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MASTER'S THESIS

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Abstract

Climate challenges have led to greater focus on renewable energy sources and the development of wind turbines has become increasingly popular. There are both advantages and disadvantages with the use of wind turbines. The wind turbines offer major advantages in the production of renewable energy, while at the same time causing interference in nature.

Wind turbines are continuously affected by varying wind forces that create cyclic stresses in the foundation. Over a typical 20-year life, a foundation can be exposed to up to 10^9 load cycles. These cyclic loads can lead to fractures in the concrete at stresses lower than the static breaking limit.

By using one year of wind measurements, a spectrum of fatigue loads applied to a foundation for a 100-meter-high wind turbine have been created. This spectrum has been used to control the concrete fatigue capacity in the foundation for compression and shear forces according to three different standards, NS-EN 1992-1-1:2004, Model Code 2010 and DNV-OS-C502. In addition, the expected lifetime of the foundation due to concrete fatigue has been calculated according to two of the standards.

After analysis of the results, it turns out that DNV-OS-C502 is the most suitable standard to use for concrete fatigue verification of wind turbine foundations, for both compression and shear forces.

Huge deviations in the results between the standards for both compression and shear were found, which indicates that concrete fatigue needs further development and better standardization.

Preface

This thesis marks the end of a two-year master's degree at the University of Stavanger. The thesis has been ongoing from the beginning of January to the middle of June 2020.

Renewable energy and wind turbines are a very relevant topic in the world today, which led me to embark on this thesis. Concrete fatigue was a topic I had little background knowledge about, which was also a factor for thesis selection, as I like learning about new topics that I find exciting. Through the work on this thesis, several people have been of great help.

I would like to thank Samindi Samarakoon at UiS for great help in defining the thesis, and good guidance throughout the duration of the thesis.

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I would like to thank Lars Tallhaug at Kjeller Vindteknikk for help in obtaining wind data that was used in the thesis.

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1. Thesis description

1.1 Introduction

Wind turbine structures are constantly exposed to a large number of cyclic loads. Varying wind loads cause the applied forces to vary widely in both size and intensity. Therefore, great demands are placed on well-designed foundations to absorb these forces so that material fractures are avoided. Several different standards have specific rules for concrete fatigue. In this thesis, a wind turbine foundation will be examined for concrete fatigue according to three different standards.

The software that have been used in this thesis are:

-Microsoft Word

-Microsoft Excel

-Autodesk Revit

-Strusoft FEM-design

1.2 Problem description

There are three main problems that will be answered in this thesis.

- Is there a difference in the results for the validation of concrete fatigue for a wind turbine foundation between the standards NS-EN 1992-1-1:2004, Model Code 2010 and DNV-OS-C502?

- Which standard is most suitable for validation of concrete fatigue for compression in a wind turbine foundation?

- Which standard is most suitable for validation of concrete fatigue for shear force in a wind turbine foundation?

1.3 Method

To be able to solve the problems regarding this thesis, foundational knowledge about concrete fatigue and wind turbine design was initially acquired. Further on, the rules for concrete fatigue described in the three different standards was investigated closely.

When this foundational knowledge was acquired, the dimensions of a wind turbine to be investigated were found and equations for wind loads on a wind turbine at a specific wind speed was derived.

The derived formulas were in combination with the measured wind data and the formulas for concrete fatigue described in the standards, inserted to Microsoft Excel for calculation of the fatigue loads. The results according to the different standards were then analyzed and compared with each other.

2 Wind turbines

2.1 Wind energy

Wind power is an emission-free method for power generation. The wind causes the turbine blades to rotate, thus creating motion energy that is converted into electricity by means of a generator. Through the generator, the energy is passed on to the consumers through cables and grids.

Wind power is a renewable energy source, which means that CO2 is not released into the air during wind power production itself. Nevertheless, there is a small environmental impact from wind power, but this is primarily related to the wind power plant itself and the electricity grid. The authorities impose strict requirements that must be met when wind farms are built and later when operated to minimize environmental impact.

Figure 1- Wind turbine in Ånstadblåheia wind farm in Sortland, Norway (Fortum, 2020)

During periods when the wind is absent, the wind turbines cannot produce energy. A minimum windspeed of 3 m/s is required for the turbine to have profitable wind production. In addition, the wind turbine must be turned off if the wind becomes stronger than 25 m/s to avoid damage to the turbine (Rosvold & Hofstad, 2019). When the wind turbines are not operating, a different energy source must be used to balance the energy demand. Wind power is particularly suitable in areas that also have access to hydropower. Hydropower can be used to regulate the irregular wind production, providing 100% renewable energy production. Norway is an example of where this combination can be used with high efficiency because of its high number of hydropower facilities and coastal topography with high annual winds. Increased use of renewable energy sources is important for solving the climate challenges we are facing (Fortum, 2020).

By 2040, population growth and a higher standard of living will contribute to increased energy consumption globally. In order to minimize negative environmental impacts, major parts of the energy should be sourced from renewable energy sources. According to Statkraft, wind power will cover 20 percent of electricity production in 2040 (Statkraft, 2019). Figure 2 below shows the expected distribution of electricity sources in the world in the period 2015 to 2040. The figure indicates that the use of renewable energy sources is rising, and especially solar and wind power are expected to have a large gradual increase. The use of fossil energy sources is declining and especially energy from oil and coal is expected to have a major decline.

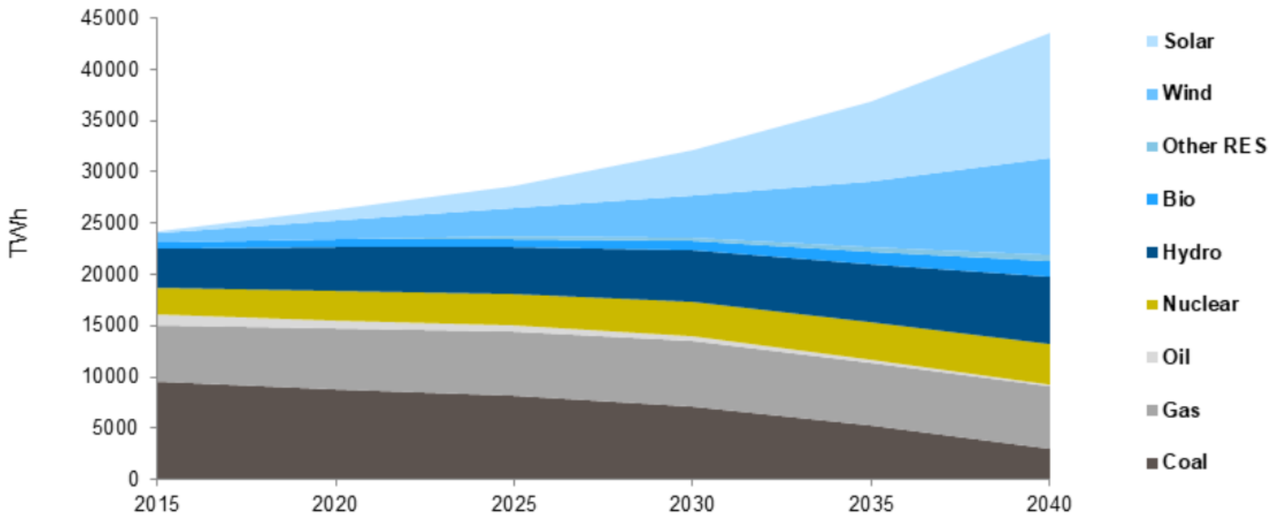

Figure 2 - Expected distribution of electricity sources in the world in the period 2015-2040 (Statkraft, 2019)

There are not exclusively positive factors related to wind turbines. In many countries, the development of wind turbines has been a highly debated topic. The most used argument against wind turbines are that they do not fit in, in a natural environment. Wind farms need to be located in areas like hillsides or open plains, where the wind is consistently strong. This makes the turbines easily visible in the landscape. The wind turbines also produce a lot of noise, which interferes with residents close to the wind farms.

Birdlife is also highly affected by wind turbines, as several million bats and birds are being killed by the rotating blades each year. Studies conducted in Spain, estimate that each turbine kills approximately 400 birds on average each year. The turbines especially pose a threat to endangered bird species (Pedersen, 2012).

Shadows caused by the wind turbines may also be a negative factor for buildings nearby, as it can reduce exposure of sunlight.

In addition to the turbine itself, road systems must be built in the wind farm areas in order to build and maintain the structures. On the other hand, these road systems make nature more accessible for cyclists, families with small children and disabled people, as the roads normally are open for non-motorized traffic.

2.2 Wind turbine designs

Since the first windmills were built in Persia around year 600, the design has evolved. In the past, wind power was mainly used to grind grain in classic windmills. Today, the wind turbines are largely used for production of electrical energy (Mæhlum & Rosvold, 2019).

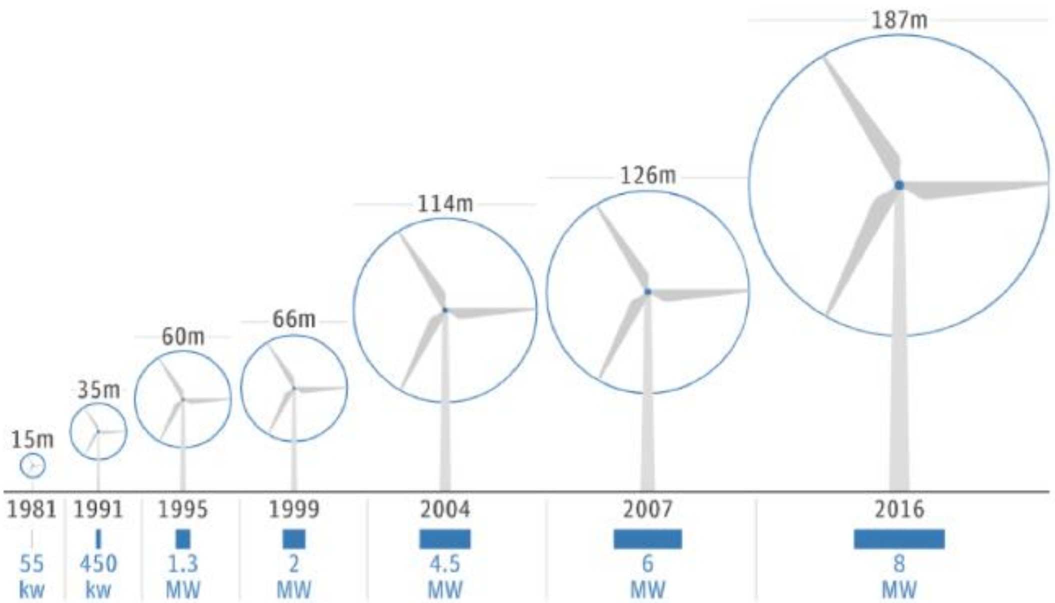

Figure 3 - Development of wind turbine size and efficiency from 1981 to 2016 (NorthSEE, 2017)

Because the wind generally is stronger at higher altitudes, it is desirable to build wind turbines as tall as possible to achieve the highest possible energy production. As shown in Figure 3 above, the development of bigger and more efficient wind turbines has escalated drastically the last 20 years. In 2021, 260-meter-tall 12 MW wind turbines are expected to be in service. Turbines of this size alone, can power up to 16 000 typical European households. As the focus on renewable energy sources has seen a great increase over the past years, further development and improvement of both design and technology is expected in the following years (Genreral Electric, 2019).

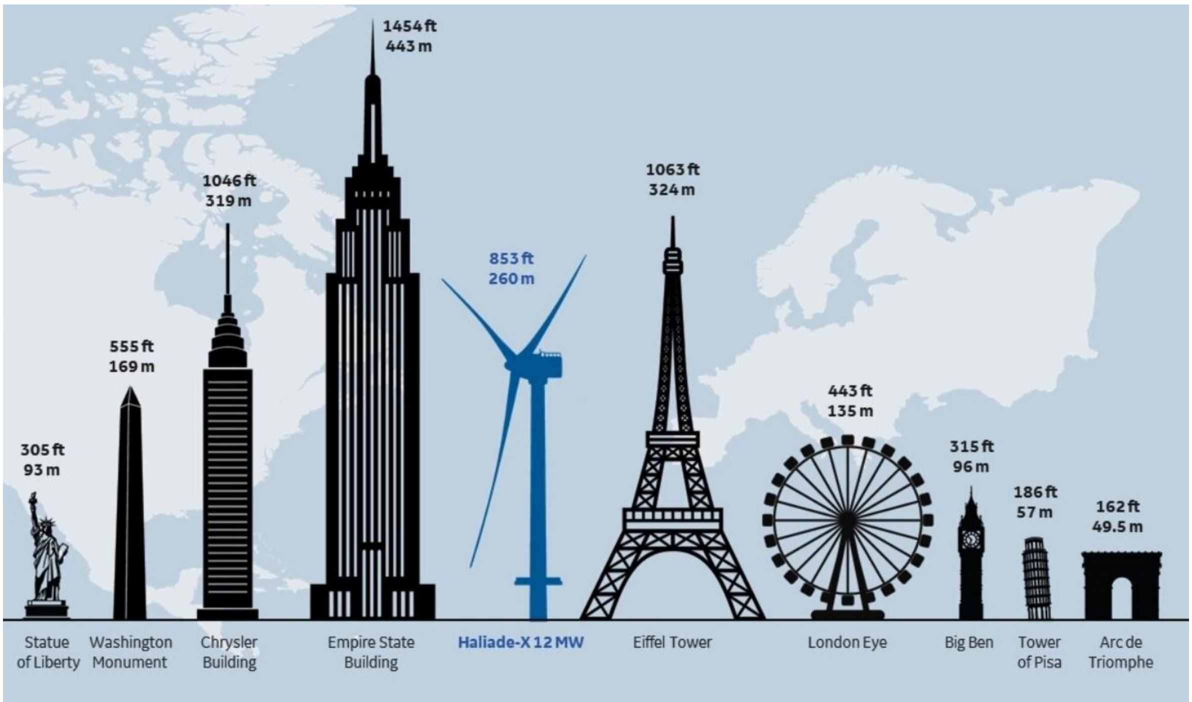

Figure 4 - The size of the planned GE Haliade-X 12 MW turbine compared to other well-known structures (Edgren, 2019)

The typical design of the wind turbine today has a horizontal axis rotor and three blades. This design has proven to be the most cost-efficient. A four-bladed turbine may be more efficient, but the cost of an additional blade will not have a high reward in relation to the small increase in efficiency. Two-bladed, and even mono-bladed turbines do exist, although it is not very common.

