on weak soils need new cost-effective foundation solutions which provide a balance of cost, speed of construction, quality, and performance.

Soft soils have an influence on the dynamic response of the structure. According to Enrique (1986) [5], soil-structure interaction had a significant effect on soils which have shear wave velocity lower than 750 m/s and this effect could lead to a reduction in response. A study on the influence of soil-structure interaction on the dynamic response of windmills showed that considering soil-structure interaction is a vital aspect in order to avoid the resonance range [6]. Therefore, the effect of using the new foundation solution on the dynamic performance of the structure considering the soil-structure interaction is investigated in this study.

The objective of this work is to conduct a comprehensive analysis of the geotechnical behaviour, dynamic behaviour, and the cost of a new foundation solution in comparison to a piled raft with long friction piles. In this work, load conditions are based on loads for a 2 MW wind turbine with 80 m tower height at the west coast of Sweden [7].

## 2 MATERIAL PROPERTIES AND LOADS

The main characteristics of soil and load sets are presented in this section. The geotechnical material model adopted in this study is the Mohr-Coulomb model [8]. The parameter values of the soil can be seen in Table 1. The undrained cohesion $c_u$ of the soil equals 30 kPa, and Poisson's ratio of the soil $\nu_s$ equals 0.3 [9]. The backfilled soil ($\gamma$ =18 kN/m$^3$) is assumed to be up to the natural ground level (NGL) as shown in Figure 1.

Two sets of loads are used in this study; serviceability limit state (SLS) loads, and ultimate limit state (ULS) loads of a Vestas V90-2.0 MW wind turbine [7, 10]. In this study, the serviceability limit state (SLS) loads are used in the FE analysis to calculate the settlement and the tilting of the foundations. However, the foundation needs to be verified in terms of bearing capacity, sliding resistance, and overturning resistance using the ultimate limit state (ULS) loads. Each set contains a centric vertical load $N$ from the tower, blades, and nacelle being the same in both cases, a horizontal load $H$, a twisting moment $M_z$, and a bending moment $M$. The load definitions are shown in Figure 1.

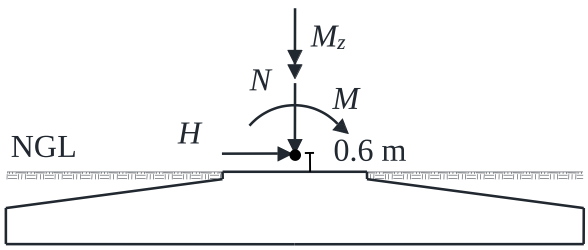


**Figure 1** Definition of load denotations [7]

The loads indicated in Table 1 are acting at 0.6 m above the upper surface of the slab as shown in Figure 1. Moreover, load conditions corresponding to a 2 MW wind turbine with 80 m tower height at the west coast of Sweden are used [7]. Fatigue is not included in this study.

**Table 1** Material properties and loads

| IV. Soil Parameters | | | | | |
|---|---|---|---|---|---|
| Description | depth $z$ (m) | Unit weight $\gamma$ (kN/m$^3$) | Young's modulus $E$ (kPa) | Internal friction angle $\emptyset$' (°) | Cohesion $c$' ( kPa ) |
| Soft clay | 40 | 17 | 5000 | 30 | 1 |
| **V.** Reinforcement concrete material parameters | | | | | |
| Concrete class is C30/37 | | | Reinforcement | | |
| Young's modulus $E_b$ (kN/m$^2$) | Poisson's ratio $v_b$ | Unit weight $\gamma_b$ (kN/m$^3$) | Yield stress $f_{yk}$ (MPa) | Design Yield stress $f_s$ (MPa) | Young's modulus $E_s$ (GPa) |
| $3.3 \times 10^7$ | 0.2 | 25 | 500 | 435 | 200 |
| **VI.** Tower loads, characteristic values | | | | | |
| Load set | Type of limit state | Load set | | | |
| | | $N$ (kN) | $H$ (kN) | $M$ (kNm) | $M_z$ (kNm) |
| I | SLS | 3510 | 482 | 35108 | 303 |
| II | ULS | 3510 | 797 | 63825 | 1642 |

## 3 FOUNDATION DESCRIPTION

The considered foundations in this work are a piled raft and a flat circular raft surrounded by an active stabilisation system using a water tank as shown in Figure 2 and 3. In the following sections, a simple description of the considered foundation systems is given.

### 3.1 Piled raft

A piled raft is a foundation having piles to stabilise the foundation. The raft foundation and the piles are designed to cooperate to ensure that the settlement does not exceed the allowable settlement value. In this study, friction piles are used to reduce the settlement and the tilting of the foundation. The geometry of the piled raft is shown in Figure 2.

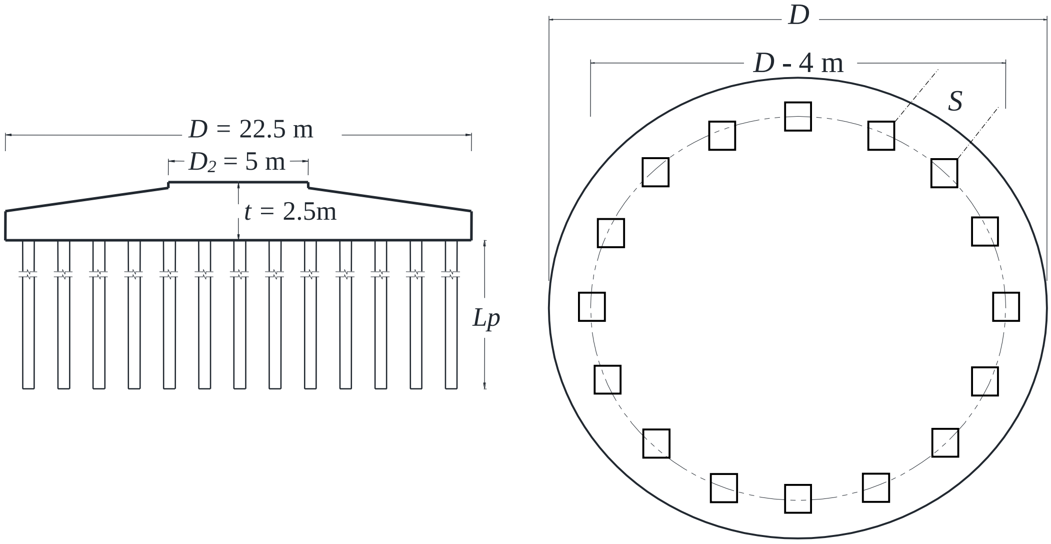


**Figure 2** Piled raft geometry.

In the following analysis, the following characteristics are kept constant; raft diameter *D* is equal 22.5 m, square piles of $a_{pile}$ = 1.0 m are used, and the spacing between piles *S* equals 2.4 m. Piles are located symmetrically along a ring with $D_{ring}$ = 18.5 m. However, the only foundation variable in this study is the pile length $L_p$ = 16, 20, 24 and 28 m.

### 3.2 Flat circular raft surrounded by an active stabilisation system

The concept of an active stabilisation system using a water tank is a novel idea using the weight of water to stabilise against the overturning moment. The water tank is divided into four compartments, and only one or two compartments will contain water, and all compartments are connected to an active system to move water between the compartments according to the wind direction. More details about the water movement is presented in the Appendix. The reasons for using water in this system as a movable load are: 1) the ease of moving water with wind turbine movement, 2) the ease of moving a specific volume of water (not all the water volume) in order to give stabilising moment almost equal to the overturning moment in case of normal wind speeds, and 3) in case of very low wind speed, the water can be distributed equally using gravity alone in all the compartments. Figure 3 shows more details of the geometry and dimensions of the new foundation solution.

