ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

STUDY ON THE DESIGN METHODS OF TALL WIND TURBINE FOUNDATION

(A CASE OF ADAMA I WIND FARM)

By

Bizu Melesse Birru

An Independent Project Submitted to School of Graduate Studies

in Partial Fulfillment of the Requirement for Degree of Master of Engineering

In

Geotechnical Engineering

Advisor

Dr.-Ing Samuel Tadesse

April, 2019

Addis Ababa University

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Dr.-Ing Samuel Tadesse	________________	________________
Advisor	Signature	Date
Dr.-Ing Henok Fikre	________________	________________
Examiner	Signature	Date

DECLARATION

I, the undersigned, declare that this project is my original work performed under the supervision of my advisor Dr.-Ing Samuel Tadesse and has not been presented as a project for a degree in any other university. All sources of materials used for this project have also been duly acknowledged.

Name: Bizu Melesse Birru

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Place: Addis Ababa Institute of Technology

Addis Ababa University,

Addis Ababa.

Date: April, 2019

Abstract

Wind energy has been deemed an appropriate means of diversifying Ethiopia's energy mix while simultaneously addressing the deficit in electricity supply which the country faces. The first wind farm in Ethiopia is Adama I wind farm, which generates a 51MW of energy using 34 sets of wind turbine generators. The foundation supporting the wind turbine generators is designed assuming the load encountered to be static; however, wind turbine structures are inherently dynamic structures, and the design of foundations for resistance of dynamic loading requires a different design approach other than static approach. Both the static and dynamic foundation design approach for wind turbine structure are studied in this project.

When designing the foundation taking into account the dynamic nature of the structure, the final design resulted in the circular gravity base foundation with a bottom diameter of 13.2 m, and depth of 3m. The reduction in dimensions of one particular foundation is interpreted by the change in volume of concrete, which showed a decrement of 124 m^3 when compared with the existing typical foundation in Adama I wind farm.

Moreover, the foundation design to support all the 31 turbines is identical across 5km stretch of wind farm land, and didn't cater to the local soil or rock condition. In reality, the ground condition is different across the wind farm land; as result, the effect of using different bearing capacity values on foundation dimension is also explored.

The final design resulted in the circular gravity base foundation with a bottom diameter of 14 m, and depth of 2.5 m for all the bearing capacity used in the study. The reduction in dimensions of one particular foundation is interpreted by the volume of concrete, volume of backfill. The volume of the concrete showed 115 m^3 decrement; the backfill material decreased by 1,137 m^3 when compared with the existing foundation in Adama I wind farm. The results of this part of study show that the increasing the bearing capacity had negligible contribution in reducing the dimensions of the foundation.

This project concludes that the bearing capacity is not the major limiting factor when designing a foundation for the Adama I wind farm. The stability against overturning is the most critical criteria for both the static and dynamic approaches. Furthermore, when designing wind turbine foundation taking into consideration the soil structure interaction and dynamic nature of the wind turbine generators, the foundation dimension showed a decrement, which emphasizes the static approach to foundation design is conservative.

Acknowledgements

This study was made possible through the continuous support of many people, of whom it is important to acknowledge and thank.

Firstly, thank you to my advisor, Dr.-Ing Samuel Tadesse, who provided valuable support and guidance throughout the period of this work for which I am extremely grateful. I am also appreciative of your boundless energy and positive attitude, which made this study enjoyable and interesting.

Thank you to Dr. Im Soo Been, for your willingness to provide guidance and thought-provoking advice, which spurred this study on. Not only have you been an integral part of this study, but you have also played a hugely influential role in my development as a geotechnical engineer and as a person.

Lastly, and most importantly, thank you to my family and friends for your unconditional love and support throughout my studies, without which this task would have been insurmountable.

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1. Chapter One - Introduction

1.1. Wind Farm Background

In an era of massive energy consumption, alternative energy sources have already been established within our conscience as the last available link between this planet's sustainability and our own very existence. The access to energy is a very important matter in the modern society, but even more important is how the energy is provided. There is almost unlimited ways of how to provide energy and each method has got their own benefits and disadvantages. The method should be efficient, and in addition not affect the environment in a bad manner. Among these sources, wind turbines are becoming all the more popular, since, with the progress of technology, such constructions promise to provide more efficient and simultaneously less expensive wind power utilization.

The use of wind to produce energy is a very old tradition. As early as 3000 years ago China and Japan built wind mills, and later on in the 13th century wind power were spread to Europe. [1] Until the 18th century wind energy was mostly used to pump water, grind grains, drain lakes, etc. The first wind mill for electricity production was built in Scotland in 1887 by Professor James Blyth in the garden of his holiday cottage house. [1] He built a 10 m tall wind turbine to produce electricity to charge accumulators and light his cottage. A year later in 1888, Charles F. Brush developed a wind mill in Cleveland, Ohio. [1] It was an 18 m tower with a rotor diameter of 17 m which was capable of producing 12 kW of electricity at that time. As time passed on, people started to develop interest in wind energy because of the scarcity and the high price of oil, gas, fuel and coal.

Observing the evolution of wind power plants, from the windmills of the ancient times to today's modern wind turbines (see Figure 1), one can easily notice the significant changes which have taken place to both the design characteristics and the overall performance of these plants.

Figure 1: Evolution of wind power plants from the ancient vertical-axis wind wheel (left) to the modern horizontal-axis wind turbine (Warrenski, 2010)

1.2. Electricity Production by Wind Turbine

The wind turbine generates electricity by converting kinetic energy of the wind into mechanical energy. The simple principle behind the operation of a wind turbine is that when wind blows past the turbine, the blades rotate and capture the energy. The rotation of the blade prompts the internal shaft connected to the gear box to spin, which is ultimately connected to a generator which produces electricity. Figure (2) shows the schematic of power generation using wind turbines.

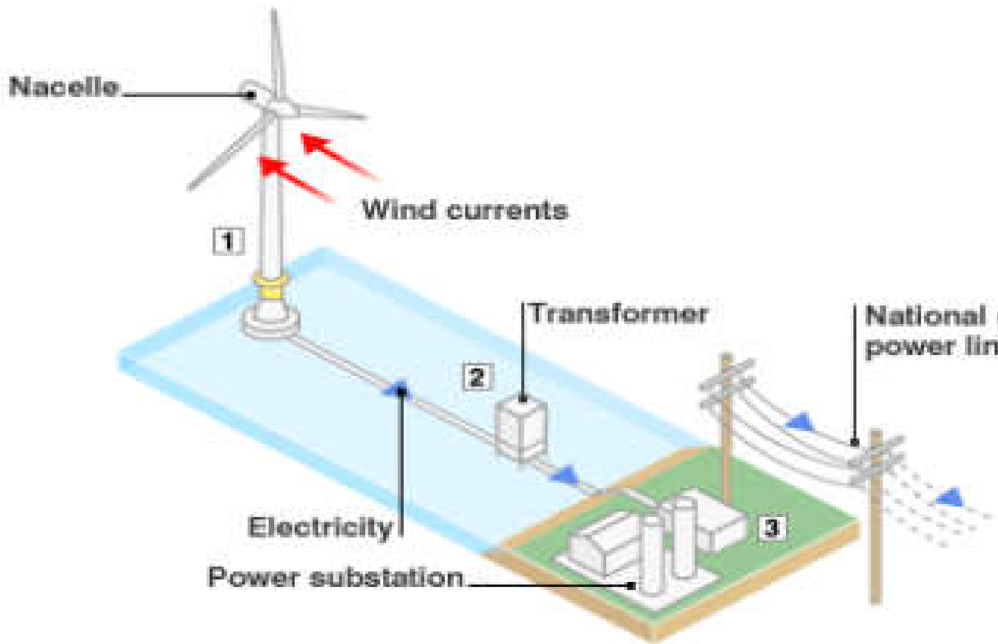

Figure 2: Process of power generation by wind turbine (BBC News, 2007)

1. Blades turn shaft inside nacelle. Generator inside nacelle uses magnetic fields to convert rotational energy into electric energy.
2. Transformer converts generated energy for distribution and sends it to substation.
3. National grid distributes power around the country.

1.3. Components of Wind Turbine

Wind turbine generally has the super structure and a sub structure. The superstructure of wind turbine generator consists of the tower, nacelle, the hub and the rotor blades. The substructure on the other hand is the foundation. A typical horizontal axis wind turbine is shown in Figure (3).

1. Foundation
2. Tower
3. Nacelle
4. Rotor blades
5. Hub
6. Transformer

Figure 3: Horizontal axis wind turbine, parts numbered (Photobucket Inc., 2009)

1. **Foundation System**

The foundation system includes the upper part of the base, which links the tower to the foundation elements transferring loads to the soil. The foundations only task is to ensure the stability for the wind turbine, and to do so over its life time. This is done by transferring and spreading the loads acting on the foundation to the ground.

2. **Tower**

The tower is part of the turbine that supports the nacelle and the rotor. It is built sufficiently high to enjoy the best wind conditions and ensure free movement of the blades. The tower usually has got the form of a hollow truncated cone, and is made of high quality steel. The wide tower base connects, the prefabricated tower, to the in situ made foundation via an interface.

3. **Nacelle**

The nacelle is located at the top of the wind turbine and houses the components generating electrical energy, as well as other components (generator, gearbox, brake, coolers, etc.).

4. Rotor Blades

The rotor is composed of a set of turbine blades and a low-speed rotor shaft. The rotor is the component that directly receives wind energy, and is connected to the high-speed shaft in the nacelle by the rotor hub.

5. Hub

The hub is the connection between the rotor blades and the rotating bar that goes into the nacelle. The steel is usually a cast iron, and must be very resistant against metal fatigue and therefore a special alloy is used, which after casting undergoes a heat treatment to get the right properties. [2]

6. Transformer

The transformer unit is not part of the wind turbine itself, but a unit necessary to transform the wind turbine output power to electric power suitable for the actual environment. [2]

1.4. Wind Turbine Generator Foundation Options

A safe and economic foundation design is an important part of all construction projects. There are several types of wind turbine generator foundation designs. The most appropriate and economical foundation type is determined based on conditions such as site-specific turbine loads and geotechnical conditions. The groundwater conditions along with the water table location below the foundation base should also be considered while determining the load carrying capacity of the top soil for the wind turbine foundation. The foundation geometry and size are then designed to minimize cost while maintaining stability through the life of the structure.

Onshore wind turbines are generally supported by a shallow foundation, deep foundation or a combined of both. A shallow foundation is used where the top-level soil is capable of adequately supporting the superstructure. In cases where the upper soil layer is weak and is not capable to carry the load, shallow foundation is impractical and uneconomical. In such situations, a deep foundation is used which transfers load to a deeper stronger soil layer or bedrock. Deep foundations are also used to reduce the excessive differential settlement.

1.4.1. Shallow Foundation

Some of the shallow foundations used for the wind turbine tower are octagonal gravity base foundations, rock anchor foundations, anchor cage foundations and mat foundations.

I. Gravity Base Foundation

This is a spread foundation that is footed some distance down in the soil, meaning that some soil has to be excavated and filling material is replaced above the foundation after it is constructed, see Figure 4. If strong and stiff soils are overlaid with top layers of soft soils such as clays it can be a good idea to excavate these layers and put the foundation on the better soil. If this is the case the weight of the filling soil above the foundation is preventing the tower from turning over and the area of the plate can be reduced. A benefit of this type of foundation is that it reduces the amount of concrete, but instead requires major excavation- and refilling work [3]

This kind of wind-turbine foundation slab is generally octagonal shaped and is similar to a circular foundation slab with the same surface area and diameter. The footings are considered to be infinitely rigid. It includes a steel pedestal section to support and anchor the tower. The foundation relies on its self-weight and soil over-burden to provide sufficient resistance to overturning, leading to them often being referred to as gravity foundations, and on the soil shear strength and compressibility characteristics for vertical resistance.

Figure 4: Octagonal gravity base foundation
(http://blog.mlive.com/watershedwatch/2008/07/construction_starts_on_thumbs.htm)

The design methods for such foundations are also universally accepted and well understood, as well as this foundation concept being applicable to a wide range of subgrade strengths, making this the simplest and most common form of foundation for wind turbines. [4]

II. Anchor Cage Foundation

The anchor cage foundation consists of a set of bolts held together by steel rings. The bolts and rings are mounted together at the site. Before assembling the steel rings and bolts, the site is prepared by excavating a hole and laying about 20 cm of concrete. [5] The anchor cage is positioned in the middle and can be assembled in the excavated area or nearby area. The reinforcement bars are also provided and finally the concrete is poured. The tower is attached to the foundation using bolts and tensioned. The shape of the foundation could be either circular of octagonal.

Figure 5: Anchor cage foundation (Miceli, 2012)

III. Rock Anchored Foundations

The use of rock bolts or micro-piles to anchor pad footings has received much attention in the field of wind turbine structures. This foundation is suitable only when strong bedrock is encountered at a shallow depth. Embedding an anchor in the bedrock, to sufficient depth, and connecting it to the pad footing improves the tensile stress capacity of the footing, as the anchor is used to mobilize the tensile stresses. This mechanism is significantly improved by post-tensioning of the anchor. [6]

Subsequently, a reduction in reinforcing steel and pad footing dimensions can be achieved, leading to cost optimization of the footing.

The load from the wind turbine is resisted by the combination of bearing pressure beneath the cap at the bearing layer and tension in the steel bars grouted into boreholes that are post-tensioned after placement. [6] The rock anchor helps to fix the foundation and prevent the uplift of the foundation. Such foundation can reduce the foundation area and minimize the use of concrete and reinforcement. The rock anchor foundation is shown in Figure (6).

Figure 6: Rock anchor foundation (Miceli, 2012)

IV. Precast Concrete Foundation

The precast foundation is similar to the gravity foundation. The advantage of such foundation is the faster construction and installation, along with the reduction in excavation volume. (Meceli, (2013)) reported that it takes only two days to complete the installation. One day to install the pieces and another day to connect the tower tensioning the bolts. The advantages of such foundation are lower costs, faster execution and precast concrete quality. A precast concrete foundation is shown in Figure (7).

Figure 7: Precast concrete foundation (Meceli, 2013)

1.5. Wind Farms in Ethiopia

Wind energy potential in Ethiopia is estimated to be enormous due to local unique landscape. The total exploitable reserve of wind energy is estimated to be 10 GW and less than 1 percent of these wind resource have been utilized thus far. [7]

Plans to develop Ethiopia's vast wind resources can be traced back to 2006 when the Ethiopian government, supported by a grant from GTZ, contracted German company Lahmeyer to conduct a feasibility study of potential wind farm sites in Ethiopia. [4] They identified several potential sites for wind farms including Ashegoda, Adama, and Messebo-Harena.

In 2008, Ethiopian Electric Power (EEP) signed an Engineering, Procurement and Construction (EPC) contract with Vergnet of France to develop a 120 MW wind farm at Ashegoda which is located a few kilometers from Mekelle. [5] The project commenced in 2009, and was inaugurated in 2011. [5]

Subsequently in 2009, HydroChina and CGC Overseas Construction Group signed an EPC contract with EEP to develop a 51 MW wind farm at Adama.[iv] Adama Wind Farm is located 95 kilometers southeast of the capital, Addis Ababa. Construction on the Adama Wind Farm commenced in June 2011, and was expected to last 12 months. However, Phase I was inaugurated in March 2012, three months ahead of schedule.

After Adama phase I inauguration in 2012, EEP signed another contract with HydroChina to add an additional 153 MW of capacity. [5] Considered Phase II, this additional capacity became available in 2015. The consultation service for the project was awarded to Addis Ababa University for the first phase, and to Mekelle University and Adama University for the second phase. Adama I Wind Farm was the first operational wind farm in Ethiopia.

As of 2015, four sites have already been identified for wind farm development: Aysha Wind Farm (300 MW), Mesebo-Herena Wind Farm (42 MW), Assela Wind Farm (100 MW), and Debre Berhan Wind Farm (100 MW). [5] The functioning wind farm in Ethiopia are illustrated in Figure (8).

Figure 8: Adama I (left), Adama II (middle), Ashagoda (right)
(http://www.vergnet.com/wpcontent/uploads/2016/01/ASHEGODA)

1.6. Adama I Wind Farm

1.6.1. Project Location

Adama I Wind Park Project site is located in the middle of Ethiopia about 95km from Addis Ababa and 3km from Nazareth with altitude 1824-1976m; the Park is located in the strip of land stretching northeast to southwest. The geographical position is shown in Figure (9).