Figure 5 below displays the main parts of a typical wind turbine. Each component has their own important function for the turbine to operate.

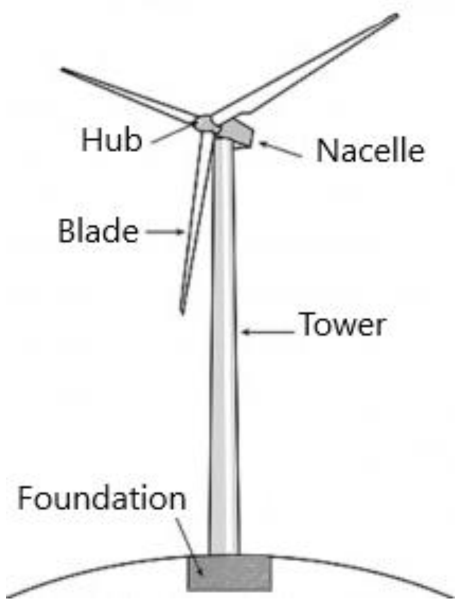

Figure 5 - Typical design of a wind turbine (Waubra, 2013)

Blades are usually made of glass fibers or a combination of both carbon fiber and glass fiber. These materials are especially suited because of their great strength combined with low density, which are important factors in a turbine blade. The weight of the blades is preferred to be as low as possible, while still being strong enough to withstand the applied forces. Glass fiber is the most cost efficient and easily accessible alternative, however carbon fiber has up to 30 % lower density than glass fiber. By reducing the weight as much as possible, the gravitational forces have less impact on the turbine, which results in higher electricity production (Langøy, 2019).

The part where the blades are connected is called the hub and is made of solid steel. The hub is attached to the nacelle and operates the generator directly.

The definition of both the blades and the hub is called the rotor. The rotor is the moving part of a wind turbine from an outside perspective. The size of a turbine is often stated as rotor diameter or swept area, which is the area covered by the rotor when rotating.

The nacelle is a platform on the top of the tower where the rotor is mounted. The generator and the gearbox for the turbine is found in the nacelle.

The size of the tower is what mainly defines the height of the turbine. The three main tower types are steel, concrete and hybrid.

The steel tower is the most commonly used tower type. There are many benefits with the use of a steel tower. Transportation and installation of the tower is simple. Instead of transporting the whole tower at once, it is divided into smaller cylinder parts, which thereafter are mounted on site. Steel towers also have good resistance against corrosion and are suitable for both onshore and offshore use. As the steel tower have been the most commonly used throughout the years, the manufacture technology is very mature. In addition, production of steel towers has a fair average price. Figure 6 below pictures a steel turbine tower located at Ånstadblåheia wind farm in Norway.

Figure 6 - Steel tower turbine at Ånstadblåheia wind farm in Norway (Fortum, 2020)

The main advantages of a concrete tower are the low cost and no thickness limit in the walls. The disadvantages, however, is long production time and they are only usable onshore. Figure 7 below shows a precast concrete tower during the installation phase. There is also an environmental issue with the concrete towers as the waste produced after demolition is difficult to dispose. This is a critical factor because the environment is a big focus point in the wind turbine industry.

Figure 7 - Precast concrete tower during installation phase (Dvorak, 2016)

Hybrid tower is a combination of steel tower and concrete tower. This type of tower does however have big construction difficulties. The tower type is only viable for turbines taller than 100 meters as the price will be too high for smaller turbines were this combination is applied. The hybrid tower can also only be used onshore (CNBM International, 2017). Figure 8 below displays a hybrid turbine tower mounted in India.

Figure 8 - Hybrid concrete and steel turbine tower (Financial Express, 2018)

To connect the tower to the foundation, a type of anchorage is necessary. There are two main types of anchorage, which are anchorage plate and anchorage cage.

The anchorage plate is a solid circular steel plate that will usually be welded to the bottom of the tower and is further bolted into to the concrete in the foundation. The thickness of the plate is usually upwards to 100 mm, depending on the tower size. An anchorage plate is shown on Figure 9 below. This type of anchorage is mostly used in smaller turbines.

Figure 9 - Anchorage plate (Ciltug Inc., 2019)

An anchorage cage is ideally used for larger turbines. The cage is a top and a bottom circular steel plate, that are joined by several thick steel bolts and are cast into the foundation (CNBM International, 2017) . In Figure 10 below an anchorage cage is displayed.

Figure 10 - Anchorage cage during the installation phase (CTE Wind International, 2020)

To prevent overturning and failure of the turbine a solid foundation is necessary. Foundations can have different shapes, either circular, squared or angular. There are several design types for the foundation, although the three most common types are gravity foundation, rock-piled foundation and the ribbed beam foundation.

Gravity foundations are based on the principle that gravity keeps the tower stable. Large amounts of concrete are used in combination with overfilling of soils to give the foundation the necessary stability. This kind of foundation requires little installation time, are easy to construct and may be used in all plain areas where the substrate consists of soil or sand. A gravity foundation is displayed in Figure 11 below.

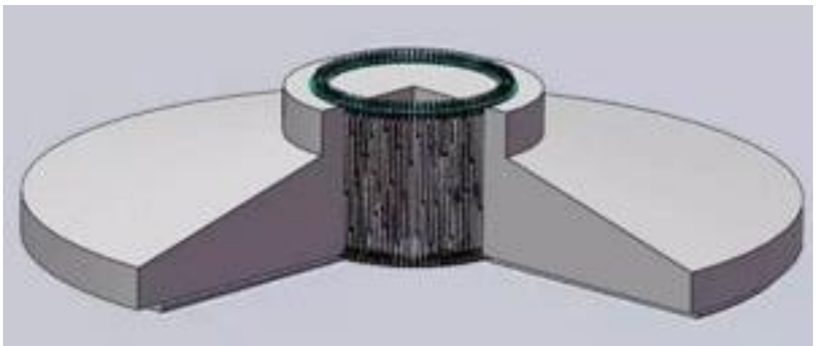

Figure 11 - Gravity foundation with anchorage cage (CNBM, 2017)

In rocky areas, a rock-piled foundation is a preferable option over the gravity foundation. The foundation is reinforced in the solid rocks underneath the ground by steel piles that are anchored in the concrete. Due to the support received from the rocks, this foundation requires less amounts of concrete and rebars. A rock foundation will always be the better option as this is a much lower in cost and results in less impact on the environment unlike a gravity foundation. Figure 12 below displays a 3D cross-sectional view of a rock-piled foundation.

Figure 12 - Cross-sectional 3D view of a rock-piled foundation with an anchorage plate (Proplate, 2020)

A third foundation type is the ribbed beam foundation. This is a variant of the gravity foundation, except beams are used to withstand the forces from the tower so the amount of concrete used can be reduced. The downside of this foundation type is that a wider surface area is needed, which means that more groundwork is necessary (CNBM, 2018). Figure 13 below shows a model of a ribbed beam foundation.

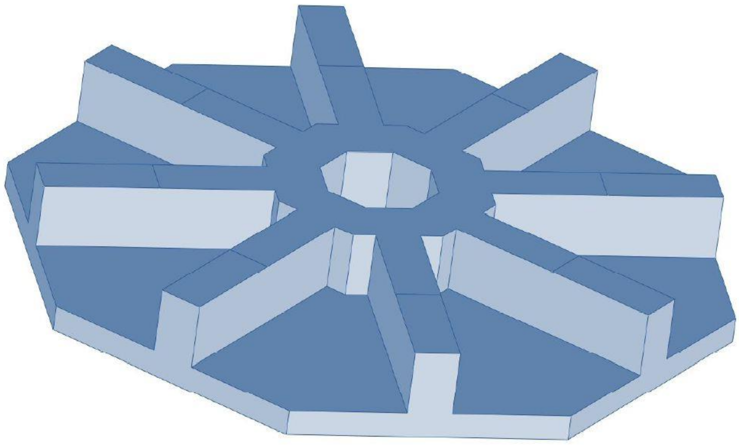

Figure 13 - The ribbed beam foundation type (Miceli, 2013)

3 Concrete fatigue

3.1 What is concrete fatigue?

Concrete fatigue can occur when a material is subjected to a high number of cyclic loads at stresses lower than the ultimate breaking limit. When a critical number of load cycles have been applied the material will fail.

This suggests that a structural part can fail to a load that is much lower than the static strength of the material, if the load is applied enough times. Material fatigue can therefore be very dangerous since no high stresses are necessarily required for the structural part to fracture.

Wind turbines, offshore structures, bridges, roads, tall buildings and airport runways are examples of structures that are affected by cyclic loadings. A wind turbine can be subjected to more than one billion cycles in 20 years. Therefore, it is important to consider the effects of fatigue when designing structures as these (Gessert, Nguyen, & Rogers, 2018).

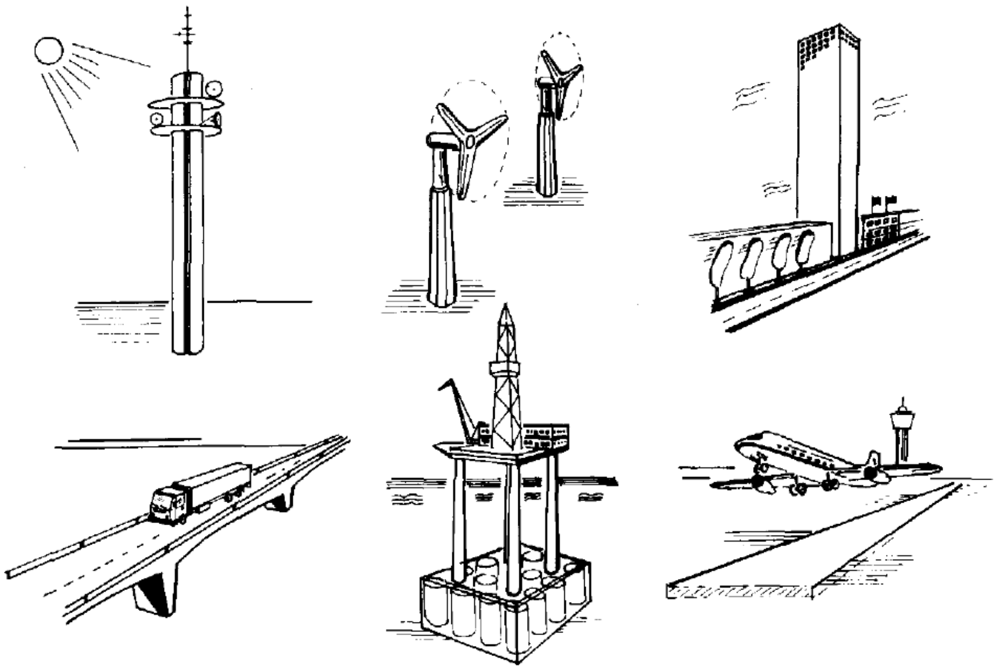

Figure 14 - Structures that are typically affected by cyclic loadings (Hordijk, 1991, s. 8)

Concrete is an inhomogeneous material, which consist of a composition of cement paste, pores and aggregates of various sizes. There is a wide inner variation in this composition, which makes it difficult to set clear theoretical rules for concrete. Guidelines for concrete fatigue are therefore mainly based on research (Andersen & Ertel, 2017).

In steel and aluminum, several research projects have been executed about fatigue. Steel is much more homogeneous than concrete, so it easier to set guidelines for steel fatigue. In addition, fatigue in metals has been the source of many serious accidents that has resulted in high attention compared to fatigue in concrete. Examples of these accidents are the bridge collapse in Genova in 2018, the Turøy accident in 2016, the Alexander L. Kielland accident in 1980 and the Havilland Comet accidents in the 1950s. These are all accidents with major loss of life and material damage.