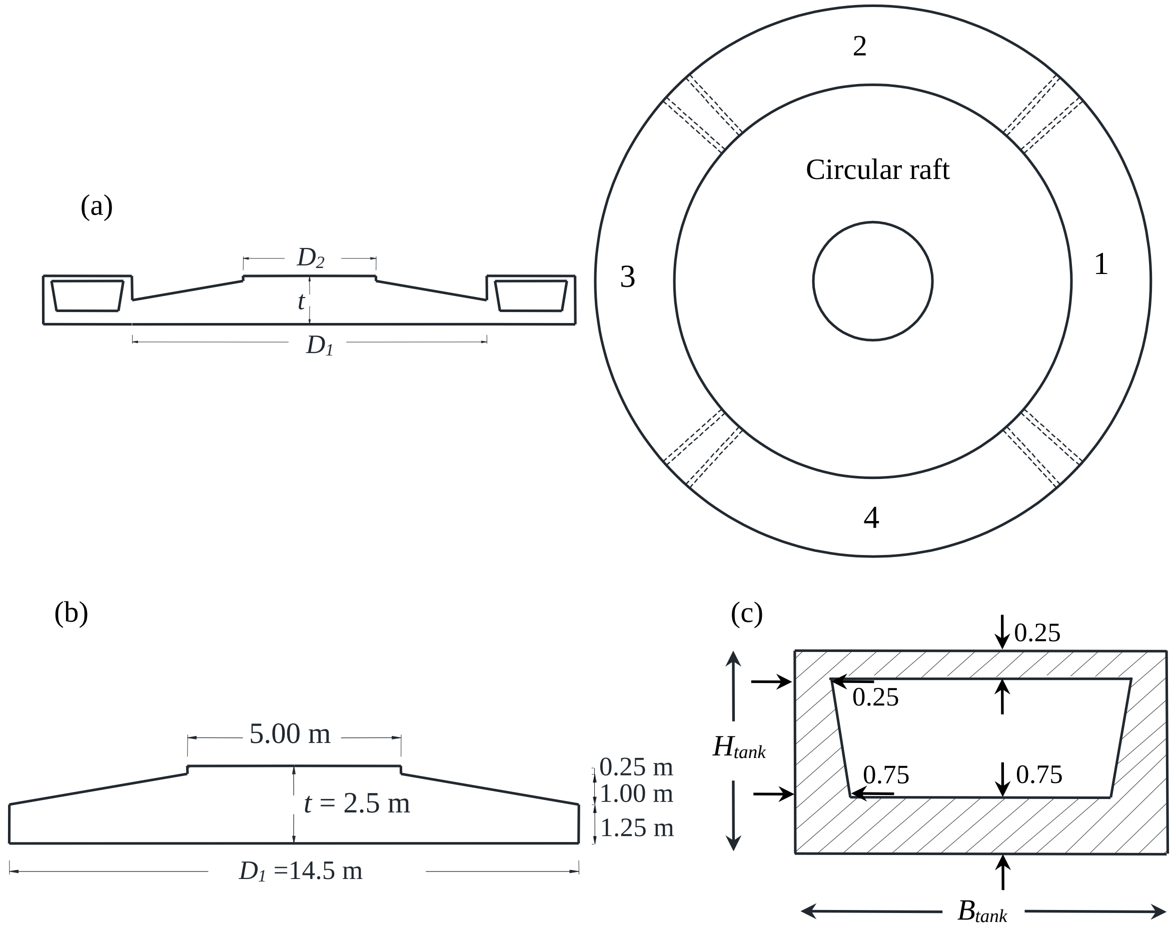


**Figure 3** (a) Circular raft surrounded by a water tank, (b) Vertical section for a circular raft, and (c) Vertical section for a water tank

In the following analysis, the following characteristics are kept constant; the diameter of the circular raft $D_1$ equals to 14.5 m, the raft thickness under the tower $t$ equals to 2.5 m, and the diameter of the upper cylinder $D_2$ is equal to 5.0 m. For the water tank, the height of the tank $H_{tank}$ is 5.0 m, the thickness of the vertical walls increase from 0.25 m in the upper part to 0.75 m in the lower part. The thickness of the upper slab is 0.25 m, and the thickness of the lower slab is 0.75 m. However, the analysis is carried out for one foundation variable, namely the tank width $B_{tank}$ = 4.0 and 5.0 m.

## 4 FE MODEL

A three-dimensional finite element model of the foundation-soil system is created using *Abaqus* [11]. For the soil model, the computational region chosen is 100$^{x}$100$^{x}$50 m where the depth in the z-direction is 50 m. The continuum C3D20R element is selected using a fine grid around the raft and a coarse grid for the far field. The geotechnical material model adopted in this work is the Mohr-Coulomb model [8] with parameters according to Table 1. A full interaction is assumed between the soil and the compression side of the foundation. However, surface contact interaction is used on the tension side of the foundation between the soil and the foundation to allow the foundation to elevate without any tension at the interface. The coefficient of friction between a clay soil and the footings was assumed to be 0.30.

### 4.1 FE verification for the shallow foundation

One case was studied and compared with the results on Svensson's work [7]. Svensson analysed a circular raft on moraine soil using 2D FE model in Plaxis. Svensson's results showed that the differential settlement of a circular raft on moraine soil is equal to 1.25 cm, the maximum settlement is equal to 1.3 cm and the minimum settlement is 0.05 cm. In this study, 3D FE model is established in Abaqus with the same foundation dimensions and soil properties. The present FE model results shows that the differential settlement is equal to 1.13 cm with 9.6% deviation, the maximum settlement is equal to 1.24 cm, and the minimum settlement is 0.11 cm.

### 4.2 FE verification for the deep foundation

One case was studied and compared with the results on Abdel Glil's work [12]. In Abdel Glil's work, an ordinary building rested on a piled raft foundation in Port-Said city is analysed using Elpla software. Abdel Glil calculates the maximum and differential settlements of a piled raft with a raft thickness equals to 1.1 m, a pile length of 24 m, a pile spacing of 2 m and a pile diameter of 0.5 m. Abdel Glil results show that the maximum settlement is equal to 12.6 cm and the differential settlement is equal to1.89 cm [12]. A 3-D FE model is established in Abaqus with the same foundation dimensions and soil properties. The results of the FE model in Abaqus are: the differential settlement equals to 2.13 cm with 11.3% deviation, and the maximum settlement equals to 13.36 cm with 5.7 % deviation.

## 5 SETTLEMENT AND DIFFERENTIAL SETTLEMENT

As mentioned, the FE model is used to calculate the maximum settlement and the differential settlement ($\Delta S = S_{max} - S_{min}$) of the new and the traditional foundations. Figure 4 shows the maximum settlement and the minimum settlement of a circular raft surrounded by a water tank and a piled raft. The following results are from the case of using one compartment to contain the whole water volume.

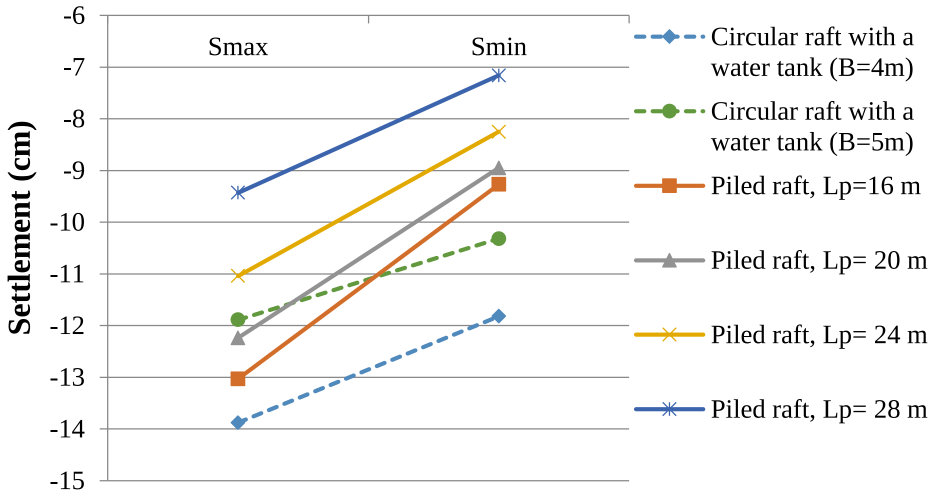


**Figure 4** Maximum settlement and minimum settlement for a piled raft and a flat circular raft with a water tank.

The results show that using a circular raft surrounded by a 5 m by 5 m water tank increases the maximum settlement by 26% compared to a piled raft with 28 m pile length. However, it is still lower than the allowable settlement for shallow foundations.

Table 2 presents the differential settlement of a circular raft surrounded by a water tank and a piled raft. Also, Table 2 presents the reduction of the differential settlement $H_s$ which is

$$H_s = (1 - (\Delta S_{rwt} / \Delta S_{pr}))\ 100 \qquad (2)$$

where $\Delta S_{rwt}$ is the differential settlement of a circular raft with a water tank, and $\Delta S_{pr}$ is the differential settlement of a piled raft.

**Table 2** Differential settlement (cm) and the reduction of the differential settlement

| $B_{tank}$ | | $L_p$ = 16 m | | $L_p$ = 20 m | | $L_p$ = 24 m | | $L_p$ = 28 m | |
|---|---|---|---|---|---|---|---|---|---|
| | $\Delta S_{rwt}$ | $\Delta S_{pr}$ | $H_s$ | $\Delta S_{pr}$ | $H_s$ | $\Delta S_{pr}$ | $H_s$ | $\Delta S_{pr}$ | $H_s$ |
| 4 m | 2.06 | 3.76 | 45% | 3.29 | 37% | 2.78 | 26% | 2.27 | 9% |
| 5 m | 1.57 | | 58% | | 52% | | 44% | | 31% |

From Table 2, it can be concluded that using the new foundation decreases the differential settlement compared to a piled raft with long friction piles. Also, using a circular raft surrounded by a 5 m by 5 m water tank decreases the differential settlement by 31% compared to a piled raft with 28 m pile length. Also, using a 4 m by 5 m water tank decreases the differential settlement by 9% compared to a piled raft with 28 m pile length.

## 6 COST COMPARISON BETWEEN THE FOUNDATION SYSTEMS

A cost compaeison is carried out to calculate the cost of the reinforcement concrete, the water movement system and the pilling for the new foundation and a piled raft. For reinforcement in piles, the calculation uses 12 bars Ø 16 mm and 125 mm for lateral tie spacing in the connection between the piles and the raft. Table 3 shows a cost comparison between the foundation systems. The comparison is depending on estimate cost comparison according to Sektionsfakta [13] and Waleed Mhanaa (Senior Estimation Engineer at Consolidated Contractors Company).