Figure 9: Diagram of geographical position of Adama (Nazret) Wind Park
(GTZ- Feasibility Study for two Wind Parks in Ethiopia, 2006)

The wind park is located in a strip of land 400m to 600m wide and 5km long. The total installed capacity of Adama I wind farm is 51MW with 34 sets of Goldwind GW77 wind turbines with unit capacity of 1.5MW. [6] The annual average electricity generation is 157.4GWh and the full load operation hour is 3087 hour. [7]

Figure 10: View of Some completed Wind Turbines
(Adama I Wind farm Project Summary Report, 2010)

1.6.2. Layout of the Project Site

The central geographical position of the wind Farm is 39°13'48"E, 8°32'41"N. Adama (Nazret) Wind Park is mountainous hilly terrain and the area is small, and the layout area is only one mountain ridge. The installed capacity of the sector is 51MW, with 34 sets of wind turbines of 1500KW. Based on the limiting conditions of sector and in order to place the turbines along the mountain ridge in lines as many as possible, the distance between the two turbines at vertical prevailing wind direction was controlled as 3 times as the rotor diameter and five times at prevailing wind direction. [8] The layout of the project is shown in Figure (11).

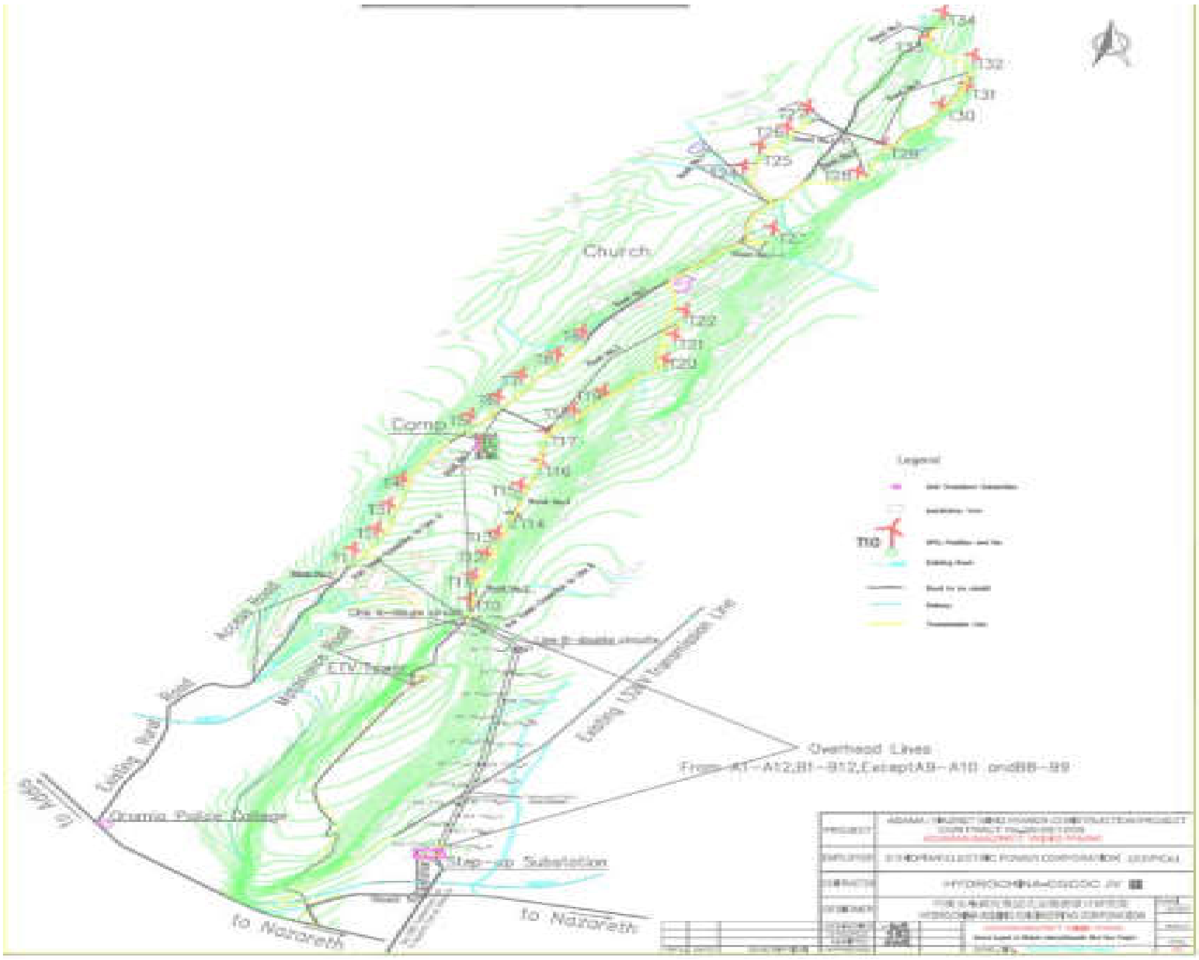

Figure 11: General layout of Adama l wind farm
(Adama I Wind farm Project Summary Report, 2010)

1.7. Purpose of this Study

The existing circular base foundation for the Adama I Warm Farm is not site specific, in a sense that, the foundation design is the same across 5km stretch of the wind farm land and didn't cater the local soil and rock condition prevalent under each wind turbine towers. In reality, the ground condition is different across the wind farm land;

as result, the effect of using different bearing capacity values on foundation dimension is explored, which is one component of this project.

Wind turbine structures are inherently dynamic structures. That is, the environmental conditions dictate the nature of loading that they must withstand, and because the loading is time-dependent the structural response is also time-dependent. The design of foundations for resistance of dynamic loading requires a different design approach other than static approach. Both the static and dynamic foundation design approach for wind turbine structure are studied in this project.

1.8. Objectives

1.8.1. General Objective

The main objective in this project is to study both the static and dynamic design approach to wind turbine foundation and also to assess the effect of using different bearing capacity vales on the dimension of wind turbine foundation for Adama I wind farm.

1.8.2. Specific Objectives

The specific aim of this study is to:

1) To study and analytically design foundation using the static approach for wind turbines.
2) To study and analytically design foundation considering the soil structure interaction and dynamic nature of wind turbine structure.
3) To study the governing factors that are applicable when designing wind turbine foundation.
4) Measure and judge the performance of foundations against criteria's such as: strength, stability, stiffness, settlement.
5) Emphasizes the importance of site-specific designed for the prevailing local site conditions in Adama I Wind Farm.

1.9. Methodology

There are a variety of different approaches that can be taken in wind turbine foundation design and accordingly there are also a number of issues that need to be taken into account. The methodological steps that is taken to address the above objectives are as follows:

Step 1: Literature Review

This section of the process deals with the review and investigation of the previous design with a similar context. The process of designing a wind turbine foundation involves the implementation of a number of requirements and guidelines to create a foundation capable of supporting the system. The general and specific suggestion of these different design codes and guidelines are reviewed.

Step 2: Data Collection

In this step, an understanding of the local soil and rock conditions are achieved through the examination of the existing geotechnical report of Adama I wind farm and the summary report. The description of the site in terms of groundwater conditions, soil and properties are obtained.

Step 3: Foundation Design

The foundations are designed using moment, horizontal and axial loads resulting from the tower and the properties of the supporting ground condition.

Step 4: Results and Conclusion

The final deliverables of this project include geotechnical design for a land based wind turbine foundation for Adama I wind farm taking into consideration the different objectives stated in **1.8.2.**

1.10. Limitations of the Study

The geotechnical investigations for Adama I wind farm project were carried out by the Addis Ababa University but the data for the hilly side of the wind farm were not accessible. The dynamic test results were also not accessible. As a result, bearing capacity values along with other design parameters that were necessary to carry out this project were assumed from the Ethiopian building code and other relevant literatures.

2. Chapter Two - Literature Review

2.1. Introduction to Wind Turbine Foundation Behavior

A substructure, or foundation, serves the purpose of providing a sufficiently safe connection between the ground and the superstructure, thereby acting as an interface between the superstructure and the ground.

Based on the complex array of loads transferred to the foundation, several different foundation concepts have been developed over the last few decades.Different foundation concepts are studied in this chapter, which is preceded by a quantitative analysis of wind turbine foundation behavior in response to the applicable loads that will be discussed. In doing so, this chapter aims to establish a base upon which foundation design principles as well as foundation behavior may be interpreted in the following chapter.

2.1.1. The Foundation Design Problem

Wind turbine towers are slender structures, with low stiffness and a rotating mass at the free-end. The nature of the structure, and hence the nature of loads transmitted to the foundation, govern the design of wind turbine foundations.

Wind turbines are subjected to loads of a cyclic nature emanating from the environment and the rotational effect of the rotor. What makes this unusual is the fluctuation of moment and horizontal loads with time, as a result of the external conditions and operation of the wind turbine.

The dynamic nature of the loading leads to the time varying response of the structure: vibrations. Vibrations are an important consideration in wind turbine design as these structures are classified as isolated columns in foundation-engineering terms. Vibrations coupled with the cyclic nature of the overturning and horizontal actions leads to the cyclic degradation of soil stiffness over time, making differential settlement a key concern.

Furthermore, the constituents of the subgrade pose important consequences on the response, that is, the natural frequencies and modes of vibration of the structure. This means the structural response of a wind turbine underlain by soft soil will differ significantly to that of a shallow soil layer underlain by bedrock. The bottom line of this is that the soil stiffness affects the response of the structure, and therefore cannot be ignored during design by assuming that the wind turbine structure has a fixed base.

2.1.2. Definition of Foundation Loading

The loading mechanisms relevant to wind turbines may be classified in terms of the soil response to such loads, as follows:

2.1.2.1. Monotonic Load

A monotonic load is a force or displacement path applied in a constant direction. When this force is applied over a sufficiently long time there is a negligible time dependent response. Hence, it is often referred to as static loading. However, although monotonic loads are constant in direction and magnitude, they may induce a transient response in the underlying strata if the rate of loading is high.

2.1.2.2. Cyclic Loading

Cyclic loads are those that involve reversals of load about a mean level and are periodic in nature. Cyclic loading may be static or transient depending on the period of loading (long or short), respectively, and nature of the soil's saturation (dry or saturated). When the number of cycles is excessively high then the soil behavior becomes essentially fatigue related generally leading to a decrease in stiffness with loading cycles.

2.1.3. Response of Foundations to Loading

The ultimate and serviceability limit state design of foundations for onshore wind turbine structures should be done within the following framework. [9]

1. Soil-foundation strength
2. Stability
3. Soil-foundation stiffness
4. Differential settlement

2.1.3.1. Strength

Foundation strength refers to the structural and geotechnical design of the foundation in terms of ensuring that the respective structural and soil elements can withstand the ultimate load cases. Thus, the soil shear strength and bearing capacity as well as the structural design of the foundation are the critical components ensuring strength. The structural design of foundations does not fall within this part of this study, but the geotechnical strength aspects are dealt with in this part of the study.

First, bearing capacity theory is presented to provide a base upon which further aspects of foundation design may be conducted, including the assessment of foundations under combined loading. This assessment included conventional means of foundation design such as the effective area method.

Generalized Bearing Capacity Theory

The Generalized Bearing Capacity Theory assumes the soil to fail plastically under shear when the ultimate bearing capacity is reached. The ultimate bearing capacities for drained conditions are calculated using the following Meyerhof's bearing capacity Equation (1).

$$q_{all} = \frac{q_{ult}}{F_S} = \frac{(CN_c\, S_c\, d_c + q\, N_q\, S_q\, d_q + ½\, B'\, \gamma N\gamma S\gamma d\gamma)}{F_S} \tag{1}$$

Where:

q_{all} = allowable bearing capacity of footing

q_{ult} = Ultimate bearing capacity of footing

F_S= Factor of safety

C = Cohesion

B = Width of footing

L= Length of footing

d= Depth of footing

q = Effective surcharge at the base level of the footing.

γ= Effective unit weight of soil below the foundation level

N_c, N_q, N_γ = Bearing capacity factors

s_c, s_q ,s_γ = Shape factors

d_c ,d_q, d_γ = Depth factors

i_c, i_q, i_γ , = Inclination factors

Undrained Failure Criterion

The bearing capacity of footings founded on clay, which are loaded rapidly is determined using a total stress analysis. This is depicted by Equation (2).

$$q_{all} = \frac{q_{ult}}{F_S} = \frac{(NcS\ s\ dc\ Cu+q)}{F_S} \quad (2)$$

Table 1: Meyerhof bearing capacity enhancement factors

Bearing Capacity Factors	$N_c = \frac{N_q - 1}{tan\phi'}$	$N_q = K_p e^{(\pi tan\phi')}$	$N_\gamma = (N_q - 1)tan\,(1.4\phi')$
Shape factors	$Sc = 1 + 0.2(B/L)$	$Sq = 1 + 0.1Kp(B/L)$	$S\gamma = Sq$
Depth factors	$dc = 1 + 0.2\,(\frac{d}{B})$	$dq = 1 + 0.1\sqrt{Kp\frac{d}{B}}$	$dq = d\gamma$

2.1.3.2. Stability

Stability is associated closely with strength, in terms of it being directly associated with the ultimate loads. Where strength refers to the ability of the foundation materials to withstand load, stability refers mainly to the geometry of the foundation. The resistance to overturning and sliding are the foremost aspects of stability. The above bearing capacity formulations are applicable to vertically loaded footings. In the wind turbine industry, where wind turbines exert vertical, horizontal and moment loading on the foundation, these formulations need to be adjusted. Therefore, methods which treat the combined loads as equivalent eccentric vertical loads have been developed.

2.1.3.2.1. The Effective Area Approach

The effective area procedure is a simplified approach to dealing with foundations subjected to combined loads. This approach operates on the principle that a vertical load acting with an eccentricity from the geometrical center of the foundation is statically equivalent to a vertical load, acting at the center, accompanied with a moment of magnitude about the center. Conversely, and more relevant to wind turbine structures, a base subjected to a moment load, and vertical load can be made statically equivalent to a base loaded with a load at eccentricity .

This is a conservative approach which has been recommended by DNV/Risø (2002) for the design of wind turbines, given the extremely large overturning moment that wind turbines are subjected to.

Firstly, the effective elliptical area is given by Equation (3), with major and minor elliptical axes defined by Equation (4) and Equation (5), respectively.

$$a' = 2\left[r^2 cos^{-1}\left(\frac{e}{r}\right) - e\sqrt{r^2 - e^2}\right] \tag{3}$$

$$l_e = 2r\sqrt{L - \left(L - \frac{b}{2r}\right)^2} \tag{4}$$

$$b_e = 2(r - e) \tag{5}$$

Subsequently, the equivalent effective area of the ellipse can be approximated by a rectangle of area A′, with sides and B′ and L′.

$$B' = \frac{L'}{l_e} b_e \tag{6}$$

$$L' = \sqrt{a' \frac{l_e}{b_e}} \tag{7}$$

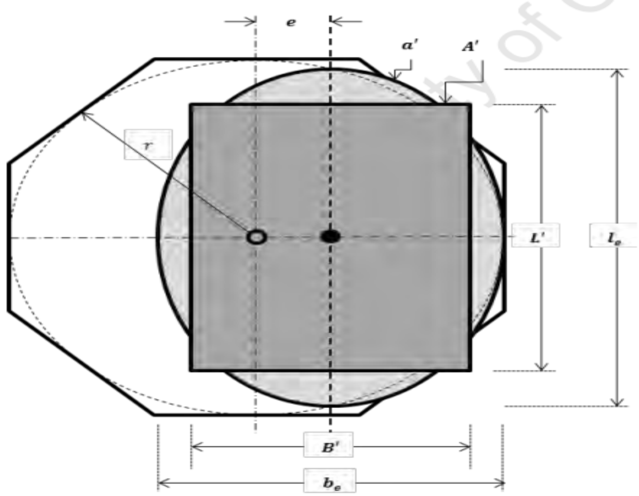

Figure 12: Circular foundation with effective elliptical and rectangular areas

2.1.3.2.2. Resistance to Overturning

The eccentricity of loading may not exceed B/6 (width) for a rectangular footing, and 0.6r (radius) of a circular footing. If these limits are exceeded the point of load application would fall outside the equivalent central third of the respective footing, which would lead to uplift on the edge away from the load.