There have been reported few accidents caused by concrete fatigue in comparison to steel. Concrete fatigue alone has not always been the cause of such accidents but have been a significant influencing factor. The most common factor is concrete fatigue failure in combination with repeated deflections, increased loads or increased load frequencies. In addition, chemical attacks such as pitting corrosion and carbonation may increase the effect of concrete fatigue. The most common problem due to concrete fatigue is cracking of concrete pavements, bridge decks and airport runways (CEB, 1988). In the following section three examples of these accidents are described.

In Japan, the Netherlands and in Sweden there have been reported of cracking in reinforced concrete bridge decks. The cracks appear near the wheel tracks and there have not been reported any damage to the steel reinforcement itself. Bridges with low traffic has shown to have longer lifetime before the initiation of cracks compared to similar bridges with more traffic. Therefore, the intensity of traffic was clearly a factor contributing to the cracking. Common in all the cases was that the engineers did not consider fatigue or the possibility of increased traffic across the bridges as contributing factors. (CEB, 1988, ss. 16-22)

On unreinforced concrete highways in the Netherlands, cracks appeared when the pavement itself had longer life span than the design life. Both longitudinally and transverse cracks was created due to fatigue from vehicles passing. (CEB, 1988, s. 25)

Cracks and deflections where found in precast, pretensioned concrete slabs in a factory in the UK. Examinations shows that heavy loads did not cause the cracking, but the forklifts repeated passing. (CEB, 1988, ss. 25-26)

3.1.1 SN-Curve

The most common way to describe concrete fatigue is as a percentage of the static concrete strength at a given number of cycles. This notates as $F_{fatigue}/F_{static}$ at N number of cycles (Ameen & Szymanski, 2006, ss. 6-7).

To display the fatigue properties of a material, a SN-curve is most used (Also commonly known as a Wöhler-curve). This type of curve was invented by railroad engineer August Wöhler in 1860 after studying fatigue in railroad axles. This is a curve showing stress level in percent of ultimate strength versus number of cycles to failure on a logarithmical scale. On Figure 15 below, an example of a SN-curve is shown.

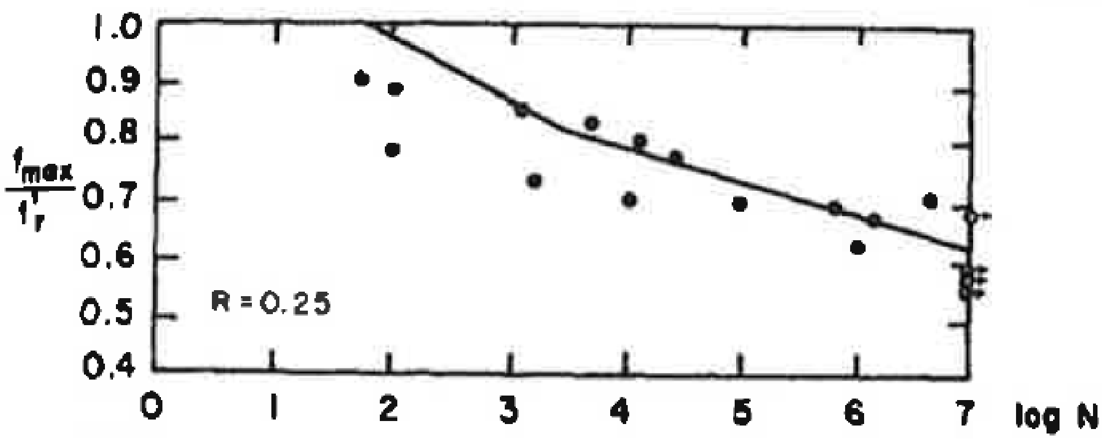

Figure 15- Example of a SN-Curve (Hsu, 1981, s. 13)..

The SN-curves have been developed through experience from results of fatigue tests. The curve is formed by plotting the test results in the diagram, then a regression line is drawn through the plots. A higher number of data will result in a higher accuracy of the curve. The figure above shows an example of a SN-curve. The vertical axis represents f_{max}/f'_{r} which is applied stress in percentage of ultimate static strength. The horizontal axis represents N, which is number of cycles until failure displayed as log N.

When an object is exposed to cyclic loads, the loads do not necessarily have the same amplitude. Therefore, a factor R can be implemented to the SN-curve to display the stress range. R is defined as, $R = \sigma_{min}/\sigma_{max}$ where σ_{min} is the average of the stress amplitudes and σ_{max} is the maximum stress applied to the material.

3.1.2 Historical review

Throughout the years, research have been conducted to identify how fatigue in concrete occurs. Already in the early 1900s, fatigue in concrete was investigated. Tests that were executed before 1930 are considered historical today, as the concrete quality has seen a great improvement in recent years (Aas-Jakobsen, 1970, s. 2).

There are four main categories that were investigated for concrete fatigue. Those four categories are concrete fatigue in compression, in flexure, in tension and fatigue of bond. The following section is a recap of the researchers results within those categories.

3.1.2.1 Concrete fatigue in compression

Between 1903 and 1907 Van Ornum executed tests with 20 000 load repetitions and found that the concrete fatigue strength in compression was about 50% of the concrete's ultimate static strength. Later, in 1934-1936, Graf and Brenner tested specimens with up to 2 000 000 load repetitions and included following variables: Age of the concrete, range of stress, concrete mix, speed of testing, cross section and curing. Their results showed that the fatigue strength is approximately 60 % of the ultimate strength at 2 million load repetitions. They also concluded that the type of specimen, the speed of testing and age of the concrete has little influence on the concrete strength (Aas-Jakobsen, 1970, ss. 2-3).

3.1.2.2 Concrete fatigue in bending

There has been a decent number of tests on concrete fatigue in bending through the 20th century. The main motivation behind these tests has been idealizing the design of concrete road pavements and airport runways (Nordby, 1958). Table 1 Below shows the results of the main tests of concrete fatigue in bending. Remember to consider that not all these tests were executed identical.

Table 1 - Concrete fatigue in bending test results (Aas-Jakobsen, 1970, s. 4)

Test	Fatigue strength in % of static strength	Number of repetitions
Clemmer (1922)	53	10 million
Hatt and Crepps (1925)	55	10 million
Le Camus (1945)	65	1 million
Kesler (1953)	55	10 million
Murdock and Kesler (1958)	60	10 million
McCall (1958)	50	10 million
Hilsdorf and Kesler (1966)	62-68	10 million

Figure 16 below shows Murdock & Kesler’s fatigue test results displayed in a SN-curve.

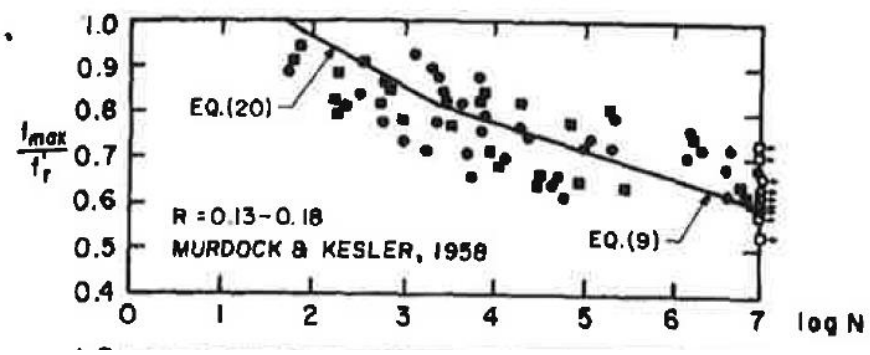

Figure 16 Murdock & Kesler's SN-curve (Hsu, 1981, s. 13)

3.1.2.3 Concrete fatigue in tension

There has been very little testing conducted of concrete fatigue in tension. One reason behind this is that the experiments is especially difficult to perform compared to compression and bending. The best solution to perform tension testing seemed to be gluing the test specimen to the load applying plate (Hordijk, 1991, s. 12). In 1898 De Joly found the fatigue strength in tension to be around 50 % of the ultimate strength. As these tests are outdated, they do not provide anything today except for historical value (Nordby, 1958, s. 203).

3.1.2.4 Concrete fatigue of bond

The bond strength of concrete is the stress that is required for det mortar or the concrete to release from the rebar. There have not been paid much attention to fatigue of bond in concrete. In 1945 Muhlenbruch conducted some testing and found the fatigue strength to be approximately 50 % of the static pull-out strength at 5 million load repetitions (Aas-Jakobsen, 1970, s. 4). On the other hand, Le Camus found that the fatigue strength at 1 000 000 load repetitions was 69 % of the ultimate pull-out strength. Therefore, the results of the testing in this category deviates a lot (Nordby, 1958, s. 204)

3.1.3 Factors affecting fatigue strength

There are various factors affecting the fatigue strength of concrete. In the following section the main factors are described.

3.1.3.1 Rest periods and load frequency

Rest periods between load cycles are found to increase the fatigue life of concrete. Up to five-minute breaks between each load sequence can extend the lifetime. Rest periods of longer than five minutes do not seem to extend the lifespan further. This was discovered by Hilsdorf & Kesler in 1966. Static load periods at low stresses also extend the fatigue life (Ameen & Szymanski, 2006, s. 7).

It has been proven that high load frequencies at high stresses increase fatigue strength. This means that fatigue tests that are performed in a short period of time can show the fatigue strength to be greater than the actual fatigue strength. It is important to have this effect in mind while performing fatigue tests at high frequencies (Ameen & Szymanski, 2006, s. 8).

3.1.3.2 Loading waveform

In 1973 Tepfers, Gørling and Samuelsson tested how fatigue strength is affected by applying the load in different waveforms. They tested sinusoidal, triangular and rectangular waveform. In Figure 17 below, their results are shown.

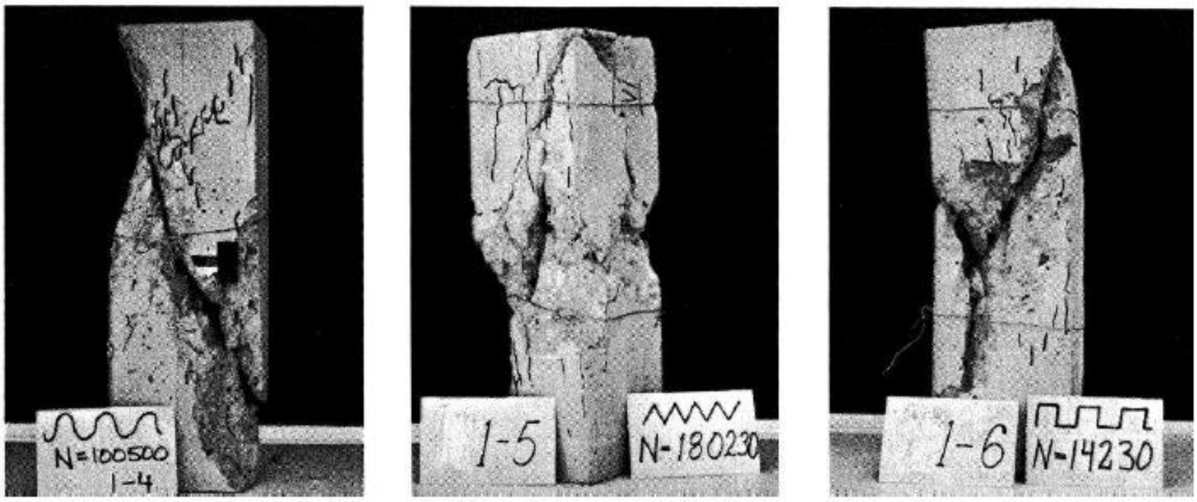

Figure 17 - Failed specimens after exposure to different waveforms. N is the number of cycles to failure (Ameen & Szymanski, 2006, s. 9)

As shown in the figure above, the specimen exposed to rectangular waveform failed at a significant less amount of cycles compared to the two other forms. At the rectangular waveform, the specimen is exposed to higher stresses over a longer period compared to the other forms. It seems that time under high stress is an influencing factor to the fatigue strength.

When a load is applied to concrete, strain is obtained in the material. The velocity of the strain's occurrence variates between the different waveforms. At the rectangular waveform, the max load is applied instantly, so the strain velocity is very high. At the sinusoidal and triangular waveform, the loads are applied gradually. The strain velocity is therefore low in these two waveforms. It seems like the strain velocity the loads cause is a factor that affects the fatigue strength as well (Ameen & Szymanski, 2006, s. 8).

3.1.3.3 Moisture

A high level of moisture in the concrete have been proven to reduce the fatigue life. Several experiments support this hypothesis. At cyclic loadings, concrete with high moisture will have increased creep, which again affects the deformations in the concrete. It is proven by Van Leeuwen & Siemes in 1979 and Waagaard in 1981, that specimens with higher moisture content have reduced fatigue strength in compression and tension. According to the research it is not the water percentage in the concrete that is crucial. It is rather if the concrete is in drying or saturation phase that affects the concrete fatigue strength (Ameen & Szymanski, 2006, ss. 9-10). In Figure 18 below the effect of moisture on concrete fatigue life is displayed. As shown, there is a significant difference between wet and dry concrete.