**Table 3** Cost comparison between a flat raft surrounded by a water tank and a piled raft.

| | Piled raft | Circular raft surrounded by a water tank | Cost in Sweden (USD) |
|---|---|---|---|
| Volume of Reinforcement concrete ($m^3$) | 665 | 717 | 170/$m^3$ |
| Steel weight (tonne) | 48 | 56 | 1625/ton |
| Pile (drilling + material) (m) | 672 | 0 | 500/m |
| Excavation ($m^3$) | 994 | 1988 | 12/$m^3$ |
| Water movement system | 0 | 32 motors + 60 pipes + 188 electric valves | 208000 |
| Total Cost (USD) | 539000 | 444750 | |

From Table 3, the cost comparison shows that using the new foundation system in Gothenburg region decreases the initial foundation costs by 17 % compared to using a piled raft with 28 m pile length.

## 7 DYNAMIC ANALYSIS

A common finding in many articles is that the dynamic response of a structure based on a weak soil is different from the response of the same structure if it is based on a strong soil [5, 14, 15]. Figure 5 shows a summary of the typical forcing frequencies applied to a Vestas V90-2MW wind turbine system [10].

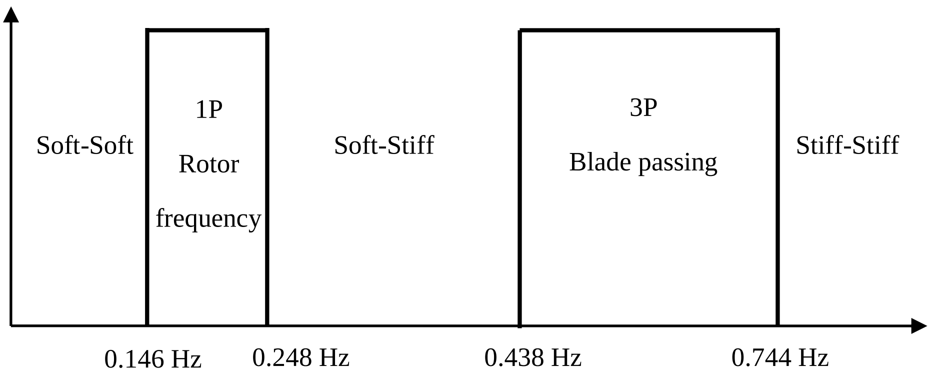


**Figure 5** Forcing frequencies plotted against power spectral density for Vestas V90-2.0 MW wind turbine.

The 1P frequency means the rotational frequency of the turbine, and the 3P frequency means the blade-passing frequency. A summary of the engineering properties of the turbine is presented in Table 4.

**Table 4** Turbine Properties.

| Property | value |
|---|---|
| Rated power | 2.0 MW |
| Cut-in wind speed | 4 m/sec. |
| Rotor diameter | 90 m |
| Tower height | 80 m |
| Lower section diameter | 4.15 m |
| Top section diameter | 3.15 m |
| Tower wall thickness | 22 mm |
| Tower mass | 156 t |
| Nacelle mass | 68 t |
| Rotor mass | 38 t |

A full 3-D finite element model of the tower-foundation-soil system is created using Abaqus [11]. Initially, the soil, foundation and tower were modelled using quadratic 3D stress elements (C3D20). Infinite elements (CIN3D12) are used for the soil boundary. Nacelle, rotor and blade masses are modelled using a mass point as shown in Figure 6. In the following analysis, a steady-state dynamics step, which can be used to analyse linear problems, has been used to calculate the first and the second natural frequency for the tower-foundation-soil system. Harmonic loads at the top of the tower in three directions are used to analyse the dynamic response of the tower-foundation-soil system.

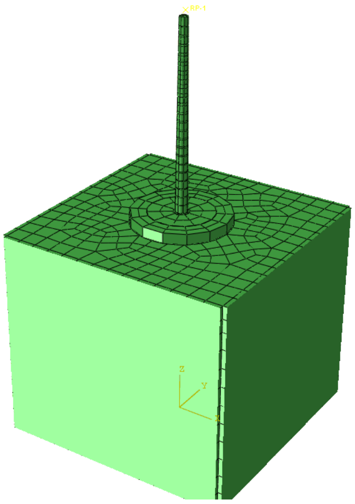

**Figure 6** Meshed 3D model using Infinite elements as a boundary condition.

Table 5 presents the frequencies of the first and second mode of vibration obtained from the dynamic analysis for a circular raft surrounded by a water tank, and a piled raft with $L_p$ = 28 m. The variation range of the frequencies is assumed to be ±10%.

**Table 5** Frequencies and periods of vibration

| Flat circular raft with a water tank | | | | | | |
|---|---|---|---|---|---|---|
| Mode of vibration | Frequency (Hz) | Frequency +10% (Hz) | Frequency -10% (Hz) | Period of vibration (s) | Rotor frequency (Hz) | Blade passing frequency (Hz) |
| 1 | 0.3 | 0.33 | 0.27 | 3.33 | 0.146-0.248 | 0.438-0.744 |
| 2 | 1.08 | 1.188 | 0.972 | 0.93 | | |
| Piled raft with $L_p$ = 28 m | | | | | | |
| 1 | 0.38 | 0.418 | 0.342 | 2.63 | 0.146-0.248 | 0.438-0.744 |
| 2 | 1.23 | 1.353 | 1.107 | 0.81 | | |

From this analysis, the structure-foundation-soil system, using the mentioned foundation systems, successfully avoids resonance during the rotor excitations. Also, using a piled raft gives larger frequency values compared to using the new foundation.

## 8 CONCLUSIONS

In this work, a new foundation solution is developed to provide a balance of cost, speed of construction, quality, and performance for windmills on weak soils. A geotechnical comparison is done between two foundation solutions using an existing soil profile in Sweden. The results show that using a circular raft surrounding by a 5 m by 5 m water tank decreases the differential settlement by 31% compared to a piled raft with 28 m pile length and a square piles of a one meter side. However, the settlement of the new foundation solution increases by 26% compared to a piled raft with 28 m pile length. Considering construction costs in Sweden, the initial foundation costs of a circular raft surrounded by a 5 m by 5 m water tank is compared to a traditional piled raft with 28 m pile length and a square piles of a one meter side. It is shown that using the new foundation system in Gothenburg region gives a significant decrease in the initial foundation costs compared to using a piled raft with long friction piles. Finally, the effect of dynamic loads on the entire system using the mentioned foundations was also investigated, and the results showed that the structure-foundation-soil system, using a circular raft surrounded by a water tank as a foundation system, successfully avoids resonance during the rotor excitations.

## REFERENCES


**[1]** Tinjum, J.M., and Christensen, R.W. (2010). "Site investigation, characterization and assessment for wind turbine design and construction." *In Wind Energy Systems, edited by John D. Sorensen and Jens N. Sorensen, 26-45. Woodhead Publishing*.

**[2]** Snyder B. and Kaiser M.J. (2009). Ecological and economic cost-benefit analysis of offshore wind energy. *Renewable Energy,* 34**,** 1567-1578.

**[3]** Engström, S., Lyrner, T., Hassanzadeh, M., Stalin, T., and Johansson, J. (2010). Tall towers for large wind turbines. Report from Vindforsk Project V-342 Höga torn för vindkraftverk.

**[4]** Manwell J.F., Macleod J., Wright S., Ditullio L. and Mcgowan J.G. (2006). Hull Wind II: A Case Study of the Development of a Second Large Wind Turbine Installation in the Town of Hull, MA. *2006 American wind energy wind association wind power conference*. Available from: http://www.ceere.org/rerl/publications/published/2006/AWEA%202006%20Hull%20II.pdf

**[5]** Enrique Luco, J. (1986) Soil-structure interaction effects on the seismic response of tall chimneys. *Soil Dynamics and Earthquake Engineering* 5(3): pp.170-177

**[6]** Olariu, C.P. (2013) Soil-structure interaction in case of a wind turbine. *Bul. Inst. Polytechnic, Iaşi, LIX (LXIII)* 1: pp.159-174.

**[7]** Svensson, H. *Design of foundations for wind turbines*. Master's dissertation for structural mechanics. 161 p. 2010 from:

< www. byggmek. lth. se/fileadmin/byggnadsmekanik/publications/tvsm5000/web5173. pdf.

**[8]** Potts D.M. and Zdravkovic L. (1999) *Finite element analysis in geotechnical engineering: theory*. Thomas Telford, London, UK, Section 7, pp.151-155.

**[9]** Olsson, M. (2010) *Calculating long-term settlement in soft clays*, Thesis for the Degree of Licentiate of Eng., Department of Civil & Environmental Eng., Chalmers University of Technology, Gothenburg, Sweden.

**[10]** Vestas Wind Systems. *Vestas V90-2 MW*. (2008) available at: http://www.vestas.com/files%2Ffiler%2Fen%2Fbrochures%2Fproductbrochurev901_8_2_0_uk.pdf.

**[11]** Hibbitt D., Karlsson B., Sorensen P. (2012) Abaqus 6.12-3 Manual.

**[12]** Abdel Glil E., El Gendy M., Ibrahim H. and Reda A. (2009) Optimization of piled raft in Port-Said. *Port-Said Engineering Research Journal*. Volume 13, No. 1, pp. 27-45.

**[13]** Sektionsfakta: kostnadsberäkningar av sektioner för byggbranschen. 13/14. (2014). Växjö: Wikells byggberäkningar. (in Swedish)

**[14]** Veletsos A.S., Meek J.W., (1974) Dynamic behaviour of building-foundation systems. Earthquake Eng Struct Dynam;3:121-38

**[15]** Moghaddasi M, Cubrinovski M, Chase J, Pampanin S, Carr A. (2011) Effects of soil-foundation-structure interaction on seismic structural response via robust Monte Carlo simulation. Eng Struct;33:1338–47.