2.1.3.2.3. Resistance to Sliding

Resistance to overturning and bearing capacity govern the geometrical and structural requirements of the foundation. Centering the design of the foundation on the above-mentioned principles generally provides a sufficient safeguard against horizontal translation due to the lateral load. However, an analysis should still be conducted to ensure that the loading, soil properties and foundation geometry satisfy the following limits for drained and undrained cases, respectively [10]:

$$H < A'c' + V tan\emptyset' \tag{8}$$

$$H < A'c_u \tag{9}$$

Note that the effective base area, is utilized in the above expressions. Additionally, the ratio of horizontal to vertical loads should not exceed 0.4.

2.1.3.3. Stiffness

Soil-foundation stiffness refers to the ability of the foundation to resist deformation under loading. One of the foremost serviceability criterion provided by wind turbine manufacturers is the minimum rotational stiffness of the foundation. The rotational stiffness of the foundation is a key factor governing soil-structure interaction and plays an important role in resisting the rocking dominated modes of vibration prevalent in shallow foundations. This criterion controls the deflection of the foundation under the overturning moment emanating from the superstructure, which at times may be extremely high in magnitude and transient in nature.

It should be noted that the magnitude of vibration satisfying serviceability usually involves relatively small dynamic displacement amplitudes. As a result, the soil-foundation system is often modelled as a linear elastic system for small amplitudes of strain.

2.1.3.4. Settlement

A shallow foundation undergoes two modes of response upon loading. The first involves the initial plastic behavior during installation and loading. This can involve significant soil compression whereby the soil and foundation establish static equilibrium. The second refers to the mechanism where, if the foundation is unloaded to a point less than the initial loading, and then reloaded, then the system responds elastically. [11]

From fundamental soil mechanics, the total settlement is comprised of three components, namely; immediate settlement, primary settlement and secondary settlement.

Immediate settlement refers to the initial elastic deformation experienced by a soil upon loading. This is due to the (often) rapid nature of the loading rate, coupled with the stress mobilization of the soil, and has the ability to occur in all soil types. Once yielding occurs the behavior becomes plastic. [11]

Primary settlement, or consolidation settlement, is a result of pore-pressure dissipation, and thus is a function of the material permeability. Thus, for saturated granular soils consolidation settlement often occurs immediately after loading (in conjunction with immediate settlement). Consolidation settlement may take months or several years to develop in saturated fine grained soils due to their inherently low permeability. Thus, this results in time-varying plastic settlements which often do not occur at the same rate across the foundation, resulting in differential settlements. [12]

Secondary settlement, also known as creep settlement, is a less well-understood topic in soil mechanics due to its complexity and interactions with primary settlement. Complexities arise in measuring creep as the differentiation between primary and secondary settlement is blurred.

2.1.3.4.1. Definition of Differential Settlement and Tilt

Differential settlement may be defined as the inconsistent deformation of a foundation-soil system, which results in differences in settlement between different parts of the structure. The consideration for differential settlement is highly applicable to sites of variable soil properties.

Wind turbine manufacturers impose stringent requirements regarding differential settlement, to ensure that a high standard of foundation quality is upheld at an international level. [13] These specifications are based on the susceptibility of the structure to over-turning and the potential damage of the superstructure due to foundation distortion.

Differential settlement considerations are associated closely with the rocking and sliding modes of vibration. The foundation-soil stiffness plays an integral role in the dynamic response of the structure, that is the relation between natural frequencies, 1P, 3P and hence amplitudes of vibration.

Isolated bases, such as shallow foundations for wind turbines are generally assessed in terms of tilt, because the rigidity of the foundation, combined with the applied loading generally produces rotation of the base. Figure (12) defines the tilt, $w*$, and differential settlement of a foundation. Furthermore, due to rotation the foundation will also undergo a translational displacement, as indicated by the deflection in Figure (12).

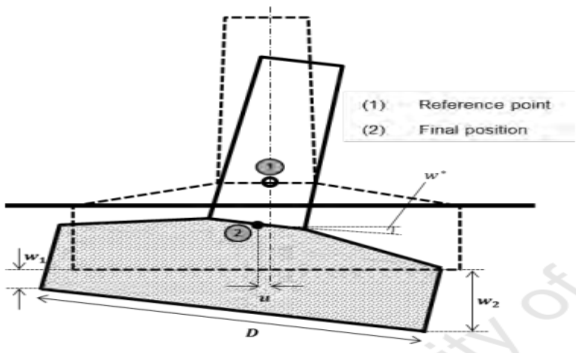

Figure 13 Differential settlement and tilting of foundation base (Bonnett, 2005)

2.1.4. Principles of Soil Structure Interaction for Elastic Foundations

Soil-structure interaction (SSI) may be defined as a general approach to foundation design, whereby the influence of the structure-foundation system and loading regime on the subgrade reaction is studied. Simultaneously, the contact pressure and its influence on the structure-foundation system, is evaluated. This is done by constructing mathematical models to encapsulate stiffness and deformation relations between the foundation and soil. In the past this method of design has been termed rational foundation design based on it representing a more realistic view of how foundation-soil systems behave under load.

2.1.4.1. Soil-structure Interaction: Governing Equation

Most shallow foundations may be modelled as beams (one-dimensional) or plates (two dimensional). The differential equations governing beam on elastic foundation mechanism is defined below. These models are based on the theory of elasticity, whereby the initial plastic deformation of the foundation-soil system is assumed to have already occurred, and therefore may be assessed as an elastic medium.

Beam on an Elastic Foundation

The theory of a beam resting on an elastic medium is applicable for the analysis of foundations undergoing one-dimensional loading and deformation. Hence, it has roots in several geotechnical engineering applications. The governing equation assumes no friction between the beam element and soil and is defined as follows:

$$EI\frac{d^4w}{dx^4} = V(x) - q(x) \tag{10}$$

Where:

EI - Flexural rigidity of the foundation

$\frac{dw}{dx}$ - Vertical deflection with respect to foundation length (x)

$V(x)$ - Vertical force applied to the footing with respect to foundation dimension (x)

$q(x)$ - Subgrade reaction with respect to foundation dimension (x).

2.1.4.2. Review of Foundation-Soil Models

The structural design of foundations hinges on the knowledge of the magnitude and nature of the contact pressure, as this directly affects the internal forces within the foundation element and the deformation behavior of the soil. Conventionally, the soil pressure has been assumed to be uniform or linear/planar and although this assumption has benefits in terms of simplicity and computing time, it results in an underestimation of foundation bending moments in cohesive soils and highly conservative bending moment values in granular soils. [14]

2.1.4.2.1. Linear or Planar Soil Pressure Distribution

As previously mentioned, the traditional method of analysis was to assume that the soil pressure beneath a rigid foundation slab varied linearly with respect to the applied loading. In doing so the soil reaction was solely a function of the external loading and foundation geometry. Thus, in essence, no soil structure interaction exists due to each system being considered in isolation, but this is included here for illustrative purposes. Equation (11) defines such a relationship for a footing under combined loads. [15]

$$q(x) = \frac{V}{A_{footing}} \pm \left(\frac{M_x}{I_x}y + \frac{M_y}{I_y}x\right) \tag{11}$$

Where:

I – Second moment of area

M - **A**pplied moment about each axis

The loading eccentricities, defined as $e_y = Mx/V$ and $ex = My/V$, may be used to evaluate the loading and geometric limits of the footing. Overturning and instability is indicated by a negative contact pressure, as a result of large eccentricity.

This method has been used extensively for more traditional relatively small foundations supporting the columns of multi-story structures based on its simplicity. However, for large flexible foundations, such as those encountered in the wind energy field, this method leads to over conservative designs. This is due to the soil stress concentrations, which lead to lower internal stresses in the foundation, being neglected. [16]

2.1.4.2.2. Modulus of Subgrade Reaction or Winkler's Model

The modulus of subgrade reaction defines a relationship between the applied loading, soil stiffness and soil deformations, and is arguably the most extensively used model for assessing SSI. [15] This is based on its adaptability to a wide array of shallow foundation problems as well as deep foundations.

Also, selecting a SSI model is a fine balance between the degree of accuracy required and the quality of input data available. Significantly more sophisticated models than the modulus of subgrade reaction are available for the prediction of soil reactions, but with increased degrees of analysis and accuracy come the complications of measuring and interpreting the additional input parameters for the respective model. Hence, the main advantage of the modulus of subgrade approach is the relative low data input required and the well accepted and common material tests which it involves.

The modulus of subgrade reaction operates on the following ratio, which stemmed from Winkler's hypothesis; the soil develops resistance to loading through discrete and independent elements, and therefore disregards shearing between elements and focuses primarily on the soil stiffness. [17] Thus, the subgrade is modelled as a series of springs with stiffness, often called the Winkler model:

$$w = \frac{q}{k} \tag{12}$$

Where:

w - Deflection

q - Contact pressure

K - Spring stiffness

2.1.5. Behavior of Foundation-soil Systems under Dynamic and Cyclic Loading

The preceding sections were concerned with soil strength and stiffness such that the failure state of the soil-foundation system could be defined and excessive deformation avoided. In these cases, the loads were considered static and induced strains within the soil mass from 10^{-3}% to several per cent at failure. [18]

Wind turbines are just one of many scenarios where dynamic loads are transferred to the soil resulting in the generation of vibrations. In the case of wind turbines, the foundation vibration response originates from variable wind conditions, rotor dynamics and out-of-balance masses acting on or within the structure, respectively. It is imperative from an economical that vibration of wind turbine foundations be assessed. There are three key reasons for this:

1. The cyclic loading of soils alters the natural frequency of the rotor-tower-foundation-soil system. Any change in natural frequency results in a change in bending moment and deflection characteristics at the top of the tower.
2. Strain and stress accumulation, emanating from vibrations, may lead to the densification of the respective soil leading to differential settlements and associated structural problems.
3. Fatigue effects result in damages such as reduced load capacity of the foundation and associated crack propagation, and thereby a loss of serviceability.

2.1.5.1. Essential Characteristics of a Dynamic Problem

The design of foundations for the resistance of dynamic loading requires a different design approach, based on the following two factors that differentiate dynamic problems from static and quasi-static, namely, the dynamic loading of soils and the strain dependency of their response.

2.1.5.1.1. Dynamic Loading of Soils

The term dynamic loading may be defined as the time-varying loading of a system, which implies that the respective loads applied to the system are a function of time. To make this definition more specific, dynamic loading may be defined in terms of two attributes (1) loading frequency and (2) the number of loading cycles. The loading frequency of wind turbine foundations is relatively low; however the number of loading cycles is disproportionately high. Problems of dynamic loading may be classified in terms of the loading frequency in three categories:

1. Impulse loads, associated with singular dynamic events.
2. Vibration or wave propagation problems. The frequency of loading typically ranges from 1- 100 Hz with the number of load cycles being between 10 and 100. [18]

3. Fatigue related behavior, where the soil experiences an extremely large number of load cycles.

Wind turbine foundation problems span all three cases defined above, where two major considerations may be made. The first regards the overall stability of the system when subjected to impulsive loads, associated with extreme wind events and possible faults. The second and most significant, regards the cyclic nature of wind turbine loads coupled with the extremely high number of loading cycles expected to be transferred to the subgrade of a wind turbine foundation.

2.1.5.1.2. Strain Dependent Behavior of Soils under Cyclic Loading

The influence of cyclic loading is fundamental to analyzing and predicting the behavior of wind turbine foundations. It has been mentioned throughout this text that wind turbines are subjected to a disproportionately high number of load cycles during their service life. Thus, fatigue considerations are of primary concern in the design of many wind turbine components.

This is analogous to accounting for the stiffness degradation of foundation-soil systems under cyclic loading. A critical level of cyclic loading may be defined where, if exceeded, stiffness degradation or strain accumulation will occur. [19]

Strain accumulation is an especially difficult process to design for and quantify. On a qualitative basis, the major consideration is the diminishing stiffness and change in natural frequency associated with strain accumulation. Thus, this is another reason for the stringent rotational stiffness measures stipulated by wind turbine manufacturers, because ensuring that the initial subgrade stiffness is sufficiently high is the primary mitigating factor for strain accumulation.

2.1.5.2. Developing a Solution to the Dynamically Loaded Foundation Problem

The preceding discussion established that the dynamic and cyclic loading of soils requires a different design approach than the traditional geotechnical approach adopted for the static loading of foundations. The classic text of soil and foundation vibration by Richart et.al. (1970) addressed this problem through four key questions which are examined below.

A. What Constitutes Failure?

Definition of failure criteria is a subjective process, which is based on the environment in which the foundation will operate. Therefore, if the wind turbine is to be situated close to human settlements then the controlling criteria will be based on limiting the transfer of vibrations to neighboring structures and ensuring that the resulting vibrations are of such a frequency that they do not disturb local inhabitants. Secondly, and more critically, the design criteria for wind turbines must ensure that the foundation is designed to withstand the resulting vibrations from a material fatigue point of view. This is in order to limit the propagation of cracks in the foundations and differential settlement of the supporting soil.

B. How is the Applied Load Related to the Deformation of the Soil-foundation System?

This is an analytical process whereby a constitutive model is selected to describe the load-deformation behavior of the foundation-soil system. Thus, addressing this question requires the cognizance of the strain-dependency of soils and the nature of the applied loadings.

C. What is the most Applicable and Appropriate Design Model?

The next step in the design process is to incorporate the applied loading and material deformation relationships into an applicable and accurate model which may be solved numerically or graphically. The lumped-parameter model is probably the most widely used model in structural and soil dynamics. It operates on the notion that the system may be simplified to three components: (1) a lumped mass, incorporating the structural loads, mass of the structural component supported and the mass of the foundation, (2) a lumped spring parameter, describing the soil stiffness and (3) a lumped damping parameter defining the damping characteristics of the system.

D. How can the Soil Parameters be Determined?

The determination of material parameters describing soil response to loading, such as $s$hear modulus G and Poisson's ratio V, must be done under anticipated conditions. Therefore, the determination of these soil parameters for dynamic loads is different to the traditional laboratory and field techniques used to determine the same parameters under static loading.

2.1.5.3. Soil Properties under Dynamic and Cyclic Loading

Wind turbine foundations are deemed to operate within two distinct domains of shear strain:

1. Low (<10^{-3}%) where the soil is considered to act as an elastic medium and

An elastic constitutive model may be adopted for this range of shear strain, whereby the shear modulus,G, and Poisson's ratio,V, are the key parameters describing the soil load-deformation behavior.

2. Intermediate (10^{-3}% - 10^{-2}%) which constitutes elasto-plastic behavior with minor stiffness degradation.

The shear modulus at very low strains has been found to be relatively constant and a maximum (G_{max}). The wave propagation techniques may be used to determine the maximum shear modulus, which may in turn may be used to determine the any stiffness degradation upon increased load cycles.

Based on the behavior of wind turbine foundations being restricted to very small and small shear strain ranges, the theory of wave propagation, and the subsequent elastic relations, is relevant and especially beneficial. A time varying force applied to a soil mass induces a seismic wave within the soil. Seismic waves are defined as the transfer of energy by means of particle motion, and are characterized with respect to the type of motion induced within the medium. [20] The induced motion occurs under constant phase and in three-dimensions. There are two broad categories of stress waves (1) body waves and (2) surface waves. Body waves propagate in all directions and surface waves propagate along the surface of the body. Body waves are more applicable to foundation design at low-levels of shear strain, whereas surface waves involve greater amounts of energy.

Body Waves (Love Waves)

The relationships governing the velocity of seismic waves propagating through elastic media are fundamental to the determination ofG_{max}. The shear wave velocity may be used to determine G_{max} , assuming an equivalent linear model. Hence, the following relationship is relevant:

$$G_{max} = \mu_{max} = \rho vs^2 \tag{13}$$

2.1.6. Analysis of Foundations under Dynamic Loads on Elastic Media

The design of foundations for dynamic loads was originally done based on increasing the mass of the foundation, or increasing the stiffness of the founding soil by means of piles. This procedure was found to work adequately, but often resulted in significant over-designs. [21] These early procedures also reduced the resonant frequency of the foundation and effective damping. [22] Both of which are undesirable consequences for wind turbine foundations. As previously shown, the natural frequency of the tower-foundation system must not intersect the relatively low 1P frequency of the turbine. Thus, the efficiency of designs for foundations under dynamic loads has been improved by the development of vibration analysis theories which occurred during the 20th century.

Solutions for the six degrees of freedom have been developed for a rigid circular surface foundation. The analysis of each degree of freedom is hardly ever carried out in practice though, because there is often insufficient information pertaining to loading regimes and material parameters to justify such an analysis, or to render the analysis accurate. Furthermore, it is often found that vertical and torsional oscillations occur as uncoupled oscillations, whereas, sliding and rocking oscillations normally occur in coupled motion. Therefore, conducting an analysis of a foundation under rocking and sliding is generally done in practice, rather than conducting the analysis for each respective degree of freedom.