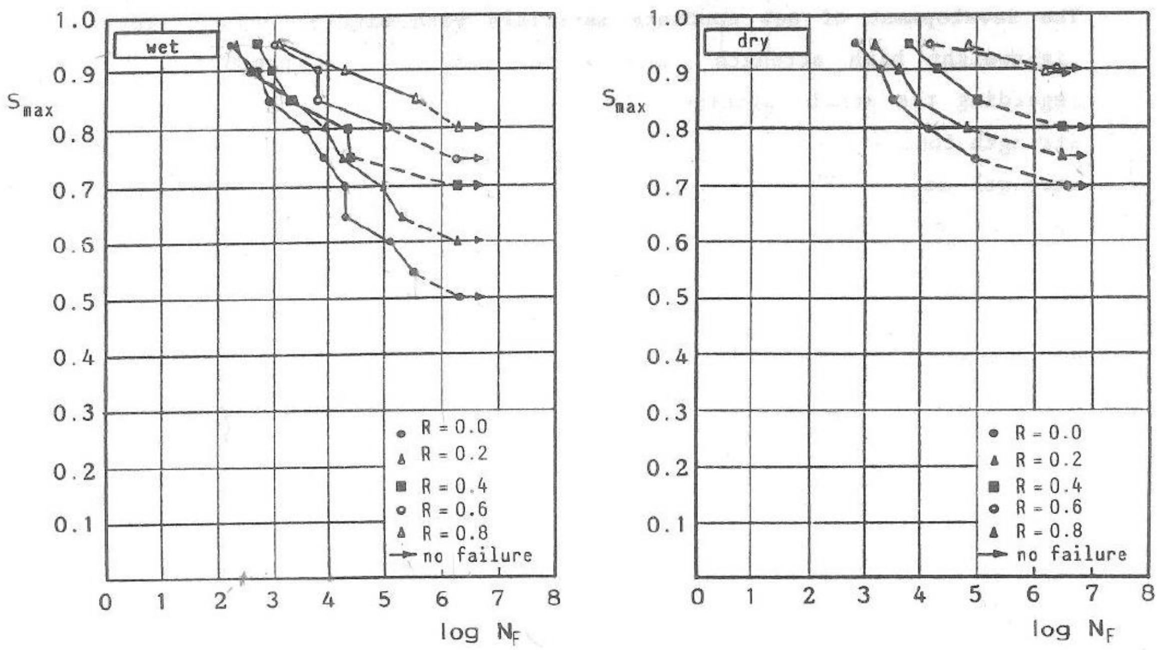

Figure 18 - The variation of fatigue life in wet versus dry concrete (Ameen & Szymanski, 2006, s. 10). R is a stress range defined as R=$\sigma_{min}/\sigma_{max}$.

3.1.3.4 Temperature

In 1990 Ohlsson, Daerga and Elfgren performed fatigue tests in +20ºC and -35ºC and compared the results. They found that low temperatures increase the fatigue strength for concrete with a high moisture content. Their explanation was that the ice formed in the concrete probably contributed in a positive way to the materials resistance against fatigue.

However, they did not test how the fatigue strength would change after affection of freeze-thaw cycles (Ohlsson, Daerga, & Elfgren, 1990).

3.1.3.5 Cement and water containment

If the fatigue strength is displayed in relation to the static strength, the cement and water containment should not affect the fatigue strength of the material. This also applies for the factors hardening properties and age (CEB, 1988).

3.2 Concrete fatigue in standards

Standardization is important to ensure that materials, products, processes and services have high enough quality for the purpose they are intended to have. A standard usually deals with a limited and very specific subject and today there are many different professional groups that must adhere to standards (Standard Norge, 2018). In this thesis, the verifications for concrete fatigue from three different standards have been investigated. These standards are NS-EN 1992-1-1:2004, Model Code 2010 and DNV-OS-C502.

3.2.1 NS-EN 1992-1-1:2004

According to NS-EN 1992-1-1:2004 (Standard Norge, 2018) a fatigue verification should be carried out for structures and structural components which are subjected to regular load cycles. This verification shall be performed separately for concrete and steel.

3.2.1.1 Compression

NS-EN 1992-1-1:2004 have two different methods for verification of concrete fatigue in compression presented in the standard. The first method is based on describing the concrete strength at N=10^6 cycles and is as follows.

A satisfactory fatigue resistance may be assumed for concrete under compression, if the following condition is fulfilled:

$$E_{cd,\max,equ} + 0{,}43\sqrt{1 - R_{equ}} \leq 1$$

Equation 1

Where:

$$R_{\text{equ}} = \frac{E_{\text{cd,min,equ}}}{E_{\text{cd,max,equ}}}$$

Equation 2

$$E_{\text{cd,max,equ}} = \frac{\sigma_{\text{cd,max,equ}}}{f_{\text{cd,fat}}}$$

Equation 3

$$E_{\text{cd,min,equ}} = \frac{\sigma_{\text{cd,min,equ}}}{f_{\text{cd,fat}}}$$

Equation 4

Where:

R_{equ}	is the stress ratio.
$E_{cd,min,equ}$	is the minimum compressive stress level.
$E_{cd,max,equ}$	is the maximum compressive stress level.
$\sigma_{cd,max,equ}$	is the upper stress of the ultimate amplitude for N = 10^6 cycles.
$\sigma_{cd,min,equ}$	is the lower stress of the ultimate amplitude for N= 10^6 cycles.
$f_{cd,fat}$	is the design fatigue strength of concrete.

The design fatigue strength of concrete can be calculated according to the following equation.

$$f_{\text{cd,fat}} = k_1 \beta_{\text{cc}}\left(t_0\right) f_{\text{cd}}\left(1 - \frac{f_{\text{ck}}}{250}\right)$$

Equation 5

Where:

k_1	is a factor that is set equal to 0,85 when the whole section is under compression.
$\beta cc(t_0)$	is a coefficient for concrete strength at first load application.
t_0	is the time of the start of the cyclic loading on concrete in days.
f_{cd}	is the design compressive strength.
f_{ck}	is characteristic compressive cylinder strength of concrete at 28 days.

$\beta cc(t_0)$ can be found by the following equation.

$$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\frac{28}{t} \right)^{1/2} \right] \right\}$$

Equation 6

Where:

s	is a coefficient which depends on the type of cement:
	= 0,20 for cement class R
	= 0,25 for cement class N
	= 0,38 for cement class S
t	is the age of the concrete in days.
exp{ }	is the same as $e^{()}$.

f_{cd} can be calculated by the following equation.

$$f_{cd} = \alpha_{cc}\, f_{ck} / \gamma_C$$

Equation 7

Where:

α_{cc}	is the coefficient taking account of long-term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied. The coefficient is determined in NA.3.1.6 and is set equal to 0,85.
f_{ck}	is characteristic compressive cylinder strength of concrete at 28 days.
γ_C	is the partial safety factor for concrete and is found in Table 2 below.

Table 2 – Values for the partial safety factors γ.

Design situations	γ_C for concrete	γ_S for reinforcing steel	γ_S for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Fatigue	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

The second method is described as followed.

A satisfactory fatigue resistance may be assumed for concrete under compression, if the following condition is fulfilled:

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0{,}5 + 0{,}45\frac{\sigma_{c,min}}{f_{cd,fat}} \begin{cases} \leq 0{,}9 \text{ for } f_{ck} \leq 50 \text{ MPa} \\ \leq 0{,}8 \text{ for } f_{ck} > 50 \text{ MPa} \end{cases}$$

Equation 8

Where:

$\sigma_{c,max}$ is the maximum compressive stress at a fiber under the frequent load combination

$\sigma_{c,min}$ is the minimum compressive stress at the same fiber where $\sigma_{c,max}$ occurs.

If the compression zone is also exposed to shear force, the concrete strength $f_{cd,fat}$ should be reduced by the strength reduction factor v.

$$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right] \qquad (f_{ck} \text{ in MPa})$$

Equation 9

3.2.1.2 Shear

For members not requiring design shear reinforcement for the ultimate limit state it may be assumed that the concrete resists fatigue due to shear effects where the following apply:

$$\text{for } \frac{V_{\text{Ed,min}}}{V_{\text{Ed,max}}} \geq 0:$$

$$\frac{|V_{\text{Ed,max}}|}{|V_{\text{Rd,c}}|} \leq 0{,}5 + 0{,}45 \frac{|V_{\text{Ed,min}}|}{|V_{\text{Rd,c}}|} \begin{cases} \leq 0{,}9 & \text{up to } C50/60 \\ \leq 0{,}8 & \text{greater than } C55/67 \end{cases}$$

Equation 10

$$\text{for } \frac{V_{\text{Ed,min}}}{V_{\text{Ed,max}}} < 0:$$

$$\frac{|V_{\text{Ed,max}}|}{|V_{\text{Rd,c}}|} \leq 0{,}5 - \frac{|V_{\text{Ed,min}}|}{|V_{\text{Rd,c}}|}$$

Equation 11

Where:

$V_{Ed,max}$ is the design value of the maximum applied shear force under frequent load combination.

$V_{Ed,min}$ is the design value of the minimum applied shear force under frequent load combination in the cross-section where $V_{Ed,max}$ occurs.

$V_{Rd,c}$ is the design value for shear-resistance and is calculated in appendix B.

3.2.2 Model code 2010

The rules found in Model code 2010 (FIB, 2012) do apply for the entire service life of concrete structures.

3.2.2.1 Fatigue in compression

There are three different methods for verification of concrete fatigue in compression in this standard. The methods are divided according to how accurate they are. The first method is the least accurate and should therefore only be used if the requirement for precision is low. Furthermore, method number two is medium precise, while method number three has the highest precision. The higher the precision of the method, the more cumbersome the calculations will become.

The first method is called "Level 1 of Approximation: the simplified procedure". This procedure is only applicable to structures subjected to a limited number ($<10^8$) of stress cycles.

Detailed fatigue design needs not be carried out if the maximum calculated stresses under the frequent combination of loads, $\sigma_{c,max}$ (compression), satisfy the following criteria:

Compression:

$$\gamma_{Sd}\sigma_{c,max}\eta_c \leq 0.45 f_{cd,fat}$$

Equation 12

Where:

$\sigma_{c,max}$ is the maximum compressive stress.

η_c is the averaging factor of concrete stresses in the compression zone considering the stress gradient.

$f_{cd,fat}$ is the design fatigue reference strength for concrete in compression.

γ_{Sd} is a partial safety factor accounting for model uncertainty with value 1,05.

$$\eta_c = \frac{1}{1.5 - 0.5|\sigma_{c1}|/|\sigma_{c2}|}$$

Equation 13

Where:

$|\sigma_{c1}|$ is the minimum absolute value of the compressive stress within a distance of 300mm from the surface under the relevant load combination of actions.

$|\sigma_{c2}|$ is the maximum absolute value of the compressive stress within a distance of 300mm from the surface under the same load combination as that for which $|\sigma_{c1}|$ was determined.

$$f_{cd,fat} = 0.85\beta_{cc}(t)\left[f_{ck}\left(1-\frac{f_{ck}}{25f_{ck0}}\right)\right]/\gamma_{c,fat}$$

Equation 14

Where:

$\beta cc(t)$ is the coefficient which depends on the age t of the concrete in days when fatigue loading starts. The value is found by applying equation 6.

f_{ck0} =10 MPa (reference strength).

f_{ck} is characteristic compressive cylinder strength of concrete at 28 days.

$\gamma_{c,fat}$ partial safety factor for concrete material properties under fatigue loading and has a value of 1,5.

The second method is called “Level 2 of Approximation: verification by means of a single load level”.

The fatigue requirements under cyclic loading will be met if the required lifetime (number of cycles) is lower than or equal to the number of cycles to failure:

$n \leq N$

Where:

n is the foreseen number of cycles during the required design life

N is the number of resisting stress cycles, to be calculated from the fatigue strength functions given below.

For $S_{cd,min}$ > 0,8, the value of $S_{cd,min}$ can be assumed to be 0,8.

For $0 \leq S_{cd,min} \leq 0,8$, the equations below apply.

$$\log N_1 = \frac{8}{Y-1}\cdot(S_{cd,\max}-1)$$

Equation 15

$$\log N_2 = 8 + \frac{8 \cdot \ln(10)}{Y - 1} \cdot (Y - S_{cd,\min}) \cdot \log\left(\frac{S_{cd,\max} - S_{cd,\min}}{Y - S_{cd,\min}}\right)$$

Equation 16

Where:

$$Y = \frac{0.45 + 1.8 \cdot S_{cd,\min}}{1 + 1.8 \cdot S_{cd,\min} - 0.3 \cdot S_{cd,\min}^2}$$

Equation 17

Where:

$$\text{(a) if } \log N_1 \leq 8 \text{, then } \log N = \log N_1 ;$$

$$\text{(b) if } \log N_1 > 8 \text{, then } \log N = \log N_2 ;$$

Where:

$S_{cd,min}$ is the minimum compressive stress level.

$$S_{cd,min} = \frac{\gamma_{Ed} \sigma_{c,min} \eta_c}{f_{cd,fat}}$$

Equation 18

$S_{cd,max}$ is the maximum compressive stress level.

$$S_{cd,max} = \frac{\gamma_{Ed} \sigma_{c,max} \eta_c}{f_{cd,fat}}$$

Equation 19

γ_{Ed} is assumed to be 1,1 according to chapter 4.5.2.3 in Model Code 2010.