**[16]** Larock, B. E., Jeppson R. W. and Watters G. Z. (1999) *Hydraulics of pipeline systems*, CRC press LLC.

## Appendix

### Water movement system

In this study, the water movement system depends on eight motors and fifteen pipes with electric valves. This system uses seven working motors and an extra motor is used for safety between every two compartments as shown in Figure 7. According to Larock et al. [16], the required motor horsepower can be calculated from

$$\text{Motor HP} = \rho\ g\ Q\ h\ /\ \mu \qquad \text{(A.1)}$$

where $\rho$ is the water density (Kg/m$^3$), $Q$ is the water discharge (m$^3$/s), $h$ is the total head (m), $g$ is the acceleration due to gravity (m/s$^2$) and $\mu$ is the motor efficiency. In this study, the motor efficiency $\mu$ is assumed to be equal to 0.80.

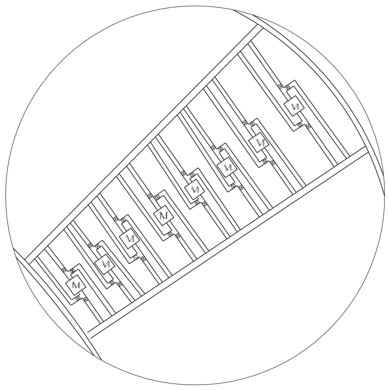

**Figure 7** Water movement system between water tank compartments.

The design of the water movement system is carried out for some constant values such as the width of the tank $B_{tank}$ is 5.0 m, and the height of the tank $H_{tank}$ is 5.0 m. The volume of water is increased using a factor of safety of 1.25 to compensate the losses that can occur from the components of the water movement system. The volume of water used in this design is 220 $\times$ 1.25 = 275 m$^3$. Between the compartments, there are two pipes to move air from the compartment which is intended to be filled to the other compartment which is intended to be emptied. The location of the water in the new foundation solution depends on the wind direction. Therefore, the control system of the water movement system uses data from the wind direction sensor and the wind speed sensor that are used in the wind turbine's yaw system. Figure 8 shows the reference line and angles which are used in the control system to move the water between compartments.

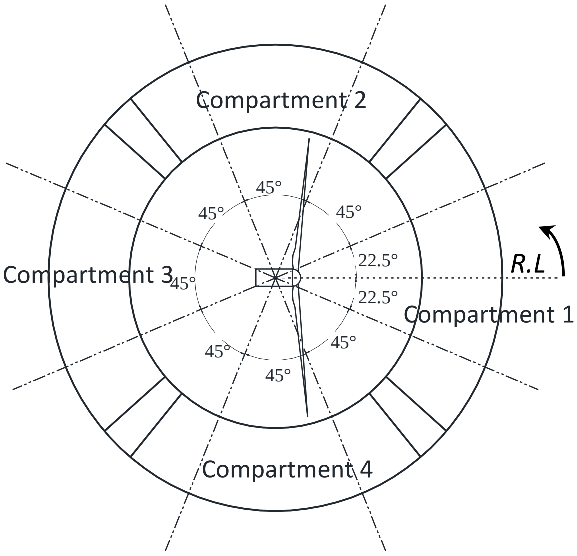


**Figure 8** Reference line and angles used to move water between compartments.

Table 6 presents the location of the water according to the rotor hub position. In this system, the water cannot be moved directly from a compartment on one side of the foundation to a compartment on the opposite side without moving through the nearest compartment first.

**Table 6** Location of the water according to rotor hub angle with the R.L.

| Rotor hub angle with the R.L. | Water position |
|---|---|
| 0.00 to 22.5 & 337.5 to 360 | Part 1 |
| 22.5 to 67.5 | Part 1 + Part 2 |
| 67.5 to 112.5 | Part 2 |
| 112.5 to 157.5 | Part 2 + Part 3 |
| 157.5 to 202.5 | Part 3 |
| 202.5 to 247.5 | Part 3 + Part 4 |
| 247.5 to 292.5 | Part 4 |
| 292.5 to 337.5 | Part 4 + Part 1 |

This system needs seven motors with a flow rate equal to 0.33 $m^3/s$ to move half of the water volume in one minute from a compartment to the nearest compartments. The diameter of the pipes between the compartments is designed to move half of the water volume in one minute using gravity.

# Paper C

## New onshore wind turbine foundation system for poor soil

Based on a paper published in the Proceeding of EWEA 2015

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**New onshore wind turbine foundation system for poor soil**

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**Abstract:** A comparative study of two foundation solutions, a flat circular raft surrounded by an active stabilisation system and a piled raft with long friction piles, has been performed using finite element simulation. The concept of an active stabilisation system surrounding a raft is a novel idea using the weight of water to stabilise against the overturning moment. A layered soil profile found in Port-Said region in Egypt is utilised in this study to investigate the geotechnical behaviour of the mentioned foundation solutions on a ground with weak properties. In terms of tilting and settlement, it is shown that using an active stabilisation system decreases the tilting of the foundation compared to using friction piles with 24 m length. However, the settlement of the new foundation solution increased compared to the settlement of a piled raft with long friction piles. Also, a cost comparison between the new foundation and a piled raft has been performed using Egyptian construction costs. It is shown that using the new foundation system gives a significant decrease in the initial foundation cost compared to using a piled raft. The effect of the dynamic loads on the tower-foundation-soil system is also investigated. The results showed that the entire windmill system successfully avoids resonance through the rotor excitations in the case of using the new foundation solution.

**Key words:** Conical raft, FE analysis, Onshore, Raft with a water tank, An active stabilisation system.

## 1. Introduction

Wind power is still in its beginning in Egypt with an installed capacity in 2014 of 550 MW, corresponding to less than 1% of the electricity consumption. The Egyptian government has specified that the Egyptian wind power share will reach 7200 MW corresponding to 12% of the electricity consumption by 2020 [1]. In order to achieve this goal, the Egyptian government must invest in all the Egyptian wind resources and build many windmills. Egypt has an excellent wind energy potential, especially in the Red Sea coast area where a capacity of 20 000 MW could be reached, as the annual average wind speed is around 10 m/s [2]. Also, the mean annual wind speed in Egypt is from 2 m/sec to 7.5 m/sec with maximum centres at Port-Said, Hurghada, Ras Benass, Elowainat, and around Nasser Lake [3].

Port-Said city in Egypt lies on the north end of the Suez Canal on the Mediterranean Sea. This area was well investigated by Goldar Associates [4], El Azab [5], El Gendy [6], and Reda [7]. These investigations showed that Port-Said area contains soft clay layers from about 12 m under the ground surface down to 50 m. Settlement and tilting problems in Port-Said have been widely observed in many buildings using flat rafts as a foundation system. Many researchers recommend that it is necessary for high-rise buildings to use a piled raft instead of a raft foundation in Port-Said city [6, 7, 19].

For onshore wind turbine foundations, as turbines reach higher rated power, the common design approaches leads to use a larger raft foundation having more concrete and reinforcing, or to use a deep foundation. Onshore wind farms installed costs in 2010 in Europe are between USD 1850 to USD 2100/kW. However, offshore wind farms are more expensive, and the installed costs in 2010 were USD 4000 to USD 4500/kW [8]. Therefore, the governments tend to invest in all the onshore wind resources before going to invest in the offshore resources.

Many countries have attractive wind spots, but these spots have poor soil conditions. It is well known that poor soils need deep foundations to fulfil the requirements of the stability analysis. Usually, the foundation cost range depends on many factors such as the soil properties, the foundation type, and the foundation dimensions. The most significant constraint on construction projects is the financial feasibility. To make sure that the construction is reasonably priced, a cost analysis of alternative designs and materials has to be performed in order to choose the best design. On poor soils, foundations make up a high percentage of the total cost. Therefore, the foundation costs can be reduced by using new cost-efficient solutions to resist the overturning moment. The main aims of this study is to decrease the cost and the tilting for wind turbine foundations on poor soils. The new foundation solution is expected to be less expensive to pass the requirements the geotechnical design.

In this study, the geotechnical performance of a flat circular raft surrounded by an active stabilisation system is investigated and compared to the geotechnical performance of the traditional foundation solution (a piled raft). The concept of an active foundation system is a novel idea using a movable load to stabilise against the overturning moment. In this study, the active stabilisation system uses water as a movable load. Also, a cost comparison between the new and the traditional foundation solutions is made. Finally, the dynamic performance of the tower-foundation-soil system, using the new and the traditional foundation solutions, is studied.

In addition to this introduction, this study is divided into seven sections. The second section presents the material properties and modelling. The third section presents the main characteristics of load sets of a 2 MW wind turbine. The fourth section gives a description of the foundation solutions that are used in this study. The fifth section describes the FE model. The sixth section shows the geotechnical comparison and a cost comparison between the mentioned foundation solutions. The seventh section presents the effect of dynamic loads on the mentioned foundations. The eighth section presents the conclusions of this study.