2.1.6.1. Coupled Rocking and Sliding

The equilibrium equation for horizontal motion and the rotational motion are coupled can be easily formulated on the basis of D'Alembert's principle. Assembled together, they result in the matrix equation of motion that has the usual general form of Equation (14).

$$[m]\{\ddot{u}(t)\} + c\{\dot{u}(t)\} + [k]\{u(t)\} = \{P(t)\} \tag{14}$$

Where the mass matrix, damping matrix, and stiffness matarix are given by the following equations;

$$[m] = \begin{bmatrix} m & 0 \\ 0 & I \end{bmatrix};\ [c] = \begin{bmatrix} c_h & -hc_h \\ -hc_h & h^2 c_h + c_r \end{bmatrix};\ [k] = \begin{bmatrix} k_h & -hk_h \\ -hk_h & h^2 k_h + k_r \end{bmatrix} \tag{15}$$

Where:

h - Vertical offset of the centroid of the machine-foundation system from the bottom of the foundation,

h and r- indices that stand for the horizontal and rocking degrees of freedom, respectively

The dynamic stiffness for horizontal and rotational stiffness are given by the following Equations (16) and (17):

$$k_\theta = \alpha_\theta K_\theta; \quad c_\theta = \frac{\beta_\theta R_\theta}{V_s} K_\theta \tag{16}$$

$$k_h = \alpha_h K_h; \quad c_h = \frac{\beta_h R_h}{V_s} K_h \tag{17}$$

Where:

K_h and K_θ - Static horizontal and rotational stiffness respectively

R_h and R_θ - Radii of the equivalent circular foundation

V_S - Half space shear-wave velocity

α_i and β_i - Dynamic multipliers that reflect the frequency dependence of the dynamic

Veletsos and Verbic (1973) have provided relations for dynamic multipliers with small hysteresis damping and for an average Poisson's ratio of 0.4:

$$\alpha_h = 1 - 0.01a_0; \qquad \alpha_\theta = 1 - 0.215a_0 \tag{18}$$

Where:

α_h *and* α_θ - Dynamic multipliers that reflect the frequency dependence of the dynamic stiffness

$a_o = \frac{\omega R}{V_S}$ - Dimensionless frequency ratio

The dynamic multipliers β is given in Figure (14) for a representative Poisson's ratio of 0.4 as a function of a_0..

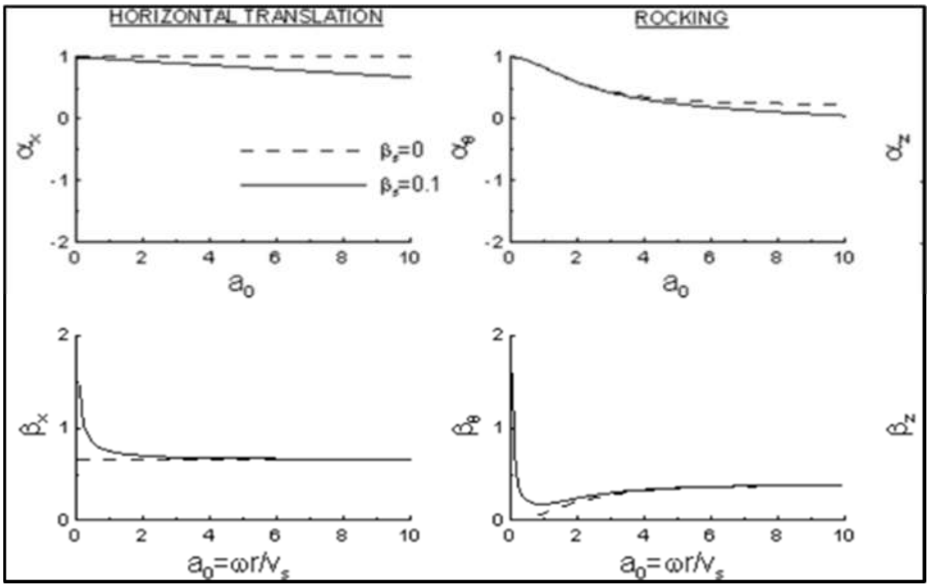

Figure 14: Dynamic multipliers (Veletsos and Verbic (1973)

The static hortanatal stiffness and rotational stiffness are calculated using Eqution (19) and Equation (20) respectively.

$$K_h = \frac{8GR_h}{2-\nu} \tag{19}$$

$$K_\theta = \frac{8GR_\theta{}^3}{3(1-\nu)} \tag{20}$$

Where: G - the shear modulus of the elastic half space

Undamped Natural Frequencies and Mode Shapes

In order to solve the forced vibration problem which is relevant to wind turbine foundation, the free-vibration problem needs to be solved first, which results in the following characteristic polynomial equation for $\boldsymbol{\lambda = \omega^2}$:

$$\lambda^2 - [\lambda_x + (1+\eta)\lambda_\theta]\lambda + \lambda_x\lambda_\theta = 0 \tag{21}$$

$$\lambda_x = \frac{k_x}{m}; \ \lambda_\theta = \frac{k_\theta}{I_y}; \quad \eta = \frac{h^2 k_x}{k_\theta} \tag{22}$$

Where η is the coupling factor expressed as a dimensionless ratio of the two stiffnesses

The two real roots of Equation (21), and hence the two natural frequencies, are found from the following Equation:

$$\lambda_{1,2} = \frac{1}{2}\left(\tilde{\lambda}_{h\theta} \mp \sqrt{\tilde{\lambda}_{h\theta}^2 - 4\lambda_{h\theta}}\right) \tag{23}$$

Where

$$\lambda_{h\theta} = \lambda_h \lambda_\theta \qquad \tilde{\lambda}_{h\theta} = \lambda_h + (1+\eta)\lambda_\theta \tag{24}$$

Response to Harmonic Loading

The solution method for the case of harmonic loads having the form $P(t) = p_o \, sin_{\omega}^{-}t$ and $M(t) = M_o \, sin_{\omega}^{-}t$ with the excitation frequency of $\bar{\omega}$.

When the force and the moment act separately, the response amplitudes are calculated using the following Equations.

- **Force only acting**

$$(u_0)_P = P_0\sqrt{\frac{(k_{22} - \bar{\omega}^2 I_r)^2 + \bar{\omega}^2 c_{22}^2}{D_r^2 + D_i^2}} \tag{25}$$

$$(\theta_0)_P = P_0\sqrt{\frac{k_{12}^2 + \bar{\omega}^2 c_{12}^2}{D_r^2 + D_i^2}} \tag{26}$$

- **Moment only acting**

$$(u_0)_M = M_0\sqrt{\frac{k_{12}^2 + \bar{\omega}^2 c_{12}^2}{D_r^2 + D_i^2}} \tag{27}$$

$$(\theta_0)_M = M_0 \sqrt{\frac{\left(k_{11} - \overline{\omega}^2 m\right)^2 + \overline{\omega}^2 c_{11}^2}{D_r^2 + D_i^2}} \tag{28}$$

The real and imaginary parts of these determinants, identified by the indices r and i, are obtained after expansion and rearranging as follows:

$$\begin{aligned} D_r &= \left(k_{11} - \overline{\omega}^2 m\right)\left(k_{22} - \overline{\omega}^2 I_r\right) - k_{12}^2 - \overline{\omega}^2\left(c_{11}c_{22} - c_{12}^2\right) \\ D_i &= \overline{\omega}\left[c_{22}\left(k_{11} - \overline{\omega}^2 m\right) + c_{11}\left(k_{22} - \overline{\omega}^2 I_r\right) - 2k_{12}c_{12}\right] \end{aligned} \tag{29}$$

Using the Prakash and Kumar simple super position, the principle horizontal amplitude and the rotational amplitude can be by the following relations.

$$\begin{aligned} U_O &= (U_O)_p + (U_O)_M \\ \theta_O &= (\theta_O)p + (\theta_O)M \end{aligned} \tag{30}$$

3. Chapter Three – Dynamic Design of Foundation

3.1. Assessment of Design Parameters and Problem Definition

3.1.1. Problem Definition

The existing design of the Adama I wind farm foundation assumed the tower to be fixed to the foundation. That is, the foundation stiffness was not taken into account, and therefore assumed not affect the response of the tower-rotor system, and vice-versa. This is not the case in reality as foundation stiffness plays a central role in the dynamic response of the system; the most significant of these being reduction in natural frequency.

In addition, the loads applied to the foundation were assumed to be static. However, wind turbine structures are inherently dynamic structures, and the design of foundations for the resistance of dynamic loading requires a different design approach. As a consequence, in this part of the study, Adama I wind farm is considered as a sample problem for design of foundations considering the dynamic nature of wind turbine structures and foundation stiffness.

3.1.2. Design Parameters

The inputs parameter for the foundation design is geotechnical parameters and design loads from the super structure.

3.1.2.1. Design Loads

The loads transferred to the foundation consists of loads from the turbine, wind, self-weight of the foundation. The vertical force acting on the foundation is mainly dead load from the tower, the nacelle and the rotor blades, but the wind may also give arise to some vertical force. The most significant loads on the foundation origins from the wind. Due to the big height of the tower, a horizontal force from the wind is giving a considerably big bending moment at the foundation. Loads from the turbine, which act at the tower top, are obtained from a structural load document provided by the turbine manufacturer.

Design loads are provided by the manufacturer of the WTG, who has standard documents with the different type of loads. The loads used for the design of the existing foundation was also used in this study. The subscript "1" refers to normal working load, and the subscript "2" refers to the extreme load. The loads are tabulated below in Table (2).

Table 2: Loads and moments provides by the manufacturer

Characteristics value load and moments (Normal working condition)			
F_{1rk}	F_{1zk}	M_{1rk}	M_{1zk}
$282\ kN$	$1{,}727kN$	$15{,}538\ kN.m$	$83\ kN.m$
Corrected value load and moments (Normal working condition), $\boldsymbol{Ko = 1.35}$			
F_{1rkx}	F_{1zkx}	M_{1rkx}	M_{1zkx}
$380.7\ kN$	$2{,}331.45\ kN$	$20{,}976.3kN.m$	$112.05\ kN.m$
Characteristics value load and moments (Extreme working condition)			
F_{2rk}	F_{2zk}	M_{2rk}	M_{2zk}
$526kN$	$1{,}693kN$	$29{,}332\ kN.m$	$140\ kN.m$
Corrected value load and moments (Extreme working condition) , $\boldsymbol{Ko = 1.35}$			
F_{2rkx}	F_{2zkx}	M_{2rkx}	M_{2zkx}
$710.1\ kN$	$2{,}285.6\ kN$	$39{,}598.2\ kN.m$	$189\ kN.m$

Where,

F_{rk} − Horizontal resultant,

F_{zk} − Vertical resultant

M_{rk} − Moment resultant

M_{zk} − Torque

K_o − Correction safety coefficient, considering of the uncertainty of wind turbine generator set's load and the load model deviation.

3.1.2.2. Geotechnical Parameters

Even though the geotechnical data for wind towers located on hilly part of wind farm land is not available, the geotechnical investigation report that was conducted for wind turbines towers located on loose/soft soil is available. The geotechnical parameter obtained from this geotechnical report is tabulated in below:

Table 3: Laboratory test results

Test pit location	Sample depth	Liquid limit	Plastic limit	Plasticity Index	Free swell	Shear strength		
						Drained		Undrained
						c'	Φ'(deg)	Cu
WTG-27	3.5 m	96%	43%	53%	140%	42 kPa	5.3	96 kPa

The laboratory test results conducted on representative samples collected from the foundation of wind turbine generator number 27 exhibit free swell of 140% with Plasticity Index and Liquid Limits values of 53% and 96%, respectively.

3.1.2.3. Design Assumptions

- **Maximum Shear Modulus**

The shear modulus at very low strains has been found to be relatively constant and a maximum (G_{max}). The wave propagation techniques may be used to determine the maximum shear modulus, which may in turn may be used to determine the any stiffness degradation upon increased load cycles. Maximum shear modules is calculated using Equation (13):

$$Gmax = 151MPa$$

Where:

Density of soil (ρ) = $1.8\ kN \frac{s^2}{m^4}$

Average shear wave velocity for stiff clay (Vs) $= 290\ m/\ s$

- **Shear Modulus Degradation**

The effect of confining pressure on modulus reduction behavior for (PI = 50) soil is illustrated in (Figure 15).

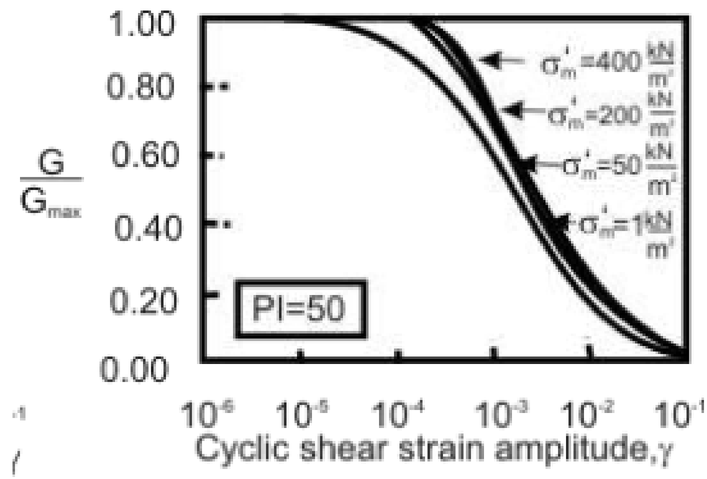

Figure 15: Influence of mean effective confining pressure on modulus reduction curves (Ishibashi, 1992)

$For\ \sigma m' = 76kPa, \frac{G}{G_{max}} = 0.65$

$\rightarrow\ G = 0.65 * Gmax = 98MPa$

- Poisson's ratio for stiff clay(υ) =0.4
- Elasticity modulus for stiff clay $(E_S) = 85MPa$

3.2. Strength and Stability

The primary objective of a foundation is to ensure a sound connection between the superstructure and soil, and in doing so ensure the stability of the structure under the respective loading. The subgrade strength dictates the type of foundation whereas the loading regime governs the proportions of the foundation to guarantee the foundation-soil system provides sufficient stability to the structure.

3.2.1. Bearing Capacity

The undrained bearing capacity is calculated using Equation (2) and the calculation for different diameters are shown below in Table (4).

Table 4: Input parameters and bearing capacity factors for undrained failure criterion

Bearing Capacity				
Input Parameters		Bearing Capacity Factors	N_c=5.14	$N_q=N_\gamma$ = 1
C_U	95.95 kPa	Shape Factors	S_C =1+0.2(B/L)=1.2	$S_q=S_\gamma$=1
$\gamma_{Backfill}$	16 kN/m^3	Depth Factors	d_c =1+0.2(D/B)	$d_q=d_\gamma$=1
γ_{bulk}	18.15 kN/m^3	Inclination Factors	i_c, i_q, i_γ , =1	
Footing Depth (D_f)	3.5 m	Effective surcharge at the base level of the footing, $\sigma o'$	63.525 kPa	

Table 5: Undrained bearing capacity for different foundation dimensions

Bearing Capacity of Proposed Foundation (Undrained)					
Diameter	**S_C**	**d_c**	**N_c**	**Q_u**	**$Q_{u,all}$**
13.2 m	1.2	1.053	5.140	686.729 kPa	343.364 kPa
13.4 m	1.2	1.052	5.140	686.261 kPa	343.130 kPa
13.6 m	1.2	1.051	5.140	685.806 kPa	342.903 kPa
13.8 m	1.2	1.051	5.140	685.364 kPa	342.682 kPa
14 m	1.2	1.050	5.140	684.936 kPa	342.468 kPa

3.2.2. The Effective Area Approach

As mentioned in this text, the conventional bearing capacity methods were not derived for combined loadings, and therefore methods which treat the combined loads as equivalent eccentric vertical loads have been developed. The calculation for effective area and dimensions for circular foundation is given in Appendix D. The corrected allowable bearing capacity calculation for foundation diameter of 13.2 m is illustrated below

Table 6: The corrected bearing capacity calculation

$S_{C,\ Corrected}$		$d_{C,\ Corrected}$		$Q_{all,\ Corrected}$	
Normal	Extreme	Normal	Extreme	Normal	Extreme
1.143	1.104	1.092	1.125	308 kPa	306 kPa

3.2.3. Resistance to Overturning

The eccentricity of loading may not exceed 0.6r for a circular footing. If these limits are exceeded the point of load application would fall outside the equivalent central third of the respective footing, which would lead to uplift on the edge away from the load. A dimeter of 13.2m is found to satisfy the overturning stability which is shown below.