$\sigma_{c,max}$ is the largest compressive stress in the compression zone at a distance no more than 300 mm away from the surface.

$\sigma_{c,min}$ is the lowest compressive stress in the compression zone at a distance no more than 300 mm away from the surface.

The third method is called “Level 3 of approximation: verification by means of a spectrum of load levels”.

This method takes account of the required service life, the load spectrum (which is divided into j blocks) and the characteristic fatigue strength functions.

Fatigue damage D is calculated using the Palmgren-Miner summation.

$$D = \sum_{i=1}^{j} \frac{n_{Si}}{N_{Ri}}$$

Equation 20

Where:

D is fatigue damage.

n_{Si} denotes the number of acting stress cycles associated with the actual stress level.

N_{Ri} denotes the number of resisting stress cycles at a given stress level. Can be calculated by the method described in level II for each stress block.

The fatigue requirement will be satisfied if $D \leq D_{lim}$.

For concrete structures onshore, D_{lim} has a value of 1.

3.2.2.2 Shear

- For members not requiring design shear reinforcement

The fatigue requirements will be met, if under cyclic loading the number of cycles corresponding to the required service life is smaller than or equal to the numbers of cycles to failure:

$n \leq N$

N should be calculated from the fatigue strength functions given below.

$$\log N = 10(1 - V_{max} / V_{ref})$$

Equation 21

Where:

V_{max} is the maximum shear force under the relevant representative values of permanent loads including prestress and maximum cyclic loading.

V_{ref} = $V_{Rd,c}$ and is the design value for the shear resistance. $V_{Rd,c}$ has a value of 1719,8 kN and is calculated in appendix B.

3.2.3 DNV-OS-C502

DNV-OS-C502 (DNV-GL, 2007) is a standard for offshore concrete structures, but it is also applicable for onshore structures.

According to the standard, all stress fluctuations imposed during the life of the structure which are significant with respect to fatigue evaluation shall be considered when determining the long-term distribution of stress range.

The stresses due to cyclic actions may be arranged in stress-blocks, each with constant amplitude and a corresponding number of stress cycles, n_i. A minimum of 8 blocks is recommended.

The design criterion is:

$$\sum_{i=1}^{k} n_i / N_i < \eta$$

Equation 22

Where:

k is the number of stress-blocks.

n_i is the number of cycles in stress-block i.

N_i is the number of cycles with constant amplitude which causes fatigue failure.

η is the cumulative damage ratio and is based on the access for inspection and repair. The value can be found in Table 3 - Cumulative Damage Ratio (η)Table 3 below.

Table 3 - Cumulative Damage Ratio (η)

Table M1 Cumulative Damage Ratios (η)		
No access for inspection and repair	*Below or in the splash zone*[1)]	*Above splash zone* [2)]
0.33	0.5	1.0

1) In typical harsh environment (e. g. the North Sea or equivalent) structural details exposed to seawater in the splash zone are normally to be considered to have no access for inspection and repair, i.e. the cumulative damage ratios is to be reduced to 0.33.
2) For reinforcement, which cannot normally be inspected and repaired; the cumulative damage ratio for reinforcement above splash zone is reduced to 0.5.

3.2.3.1 Compression

The design life of concrete subjected to cyclic stresses may be calculated from the following equation:

$$\log_{10} N = C_1 \left(1 - \sigma_{max}/f_{rd}\right) / \left(1 - \sigma_{min}/f_{rd}\right)$$

Equation 23

Where:

f_{rd} is the compressive strength for the type of failure in question. For concrete subjected to compression, f_{rd} is taken equal f_{cd}.

σ_{max} is the numerically largest compressive stress.

σ_{min} is the numerically least compressive stress.

C_1 is a factor that shall be taken as 12.0 for structures onshore.

If the calculated design life log N is larger than the value of X given by the expression.

$$X = C_1 / \left(1 - (\sigma_{min}/f_{rd}) + 0.1 \cdot C1\right)$$

Equation 24

The design life may be increased further by multiplying the value of log10 N by the factor C_2 where this is taken as.

$$C_2 = (1 + 0.2 (\log_{10} N - X)) > 1.0$$

Equation 25

3.2.3.2 Shear

- For members not requiring design shear reinforcement

The design life at tensile failure of concrete without shear reinforcement can be calculated in accordance with following equation.

$$\log_{10} N = C_1 (1 - V_{max}/V_{cd}) / (1 - V_{min}/V_{cd})$$

Equation 26

For those stress-blocks where the shear force changes sign, the following equation for log N should be used.

$$\log_{10} N = C_1 (1 - V_{max}/V_{cd}) / (1 + V_{min}/V_{cd})$$

Equation 27

If the shear force changes sign the calculation shall, if necessary, be performed with both the positive and negative values for V_{max} and V_{min} respectively in the equation above.

The factor C_1 shall be taken as

12.0 for structures onshore where the shear force does not change sign.

10.0 for structures onshore where the shear force changes sign.

4 Turbine dimensions and forces

4.1 Foundation and Turbine dimensions

The dimensions of the foundation that is being controlled for concrete fatigue in this thesis are provided by Anton Magne Gjørven at Norconsult. He has previously participated in several wind turbine foundation projects in Norway. The foundation is a circular gravity foundation with concrete quality of B45. A model of the foundation with corresponding dimensions are shown in Figure 19 below.

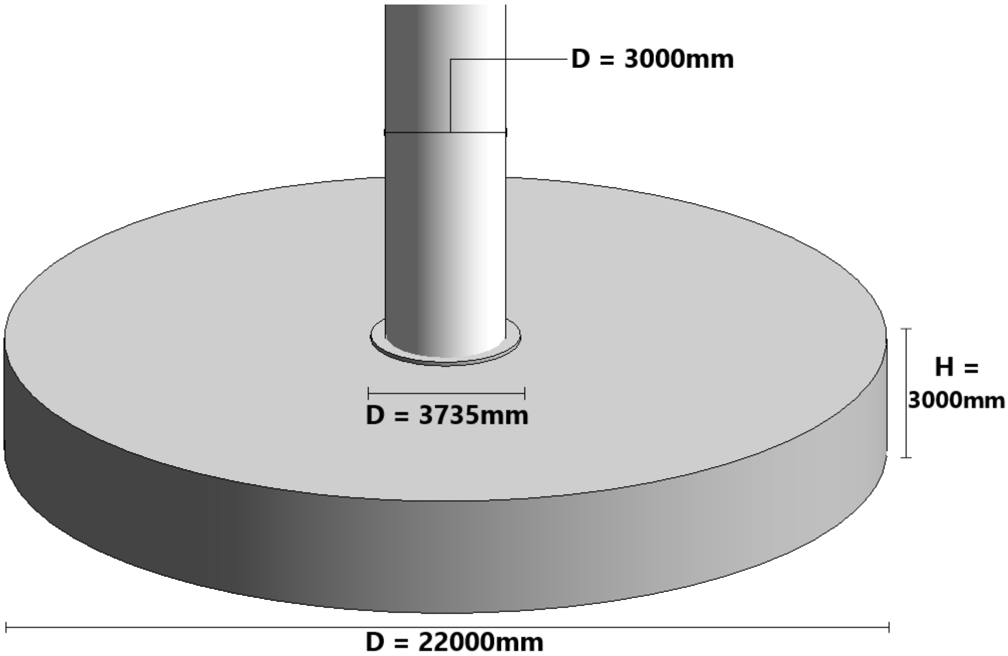

Figure 19 – 3D model of the foundation with corresponding dimensions. The foundation is modeled in Autodesk Revit.

A General Electric 5MW-158 turbine has been chosen for calculation of the forces that act on the foundation. The turbine has a rotor diameter of 158 meter which results in a swept area of over 19 600 m^2 (General Electric, 2020). The tower has a height of 100 meter, a diameter of 3 meter and a wall thickness of 65 mm. Below, in Figure 20 a model of the turbine with dimensions is displayed.

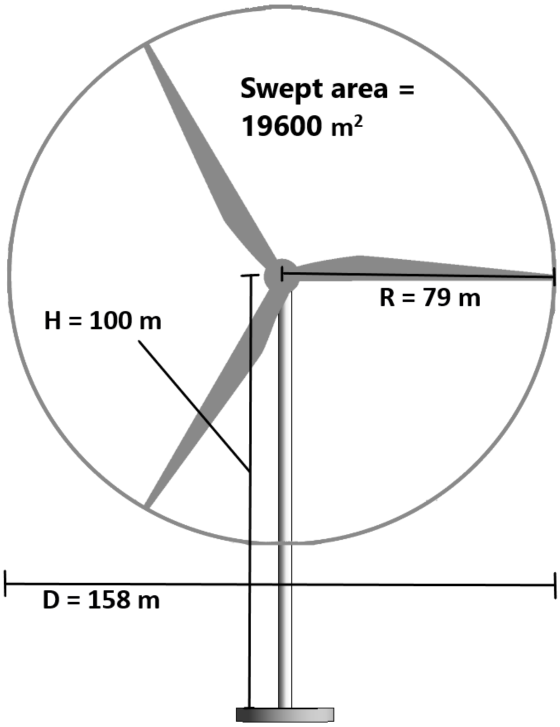

Figure 20 - Model of the turbine with dimensions.

One GE 5MW-158 turbine can produce up to 20 GWH per year and power up to 5200 residential homes in Europe (General Electric, 2020). Figure 21 below shows a GE 5MW-158 turbine located in the Netherlands.

Figure 21 - GE 5MW-158 turbine located in the Netherlands (General Electric, 2020)

General Electric do not want detailed information about their products to be published in order to avoid competing companies to copy their designs. Therefore, a close approximation of the GE 5MW-158 turbine's dimensions and weights have been used for calculations in this thesis. This will however lead to very little impact on the conclusions of the concrete fatigue analysis of the foundation. The values that are adjusted are the weights of the blades, hub and nacelle, thickness and diameter of the tower and the blade dimensions.

The mass of the nacelle, blades and hub are estimated values but are based on a similar turbine type with 110m hub height, which is safer in terms of design. The mass of the tower is calculated by multiplying the total volume of the tower by the density of steel.

$$M_{tower} = A_{tower} \cdot H_{tower} \cdot \rho$$

Equation 28

Where:

M_{tower} is the mass of the tower

A_{tower} is the area of the tower cross-sections

H_{tower} is the height of the tower

ρ is the density of steel and equals 8050 kg/m^3

$$M_{tower} = 0{,}6\ m^2 \cdot 100\ m \cdot 8050\ kg/m^3 = 483\ 000\ kg$$

Equation 28

In Table 4 below the total mass of all the turbine parts is summed.

Table 4 - Total mass of turbine

Part	Mass (kg)
Tower	483 000
Nacelle	84 000
Blades	61 800
Hub	37 000
Total	**665 800**

An anchorage plate has been chosen as the solution to connect the tower to the foundation. The plate is made of S500 quality steel with a thickness of 100 mm. The outer radius of the plate is 1867,5 mm and has a width of 800 mm. The forces acting from the tower wall is assumed to be evenly distributed throughout the anchorage plate given that the plate itself has adequate capacity.

4.2 Wind data

Recorded wind data over a one-year period is used to calculate the fatigue loadings the wind turbine foundation is expected to be exposed to. The wind data used are measured at Kvitneset which is located south-west of Molde in the western part of Norway. Kvitneset is located on the coast, so it is considered a windy area. The map in Figure 22 below show the location of Kvitneset.

Figure 22 - The location of Kvitneset marked on a map of southern Norway (Google Maps, 2020)

The wind measurements were conducted by Kjeller Vindteknikk in collaboration with Statens Vegvesen. A 100-meter-high modern measuring tower were used to perform the measurements. The tower base is located 6 meters above sea level in an open coastal landscape. Figure 23 below displays the measuring tower at Kvitneset.

Figure 23 - The measuring tower at Kvitneset (Google Maps, 2020).

The tower continuously recorded wind speed and wind direction at three different heights, 92.5 meters, 71.5 meters and 44.5 meters. The measurements were logged for every 10-minute interval. The parameters measured for each interval were average wind speed, standard deviation, minimum wind speed and maximum wind speed. The wind data were measured continuously from 01.01.2019 to 31.12.2019. There was one day with maintenance on the tower where no recordings were made. The total amount of measured ten-minute intervals is 52402 throughout the whole year. Table 5 below is an excerpt of the first hour of wind measurements and Figure 24 further below is a plot of all the measured maximum values at 92,5 meters.