## 2. Material properties and modelling

The main characteristics of the soil parameters are defined in this section. The soil profile and the parameter values of the soil are shown in Figure 1. The soil parameter values are summarised from Goldar Associates report [4]. Poisson's ratio of the soil is equal to $\nu_s = 0.25$. The modulus of compressibility for the soft clay layer on Port-Said soil profile is not constant [7]. Reda (2009) [7] determined the modulus of compressibility of Port-Said lower clay. $E_s$ increases with depth and can be approximated by the following linear formula

$$E_s = E_{so}\left(1 + 0.06\,z\right) \tag{1}$$

where $E_s$ is the modulus of compressibility, $E_{so}$ = 2 MN/m$^2$ is the initial modulus of compressibility, and $z$ is the depth measured from the top surface of the clay layer, (m). The modulus of elasticity (Young's modulus) $E$ is also required as input to soil models in Abaqus [16]. The modulus of elasticity $E$ can be obtained from the following equation using Poisson's ratio $v$ [23]

$$E = E_s \frac{1-v-2v^2}{1-v} \tag{2}$$

The geotechnical material model adopted in this work is a simple elastoplastic constitutive model. The Mohr-Coulomb elastic perfectly plastic model is used [9]. The Mohr-Coulomb criterion in terms of principal stresses is

$$\sigma_1' - \sigma_3' = 2c' \cos \emptyset' + (\sigma_1' + \sigma_3') \sin \emptyset' \tag{3}$$

where $\sigma_1'$ and $\sigma_3'$ is the principal maximum and minimum effective stresses ($\sigma_1' > \sigma_3'$), ċ is the effective cohesion, and ∅' is the effective internal friction angle [10]. The undrained cohesion cu of the lower clay layer increases from 15 kN/m2 at level -12.00 to 40 kN/m2 at level -50.00. The backfilled soil is exactly at natural ground level (NGL); Characteristics of the backfilled soil are γ =18 kN/m3, and γ´=12 kN/m3).

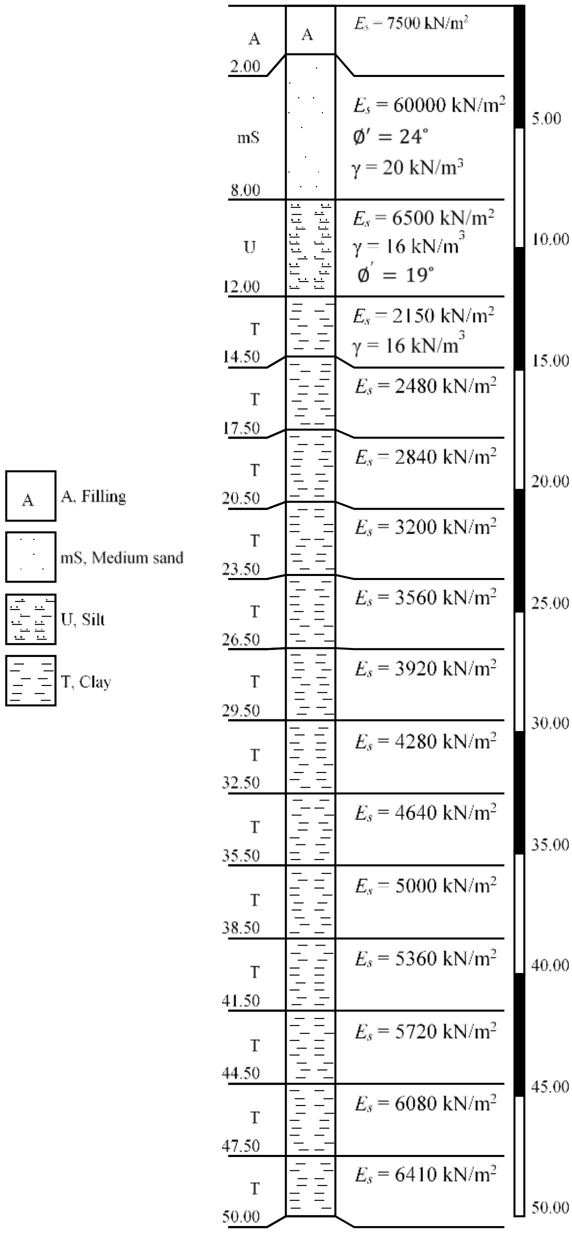


**Figure 1** Main soil profile of Port-Said, Egypt.

The concrete and the reinforcement parameter values of the foundation slab are presented in Table 1. The concrete class is 30/37 and the reinforcement class is B500B. The concrete slabs are modeled as linear elastic three-dimensional structure.

**Table1** Concrete and the reinforcement parameter values.

| **Concrete** | | |
|---|---|---|
| Young's modulus $E_b$ (GPa) | Poisson's ratio $\nu_b$ | Unit weight $\gamma_b$ (kN/m$^3$) |
| 33 | 0.2 | 25 |
| **Reinforcement** | | |
| Yield stress $f_{yk}$ (MPa) | Design Yield stress $f_s$ (MPa) | Young's modulus $E_s$ (GPa) |
| 500 | 435 | 200 |

## 3. Loads

The main characteristics of load sets are defined in this section. This study uses two different realistic sets of loads; serviceability limit state (SLS) loads, and ultimate limit state (ULS) loads for a 2 MW wind turbine with 80 m tower height [11]. Each set comprises a vertical load $N$, a horizontal load $H$, a bending moment $M$ and a twisting moment $M_z$. The turbine is three-bladed with a blade length of 44 m which gives a rotating diameter of 90 m (including the generator) [12]. The loads given here in Table 2 are a real loads for a 2 MW turbines on the west coast of Sweden [11]. Note that all the loads including the load safety factors according to the Swedish standards and acting 0.6 m above the tower base [11].

**Table 2** Main characteristics of load sets.

| Load set | Tower loads | | | |
|---|---|---|---|---|
| | $N$(kN) | $H$(kN) | $M$(kNm) | $M_z$(kNm) |
| SLS | 3510 | 482 | 35108 | 303 |
| ULS | 3510 | 797 | 63825 | 1642 |

## 4. Foundation description

Two types of foundation solutions have been analyzed as shown in Figure 2. In this study a piled raft, and the proposed flat raft surrounded by a water tank are analyzed to check the validity of using the new solution as a cost-efficient foundation system for poor soils. The idea of using a raft surrounded by a water tank concerns using the water load to give a stabilising moment to resist the overturning moment. The reason of using water is the ease of moving it with the wind turbine movement.

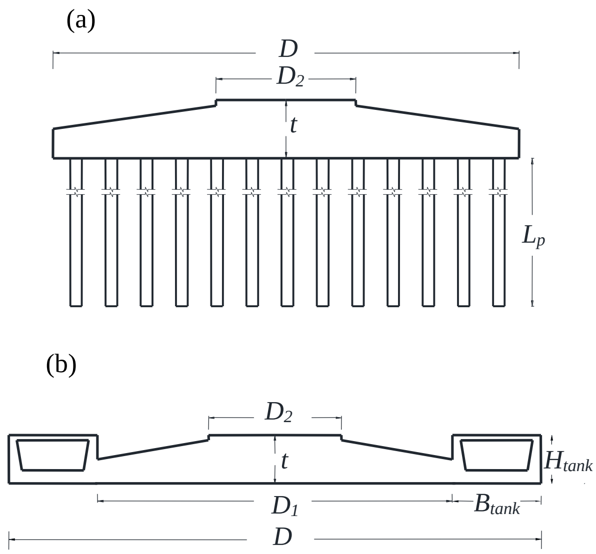


**Figure 2** Foundation types (a) a piled raft, (b) a flat circular raft with a water tank.

The water movement system depends on motors and pipes with electric valves. To show how the water movement system works, consider a case where the whole volume of water is in one compartment and all this water should be moved to the nearest appropriate compartment: Half the volume is first moved (using gravity alone) by opening the electrical valves to the nearby compartment. This takes one minute. The valves are then closed and the pumps between the compartments are started in order to move the other half of the water. Energy is saved due to the electrical valves, enabling the use of pumps for only half of the volume. It turns out that it is less expensive to use more pumps with less flow rate. For more details see the Appendix, where two pump systems are compared.

The wind turbines already have a control system. Therefore, the water movement control system will use the wind direction sensor and the wind speed sensor that are already in the yaw system in the wind turbines.

### 4.1 Geotechnical design

The geotechnical design considers many aspects such as determination of the required foundation dimensions and the required foundation weight to remove the failure probability of the foundation considering soil bearing capacity, sliding, overturning, settlement, and tilting [13]. A calculation can be done to check soil bearing capacity, sliding, and overturning aspects using simple equations. However, checking settlement and tilting aspects need special geotechnical programs which incorporate nonlinear soil models. Only the bearing capacity aspect is used to calculate the required diameter of a flat circular raft *D*. While the overturning resistance, sliding resistance, settlement and tilting are used for checking the validity of using the mentioned foundations on a layered soil containing a deep soft clay layer.

In this study, soil bearing capacity, sliding, and overturning aspects are done first by using hand calculation formulas to check the required diameter of the foundation. Then a FE program-Abaqus-is used to analyse the settlement and tilting of the foundations.The serviceability limit state (SLS) loads are used in the FE analysis to calculate the settlement and tilting. The maximum settlement of a raft foundation should be less than 15 cm on clay or 10 cm on sand according to the Egyptian code [14]. The maximum tilting of a foundation should not exceed 1mm/m according to Szerzo [15].

A manual calculation is used to calculate the required diameter of a flat raft foundation. The required diameters of the foundations are calculated by putting the maximum compressive stress under the foundation Eq.(4) equal to the allowable bearing capacity

$$\sigma = \frac{V}{b_{eff} l_{eff}} + \frac{6M_t}{b_{eff} l_{eff}^2} \tag{4}$$

where $M_t$ is the total bending moment at the foundation base level, $V$ is the sum of all the vertical loads, $b_{eff}$ is the effevtive foundation width and $l_{eff}$ is the effective foundation length. The results show that the required raft diameter for Port-Said region is 24 m to support a 2 MW windmill. Also, using this diameter fulfil the requirements of the geotechnical design to avoid the failure from overturning and sliding.