Table 7: Overturning stability calculation

Overturning stability, e < 0.6r =3.96m			
Eccentricity(e)	Normal	2.127 m	**Satisfied**
	Extreme	3.809 m	**Satisfied**

3.2.4. Resistance to Sliding

An analysis is conducted to ensure that the loading, soil properties and foundation geometry satisfy the sliding stability for undrained cases. This is done by employing Equation (9).

Table 8: Calculation for sliding stability for undrained case

Sliding stability			
	H < A'Cu	H/V <0.4	
Normal	381 KN < 7,836.332kN	0.163	Satisfied
Extreme	710 KN < 5,787.128kN	0.311	Satisfied

3.3. Settlement and Stiffness

Wind turbine manufacturers impose strict specifications on the serviceability conditions of wind turbines to ensure that settlements of the structure do not affect the operation of the turbine and rotor, or lead to a loss in stability due to the slender nature of wind turbine structures.

The influence of the structure-foundation system and loading regime on the subgrade reaction is assessed. Simultaneously, the contact pressure and its influence on the structure-foundation system, is evaluated. This is done by constructing mathematical models to encapsulate stiffness and deformation relations between the foundation and soil.

3.3.1. Calculating Subgrade Reaction using Winkler's Model

The subgrade is modelled as a series of springs with stiffness using the input parameters listed in Table (9). The detail calculation is shown in Appendix D.

Table 9: Input parameters to calculate the subgrade reaction

Input prameters	
B (per meter)	1,000 mm
Diameter	13,200 mm
υ	0.4
E_{soil}	85 MPa
$E_{concrete}$	30,000 MPa
Radius	6,600 mm
P	2,331.450 kN
M	20,976.300 kN.m
q	63.483 kN/m^2

A. Using Vesic's Relation for k_s

Find $(EI)_b$: circular $(EI)_b = \frac{Er^4}{4}$ (31)

Find k_s: $k_s = \frac{0.65E_s}{B(1-\nu^2)} \sqrt[12]{\frac{E_sB^4}{(EI)_b}}$ (32)

B. Calculate λL:

$$\lambda = \sqrt[4]{\frac{K_sB}{4(EI)_b}} \quad (33)$$

C. Calculate dimensionless coefficients

The dimensionless coefficients different locations along the section of the foundation can be determined using Equation (34).

$$A(\lambda x) = e^{-\lambda x}(cos\lambda x + sin\lambda x) \qquad B(\lambda x) = e^{-\lambda x}sin\lambda x$$

$$C(\lambda x) = e^{-\lambda x}(cos\lambda x - sin\lambda x) \qquad D(\lambda x) = e^{-\lambda x}cos\lambda x \quad (34)$$

D. Calculation of the bending moment and shear force values

i. Due to concentrated force:

Moments

$$M_A^P = \frac{P}{4\lambda} C(\lambda x) \qquad M_B^P = \frac{P}{4\lambda} C(\lambda x) \tag{35}$$

Shear Forces

$$V_A^P = +\frac{P}{2} D(\lambda x) \qquad V_B^P = -\frac{P}{2} D(\lambda x) \tag{36}$$

ii. Due to Moment:

Moments

$$M_A^M = -(-\frac{M}{2} D(\lambda x)) \qquad M_B^M = -\frac{M}{2} D(\lambda x) \tag{37}$$

Shear Forces

$$V_A^M = -(-\frac{M\lambda}{2} A(\lambda x)) \qquad V_B^M = -(-\frac{M\lambda}{2} A(\lambda x)) \tag{38}$$

E. Method of Superposition

$$M_A' = \frac{1}{2}[M_A + M_B]$$

$$M_A'' = \frac{1}{2}[M_A - M_B] \tag{39}$$

$$S_A' = \frac{1}{2}[V_A - V_B]$$

$$S_A'' = \frac{1}{2}[V_A + V_B] \tag{40}$$

F. Continuing with superposition

$$E_1 = \left[2\left(1 + D(\lambda L)\right)\left(1 - D(\lambda L)\right) - \left(1 - A(\lambda L)\right)\left(1 + C(\lambda L)\right)\right]^{-1}$$

$$E_2 = \left[2\left(1 + D(\lambda L)\right)\left(1 - D(\lambda L)\right) - \left(1 + A(\lambda L)\right)\left(1 - C(\lambda L)\right)\right]^{-1} \tag{41}$$

$$P_o' = 4E_1\left[S_A'\left(1 + D(\lambda L)\right) + \lambda M_A'\left(1 - A(\lambda L)\right)\right]$$

$$P_o'' = 4E_2\left[S_A''\left(1 - D(\lambda L)\right) + \lambda M_A''\left(1 + A(\lambda L)\right)\right] \tag{42}$$

$$M_o' = \frac{-2E_1}{\lambda}\left[S_A'\left(1 + C(\lambda L)\right) + 2\lambda M_A'\left(1 - D(\lambda L)\right)\right]$$

$$M_o'' = \frac{-2E_2}{\lambda}\left[S_A''\left(1 - C(\lambda L)\right) + 2\lambda M_A''\left(1 + D(\lambda L)\right)\right] \quad (43$$

G. Calculate the end conditioning shear forces and moments

i. Shear forces:

$$V_{oA} = P' + P'' \qquad V_{oB} = P' - P''$$

ii. Bending Moments:

$$M_{oA} = M_o' + M_o'' \qquad M_{oB} = M_o' - M_o''$$

H. Deflection

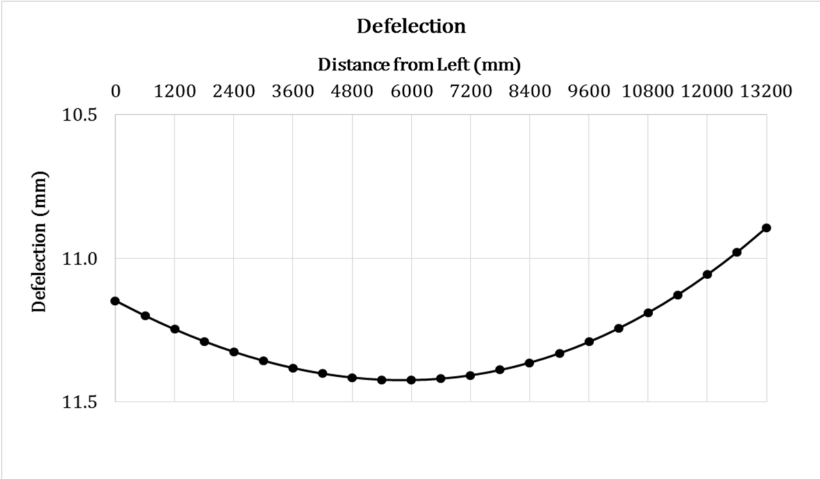

Figure 16: Deflection graph

Note: the maximum deflection is 11.424mm which is less than the specified requirement by the manufacturer (100mm).

I. Contact pressure graph

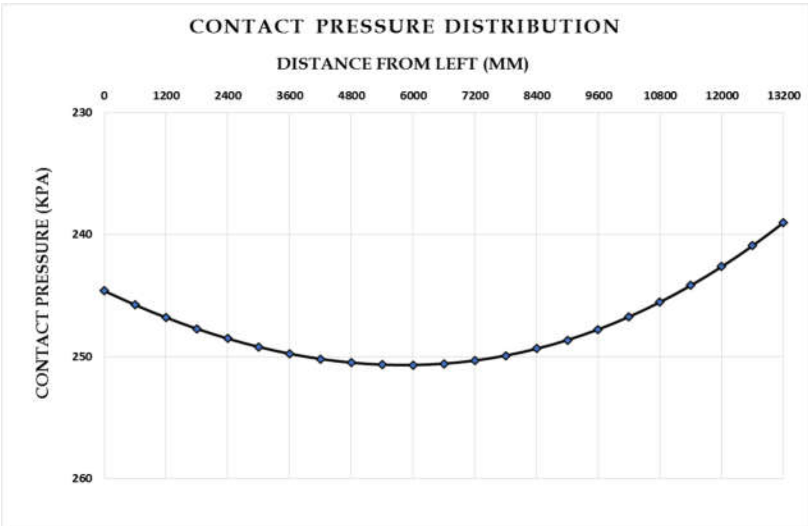

Figure 17: Contact pressure graph

Note: the maximum contact pressure is 250.687kPa which is less than the calculated bearing capacity (306kPa).

3.3.2. Differential Settlement and Tilt

Differential settlement considerations are associated closely with the rocking and sliding modes of vibration. The coupling between rotational and lateral deformation under differential settlement is generally accounted for by the specifications enforced by wind turbine manufacturers. The detail calculations are illustrated in Appendix D.

It may be summarized by the following points:

1. The most important consideration in the design of wind turbine towers is to ensure that the natural frequency of the tower avoids the blade passing frequency (3P) or a multiple thereof. The constant rotor rotational frequency (RPM or 1P frequency) must also not coincide with the natural frequency of the tower. The 1P and 3P frequencies are pre-determined by the wind turbine manufacturer using the rotor revolution per minute. The 1P and 3P frequencies are:

1P	0.283 Hz
3P	0.850 Hz

2. The stiffness of the structure is a function of the tower and equivalent minimum rotational and lateral stiffness of the foundation-soil stiffness, the value of which needs to be less than the 1P and 3P frequencies determined in order to avoid resonance. Thus, it is imperative that the natural frequencies of wind turbine towers are tuned to ensure that the relevant rotational frequencies are avoided to ensure the safety of the structure.

Natural Frequencies	
ω_1	0.342
ω_2	0.758

Note: both natural frequencies of the system are in the range between the 1P and 3P frequencies which satisfies the criteria.

3. Isolated bases, such as shallow foundations for wind turbines are generally assessed in terms of tilt. Rotation of the foundation must comply with the requirement of the manufacturer, which is $(\theta_o)_{Total} < 0.005$.

Force Only Acting		**Moment Only Acting**		**Total**		
$(\theta_o)_P$	4.21E-08	$(\theta_o)_M$	4.61E-04	$(\theta_o)_{Total}$	0.000461	**satisfied**

The detail calculation is carried out using a excel spread sheet which is presented in Appendix D

3.4. Result

A design of a land-based wind turbine considering the dynamic nature of the wind turbine structure was performed. The outcome of this study along with the already adopted foundation for Adama I wind farm is given in Table 10.

Table 10: Results and comparsion of the foundation design

Comparision of results		
	Dynamic	**Static**
Bearing Capacity	305 kPa	250 kPa
Foubdation Ring Diameter (D)	4.2 m	4.2 m
d	0.7 m	1.1 m
Bottom Slab Diameter (L)	13.2 m	15.6 m
Circular Section (h_1)	0.8 m	1 m
Oblique Section (h_2)	0.8 m	1 m
Pedstal Section below backfill (h_3)	1.2 m	0.9 m
Pedstal above backfill (h_4)	0.1 m	0.1 m
Above Pedstal (h_5)	0.4 m	0.4 m
Top Pedstal Diameter (D_1)	5.6 m	6.4 m
Total Height of Concrete(H)	3 m	3.4 m
Volume of concrete	200.04 m^3	324 m^3

The foundation bottom slab and the diameter of the pedestal are found to be 13.2 m and 5.6m respectively. In addition, the total thickness (i.e. pedestal and slab) is 3 m.

The reduction in dimensions of the foundation is interpreted by the volume of concrete which resulted in decrement of 124 m^3 volume of concrete.

The reduction in dimensions of the foundation is also quantified monetarily according to current price of materials specified by Addis Ababa city government finance & economy development bureau (2018). For a one typical foundation, the saved concrete price is estimated to be 310,000 Birr

4. Chapter Four – Static Design of a Wind Turbine Foundation

4.1. Assessment of Design Parameters and Problem Definition

4.1.1. Problem Definition

In this study, Adama I wind farm was considered as a sample problem for the design and analysis of site specific foundations. The existing circular base foundation for the Adama I Warm Farm foundation was designed considering the worst soil condition that was evident at site during construction. Furthermore, the foundation design for the weakest soil was implemented for all of the 34 wind turbine generators. However, the reality is that most of the wind turbines are located on the southwest part of the windfarm which is on the hillside, and the bearing capacity of this hillside land is unquestionably larger than what was used for the design. As a consequence, the effect of using a different values of bearing capacity on the geotechnical and structural design of foundation is assessed in this study.

4.1.2. Design Parameters

The inputs parameter for the foundation design is geotechnical parameters and design loads from the super structure.

4.1.2.1. Design Loads

Design loads are provided by the manufacturer of the WTG, who has standard documents with the different type of loads. In this particular wind farm project, the manufacture specifies wind turbines should be design for two types of working conditions namely:

- Normal working conditions which the wind turbine will be subjected to most of its design life.
- Extreme working conditions which consider factors such as extreme wind speed, extreme direction change, extreme dynamic shear.

The loads used in the previous design of foundation chapter is also used in this part of the study.

4.1.2.2. Design Code

To design structures in a correct manner there are design codes available, which specifies values for safety factors, values for material parameters, designing criterions and other rules and regulations that governs the design. If the code is followed when designing, it is ensured that the design is performed properly and does not violate rules and regulations. To design geotechnical and structural elements it is advantageous to use designing codes. There are several different codes that can be used and often each country has got its own code. In this study, the Chinese standards provided by the manufacturer are used.

- Design regulations on subgrade and foundation for wind turbine generator system - FD 003－2007
- Code for design of building foundation - GB 50007－2002
- Classification and design safety standard of wind power projects - FD 002-2007
- Code for design of concrete structures - GB 50010-2002
- Load code for the design of building structure - GB 50009-2001

4.1.2.3. Design Assumptions

As mentioned in the limitations of this study, a geotechnical data for Adama I wind farm was not accessible during the course of this study; therefore, the presumed value is taken from Ethiopian building code. The bearing capacity specified by building code for weak rock and moderately weak rock with large discontinuity was selected to reflect the highly weathered state of the rock. Design values of 500kPa, 1000kPa, 1500kPa, 2000kPa 2500kPa are used for uniaxial compressive strength of the rock.

These estimations were roughly based on the general data collected from the summary report of the project: 'the quaternary sediment in southwest of the site consists of light grey and dense sandy silt and weathered basalt rubble. The general thickness of the quaternary sediment is 1.00m. The bed rock under the quaternary sediment is basalt with porphyritic texture and block structure. On the top of the bed rock is highly weathered zone with developed weathering fissures, rock mass is fragmented and partly filled with silt. [2]

4.2. Theory

In this chapter, the design steps of the wind turbine foundation using static approach will be discussed. The design and optimization of a gravity base foundation involves many procedures and controls. The details of the geotechnical design and structural design of gravity base foundation turbine along with excel spread sheet are presented in Appendix A, B and C respectively.

The controls and designing steps can be arranged in many ways and the order is often important. The basic order is that the geotechnical design is done first controlling the stability and the bearing capacity. Then the structural design begins, determining the sectional forces and the required bending reinforcement area and the need of shear stirrups.

4.2.1. Geotechnical Design

4.2.1.1. Design of Foundation for Bearing Capacity

The foundation should be checked for axial capacity and eccentricity due to moment to ensure that it has enough bearing capacity to resist the design loads. In order to do that, all the loads that are expected to be encountered at the foundation base should be estimated. The loads from the super structure including the tower load, wind load and dynamic load are already obtained from the structural load document. The additional loads from the substructure should be calculated based on the assumed dimensions of the foundation.

I. Propose Dimension of the Foundation

The dimensions of a foundation are of course dependent on the foundation type and the loads that act on the foundation. The proposed geometry for the footing can be seen in Figure (18).