Table 5 - Logged wind data of the first recorded hour at Kvitneset

		Wind speed (m/s)											
		92,5m				71,5m				44,5m			
Date	Time	AVG	STD	MIN	MAX	AVG	STD	MIN	MAX	AVG	STD	MIN	MAX
2019-01-01	00:00:00	10.39	2.80	3.72	17.64	10.42	2.96	3.22	17.54	10.23	3.85	2.18	17.83
2019-01-01	00:10:00	7.03	2.06	2.32	13.89	6.57	1.77	2.64	12.26	5.70	1.54	2.16	9.46
2019-01-01	00:20:00	8.32	2.72	2.23	15.80	7.15	2.40	0.67	13.76	6.54	2.18	1.76	14.16
2019-01-01	00:30:00	9.44	3.20	2.36	15.79	8.20	2.68	3.03	15.53	8.18	2.21	2.91	13.89
2019-01-01	00:40:00	10.09	2.49	4.45	16.72	8.63	2.17	4.26	14.10	8.21	2.30	3.40	14.14
2019-01-01	00:50:00	10.54	2.78	3.50	16.58	9.16	2.29	4.13	15.05	7.58	2.22	2.18	16.30
2019-01-01	01:00:00	10.18	2.39	4.79	17.30	9.21	2.29	4.42	15.69	7.72	2.18	3.13	13.76

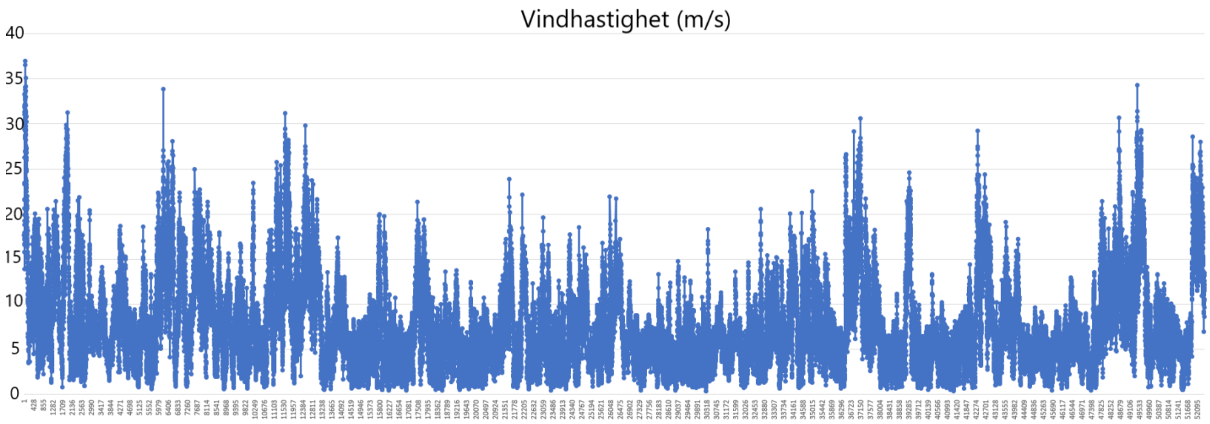

Figure 24 - A plot of all the measured maximum values at 92,5-meter height

Because the wind power fluctuates depending on the season, wind data for an entire year is ideal to map the wind situation. Wind speeds vary somewhat from year to year, but it will still provide a very good estimate for annual winds.

During the measurement period, the maximum measured wind speed was 36.99 m/s and the average wind speed was 7,67 m/s at 92,5 meters. The lowest recorded max value of a 10-minute interval at 92,5 meters was 0,34 m/s.

4.3 Load factors and load combinations

IEC 61400-1:2019 (IEC, 2019) is a standard that describes design requirements for wind turbines. This standard is used for obtaining the load factors and load combinations. Chapter 7.6.3 in the standard describes fatigue failure. According to this chapter the load combination for fatigue calculation should be as shown below.

$$\gamma \cdot DL + \gamma \cdot WL$$

Where:

DL is dead load

WL is wind load

γ is the partial safety factor for fatigue loads and has a value of 1,0

There are some exceptions from the load combination above regarding the NS-EN 1992-1-1:2004. In the second method for fatigue in compression (Equation 8), the maximum compressive stress is required to be in the frequent load combination. For the calculation of fatigue in shear, the shear force is also required to be in the frequent load combination (Equation 10 and 11) . For these specific cases the load combination will be as shown below.

$$\gamma \cdot DL + \gamma \cdot \Psi_1 \cdot WL$$

Where:

DL is dead load

WL is wind load

γ is the partial safety factor for fatigue loads and has a value of 1,0

Ψ_1 is a factor found in Table 6 below and equals 0,2 for wind loads.

Table 6 - Values for ψ - factors for structures. Retrieved from Table NA.A1.1 in NS EN 1990:2002 National Annex (Standard Norge, 2016)

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

4.4 Loads on the turbine

Loads that affect fatigue in a wind turbine foundation are wind load on the rotor, wind load on the tower and own load of the turbine. There are also other loads that can influence a wind turbine such as earthquake loads, snow loads and ice loads. Earthquake loads are chosen to be disregarded in this task. Snow loads and ice loads normally have very little impact on wind turbine structures and are also chosen to be disregarded in this thesis.

According to NS-EN 1993-1-1:2005 torsion can be disregarded as the tower has a closed cross section (Standard Norge, 2015, s. 53).

4.4.1 Wind load

Air moving in relation to an object exerts force on the object. The pressure applied to the object is dependent on the windspeed, in addition to the air pressure and the shape and size of the object. Wind load is a natural load with great variation in its impact on an object. Cyclic loads are applied to the object as the loads alternate with the variable wind speed (Harstveit, 2009).

In the calculations of the wind loadings, two assumptions have been made. The wind direction is assumed to be constant and that the wind is acting perpendicular to the structural lateral surface.

To configure the fatigue loadings due to the measured winds, formulas for the wind load is needed as a function of the wind speed. These formulas are derived in the next subsections.

For each measured ten-minute interval, the maximum wind speed at the height of 92,5 meter have been chosen as input in the calculations. In terms of design, the maximum wind speed will result in the most unfavorable stresses in the foundation.

For calculation of the wind load on the rotor, the wind speed at hub height is required in the formula which is 100 meter above ground level. The wind measurements were made at 92,5 meters which will lead to a deviation in the results. There is expected to be very little difference in the wind speed at 92,5 compared to 100 meters, therefore the impact on the results will be small. On the contrary, the load of the wind on the tower has been calculated by using the wind speed at 92,5 meters for the whole tower.

4.4.1.1 Wind load on the rotor

The wind force acting on a turbine blade can be calculated by the following equation.

$$F_R = \frac{1}{2} \cdot \rho \cdot v^2 \cdot A \cdot C_D$$

Equation 29

Where:

ρ is the density of air which is set to 1,225 kg/m^3

v is the wind speed in m/s at hub height

C_D is the drag coefficient of the blades

A is the area of the blade exposed to wind load perpendicular to the rotor.

(Det Norske Veritas, Wind Energy Department, & Risø National Laboratory, 2002)

The total area that is exposed to wind load perpendicular to the rotor for one blade is 131 m^2 and is calculated in appendix 1.

The value of C_D is dependent on the wind angle, blade shape, blade length and the twisting of the blade. As the calculation of the exact value of C_D is a very complex procedure, the maximum possible value of C_D is therefore assumed for all the measurements. This assumption will give more unfavorable loads, so it is safer in terms of design. $C_{D,max}$ for a turbine blade is calculated by the formula below.

$$C_{D,max} = 1{,}11 + 0{,}018 \cdot AR$$

Equation 30

Where:

AR is the aspect ratio of the blade and can be calculated by the following equation.

$$AR = \frac{L}{C}$$

Equation 31

Where:

L is the length of the blade

C is the length of a representative chord

The value of AR for the blade is calculated in appendix A and has a value of 17,86.

This results in the following value of C_D.

$$C_D = 1.11 + 0,018 \cdot 17,86 = 1,4315$$

Equation 30

The formula for wind load on all three blades is then expressed as follows.

$$F_R = 3 \cdot \frac{1}{2} \cdot 1,225 \cdot v^2 \cdot 131 \cdot 1,4315 = 344,58 \cdot v^2 \text{ (N)}$$

Equation 29

Figure 25 below show the wind load from the rotor acting on the tower.

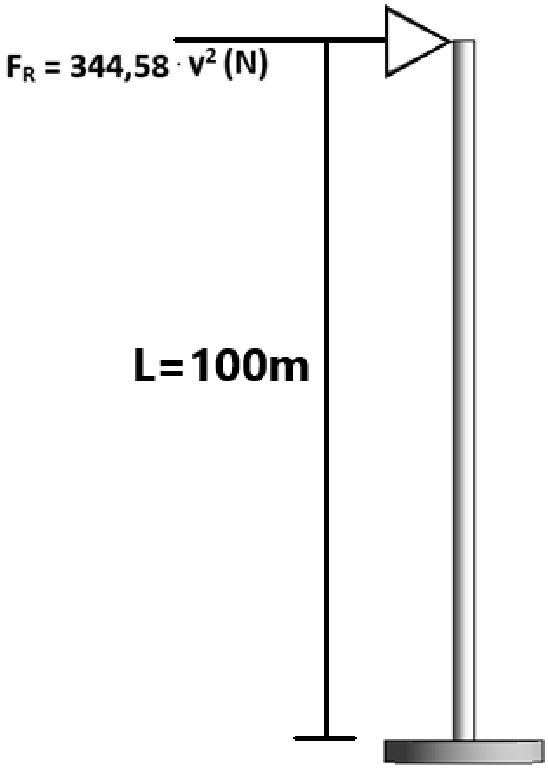

Figure 25 – Wind load from rotor. The tower is modelled in Autodesk Revit.

4.4.1.2 Wind load on the tower

The formula for calculation of wind force on the tower is described below.

$$F_T = \frac{1}{2} \cdot \rho \cdot v^2 \cdot d \cdot h \cdot C_D$$

Equation 32

Where:

ρ is the density of air which is set to 1,225 kg/m^3

v is the wind speed in m/s

d is the diameter of the tower

h is the height of the tower

C_D is the drag coefficient of the tower

(Det Norske Veritas, Wind Energy Department, & Risø National Laboratory, 2002)

The drag coefficient of the tower is found in the same way as the drag coefficient of a cylinder. The coefficient is dependent on the cylinder length/width ratio, shape and Reynolds number. The tower has a length/width ratio of 33,3 which results in a C_D value of approximately 0,95. The value of C_D is set to 1,20 for insurance as this is the maximum value of C_D for a cylinder (Blevins , 2013).

The formula for wind load on the tower is then expressed as follows.

$$F_T = \frac{1}{2} \cdot 1{,}225 \cdot v^2 \cdot 3 \cdot 100 \cdot 1{,}20 = 220{,}5 \cdot v^2 \text{ (N)}$$

Equation 32

This is the resultant of the wind force acting on the center of the tower. By dividing the resultant force with the tower length, the wind load is obtained as a uniformly distributed load acting parallelly with the tower.

$$F_T = \frac{220{,}5 \cdot v^2}{100} = 2{,}205 \cdot v^2 \text{ (N/m)}$$

Equation 32

In Figure 26 below both the resultant wind load and the uniformly distributed load on the tower is displayed.

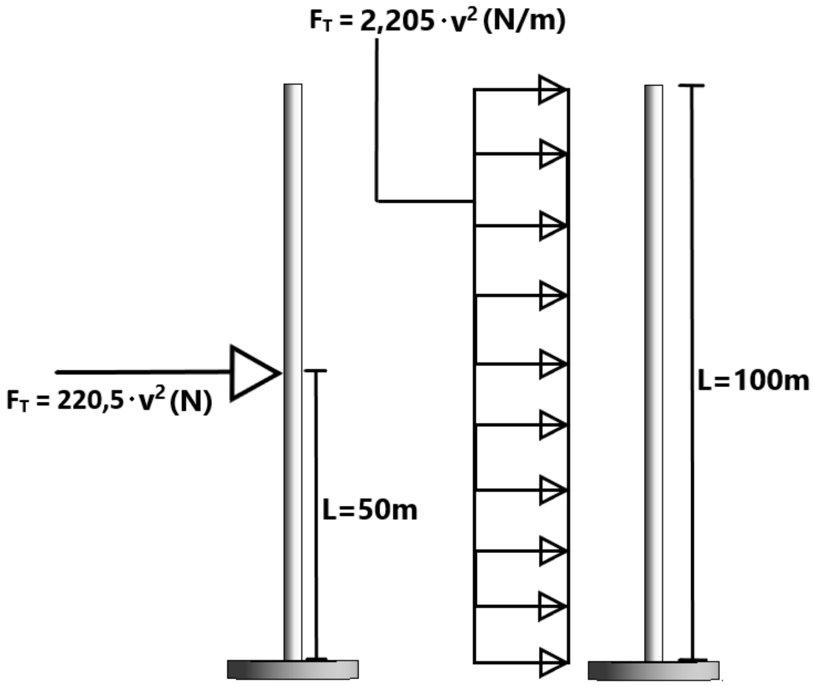

Figure 26 - Model of the wind load acting on the tower. The towers are modelled in Autodesk Revit.