### 4.2 Constant characteristics

In the following analysis, the following characteristics are kept constant: total diameter is equal to 24 m, the total thickness of the inner cylinder $t$ = 2.50 m and the diameter of upper cylinder $D_2$ = 5 m. For the piled raft the spacing between piles $S$ is equal to 2.5 m and the pile diameter $D_p$ is equal to 0.5 m.

## 5. FE model and Verification

### 5.1 FE models

A full 3-D finite element model of the foundation-soil system was created using both Abaqus and Elpla [16, 23]. The soil, foundations and tower were modeled using quadratic 3D stress elements using a fine grid around the raft and a coarse grid for the far field. For the soil model, the computational box region chosen is 100$^{x}$100$^{x}$50 m where the depth in the $z$-direction is 50 m. The Mohr-Coulomb elastic perfectly plastic model is used. The concrete slabs are modeled as linear elastic three-dimensional structures with Young's modulus $E$ = 33 GPa, unit weight $\gamma$ = 24 kN/m$^3$ and Poisson's ratio $\nu$ = 0.2. Soil meshes with different element sizes under the foundation were used to check that the results are independent of the mesh. The results were shown to be convergent. Element size around 0.5 m were compared to element size 0.25 m without any change in the results for both FE models.

### 5.2 Abaqus FE model verification

In this section one case is used to verify the Abaqus FE model. In the first case, the present Abaqus FE model results were compared with the results of the work by Svensson [11]. In his work Svensson finds that the differential settlement of a circular raft supporting a 2 MW turbine on moraine soil is equal to 1.25 cm, the maximum settlement is 1.30 cm and the minimum settlement is 0.05 cm [11]. The present Abaqus FE model result gives tilting equal to 1.13 cm with 9.6% deviation, the maximum settlement is equal to 1.24 cm, and the minimum settlement is 0.11 cm.

### 5.3 Elpla FE model verification

In this section two cases were used to verify the Elpla FE model. In the first case, the FE model results were compared with the results in Abdel-Glil [20]. Abdel-Glil analyse an ordenary building using a piled raft as a foundation system with a raft thickness of 1.1 m, pile length of 24 m, pile spacing of 2 m and pile diameter of 0.5 m. Abdel-Glil calculates the maximum settlement and the differential settlement. Abdel-Glil's results show that the maximum settlement is equal to 12.6 cm and the differential settlement is equal to 1.89 cm [20]. The present FE model the result is the maximum settlement is equal to 13.1 cm with deviation 4%, and the differential settlement is equal to 1.95 cm with 3% deviation. In the second case, the experimental results of El-Garhy's work [21] are compared with the Elpla FE model results using the same geometry and soil properties. El-Garhy finds that the maximum settlement is 2.65 cm for using 16 piles under the footing [21]. The present FE model finds that the maximum settlement is 2.37 cm with 10.6% deviation.

## 6. Comparison between a circular raft with a water tank and a piled raft

In this section, the geotechnical behavior of the new system and a piled raft are analysed and compared. In the following analysis, the new foundation solution is analysed by using Abaqus software [16]. While, a piled raft is analysed in Elpla software [18].

### 6.1 Analysis of a flat circular raft with a water tank using Abaqus

The concept of an active system is a novel idea using a movable load to stabilise against the overturning moment. In this study, the water tank is divided into four compartments, and only one or two compartments will contain water, and all compartments are connected to an active system to move water between the compartments according to the wind direction. Figure 3 shows the geometry of the circular raft surrounded by a water tank.

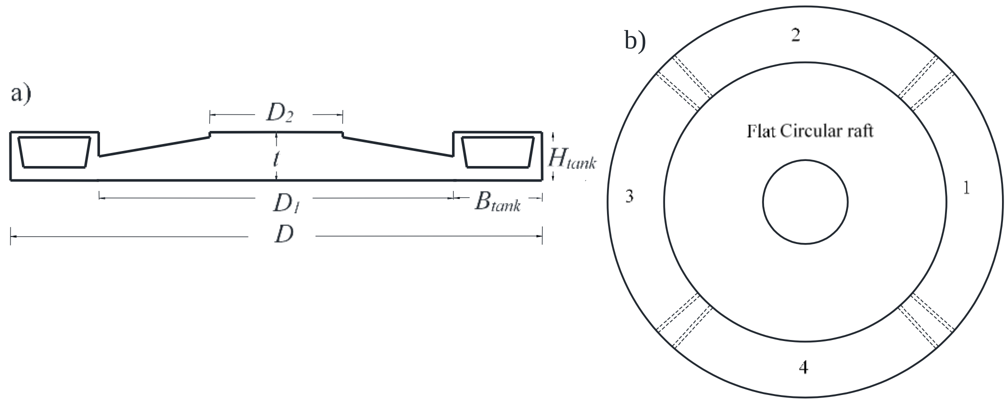


**Figure 3** Flat circular raft with a water tank a) a cross-section, and b) plan view.

In the following analysis, the total height $H_{tank}$ of the water tank is equal to 2.50 m, the thickness of the vertical wall increase from 0.25 m in the upper part to 0.40 m in the lower part. The thickness of the upper slab is 0.25 m, and the thickness of the lower slab is 0.75 m. Only one foundation variable is used in the following analysis, namely the width of the tank $B_{tank}$= 3.50, 4.00 and 4.50 m.

As mentioned, the maximum settlement and tilting for the new system are calculated using FE model in Abaqus. An existing layered soil with poor properties near Port-Said city in Egypt is used. Figure 4 shows the maximum settlement and the minimum settlement for a circular raft surrounded by a water tank with the mentioned water tank widths in the case of having the whole water volume in one compartment.

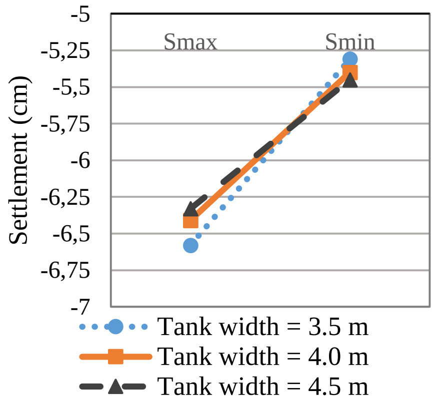


**Figure 4** Maximum settlement and minimum settlement of a flat circular raft with a water tank.

From the results, the settlement decreases with increasing the width of the tank due to the increase in the stability moment. Also, the maximum settlement in all the cases is smaller than the allowable settlement of a raft foundation on sand. Using a 2.5 m by 4.5 m water tank decreases the maximum settlement by 4% compared to using a 2.5 m by 3.5 m water tank. Concerning the tilting, Table 3 shows the tilting of the circular raft surrounded by a water tank for the same case mentioned.

**Table 3** Tilting of a flat circular raft with a water tank

| Tank width (m) | Calculated tilting (mm/m) | Allowable tilting (mm/m) |
|---|---|---|
| 3.5 | 0.76 | |
| 4.0 | 0.61 | 1 |
| 4.5 | 0.53 | |

It is shown that tilting decrease with increasing tank width due to the increased stability moment. Also, for all the cases, the tilting is lower than the allowable tilting value (1 mm/m). Using a 2.5 m by 4.5 m water tank decreases the tilting by 30% compared to using a 2.5 m by 3.5 m water tank.

### 6.2 Analysis of pile foundation using Elpla 9.2

The problem of soil containing clay layers extended to a large depth such as in Port-Said region is found in many cities around the world like London and Frankfurt. The behavior of Frankfurt clay and London clay using a piled foundation are well documented by several researchers. It was found that the piled raft foundation in such case of extended clay layers was the suitable foundation system for high-raising buildings. Depending on this philosophy, Reda [7] studied the piled raft foundation in Port-Said area for the first time, to get the optimal elements of the piled raft in the city. Reda [7] recommended to use a piled raft foundation in Port-Said city for high-raising buildings. El Kamash [19] extended the Reda's work [7], and studied the effect of 3D-space structure on a piled raft under seismic loads. Reda and El Kamash observed that the influence of a pile length in settlement reduction starts for a pile length more than 12.0 m [7,19].

This section concerns the effect of one foundation variable, namely the pile length $L_p$. The values of the pile length tested are 16 m, 20 m, and 24 m. Some constant values such as the pile spacing $S$ = 2.5 m, the raft diameter $D$ = 24 m, and the pile diameter $D_p$ = 0.5 m are used. In this study, the behavior of the piled raft in Port-Said area under 2 MW turbine loads is investigated. Two different models of a piled raft were considered in the following analysis. In model 1, piles are placed symmetrically along a single ring with $D_{ring}$ = 20 m, see Figure 6. However, in model 2, piles are placed symmetrically along two rings with $D_{ring\ 1}$ = 20 m and $D_{ring\ 2}$ = 8 m as shown in Figure 7.