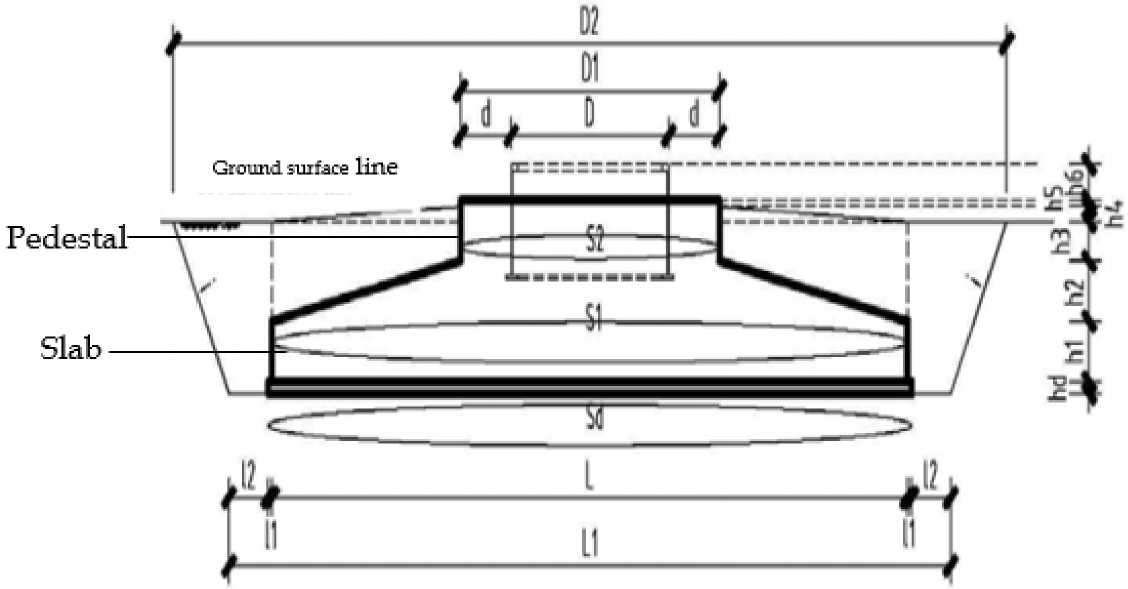

Figure 18: Geometry of the Foundation

II. Foundation Dead Load Calculation

After proposing the dimensions of the foundation, the next step is to calculate the foundation weight including possible weight from overlaying soil, the foundation depth and the foundation area. The weight of the foundation, the soil above the foundation, and the total weight are calculated by the geometric relations of Figure (18).

$$G_{foundation} = \left(\left(\pi \frac{L^2}{4}\, x\, h1 \right) + \left(\pi \frac{D1^2}{4}\, x (h3 + h4 + h5) \right) + \left(\frac{\pi \frac{L^2}{4} + \pi \frac{D1^2}{4} + \sqrt{\pi \frac{L^2}{4}\, x\, \pi \frac{D1^2}{4}}}{3} \right) \right) x\, \gamma_{gh}$$

$$G_{backfill} = \left(\left(\frac{\pi L^2}{4} - \frac{\pi D1^2}{4} \right) x\, (h3 + h4) + \left(\left(\frac{\pi L^2}{4}\, x\, h2 \right) - \left(\frac{\pi \frac{L^2}{4} + \pi \frac{D1^2}{4} + \sqrt{\pi \frac{L^2}{4}\, x\, \pi \frac{D1^2}{4}}}{3} \right) \right) \right) x\, \gamma_t$$

$$G_k = G_{foundation} + G_{backfill} + G_{ring}$$

Where:

γ_{gh} – Unit weight of the foundation

γ_t – Unit weight for the backfill material

G_{ring} – Weight of the foundation ring

$h_1, h_2, h_3, h_4, h_5, h_6, L, D_1$ – Dimensions related to those in Figure (18)

III. Foundation Bearing Pressure

The checking of the foundation bearing capacity of spread foundation is carried out to assess the effect of the total moment at the base of the foundation. The design moment (M_k) is the moment at the base of the foundation and is calculated as the sum of the bending moment and the horizontal force multiplied with the distance to the bottom of the foundation. The vertical load(N_k) is the sum of the vertical force from the tower, the foundation ring, the weight of the concrete and the soil above the foundation.

Maximum gross pressure at the bottom of the slab is calculated by Equation (44)

$$P_{kmax} = \left(\frac{N_{kx}}{A} + \frac{M_{kx}}{Z} \right) < f_{ak} \tag{44}$$

Where:

P_{kmax} – Maximum gross pressure under the standard combination of loads axial load and moment

N_{kx} – Corrected standard value of axial load applied on the top surface of foundation transmitted from the super structure.

A – base area of spread foundation

M_{kx} – Corrected standard value of moment applied at the bottom of the foundation

Z – Section modulus of the circular foundation

$f_{ak} = 1.2fa$, is the corrected value of the bearing strength of wind turbines generator set foundation.

$f_a = 500kPa$, is the characteristic value of the bearing strength of wind turbines generator set foundation.

4.2.1.2. Eccentricity Check

The foundation is subjected to a significant amount of horizontal load and moment, thus the eccentricity as result of these factors is calculated.

According to the Design Regulations on the Foundation set specified in the design document, the foundation base shall not come off under the normal working load, and this criteria is satisfied if the eccentricity is less than one six of the foundation diameter; the come-off area of foundation base shall not be larger than 25% under the extreme load condition. The eccentricity is calculated with Equation (45).

$$e = \frac{M}{N} \tag{45}$$

Where:

$M -$ Total moment at the base of the foundation

$N -$ Total Vertical load taken at the base of the foundation

4.2.1.3. Foundation Stability

Requirements for foundation stability often constitute the most decisive factors as regards determination of foundation area, foundation embedment and necessary weight for a structure with a gravity-based foundation. It is therefore essential in an optimal design process to give high emphasis to foundation stability calculations.

Stability calculation includes the sliding stability calculation and overturning calculation. The question of foundation stability is most commonly solved by limiting equilibrium methods, i.e. by ensuring equilibrium between driving and resisting forces.

I. Check Against Sliding

When the loading is not normal to the foundation base, foundations shall be checked against failure by sliding. Calculation under the normal working load condition and the extreme load working condition is carried out separately. Sliding stability is calculated by Equation (46).

$$\frac{F_R}{F_S} > 1.5 \tag{46}$$

Where:

F_R— sliding resistance force under the load effect basic combination and adopt the basic combination with the partial factor of 1.0. [23]

F_S —Sliding force under the load effect maximum combination

μ— Friction coefficient of the base bottom surface to the foundation: μ:=0.3

II. Stability Against Overturning

In the calculation of overturning stability along the foundation base, the most extreme working condition shall satisfy the Equation (47). [24]

$$\frac{M_R}{M_S} \geq 1.6 \tag{47}$$

Where:

M_R − Resisting moment under the basic load effect combination

M_S − Overturning moment under the maximum load effect combination

.

4.2.2. Structural Design

When the safety margin in the geotechnical design is at a reasonable level the structural design can begin. This is also an iterative process starting with a proposed geometry and material qualities for the foundation. Input at this stage is the weight of the foundation, the foundation area. To find a good geometry there are some guideline. [25]

- The size of the sectional forces will limit the thickness of the foundation outside the embedded ring.
- If the upper part of the foundation having a slope, limit the slope to what's possible for the construction workers to perform when they are casting the concrete.
- The concrete quality affects the amount of reinforcement, the crack width and the life length of the structure
- The reinforcement quality affects the required amount of it
- The diameter of the reinforcement affects the crack width
- Sufficient concrete thickness under- and above the flange of the embedded ring

4.2.2.1. Flexural Capacity

As the wind is acting on the tower a huge bending moment occurs in the plate. This moment must be evaluated and those parts that will be exposed to tension stress must be reinforced. This results in reinforcements bars mainly in the bottom of the plate. In order to simplify the calculation of the bending resistance of circular foundation, equivalent tetragon is utilized as shown in Figure (19).

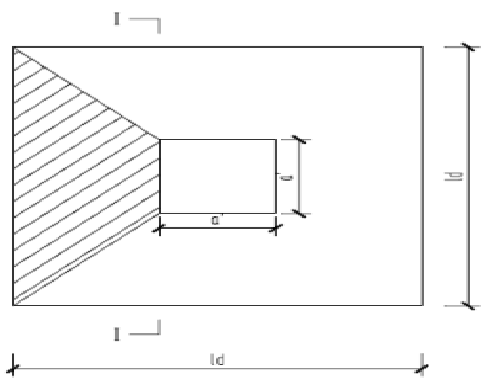

Figure 19: Equivalent tetragon of the circular foundation

Under the unidirectional eccentric load, for the square foundation, when the ratio of the step width to the height is no larger than 2.5 and the eccentricity is large than 1/6 of foundation width, the bending moment at the variable height section may be calculated by the simplified Equation (48):

$$M_I = \frac{1}{6}.a1^2.(2Id + a').\left(P_{max} - \frac{G_k}{A}\right).Al \tag{48}$$

Where:

M_I – Bending moment design value at a random section I-I under the load effect basic combination

P_{max} –Design value of maximum foundation reaction at the edge of foundation bottom under the load effect basic combination

G_k— Foundation dead weight and its covering soil dead weight considering load partial factor

a_1— The distance from a random section I-I to the foundation bottom edge with the maximum reaction force

A_w – Partial base area in bending resistance calculation

l_d – Edge length of foundation base

I. Reinforcement Calculation

A sufficient amount of reinforce needs to be provide for areas that are exposed to tension stress deduced from bending moment. The main reinforcement area of base bearing platform of wind turbine generator set is calculated by Equation (49).

$$A_{SS} = \frac{M_I}{f_y \cdot \gamma_S \cdot h_o} \tag{49}$$

Where:

A_{ss}: The main reinforcement area of base bearing platform

h_O – is the effective height of foundation (i.e. pedestal and slab)

f_y – Reinforcing steel tensile strength:

$$\gamma_S = 0.5\left(1 + \sqrt{1 - 2\alpha_S}\right) \tag{50}$$

$$\alpha_S = \frac{MI}{f_c \cdot I_d \cdot h_o^2} \tag{51}$$

4.2.2.2. Shear Resistance

Shear force is acting on the plate as a result of the underneath ground pressure. The thickness of the foundation is an essential parameter for the shear strength, and for thick structures the concrete itself can be enough to handle the shear stress. However, if the structure is not sufficiently thick reinforcements stirrups must be put in.

I. Punching Shear Resistance

The punching shear is analyzed at two different critical sections: intersection between foundation ring and pedestal, and intersection of foundation platform column edge (pedestal) and foundation slab. These sections are illustrated in Figure (20). For every section the apparent section force is compared with the resistance for the specified amount of reinforcement.

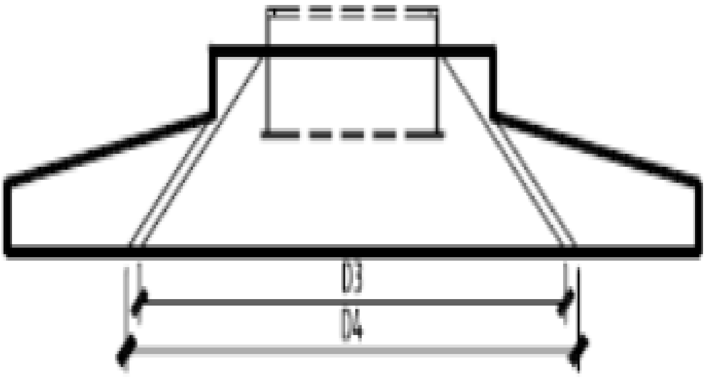

Figure 20: Area of punching shear strength

The effect of punching shear at the intersection of foundation ring and the pedestal, and also the intersection of pedestal and slab is calculated by Equation (52).

$$\gamma_O . F_l \leq 0.7 . \beta_{hp} . f_t . S_m . h_{cq} \quad (52)$$

Where:

F_l—Design value of the foundation net reaction applied on A1 under the load effect basic combination;

$\gamma_o - 1.0$

$f_t -$ design value of concrete tensile strength (kPa);

S_t – upper edge perimeter of oblique section of the cone damaged by punching shear strength

S_b – bottom edge perimeter of oblique section of the cone damaged by the punching shear strength (m);

A_l—Partial foundation base area applied in the punching check calculation.

P_{max} – Design value of maximum reaction of the foundation base edge under the load effect basic combination;

h_{cq} – the height of foundation that is affected by the punching shear.

β_{hp} – height influence coefficient of the section applied punching shear strength:

When $h_{cq} < 800mm$, adopt 1.0;

When $h_{cq} \geq 2000mm$, adopt 0.9, and the value between them shall be achieved with the linear interpolation method.

II. Oblique section shear capacity

The thickness of the foundation is not the same throughout the whole section; it decreases as the foundation goes from the slab to the pedestal that means the amount of concrete also decreases affecting the shear capacity of foundation. As a consequence the oblique section of the foundation is checked for adequacy using Equation (53).

$$\gamma_o . V_j \leq 0.7 . \beta_{hp} . f_t . S_m . h_{cq} \tag{53}$$

Where:

V_j – Design value of maximum shear on the oblique section of structural member under the load effect basic combination;

h_{cq} is the effective height of foundation (i.e. the pedestal and slab)

β_{hp} – Height influence coefficient of sheared section calculated by:

$When\ h_{oo} < 800mm, h_{oo} = 800mm;$

$When\ h_{oo} > 2000mm, h_{oo} = 2000mm.$

4.2.2.3. Crack Width

The final step in this design process is the crack width calculation which often, for structures with large reinforcement diameter and a thick concrete cover, results in too large crack widths. A concrete exposed to bending, shear, twisting or tension has cracks. These cracks must not be too big to maintain the structure's function over its lifetime. The acceptable crack width is dependent on the environmental exposure of the structure, the concrete quality the diameter of the reinforcement. The maximum allowed crack width is specified with respect to the exposure class and the design life and is specified to be 0.3mm. Equation (54) is used to calculate the crack width.

$$\omega_{max} = \alpha_{cr} \cdot \psi 1 . \frac{\sigma sk}{Es}\left(1.9 . C_c + 0.08 \frac{deq}{\rho_{te}}\right) < 0.3mm \tag{54}$$

Where:

α_{cr} — Force characteristic coefficient of structural member, reinforced concrete bearing bending and eccentricity pressure adopts the value of 2.1;

ψ_1 — Uneven coefficient of longitudinal tension bar strain among cracks:

When $\psi_1 < 0.2$, $\psi_1=0.2$;

When $\psi_1>1$, $\psi_1=1$;

σ_{sk} — Longitudinal tension bar strain of reinforced concrete structural member calculated according to the standard combination of load effect for the structural member bearing the bending force.

E_s — Elastic modulus of reinforced steel

Cc — The distance (mm) from the outer edge of longitudinal tension bar of outermost layer to the bottom edge of tensile region:

When $Cc < 20$, Cc =20 mm;

When $Cc > 65$, Cc =65 mm;

A_{te} — Effective section area of tensile concrete

A_s — Section area of vertical reinforced steel in tensile region

ρ_{te} — Reinforcement ratio of longitudinal tension bar calculated according to the effective area of tension concrete

$$If \quad \frac{As}{Ate} < 0.01\,, \rho te < \ 0.01, \rho_{te} = 0.01;$$

$$\frac{As}{Ate}\ otherwise$$

d_{eq} — Equivalent diameter (mm) of vertical reinforced steel in tensile region;

d_i — Nominal diameter (mm) of vertical reinforced steel in tensile region;

4.3. Geotechnical Design for Different Bearing Capacity Values

As mentioned before, the design of a wind turbine foundation is a very iterative process. The calculation for the selected values of bearing capacity is illustrated below:

Table 11: Geotechnical Design for Different Bearing Capacity Values

Bearing capacity	Proposed dimensions			Bearing pressure, $P_{kmax}<f_{ak}$		Eccentricty		Overturning stability
Corrected, f_{ak}	Slab diameter, L	Pedstal diameter, D	Height, H	Normal	Extreme	Normal < L/6	Extreme < 25%	Extreme < 1.6
600 kPa	9.3 m	5.6 m	1.7 m	345.959kPa	588.191kPa	4.413 m	42.25%	0.486
1,200 kPa	7.3 m	5.6 m	1.7 m	662.69kPa	1,163.845kPa	5.353 m	44.332%	0.304
1,800 kPa	6.1 m	5.6 m	1.7 m	1,094.262kPa	175.489kPa	5.972 m	45.751%	0.223
2,400 kPa	5.7 m	5.6 m	1.7 m	1,326.396kPa	2,379.629kPa	6.182 m	46.165%	0.200
3,000 kPa	5.3 m	5.6 m	1.7 m	1,632.721kPa	2,934.08kPa	6.392 m	46.553%	0.178

As shown in the above calculation, the proposed foundation dimensions for each of the selected bearing capacity satisfy the criteria of the bearing pressure but did not satisfy the eccentricity and stability requirement. Therefore, new dimension to satisfy primarily the overturning stability is chosen. It is clearly shown how eccentricity and moment are the first criteria that should be satisfied before moving further into the design. As a result, a new foundation dimensions are proposed to satisfy overturning moment criteria.