4.4.3 Total forces from tower

In total, there will be three forces acting from the tower. Those three forces are axial force from the self-weight of the turbine, shear force from the wind loads and bending moment from wind loads. Figure 27 below shows the forces from the tower acting on the foundation.

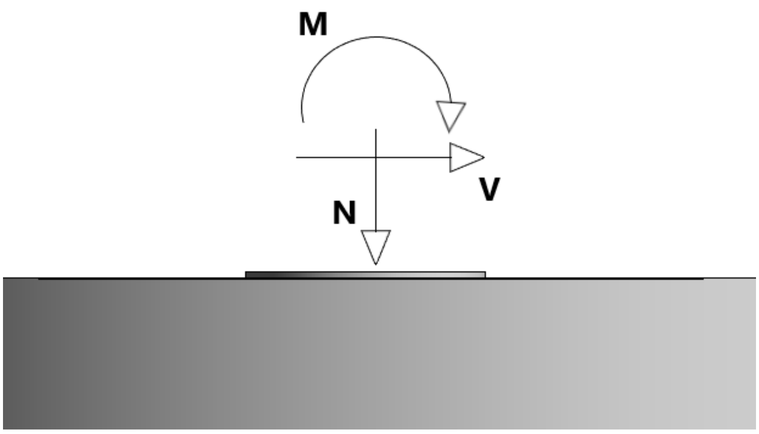

Figure 27 - Forces acting on the foundation

The axial force from the turbine acting on the foundation is calculated by multiplying the weight of the turbine with the gravitational acceleration.

$$N = 665\ 800\ kg \cdot 9{,}81\ m/s^2 = 6531{,}50\ kN$$

Equation 33

The axial force is a permanent load and will not change throughout the following fatigue calculations.

The shear force from the tower is calculated by summing the resultant of the wind forces on both the rotor and the tower.

$$V = 344{,}58 \cdot v^2 + 220{,}5 \cdot v^2 = 565{,}08 \cdot v^2 \text{ (N)}$$

Equation 34

The maximum value of the shear force that occurred at the wind speed of 36,99 m/s was 773,18 kN. The minimum value of the shear force that occurred at the wind speed of 0,34 m/s was 0,0653 kN.

The bending moment acting from the tower is calculated by adding the bending moments from the wind force on the rotor to the wind force on the tower. Each of the bending moments is found by multiplying the resultant force by the distance between where the force is acting and the foundation.

$$M = 344{,}58 \cdot v^2 \cdot 100 \cdot 10^{-3} + 220{,}5 \cdot v^2 \cdot 50 \cdot 10^{-3} = 45{,}483 \cdot v^2 \text{ (kNm)}$$

Equation 35

The maximum value of the bending moment that occurred at the wind speed of 36,99 m/s was 62232,57 kNm. The minimum value of the bending moment that occurred at the wind speed of 0,34 m/s was 5,2578 kNm.

4.4.4 Stress applied to the concrete

When bending moment is applied to the foundation, it is exposed to both compressive and tensile stresses. As concrete do not have any significant strength for tensile stresses, it is assumed that the reinforcement will absorb the tensile stresses. Therefore, only the compressive stresses will be necessary to control for concrete fatigue.

The forces from the tower are applied to the foundation through the anchorage plate. The total compressive stress is obtained by summing the stress from the axial force and the stress from the bending moment. The formula for the compressive stress in the concrete is described as follows.

$$\sigma_{\mathrm{c}} = \frac{N}{A} + \frac{M}{W}$$

Equation 36

Where:

N is the axial force

A is the area of the anchorage plate

M is the bending moment

W is the elastic section modulus of the anchorage plate

The area A of the anchorage plate is calculated as follows.

$$A_{plate} = \pi \cdot (r_1^2 - r_2^2) = \pi \cdot (1867{,}5^2 - 1067{,}5^2) = 7376459{,}55 \text{ mm}^2$$

Equation 37

Where:

r_1 is the outer radius of the plate

r_2 is the inner radius of the plate

The elastic section modulus W of the anchorage plate is calculated as follows.

$$W_{plate} = \frac{\pi}{4 \cdot r1} \cdot (r_1^4 - r_2^4) = \frac{\pi}{4 \cdot 1867{,}5} \cdot (1867{,}5^4 - 1067{,}5^4) = 4569171110 \text{ mm}^3$$

Equation 38

Where:

r_1 is the outer radius of the plate

r_2 is the inner radius of the plate

$\sigma_{c,min}$ is the lowest obtained compressive stress in the concrete and occur at the lowest wind speed. The lowest value of σ_c at the wind speed of 0,34 m/s is calculated below.

$$\sigma_{c,min} = \frac{6531500}{7376459{,}551} + \frac{5{,}2578 \cdot 10^6}{4569171110} = 0{,}8866 \text{ MPa}$$

Equation 36

$\sigma_{c,max}$ is the highest obtained compressive stress in the concrete and occur at the highest wind speed. The highest value of σ_c at the wind speed of 36,99 m/s is calculated below.

$$\sigma_{c,max} = \frac{6531500}{7376459{,}551} + \frac{62232{,}57 \cdot 10^6}{4569171110} = 14{,}5056 \text{ MPa}$$

Equation 36

In the next chapter σ_c for each load cycle is required in the calculations. The general formula for σ_c is shown below.

$$\sigma_c = \frac{6531500}{7376459{,}551} + \frac{45{,}483 \cdot v^2 \cdot 10^6}{4569171110} \text{ (MPa)}$$

Equation 36

Table 7 below displays the calculation of the forces and σ_c applied to the foundation for the first- and last five wind measurements. The calculations were made for all wind measurements.

Table 7 - The calculated forces and concrete compression stress for the first- and last five measurements

Measurement number	Wind speed (m/s)	V (kN)	M (kNm)	σ_c from M (Mpa)	σ_c from M+N (Mpa)
1	17,64	175,84	14152,93	3,097	3,983
2	13,89	109,02	8775,13	1,921	2,806
3	15,80	141,07	11354,37	2,485	3,370
4	15,79	140,89	11340,01	2,482	3,367
5	16,72	157,97	12715,15	2,783	3,668
…	…	…	…	…	…
52398	11,79	78,55	6322,32	1,384	2,269
52399	13,43	101,92	8203,54	1,795	2,681
52400	12,57	89,29	7186,54	1,573	2,458
52401	13,16	97,86	7877,00	1,724	2,609
52402	13,44	102,07	8215,76	1,798	2,684

5 Verification of concrete fatigue

According to DNV-GL a wind turbine experience 1 000 000 000 or 10^9 cycles during a typical 20 years lifetime (Gessert, Nguyen, & Rogers, 2018). This means that in one year the foundation experiences up to $5 \cdot 10^7$ cycles. It is crucial to control a structure for concrete fatigue when it is exposed to a such great amount of cycles. In this chapter, the verification of concrete fatigue for the wind turbine foundation according to the three standards previously reviewed will be calculated.

By analyzing an arbitrary wind turbine foundation in FEM-design, it is found that the largest compressive stresses and shear forces in the foundation will act right under the anchorage plate. Therefore, this is the only area that is necessary to be controlled for concrete fatigue as the whole foundation is exposed to the same number of load cycles.

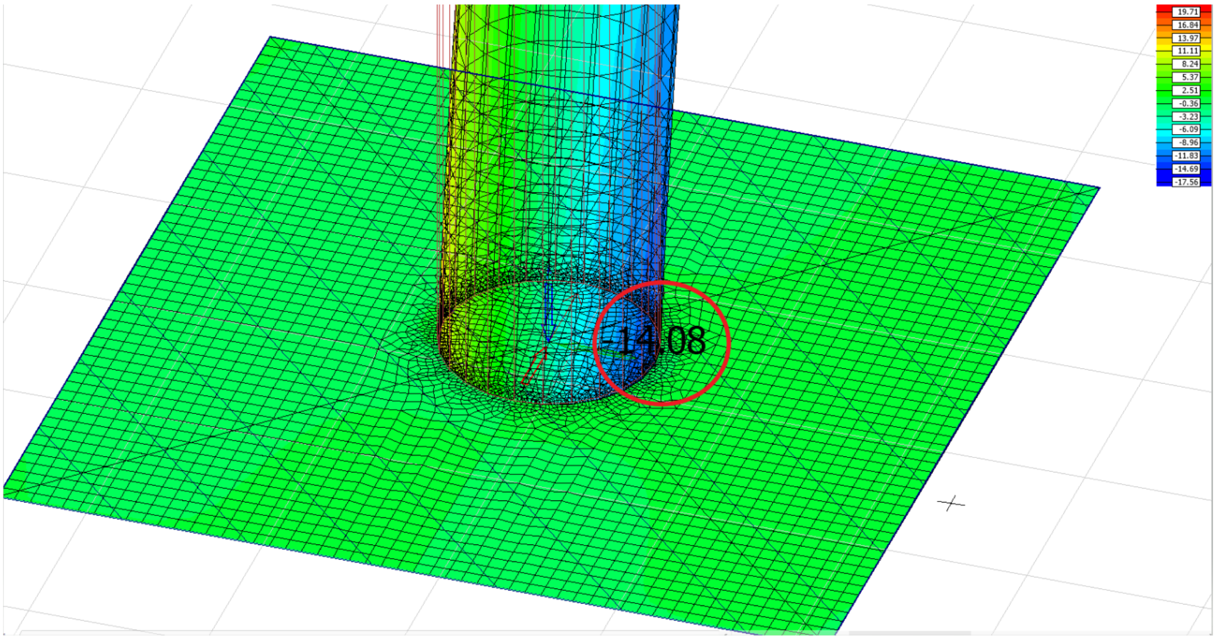

Figure 28 below displays the analyzed foundation. The red circle indicates where the maximum stress occurs.

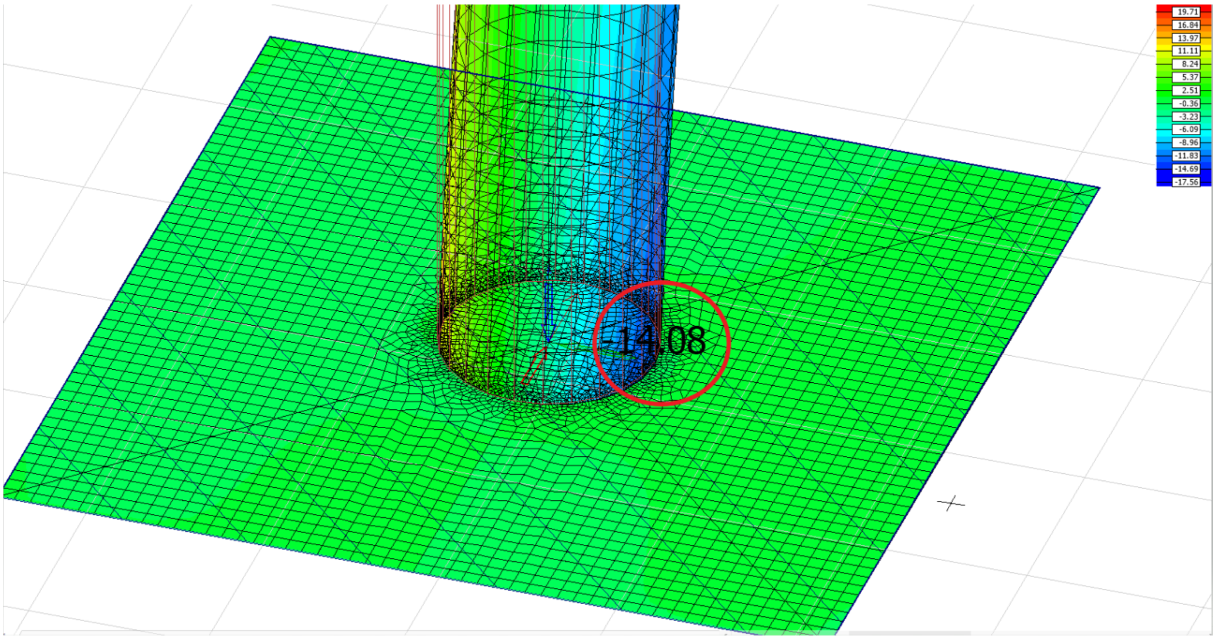

Figure 28 – FEM analysis of an arbitrary wind turbine foundation. The marked area indicates where in the foundation the largest compressive stress appears. The foundation is modeled in FEM-design.

For the calculations of the concrete fatigue in shear, it is assumed that there is no shear reinforcement in the foundation, which means that the fatigue capacity of the concrete alone is controlled.

5.1 Verification of concrete fatigue according to NS-EN 1992-1-1:2004

5.1.1 Compression

There are two methods used for verification of concrete fatigue in compression. Adequate capacity for concrete fatigue can be assumed for up 10^6 cycles if the conditions in the two methods are fulfilled.