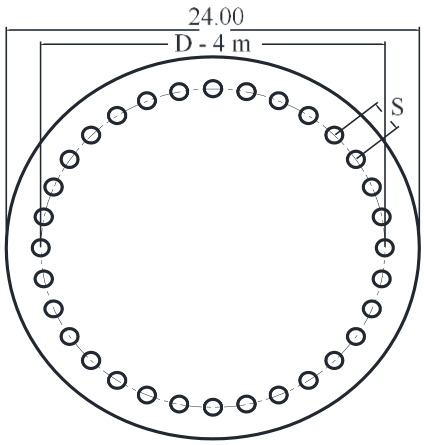


**Figure 6** The piled raft (model 1).

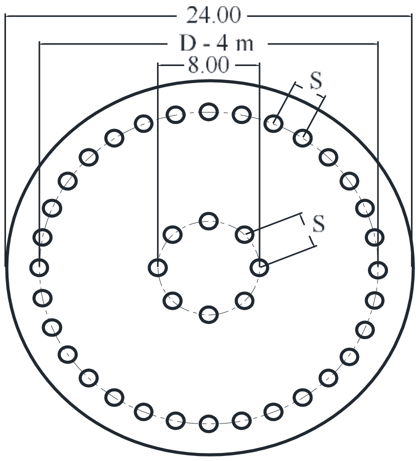


**Figure 7** The piled raft (model 2).

The maximum settlement and the minimum settlement of the model 1 and model 2 are calculated using the FE model and shown in Figure 8 and Figure 9, respectively.

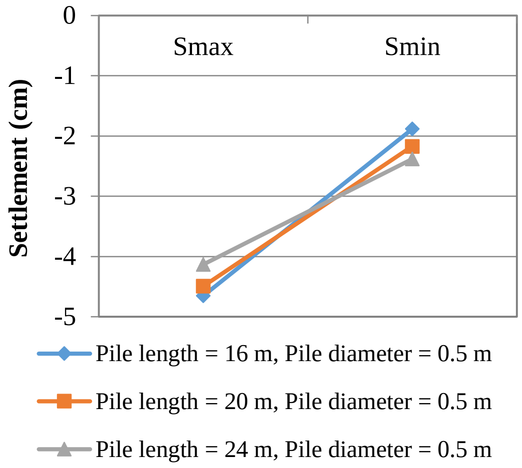


**Figure 8** Maximum settlement and minimum settlement for the piled raft (model 1).

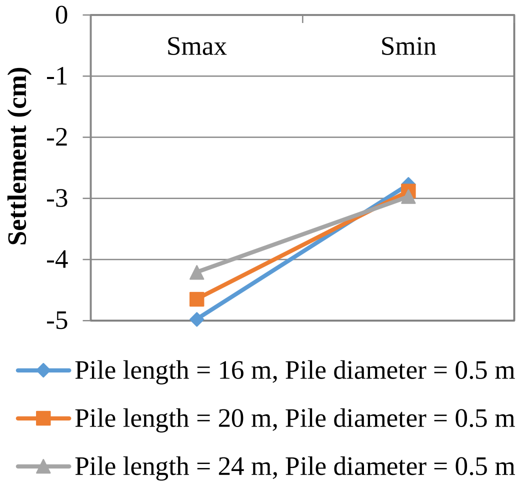


**Figure 9** Maximum settlement and minimum settlement for the piled raft (model 2).

From the results, it is shown that the suitable pile length for model 1 is more than 20 m, where every 1.0 m increase in pile length reduces the tilting about 5% for a spacing of 2.5 m. in the case of model 1 a 24 m pile length is used to get tilting close to the tilting value of the raft surrounded by a 4.0 m by 2.5 m water tank. However, a 20 m pile length is used to get tilting close to the tilting value of the raft surrounded by a 4m by 2.5 m water tank for model 2. The reduction of the tilting by using model 2 ranges from 20% to 29% compared to the model 1.

### 6.3 Comparison between the foundation systems

In this section, a geotechnical comparison and also a cost comparison are presented. Figure 10 shows the maximum settlement and the minimum settlement of a piled raft with 24 m pile length (model 1) and a flat raft with a water tank using a 4 m by 2.5 m water tank. The analysis is carried out to calculate the volume of reinforcement concrete, the steel weight, the water system cost and pilling cost for the mentioned foundations as shown in Table 4. For reinforcement in the intersection between the piles and the raft, the calculation use the minimum amount of longitudinal reinforcement that should be 1.5% of the concrete section (at least 5 bars Ø 12 mm have to provide in a symmetrical pattern) and 250 mm for lateral tie spacing according to the Egyptian code [14]. Comparison between the foundation systems will depend on estimated costs according to Associated-Consultant Engineers in Port-Said (June 2014), Ezaat Salam (Geo-constructor at Makkah office in Port-Said) and Mohamed Ibrahim (owner of Trust office in Ismailia).

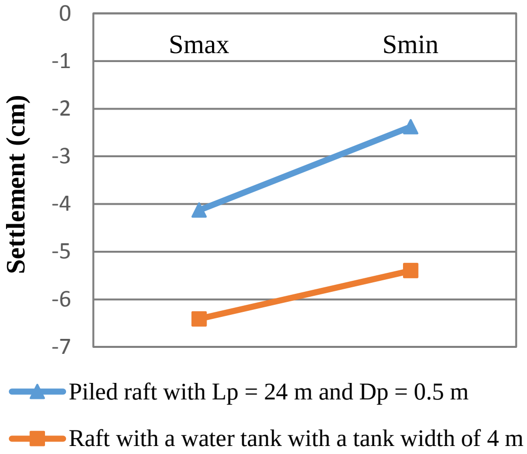


**Figure 9** The maximum settlement and the minimum settlement for the piled raft with a pile length of 24 m and a pile diameter of 0.5 m and the raft surrounding by a water tank with a tank width of 4 m.

**Table 4** Cost comparison between a raft with a water tank and a piled raft

| | Piled raft | Raft surrounded by a water tank | Cost in Egypt [USD] |
|---|---|---|---|
| Volume of concrete ($m^3$) | 771 | 426 (raft) + 313 (tank) | 180/$m^3$ |
| Steel weight (ton) | 60 | 56 | 1325/ton |
| Piling (m) | 600 | 0 | 380/m |
| Water system | 0 | 24 Motors + 32 Pipes + 128 valves | 110400 |
| Excavation ($m^3$) | 1131 | 1131 | 12.5/$m^2$ |

The total foundation cost for the piled raft is almost USD 460,000 and for the raft with a water tank is almost USD 332,000. Using the new foundation system in Port-Said city decreases the foundation cost by 28 % compared to using a piled raft with a pile length of 24 m and a pile diameter of 0.5 m. The total cost after 20 years will include the power cost and the maintenance cost. The annual maintenance cost for the pumps in this study will be 5% of its original cost. The maintenance cost includes lubrication of the pumps every one month and changing the van and the shaft every year. The total foundation cost after 20 year for the new system is less than the piled raft by almost 4%. Using a raft with a 4 m by 2.5 m water tank decreases the tilting by almost 16% compared to a piled raft with a pile length of 24 m and a pile diameter of 0.5 m (model 1). This study did not cover all the aspects that affect the foundation cost like the construction time. However, the main aim of this cost comparison is to show the main cost difference.

## 7. Dynamic analysis

Much research is currently conducted in order to determine the effect of dynamic loads on the wind turbine supporting structure systems. Moreover, using soil structure interaction in the analysis has become increasingly important. The following linear dynamic analysis for the tower-foundation-soil system is used to check the validity of using the piled raft and the raft with a water tank (for Port-Said soil). A full 3D finite element model of the tower-foundation-soil system was created using Abaqus. The soil, foundations and tower were modeled using quadratic 3D stress elements (C3D20). Infinite elements (CIN3D12) were used for the soil boundary. Nacelle, rotor and blade masses modeled using mass point. . Nacelle, rotor and blade masses are in total 106 tons. In the analysis, a steady-state dynamics step has been used to calculate the natural frequencies for the whole system by putting harmonic loads in three directions at the top of the tower. In this analysis, the tower height is 80 m with 22 mm wall thickness, the lower section diameter of 4.15 m, and the upper section diameter of 3.15 m. The rotational frequency of the turbine is from 0.146 Hz to 0.248 Hz, and the blade-passing frequency is from 0.438 Hz to 0.74 Hz. Table 5 presents the frequencies for the first and second mode of vibration obtained from the dynamic analysis for a circular raft with a water tank using 4 m tank width, and a piled raft with $L_p$ = 24 m.

**Table 5** Frequencies for a piled raft with 24 m pile length and a raft with a 4 m by 2.5 m water tank

| | Mode of vibration | Frequency (Hz) |
|---|---|---|
| Raft with a water tank | 1 | 0.295 |
| | 2 | 1.05 |
| Piled raft | 1 | 0.36 |
| | 2 | 1.16 |

The results show that the structure-foundation-soil system, using the new foundation system, successfully avoids resonance during the rotor excitations.

## 8. Conclusions

In this study, an innovative solution was developed to decrease the costs of wind turbine foundations using a circular raft surrounded by a water tank. The new foundation solution is compared to a piled raft with a various pile lengths. The maximum settlement for the new solution was increased by 37% but still in the allowable range for a raft foundation on sand. Using a flat raft with a water tank decreases the tilting by 16% compared to a pile raft in the case of piles are placed symmetrically along a single ring. Flat circular raft surrounded by a water tank is a suitable foundation system for a 2 MW wind turbine in Port-Said area. Also, this study shows that using the new foundation system in Port-Said city decreases the initial foundation cost by 28 % compared to using the piled raft in the case of piles are placed symmetrically along a single ring. This study did not consider the construction time in the cost comparison. However, deep foundations always need more construction time than shallow foundation, and more time may lead to more costs. Future studies will try to cover all aspects that affect the construction costs. Finally, the new system using the new solution successfully avoids resonance through the rotor excitations.