- **New proposed dimension**

  Slab diameter, L=14m

  Pedestal diameter, D_1=5.6m

  Total height of the foundation, H=2.5m

I. Eccentricity Check

Normal working condition

$N_1 = F_{1zkx} + G_k = 2331.45kN + 7924.849\ kN = 10{,}256.299\ kN$

$M_1 = M_{1zrk} + F_{1rkx} * H = (380.7\ kN.m * 2.9\ m)15538\ kN.m = 22080.33\ kN.m$

$$e = \frac{22080.33\ kN.m}{10256.299\ kN} = \mathbf{2.153\ m} < \frac{L}{6} = \mathbf{2.333\ m} ------------ satisfied$$

Extreme working condition

$N_2 = F_{2zkx} + GK = 2285.55\ kN + 7924.849\ kN = 10{,}256.299\ kN$

$M_2 = M_{2zrk} + F_{2rkx} * H = 526\ kN.m * 2.9\ m + 39598.2\ kN.m = 41{,}657.49 kN.m$

$e = \frac{41{,}657.49 \text{ kN.m}}{10256.299 kN} = 4.080\ m > \frac{L}{6}$ ---------- Check the Disconnected Area!

The disconnected area= 23.556 % $< 25\%$ $-----------------$ $satisfied$

II. Stability Against Overturning

$M_R = (F_{1zk} + G_k) * \frac{L}{2} = 9651.849kN * 7\ m = 67562.943\ kN.m$

$M_S = M_{2rkx} + F_{2rkx}.H = 41657.49\ kN.m$

$$F_S = \frac{67562.943\ kN.m}{41657.49\ kN.m} = 1.627 > 1.6 \ --------------\ Satisfied$$

The result of this calculation resulted in a value of 1.627 which is almost on the border line, but still satisfies the requirement. The enormous amount of moment resulting from the super structure was the main reason. This shows that overturning criteria is the most governing critical criterion. The detail calculation is presented in Appendix A.

4.4. Results

A design of a land-based wind turbine considering different bearing capacity values is performed. The outcome of this study along with the already adopted foundation for Adama I wind farm is given in Table 12.

Table 12: Results of the existing foundation and newly proposed foundation

Circular Gravity Based Foundation	Bearing Capacity (kPa)	Bottom Diameter (L)	Pedestal Diameter (D_1)	Depth	Volume of Concrete	Volume of Backfill
Existing Foundation	250	15.6 m	6.4m	3 m	324 m^3	3,738m^3
New Foundation	500	14m	5.6m	2.5 m	209 m^3	2,601 m^3
	1000	14m	5.6m	2.5 m	209 m^3	2,601 m^3
	1500	14m	5.6m	2.5 m	209 m^3	2,601 m^3
	2000	14m	5.6m	2.5 m	209 m^3	2,601 m^3
	2500	14m	5.6m	2.5 m	209 m^3	2,601 m^3

After carrying out foundation design for the specified bearing capacity values, the foundation design resulted in the same foundation dimensions for all the bearing capacity values. The new foundation bottom slab and the diameter of the pedestal are found to be 14 m and 5.6m respectively. In addition, the total thickness (i.e. pedestal and slab) is 2.5 m.

The reduction in dimensions of the foundation is interpreted by the volume of concrete, volume of backfill.

- → The volume of the concrete showed 115 m^3 decrement
- → The backfill material decreased by 1,137 m^3.

The reduction in dimensions of the foundation is also quantified monetarily according to current price of materials specified by Addis Ababa city government finance & economy development bureau, 2018). For a one typical foundation

→ Concrete -287,500 Birr

→ Backfill - 90, 960 Birr

→ Total=387,460 Birr

4.5. Conclusion

The purpose of this project was is to study both the static and dynamic design approach to wind turbine foundation and also to assess the effect of using different bearing capacity vales on the dimension of wind turbine foundation for Adama I wind farm. Based on the study conducted the following conclusions are made:

1. The existing design of the Adama I wind farm foundation assumed the tower to be fixed to the foundation. That is, the foundation stiffness was not taken into account, and therefore assumed not affect the response of the tower-rotor system, and vice-versa. This is not the case in reality as foundation stiffness plays a central role in the dynamic response of the system; the most significant of these being reduction in natural frequency.
2. In addition, the loads applied to the foundation were assumed to be static. However, wind turbine structures are inherently dynamic structures, and the design of foundations for the resistance of dynamic loading requires a different design approach.
3. When designing wind turbine foundation taking into consideration the soil structure interaction and dynamic nature of the wind turbine generators. The foundation dimension showed a decrement, which emphasis the static approach to foundation design is conservative.
4. When measuring and judge the performance of foundations against criteria's such as: strength, stability, stiffness, settlement. The stability criteria is the major criteria that governs the dimensions of the foundation as a result of the overturning moment.
5. As mentioned above, this study is carried out with the goal of assessing the effect of using different bearing capacity on the dimensions of the foundation. While doing this, the effect was not as significant as expected, nonetheless there was some change. Note here that most of wind turbine generators (WTG) are located on the hillside of the site, and the foundation used for them was not efficient. As a consequence, all the changes encountered will have a significant effect on the overall cost of the project.

6. The bearing capacity is not the major limiting criteria for the stated project. The stability against overturning is found out to be the most critical criteria. The foundation is subjected to a very high moment as a result of the loads from the superstructure. As a consequence, a sufficient amount of resisting moment is needed to maintain the stability of the overall structure, and since the foundation dead load is the only design parameter that could be manipulated in order to achieve stability, the dimension of the foundation cannot be further reduced.

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APPENDIX A: GEOTECHNICAL DESIGN

A.1. Design load

Characteristics value load and moments (Normal working condition)			
F_{1rk}	F_{1zk}	M_{1rk}	M_{1zk}
$282\ kN$	$1{,}727N$	$15{,}538\ kN.m$	$83\ kN.m$
Corrected value load and moments (Normal working condition), $Ko = 1.35$			
F_{1rkx}	F_{1zkx}	M_{1rkx}	M_{1zkx}
$380.7\ kN$	$2{,}331.45\ kN$	$20{,}976.3kN.m$	$112.05\ kN.m$
Characteristics value load and moments (Extreme working condition)			
F_{2rk}	F_{2zk}	M_{2rk}	M_{2zk}
$526kN$	$1{,}693kN$	$29{,}332\ kN.m$	$140\ kN.m$
Corrected value load and moments (Extreme working condition) , $Ko = 1.35$			
F_{2rkx}	F_{2zkx}	M_{2rkx}	M_{2zkx}
$710.1\ kN$	$2{,}285.6\ kN$	$39{,}598.2\ kN.m$	$189kN.m$

A.2.The assumed dimension of foundation

D=4.2 m

d=0.7m

L=14m

h_1=0.8m

h_2=0.8m

h_3=0.8m

h_4=0.1m

h_5=0 m

h_6=0.4

A.3. Foundation dead load calculation

- The weight of the foundation

$$Gf = (\ (\frac{\pi * 14^2}{4}\ x\ 0.8)\ +\ \left(\frac{\pi\ * 5.6^2}{4}\right) x\ 0.8) + (\frac{1}{3}\pi\ x\ (7^2 + 7 * 2.8 +\ )\ x\ 0.8))\ x\ 25kN/m3$$

$$=\ 5233.906\ kN$$

- Weight of back fill

$$G_{backfill}\ =\ ((153.938\ -\ 24.630)x0.8)\ +\ (153.938\ x\ 0.8)\ - 64.038))\ x\ 16\ kN/m3$$

$$=\ 2600.943\ kN$$

- Weight of foundation ring, G_{ring} = 90kN
- Gross weight

$$G_k = G_{foundation} + G_{backfill} + G_{ring}$$
$$= 5233.906\,kN + 2600.943\,kN + 90kN$$

$$G_k = 7924.849\,kN$$

A.4. Calculation for axial load

$$P_{1k} = \frac{N_{1k} + G_k}{A} = \frac{2331.45kN + 7924.849kN}{153.938\,m^2} = 66.626\,kPa$$
$$< 500\,kPa - Satisfied$$
$$P2k = \frac{N2k + Gk}{A} = \frac{2285.55kN + 7924.849\,kN}{153.938\,m^2} = 66.328\,kPa$$
$$< 500\,kPa - Satisfied$$

A.5. Foundation bearing pressure check

Normal working condition

$$P_{1kmax} = 148.590\,kPa < 600\,kPa ----------------\ ----satisfied$$

Extreme condition

$$P_{2kmax} = 220.963\,kPa < 600\,kPa --------------------satisfied$$

A.6. Eccentricity check.

Normal working condition

$$N_1 = F_{1zkx} + G_K = 2331.45kN + 7924.849\,kN = 10{,}256.299\,kN$$

$$M_1 = M_{1zrk} + F_{1rkx} * H = (380.7\,kN.m * 2.9\,m)15538\,kN.m = 22080.33\,kN.m$$

$$e = \frac{22080.33\,kN.m}{10256.299\,kN} = 2.153\,m < \frac{L}{6} = 2.333\,m ----------- satisfied$$

Extreme working condition

$$N_2 = F_{2zkx} + G_K = 2285.55\,kN + 7924.849\,kN = 10{,}256.299\,kN$$

$$M_2 = M_{2zrk} + F_{2rkx} * H = 526\,kN.m * 2.9\,m + 39598.2\,kN.m = 41{,}657.49kN.m$$
$$e = \frac{41{,}657.49\,kN.m}{10256.299\,kN} = 4.080\,m > \frac{L}{6} ----\ \textbf{check the disconnected area}!$$

The disconnected area is calculated by:

$S_{wt} = \int_{-min(a-\frac{L}{2}, \frac{L}{2})}^{\frac{L}{2}} 2. Bound8(x). dx$, Where, Bound8(x) = $\sqrt{\frac{L^2}{4} - X^2}$ and $a - $ = 3.997m

S_{wt} = 117.6772 m^2

$$\frac{S_1 - S_{wt}}{S_1} = \frac{153.938 - 117.677}{153.938} = 23.556\ \% < 25\% \text{ ------------- } satisfied$$

A.7. Check against sliding

Normal working condition

$$F_{1R} = (F_{1zk} + G_k) * \mu = 9651.849 kN * 0.3 = 2895.555\ kN$$

$$F_{1S} = F_{1rkx} = 380.7\ kN$$

$$\frac{F_{1R}}{F_{1s}} = \frac{2895.555\ kN}{380.7\ kN} = 7.606 > 1.5 \text{ ------------------ } satisfied$$

Extreme working condition

$$F_{2R} = (F_{2zk} + G_k) * \mu = 9617.849 * 0.3 = 2885.355\ kN$$

$$F_{2S} = F_{2rkx} = 710.1 kN = 1081.835\ kN$$

$$\frac{F_{2R}}{F_{2S}} = \frac{2885.355\ kN}{1081.835\ kN} = 4.063 > 1.5 \text{ ---------------- } satisfied$$

A. 8. Stability against overturning

Extreme working condition

$$M_R = (F_{1zk} + G_k) * \frac{L}{2} = 9651.849 kN * 7\ m = 67562.943\ kN.m$$

$$M_S = M_{2rkx} + F_{2rkx}.H = 41657.49\ kN.m$$

$$F_S = \frac{67562.943\ kN.m}{41657.49\ kN.m} = 1.616 > 1.6 \text{ ---------------- } satisfied$$

APPENDIX B: STRUCTURAL DESIGN

B.1. Calculation of Flexural Capacity

$Id = \sqrt{S1} = 12.407$ m

$A_W = \frac{(Id^2 - a\prime^2)}{4} = 32.327\ m^2$

$a' = \sqrt{S2 = 4.963}\ m$

$a1 = \frac{(Id - a')}{2} = 3.722\ m$

$$M_I = 13429.143\ kN.m$$

B.2 Renforcement Calculation

$\alpha_S = \frac{MI}{fc.Id.ho^2} = 0.028$

$A_{SS} = \frac{MI}{fy.\gamma s.ho} = 29874.958 mm^2$

$\gamma_S = 0.5(1 + \sqrt{1 - 2\alpha s}) = 0.986$

$\rho_{min} = 0.2\%$

$A_d = I_d . h1 + \frac{D1 + Id}{2} . h2 = 17.129\ x10^6\ m^2$

$A_d . \rho_{min} = 0.002 * 17.129\ x10^6\ m^2 = 34257.259 m^2$

$A_{SS} = A_{SS}\ if\ Ass \geq Ad.\rho_{min}$

$A_{SS} = A_d . \rho \min otherwise$

$$Since\ 29874.958\ m^2 < 34257.259 mm^2, use\ Ad.\rho min = 34257.259 mm^2$$

B.3. Calculation of Punching Shear Resistance

I. Intersection of Foundation Ring and Foundation

$D3 = D + 2(h1 + h2 + h3 + h4) = 9.2m$

$h_{cq} = h1 + h2 = 1.6\ m$ ----------- $h_{cq} = 0.933$

$S_t = \pi D = 13.19472$ m

$S_b = \pi D_3 = 28.903$ m

$S_m = \frac{S_t + S_b}{2} = 21.049\ m^2$

$A_l = \frac{1}{2} . \left(\frac{L}{1+\sqrt{2}} + \frac{L}{1+\sqrt{2}} + L - D\right) . L - D = 325.678 m^2$

$N = 1.2(F_{2zk} + G_k) = 11541.419 kN$

$M = 1.5(M_{2rk} + F_{2rk}'.H) = 46286.100\ kN.m$

$P_{max} = \left(\frac{N}{A} + \frac{M}{Z}\right) = 246.791$ kN

$F_l = \left(P_{max} - \frac{G_k}{A}\right).Al = 5077.574$ kN

$0.7.\beta_{hp}.f_t.S_m.h_c = 52048.222\ kN$

$\gamma_o.F_l < 0.7.\beta_{hp}.f_t.S_m.h_{cq} = 5077.574\ kN < 520428.222\ kN - - satisfied$

II. The intersection of foundation platform column edge and foundation base

$\mathrm{D}_4 = \mathrm{D}_1 + 2(\mathrm{h1} + h2) = 8.8\ m$

$S_t = \pi.\mathrm{D}_1$ = 17.593 m

$S_b = \pi.D_4 = 27.646m$

$S_m = \frac{S_t + S_b}{2}$ = 22.620 m²

$A_l = \frac{1}{2}\left(\frac{L}{1+\sqrt{2}} + \frac{L}{1+\sqrt{2}} + L - D\right).L - D_4 = 25.997\ m2$

$P_{max} = \left(\frac{N}{A} + \frac{M}{Z}\right) = 246.791\ kN$

$F_l = \left(P_{max} - \frac{G_k}{A}\right).Al = 5015.113\ kN$

$0.7.\beta hp.ft.Sm.hcq = 37109.29498\ kN$

$\gamma_o.Fl < 0.7.\beta_{hp}.f_t.S_m.h_{cq} = 5{,}015.113\ kN < 37{,}109.29498kN - - satisfied$

B.4. Calculation of Oblique Section Shear Capacity

$h_{oo} = h_{cq}$= 1600 mm

$\beta_{hp} = \sqrt[4]{\frac{800mm}{h_{oo}}}$ =0.841

$P_{max} = 246.791$ kN

$V_j = \left(P_{max} - \frac{G_k}{A}\right).a1.I_d$ = 9,019.731 kN

$0.7.\beta hp.ft.I_d.ho$ =18,345.674 kN

$Vj < 0.7.\beta_{hp}.f_t.Id.h_{oo} = 9{,}019.731$ k$N < 18{,}345.674$ kN $- - - -satisfied$

B.5. Calculation of Crack Width

$\alpha_S = 2.1$

$E_S = 200$

$Cc = \ 65mm$

$D_{eq} = 25mm$

$A_{te} = 0.5L(h1 + h2) = 11.2\ x10^7\ mm^2$

$A_S = As. P_{min} = 34257.259mm^2$

$\frac{A_S}{A_{te}} < 0.01 - - - - - \rho_{te} = 0.01$

$h_o = 1.52\ m$

$f_{tk} = 2.01\ N/mm^2$

$P_{1kmax} = 148.590\ kN$

$M_k = \frac{1}{6}. a1^2. (2I_d + a'). (P_{1max}) = 10216.710\ kN. m$

$\sigma_{sk} = \frac{M_k}{0.87\ h_o.A_S} = 22552.7kPa$

$\psi_1 = 1.1 - 0.6\frac{f_{tk}}{\rho te.\sigma sk} = -4.693 < 0.2 - - - - - - \psi_1 = 0.2$

$$\omega_{max} = \alpha_{cr}.\psi_1.\frac{\sigma_{sk}}{E_S}\left(1.9. Cc + 0.08\frac{d_{eq}}{\rho_{te}}\right) = 0.153\ mm < 0.3\ mm - - - satisfied$$

APPENDIX C: SPREADSHEET FOR CALCULATION OF WIND TURBINE FOUNDATION (STATIC APPROACH)

C.1. Design loads (at the top of foundation)

Design loads			
Normal working condition			
Charactrestic Loads		Factored Loads, Ko=1.35	
F_{1rk}	282 kN	F_{1rkx}	380.7
F_{1zk}	1,727 kN	F_{1zkx}	2,331.45 kN
M_{1rk}	15,538 kN	M_{1rkx}	20,976.3 kN
M_{1zk}	83 kN	M_{1zkx}	112.05
Extreme working condition			
Charactrestic Loads		Factored Loads, Ko=1.35	
F_{2rk}	526 kN	F_{2rkx}	710 kN
F_{2zk}	1,693 kN	F_{2zkx}	2,285.55 kN
M_{2rk}	29,332 kN	M_{2rkx}	39,598 kN
M_{2zk}	1,400 kN	M_{2zkx}	1,890 kN

C.2. Input parameters

Input Values	
Bearing Capacity	**500 kPa**
Foubdation Ring Diameter (D)	4.2 m
d	0.7 m
Bottom Slab Diameter (L)	**14.0 m**
Circular Section (h_1)	0.8 m
Oblique Section (h_2)	0.8 m
Pedstal Section (h_3)	0.8 m
Above Backfill (h_4)	0.1 m
Above Pedstal (h_5)	0.4 m
Top Pedstal Diameter (D_1)	5.6 m
Total Height of Concrete(H)	**2.5 m**
Effective Height of Slab (h_o)	1.52 m
Bottom Slab Radius	7.0 m
Pedstal Radius	2.8 m
Section Modulus (Z)	269.392 m

C.3. Calculation of dead load

Dead load Calculation					
Area (m^2)		Volume (m^3)		Weight (kN)	
Bottom Slab Area (S_1)	153.938	Circular Slab Volume(V_1)	123.151	Weight of Concrete	5233.906
Pedstal Area (S_2)	24.630	Oblique Slab Volume (V_2)	64.038	Weight of Backfill	2600.943
		Pedstal Volume (V_3)	22.167	Weight of Ring	90
		Total Volume (V_t)	209.356	Total DL (G_k)	7924.849

C.4. Design loads (at the bottom of foundation)

Combination of vertical forces			
Standard Value		Corrected Value	
N_{1k}	9,651.849 kN	N_{1kx}	10,256.299 kN
N_{zk}	9,617.849 kN	N_{2kx}	10,210.399 kN
Combination of moments			
Standard Value		Corrected Value	
M_{1k}	16,243 kN-m	M_{1kx}	21,928 kN-m
M_{2k}	30,647 kN-m	M_{2kx}	41,373 kN-m

C.5. Foundation design for bearing capacity

Bearing capacity				
		Maximum	**Minimum**	
Bearing Pressure(P_{kmax})	Normal	148.024 kPa	-14.772 kPa	Satisfied
	Extreme	219.909 kPa	-87.253 kPa	Satisfied

C.6. Foundation design for eccentricity

Eccentricity(e)	Normal	2.138 m	Satisfied
	Extreme	4.052 m	**Not Satisfied**

C.6.1. Disconnected area

Come-off area		
Come-off length	3.977 m	
Radius(L/2)	7. m	
a_c-L/2	-3.023 m	
S_w	117.938 m^2	
$(S_1-S_w)/S_1$	23.386 %	**Satisfied**

C.7. Stability calculation

Stability			
Sliding	Normal	7.606	Satisfied
	Extreme	4.063	Satisfied
Overturning	Extreme	1.627	**Satisfied**

C.8. Shear capacity calculation

C.8.1. Input values

Punching shear calculation		
	Normal	**Extereme**
N	11,582.219 kN	11,541.419 kN
M	24,365 kN.m	45,971 kN.m
P_{max}	165.682 kN	245.62 kN
Y_o	1	1

	Punching Shear 1		Punching Shear 2		Oblique Section Shear	
	f_t	1,570 kPa	f_t	1,570 kPa	f_t	1,570 kPa
	h_{cq}	2.5 m	h_{cq}	1.6 m	h_{oo}	1.6 m
	B_{hp}	0.9	B_{hp}	0.933	B_{hp}	0.841
	D_3	9.2 m	D_4	8.8 m	Id	12.407 m
	S_t	13.195 m	S_t	17.593 m		
	S_b	28.903 m	S_b	27.646 m		
	S_m	21.049 m^2	S_m	22.62 m^2		
	A_1	25.678 m^2	A_1	25.997 m^2		
F1	Normal	2,968.939 kN	Normal	2,932.417 kN		
	Extreme	5,047.117 kN.m	Extreme	4,985.031 kN.m		

C.8.2. Punching Shear capacity calculation

Shear capacity			
Punching Shear 1	Normal	2,932.42 kN	Satisfied
	Extreme	4,985.03 kN	Satisfied
	Resistance=	**52,048.22 kN**	
Punching Shear 2	Normal	2,968.94 kN	Satisfied
	Extreme	5,047.12 kN	Satisfied
	Resistance=	**37,109.29 kN**	
Oblique Section Shear	Normal	5,273.98 kN	Satisfied
	Extreme	8,965.63 kN	Satisfied
	Resistance=	**18,345.67 kN**	

C.9. Flexural capacity

Moment		
	Id	12.407 m
	a'	4.963 m
	a1	3.722 m
	A_w	32.327 m^2
	f_c	16,700 kPa
	h_o	1.52 m
	f_y	3.00E+05 kPa
Moment	Extreme	**13,348.592 kN.m**

Reinforcement			
	αs	0.028	
	γs	0.986	
Actual	Ass	2.97E+04 m^2	
	A_d	1.71E+07 m^2	
Minimum	Ad. P_{min}	3.43E+04 m^2	Satisfied

C.10. Calculation for crack width

Crack width			
	x_{cr}	2.1	
	E_s	200 kN/mm^2	
	C_c	65 mm	
	A_{te}	1.120E+08 mm^2	
	A_s=A_{sp}	3.426E+04 mm^2	
	f_{tk}	20 N/mm^2	
	deq	25 mm	
ψ	1.100	0.2	
P_{te}	0.0003	0.01	
	h_o	1.52 m	
	M_k	10177.84337	
	ssk	2.25E+07 kPa	
Crack Width	w_{max}	**0.153**	**Satisfied**

APPENDIX D: SPREADSHEET FOR CALCULATION OF WIND TURBINE FOUNDATION (DYNAMIC APRROACH)

D.1. Input parameters

Input prameters	
B (per meter)	1,000 mm
Diameter	13,200 mm
υ	0.4
E_{soil}	85 MPa
$E_{concrete}$	30 GPa
Radius	6,600 mm
P	2,370.570 kN
M	20,976.300 kN.m

D.2. Calculating stiffness using Vesic's Relation

Vesic	
$(EI)_b$	4.47E+19 N-mm^4
K_s	0.02194 N/mm
λ	1.87E-05 /mm
λl	0.2471

Since, $\lambda L = 0.2741 < 6$, the beam is Finite Beam. So, use finite beam analysis

D.4. Dimensionless coefficients for different locations

Dimensionless Cofficients					
$\lambda =$	1.87E-05				
x (mm)	λX	$A(\lambda X)$	$B(\lambda X)$	$C(\lambda X)$	$D(\lambda X)$
0	0.0000	1.0000	0.0000	1.0000	1.0000
600	0.0112	0.9999	0.0111	0.9777	0.9888
1200	0.0225	0.9995	0.0220	0.9556	0.9775
1800	0.0337	0.9989	0.0326	0.9337	0.9663
2400	0.0449	0.9980	0.0429	0.9122	0.9551
3000	0.0562	0.9970	0.0531	0.8908	0.9439
3600	0.0674	0.9957	0.0629	0.8698	0.9327
4200	0.0786	0.9941	0.0726	0.8489	0.9215
4800	0.0898	0.9924	0.0820	0.8284	0.9104
5400	0.1011	0.9905	0.0912	0.8080	0.8992
6000	0.1123	0.9883	0.1002	0.7880	0.8881
6600	0.1235	0.9860	0.1089	0.7681	0.8770
7200	0.1348	0.9834	0.1174	0.7486	0.8660
7800	0.1460	0.9807	0.1257	0.7292	0.8550
8400	0.1572	0.9778	0.1338	0.7102	0.8440
9000	0.1685	0.9747	0.1417	0.6913	0.8330
9600	0.1797	0.9714	0.1493	0.6727	0.8221
10200	0.1909	0.9680	0.1568	0.6544	0.8112
10800	0.2022	0.9644	0.1640	0.6363	0.8003
11400	0.2134	0.9606	0.1711	0.6184	0.7895
12000	0.2246	0.9567	0.1779	0.6008	0.7787
12600	0.2359	0.9526	0.1846	0.5834	0.7680
13200	0.2471	0.9484	0.1910	0.5663	0.7574

D.5. Calculation of the bending moment and shear force values

Bending Moment and Shear Force values produced due to:						
	Concentrated Load, P		Moment to the right, M_1		Summation	
Moment	$M^{P}A$	24,319.735 kN.m	$M^{M}A$	-9,198.608 kN.m	M_A	15,121.127 kN.m
	$M^{P}B$	24,319.735 kN.m	$M^{M}B$	9,198.608 kN.m	M_B	33,518.343 kN.m
Shear	$V^{P}A$	1,039.552 kN	$V^{M}A$	-193.567 kN	V_A	845.985 kN.m
	$V^{P}B$	-1,039.552 kN	$V^{M}B$	-193.567 kN	V_B	-1,233.118 kN.m

D.6. Method of Superposition

Superposition	
M_A'	23,918.399 kN.m
M_A''	-9,207.261 kN.m
S_A'	1,022.764 kN
S_A''	-193.567 kN

D.7. Continuing with superposition

E_1	1.2953
E_2	18.3198
P_o'	9587.4095
P_o''	-28025.7854
M_o'	-255931.4635
M_o''	674094.8400

D.8. Calculation of end conditioning shear forces and moments

End Conditioning	
V_{OA}	-18,438.376 kN
V_{OB}	37,613.195 kN
M_{OA}	418,163.377 kN.m
M_{OB}	-930,026.303 kN.m

D.8. Deflection and contact pressure calculation

Calculation of Defelection and contact pressure							MDA λ =	1.87E-05 /mm
							K_s=	0.02194 N/mm
ocatio	Deflection due to:							
x (mm)	V_{oA}	M_{oA}	P	M_1	V_{oB}	M_{oB}	Total	Pressure (kPa)
0	-7.864	0.000	0.997	-0.036	15.214	2.837	11.147	244.621
600	-7.863	0.074	0.999	-0.034	15.282	2.741	11.200	245.767
1200	-7.860	0.147	1.001	-0.031	15.347	2.642	11.247	246.802
1800	-7.855	0.217	1.003	-0.027	15.410	2.541	11.289	247.722
2400	-7.849	0.287	1.005	-0.024	15.470	2.436	11.325	248.524
3000	-7.840	0.354	1.007	-0.021	15.528	2.328	11.356	249.205
3600	-7.830	0.420	1.008	-0.018	15.583	2.218	11.382	249.762
4200	-7.818	0.485	1.009	-0.014	15.636	2.104	11.401	250.192
4800	-7.804	0.548	1.010	-0.011	15.685	1.987	11.415	250.491
5400	-7.789	0.609	1.011	-0.007	15.732	1.867	11.422	250.658
6000	-7.772	0.669	1.011	-0.004	15.776	1.744	11.424	250.687
6600	-7.754	0.727	1.011	0.000	15.817	1.617	11.419	250.577
7200	-7.734	0.784	1.011	0.004	15.854	1.487	11.407	250.321
7800	-7.712	0.839	1.011	0.007	15.889	1.354	11.389	249.916
8400	-7.689	0.893	1.010	0.011	15.920	1.218	11.363	249.359
9000	-7.665	0.946	1.009	0.014	15.948	1.078	11.331	248.645
9600	-7.639	0.997	1.008	0.018	15.972	0.935	11.291	247.773
10200	-7.612	1.047	1.007	0.021	15.993	0.788	11.244	246.739
10800	-7.584	1.095	1.005	0.024	16.011	0.638	11.189	245.541
11400	-7.554	1.142	1.003	0.027	16.024	0.484	11.127	244.174
12000	-7.523	1.188	1.001	0.031	16.034	0.326	11.057	242.637
12600	-7.491	1.232	0.999	0.034	16.040	0.165	10.979	240.925
13200	-7.458	1.276	0.997	0.036	16.042	0.000	10.893	239.036

D.9. An Analytical Method of Dynamic Analysis of Block Machine Foundations

D.9.1. Input values

Input Values	
γ_s	18 kN/m^3
v	0.4
g	10 m/s^2
$G_{degraded}$	98 MPa
γ_c	25 kN/m^3
e(Embedment depth)	3.30 m
L	13.2 m
B	13.2 m
H	3.5 m
d_s	6.3 m
ρ_s	1.8 kN (s^2)/m^4
CG $_{\text{from the top}}$	2.5 m
v_s	7.394 m/s

D.9.2. Calculation of the foundation mass moment of inertia

A_f	136.848 m^2
V_f	221.043 m^3
W_f	5526.074 kN
M_f	552.607 kg
$M_{Turbine}$	127,000kg
Equivalent Radius, $R_{o,\ Translati}$	6.6 m
Equivalent Radius, $R_{\theta,\ Rocking}$	6.6 m
Mass Moment of Inertia, $I_{\theta F}$	8,274.38 kNms^2
Mass Moment of Inertia, $I_{\theta m}$	280,035.00 kNms^2
$I_{\theta(\text{foundation and turbine})}$	288,309.38 kNms^2

D.9.3. Calculation of static stiffnes considering embedement effect

Static Stiffness Calculation	
k_h	3,247 kN/m
k_θ	125,728 kN.m
Accounting the Embedment Effect	
$(K_u)_{FL\backslash E}$	10,917 kN/m
$(K_\theta)_{FL\backslash E}$	403,659 kN.m
v_s	290m/s
Frequency Ratio, a_o	0.023 ω

D.9.4. Calculation of frequency independent stiffnes

Frequency independent coefficient	
k	3,281 kN/m
M_{Total}	127,553kg
ω	0.342
a_o	0.00778

D.9.5. Calculation of dynamic coefficient and dynamic stiffness

Dynamic Coefficient	
α_h	0.999922
α_θ	0.998327
β - Read from Graph	
β_h	0.8
β_θ	0.1

Dynamic stiffness	
k_h	10,916 kN/m
k_θ	402,984 kN.m

D.9.6. Calculation of the natural frequencies

Coupling Factor, η	0.025
λ_h	0.086 /s^2
λ_θ	1.398 /s^2
$\lambda_{h\theta}$	0.120 /s^4
$\lambda'_{h\theta}$	1.518 /s^4
λ_1	0.117
λ_2	0.574
Natural Frequencies	
ω_1	0.342
ω_2	0.758

D.10. Calculation of response to Harmonic Loading

D.10.1. Input values

Load & moment	
P_o	282.00 kN
M_o	16,412.20 kN.m
ω'	1.7793 rad/sec

D.10. 2. Stiffness and mass matrix

Stiffness matrix	
k_{11}	10,916 kN/m
k_{12}	-10,514 kN
k_{21}	-10,514 kN
k_{22}	413,110 kN.m

Damping matrix	
c_h	7795.539
c_θ	35973.012
c_{11}	7,795.539 kNs/m
c_{12}	-7,508.400 kN.s
c_{21}	-7,508.400 kN.s
c22	43,204.848 kNm/s

D.10.3. Response to Harmonic Loading

Force Only Acting		
$(u_o)_P$	1.77E-02	
$(\theta_o)_P$	4.21E-08	
Moment Only Acting		
$(u_o)_M$	2.45E-03	
$(\theta_o)_M$	4.61E-04	
Total		
$(u_o)_{Total}$	0.020169307	
$(\theta_o)_{Total}$	0.000461	**satisfied**