## References


[1] The Egyptian Minister of Electricity and Energy. The annual report 2013/2014. Available at: http://www.moee.gov.eg/english_new/EEHC_Rep/REP-EN2013-2014.pdf

[2] Mortensen NG el al. Wind atlas for Egypt: Measurements, miceo-and mesoscale modelling. Proceedings of the 2006 European wind energy conference and exhibition, Athens, Greece, February, 2006. 656.

[3] El-Asrag A, Sayed M, El-Reheem, MA, Awad A. Feasibility of clean energy from wind over Egypt. ICEHM2000, Cairo University, Egypt, 2000; 124-133.

[4] Golder Associates. Geotechnical report for Port-Said area. Government of Port-Said, Egypt. 1979

[5] El Azab M. Engineering properties of geotechnical properties of soil. Master's Thesis, Faculty of Engineering, Suez Canal University, Egypt. 1986

[6] El Gendy M. Deformation characteristics of Port-Said clay. Master's Thesis, Faculty of Engineering, Suez Canal University, Egypt. 1988

[7] Reda A. Optimization of reinforced concrete piled raft. Master's Thesis, Faculty of Engineering, Suez Canal University, Egypt. 2009

[8] The International Renewable Energy Agency. Renewable energy technologies: cost analysis series. Volume1: Power sector. 2012; Issue 5/5. Available at: https://www.irena.org/DocumentDownloads/Publications/RE_Technologies_Cost_Analysis-WIND_POWER.pdf

[9] Ottosen N. and Ristinmaa M. *The Mechanics of Constitutive Modeling*. Elsevier Ltd. 2005; Chapter 8, pp 165-173.

[10] Potts DM, Zdravkovic L. *Finite element analysis in geotechnical engineering: theory.* Thomas Telford, London, UK, 1999; Section 7, pp.151-155.

[11] Svensson H. ‘’Design of foundations for wind turbines’’. Report TVSM-5173, Master's dissertation, structural mechanics. Lund University, Sweden. 2010

[12] Vestas wind systems. Vestas V90-1.8 MW 2008 [Online]. Available at: http://www.vestas.com/files%2Ffiler%2Fen%2Fbrochures%2Fproductbrochurev901_8_2_0_uk.pdf

[13] Det Norske Veritas. Guidelines for Design of Wind Turbines, Risø National Laboratory, Copenhagen. 2002

[14] Egyptian code of soil mechanics and foundation engineering. Egypt. 2005 (in Arabic)

[15] Szerzo A. ‘’Optimization of foundation solutions for wind turbines’’. *Mathematical Modeling in Civil Engineering*, 2012; 4**,** 215-224.

[16] Hibbitt D, Karlsson B, Sorensen P. Abaqus 6.12-3 Manual. 2012.

[17] Shaligram PS. Behavior of triangular shell strip footing on georeinforced layered sand. *International Journal of Advanced Engineering Technology*, 2011; Vol.II, Issue II,192-196.

[18] El Gendy M, El Gendy A. Tutorial manual for the program Elpla. Available at: http://elpla.com/PDF/EN/Tutorial_EN.pdf

[19] El Kamash W. Analysis of 3D-Structures Resting on Piled Raft, Doctoral Thesis, Suez Canal University, Egypt. 2009
[20] Abdel Glil E, El Gendy M, Ibrahim H and Reda A. Optimization of piled raft in Port-Said. Port-Said Engineering Research Journal. 2009; Volume 13, No. 1, pp. 27- 45.
[21] El-Garhy, B. el al. Behavior of raft on settlement reducing piles: Experimental model study. Journal of Rock Mechanics and Geotechnical Engineering. 2013); Volume 5. Issue 5, 389-399.
[22] Larock, B. E., Jeppson R. W. and Watters G. Z. (1999) *Hydraulics of pipeline systems*, CRC press LLC.
[23] Kany, M., El Gendy, M., and El Gendy A. Computer Analysis and Design of Foundations. Volume I: Theory used in the formulation of ELPLA. Chapter 8.

## Appendix

### Water movement system

The design of the water movement system is carried out for some constant values such as the width of the water tank $B_{tank}$ = 4.0 m, and the depth of the water tank $H_{tank}$ = 2.5 m. The volume of water is increased using a factor of safety of 1.25 to compensate the losses that can occur from the components of the water movement system. The volume of water used in this design is 71 $\times$ 1.25 = 90 m$^3$. Between all the compartments there are two pipes to move air from the compartment which is intended to be filled to the other compartment which is intended to be emptied. Water movement is depending on rotor hub position which depends on the wind direction in Port-Said city. Figure A-1 shows the reference line *RL* and angles which are used when moving water from one compartment to the other compartments.

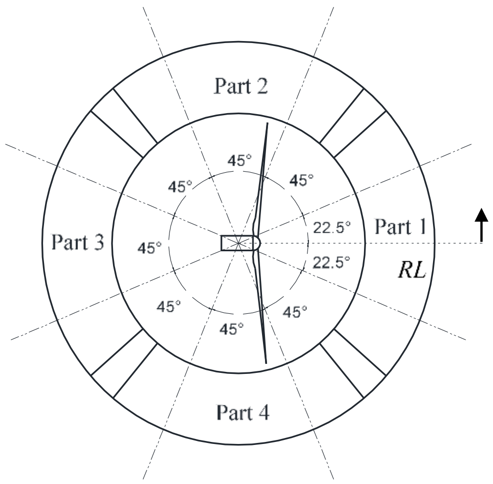


**Figure A-1** Reference line and angles used to move water from one part to the other parts.

Control systems for all the water movement systems deal with the wind direction sensor and the wind speed sensor used in the yaw system in the wind turbines. Table A-1 presents the location of the water according to the rotor hub position.

According to Larock et al. (1999) [22] the required motor horsepower is calculated from the following equation:

$$Motor\ HP = \rho\ g\ Q\ H/\mu \quad (3)$$

where $\rho$ is water density (kg/m$^3$), $Q$ is water discharge (m$^3$/s), $H$ is the total head (m), $g$ is the acceleration due to gravity (m/s$^2$) and $\mu$ is motor efficiency. The allowable water speed in any pipe systems is 3 m/s. In this study the maximum water speed in pipes is 2.5 m/s. The required diameter for the pipes are calculated from the following equation:

$Q = A\ v$ (4)

where $Q$ is water discharge (m$^3$/s), $v$ is water speed and $A$ is pipe area ($A = \pi\ D^2/4$).

**Table A-1** Location of the water according to rotor hub angle with the *RL*.

| Rotor hub angle with the reference line | Water position |
|---|---|
| 0.00 to 22.5 or 337.5 to 360 | Part 1 |
| 22.5 to 67.5 | Part 1 + Part 2 |
| 67.5 to 112.5 | Part 2 |
| 112.5 to 157.5 | Part 2 + Part 3 |
| 157.5 to 202.5 | Part 3 |
| 202.5 to 247.5 | Part 3 + Part 4 |
| 247.5 to 292.5 | Part 4 |
| 292.5 to 337.5 | Part 4 + Part 1 |

Two water systems will be designed and compared in this study to find out the effect of changing the motor flow rate on the total cost. The first system depends on six electric pumps and eight pipes with electric valves between every two parts. The system is designed with five working pumps and one extra pump will be added for safety. The pumps is designed to move the half volume of water and pipes will move the other half. When there is very low wind speed, there are eight pipes (10 inches) with electric valves between every two compartments will be opened to distribute water in all compartments. Figure A-2 presents the solution to make the pump moves water in two directions between every two compartments. This system needs five pumps with flow rate 0.15 m$^3$/s to move half of the water volume in one minute from one compartment to another compartment.

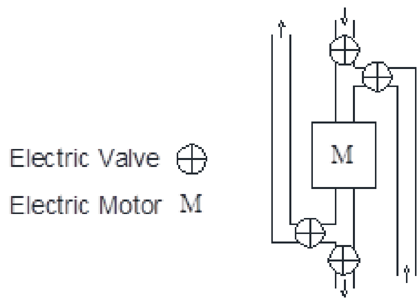


**Figure A-2** Solution to make the motor moves water in two directions between every two compartments.

The second system depends on four electric pumps between every two compartments and 12 pipes with electric valves. The system is designed with three working pumps to move the half volume of water and as mentioned before one extra motor will be added for safety. The pipes will move the other half. In case of very low wind speed, all electrical valves for all (10 inches) pipes will be opened to distribute water in all compartments. This system needs three motors with flow rate 0.25 $m^3/s$ to move half of water volume in one minute from one compartment to another compartment. Table A-2 shows the elements of the water movement systems and the total initial cost.

**Table A-2** Total cost of the water movement systems

| | System 1 | System 2 |
|---|---|---|
| No. of pumps | 24 | 16 |
| Pump flow rate($m^3/s$) | 0.15 | 0.25 |
| No. of valves | 128 | 112 |
| Pipe length (m) | 110 | 125 |
| Pipe diameter | 10 in | 10 in |
| Cost [USD] ± 5% | 110 400 | 137 250 |

The main conclusion from the water systems comparison are pump flow rate has the biggest effect on cost, and using pipes to move half of the water volume is very effective in cost issue.