5.1.1.1 Compression, 1st method

In the 1st method, the equations that is used for verification of concrete fatigue is equation 1 and equation 36.

$$E_{cd,\max,equ} + 0{,}43\sqrt{1 - R_{equ}} \leq 1$$

Equation 1

$$\sigma_c = \frac{6531500}{7376459{,}551} + \frac{45{,}483 \cdot v^2 \cdot 10^6}{4569171110} \text{ (MPa)}$$

Equation 36

Pre calculated values used in equation 1 is as follows.

$$f_{cd,fat} = 0{,}85 \cdot 1 \cdot 25{,}5 \cdot \left(1 - \frac{45}{250}\right) = 17{,}77 \text{ MPa}$$

Equation 5

$$\beta_{cc}(t) = e^{\left\{0{,}20\left[1 - \frac{28}{28}\right]^{1/2}\right\}} = 1{,}0$$

Equation 6

$$f_{cd} = \frac{0{,}85 \cdot 45}{1{,}5} = 25{,}5 \text{ MPa}$$

Equation 7

$$\sigma_{c,min} = \frac{6531500}{7376459{,}551} + \frac{5{,}2578 \cdot 10^6}{4569171110} = 0{,}8866 \text{ MPa}$$

Equation 36

The method is applied for every measured wind speed. If the left side (LS) of Equation 1 exceeds the value of 1,0, the concrete fatigue capacity for compression is not ok for 10^6 cycles for the calculated wind measurement. In Table 8 below is the results of the capacity control of concrete fatigue in compression for the first- and last five measurements according to the 1st method shown. The calculation was made for all measurements.

Table 8 - Results of the capacity control of concrete fatigue in compression for the first- and last five measurements according to the 1st method.

Measurement number	Wind speed (m/s)	$\sigma_{c,max}$ (Mpa)	$E_{cd,max}$	R_{equ}	Equation 1 (LS)	Capacity
1	17,64	3,983	0,2241	0,2226	0,6032	OK
2	13,89	2,806	0,1579	0,3160	0,5135	OK
3	15,80	3,370	0,1896	0,2631	0,5588	OK
4	15,79	3,367	0,1895	0,2633	0,5585	OK
5	16,72	3,668	0,2064	0,2417	0,5808	OK
...	...	...	...	...	...	...
52398	11,79	2,269	0,1277	0,3907	0,4633	OK
52399	13,43	2,681	0,1508	0,3307	0,5026	OK
52400	12,57	2,458	0,1383	0,3607	0,4821	OK
52401	13,16	2,609	0,1468	0,3398	0,4962	OK
52402	13,44	2,684	0,1510	0,3304	0,5029	OK

Out of 52402 controlled measurements, 52371 cases had adequate fatigue capacity. 31 cases did not fulfil the conditions in equation 1, which means that these cases did not have adequate fatigue capacity for 10^6 load cycles. The case that fulfilled the conditions at the highest wind speed, was at 30,99 m/s. This means that adequate fatigue capacity for concrete for up to 10^6 load cycles can be assumed for all wind speeds below 31 m/s.

5.1.1.2 Compression, 2nd method

The equation used for the verification of concrete fatigue in the 2nd method, is equation 8 and equation 36.

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0,5 + 0,45 \frac{\sigma_{c,min}}{f_{cd,fat}} \begin{cases} \leq 0,9 \ \text{ for } f_{ck} \leq 50 \text{ MPa} \\ \leq 0,8 \ \text{ for } f_{ck} > 50 \text{ MPa} \end{cases}$$

Equation 8

$$\sigma_c = \frac{6531500}{7376459{,}551} + \frac{45{,}483 \cdot v^2 \cdot 10^6}{4569171110} \text{ (MPa)}$$

Equation 36

Pre calculated values used in the calculations is as follows.

$$\beta_{cc}(t) = e^{\left\{0{,}20\left[1-\frac{28}{28}\right]^{1/2}\right\}} = 1{,}0$$

Equation 6

$$f_{cd} = \frac{0{,}85 \cdot 45}{1{,}5} = 25{,}5 \text{ MPa}$$

Equation 7

$$\sigma_{c,min} = \frac{6531500}{7376459{,}551} + \frac{5{,}2578 \cdot 10^6}{4569171110} = 0{,}8866 \text{ MPa}$$

Equation 36

The compression zone is exposed to both compressive and shear forces at the same time, so $f_{cd,fat}$ should be reduced by the stress reduction factor v when used in Equation 8.

$$v = 0{,}6 \cdot \left(1 - \frac{45}{250}\right) = 0{,}492$$

Equation 9

This results in the value of $f_{cd,fat}$ to be as follows.

$$f_{cd,fat} = 0{,}85 \cdot 1 \cdot 25{,}5 \cdot \left(1 - \frac{45}{250}\right) \cdot 0{,}492 = 8{,}74 \text{ MPa}$$

Equation 5

Adequate concrete fatigue capacity is achieved if the value on the left side (LS) in equation 8 is lower than the value on the right side (RS). The right side had the same value for all the measurements, as $\sigma_{c,min}$ and $f_{cd,fat}$ are unchanged values. The criterion for the right side to be lower than 0,9 for f_{ck} < 50 MPa is fulfilled for all measurements.

In Table 9 below the results of the capacity control of concrete fatigue in compression for the first- and last five measurements according to method number 2 is shown. The calculation was made for all measurements.

Table 9 - Results of the capacity control of concrete fatigue in compression for the first- and last five measurements according to the 2nd method.

Measurement number	Wind speed (m/s)	$\sigma_{c,max}$ (Mpa) Frequent load combination	Equation 8 (LS)	Equation 8 (RS)	Capacity
1	17,64	0,797	0,0911	0,5456	OK
2	13,89	0,561	0,0642	0,5456	OK
3	15,80	0,674	0,0771	0,5456	OK
4	15,79	0,673	0,0770	0,5456	OK
5	16,72	0,734	0,0839	0,5456	OK
...	...	...	...	...	...
52398	11,79	0,454	0,0519	0,5456	OK
52399	13,43	0,536	0,0613	0,5456	OK
52400	12,57	0,492	0,0562	0,5456	OK
52401	13,16	0,522	0,0597	0,5456	OK
52402	13,44	0,537	0,0614	0,5456	OK

Adequate capacity for concrete fatigue is achieved for all cases when the 2nd method is used. Largest value on the left side was calculated to be 0,3318, while the limit value for adequate fatigue capacity is 0,5456.

5.1.2 Shear force

There are two methods in the NS-EN 1992-1-1:2004 for control of concrete fatigue due to shear forces. If the direction of the shear force changes, the sign of the shear force may also change. The first method does not take change of signs of the shear force into account, while the second method do take change of signs of the shear force into account. Both methods require the shear force to be in the frequent load combination.

5.1.2.1 Concrete fatigue capacity for shear force without change of signs included.

The equations used for the verification of concrete fatigue for shear force without change of signs included, is equation 10 and 34.

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} \geq 0:$$

$$\frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0,5 + 0,45\frac{|V_{Ed,min}|}{|V_{Rd,c}|}\begin{cases} \leq 0,9 \text{ up to } C50/60 \\ \leq 0,8 \text{ greater than } C55/67 \end{cases}$$

Equation 10

$$V = 565,08 \cdot v^2 \text{ (N)}$$

Equation 34

$V_{rd,c}$ is the design value for the shear resistance and has a value of 1719,8 kN. $V_{rd,c}$ is calculated in appendix B.

$V_{Ed,min}$ is precalculated as follows.

$$V_{Ed,min} = 565,08 \cdot 0,34^2 = 0,0653 \text{ (kN)}$$

Equation 34

In Table 10 below is the results of the capacity control for concrete fatigue for shear force without change of signs included for the first- and last five measurements displayed. The calculation was done for all measurements.

Table 10 - Results of the capacity control of concrete fatigue for shear force without change of signs included for the first- and last five measurements

Measurement number	Wind speed (m/s)	V (kN) Frequent load combination	Equation 10 (LS)	Equation 10 (RS)	Capacity
1	17,64	35,17	0,0204	0,500003	OK
2	13,89	21,80	0,0127	0,500003	OK
3	15,80	28,21	0,0164	0,500003	OK
4	15,79	28,18	0,0164	0,500003	OK
5	16,72	31,59	0,0184	0,500003	OK
...	...	...	...	...	...
52398	11,79	15,71	0,0091	0,500003	OK
52399	13,43	20,38	0,0119	0,500003	OK
52400	12,57	17,86	0,0104	0,500003	OK
52401	13,16	19,57	0,0114	0,500003	OK
52402	13,44	20,41	0,0119	0,500003	OK

Adequate capacity for shear force not including change of signs is assumed for all cases. The largest value on the left side was calculated to be 0,0899 while the limit value for adequate fatigue capacity is 0,50003.

5.1.2.2 Concrete fatigue capacity for shear force with change of signs included.

The equation used for verification of concrete fatigue for shear force with change of signs included, is equation 11 and 34.

$$\text{for } \frac{V_{Ed,min}}{V_{Ed,max}} < 0:$$

$$\frac{|V_{Ed,max}|}{|V_{Rd,c}|} \leq 0,5 - \frac{|V_{Ed,min}|}{|V_{Rd,c}|}$$

Equation 11

$$V = 565{,}08 \cdot v^2 \text{ (N)}$$

Equation 34

$V_{rd,c}$ is the design value for the shear resistance and has a value of 1719,8 kN. $V_{rd,c}$ is calculated in appendix B.

In Table 11 below, the results of the capacity control of concrete fatigue for shear force with change of signs included, for the first- and last five measurements is displayed. The calculation was done for all the measurements.

Table 11 - Results of the capacity control of concrete fatigue for shear force with change of signs included for the first- and last five measurements.

Measurement number	Wind speed (m/s)	V (kN) Frequent load combination	Equation 11 (LS)	Equation 11 (RS)	Capacity
1	17,64	35,17	0,0204	0,410085	OK
2	13,89	21,80	0,0127	0,410085	OK
3	15,80	28,21	0,0164	0,410085	OK
4	15,79	28,18	0,0164	0,410085	OK
5	16,72	31,59	0,0184	0,410085	OK
...	...	...	...	...	...
52398	11,79	15,71	0,0091	0,410085	OK
52399	13,43	20,38	0,0119	0,410085	OK
52400	12,57	17,86	0,0104	0,410085	OK
52401	13,16	19,57	0,0114	0,410085	OK
52402	13,44	20,41	0,0119	0,410085	OK

Adequate capacity for shear force not including change of signs is assumed for all cases. The largest value on the left side was calculated to be 0,0899 while the limit value for adequate fatigue capacity is 0,410085.

5.2 Verification of concrete fatigue according to Model Code 2010

5.2.1 Compression

The first method in the Model Code 2010 standard is only applicable to structures subjected to less than 10^8 cycles. As wind turbines are subjected to 10^9 cycles in 20 years, this method is not suitable for wind turbine foundations.

The second and third method is applicable for verification of concrete fatigue in the foundation. The equations used in the calculations are equation 13,15,16, 17, 18, 19 and 36.

$$\eta_c = \frac{1}{1.5 - 0.5|\sigma_{c1}| / |\sigma_{c2}|}$$

Equation 13

$$\log N_1 = \frac{8}{Y-1} \cdot (S_{cd,\max} - 1)$$

Equation 15

$$\log N_2 = 8 + \frac{8 \cdot \ln(10)}{Y-1} \cdot (Y - S_{cd,\min}) \cdot \log\left(\frac{S_{cd,\max} - S_{cd,\min}}{Y - S_{cd,\min}}\right)$$

Equation 16

$$Y = \frac{0.45 + 1.8 \cdot S_{cd,\min}}{1 + 1.8 \cdot S_{cd,\min} - 0.3 \cdot S_{cd,\min}^2}$$

Equation 17

$$S_{cd,min} = \frac{\gamma_{Ed}\sigma_{c,min}\eta_c}{f_{cd,fat}}$$

Equation 18

$$S_{cd,max} = \frac{\gamma_{Ed}\sigma_{c,max}\eta_c}{f_{cd,fat}}$$

Equation 19

$$\sigma_c = \frac{6531500}{7376459{,}551} + \frac{45{,}483 \cdot v^2 \cdot 10^6}{4569171110} \text{ (MPa)}$$

Equation 36

Pre calculated values used in the calculations is as follows.

$$fcd,fat = 0{,}85 \cdot 1 \cdot (45 \cdot \left(1 - \frac{45}{25 \cdot 10}\right))/1{,}5 = 20{,}91 \text{ MPa}$$

Equation 14

$$\sigma_{c,min} = \frac{6531500}{7376459{,}551} + \frac{5{,}2578 \cdot 10^6}{4569171110} = 0{,}8866 \text{ MPa}$$

Equation 36

In Table 12 below, the results of the calculations of the accepted number of load cycles and the fatigue damage inflicted to the concrete from compression stresses, for the first- and last five measurements are displayed. The calculations were conducted for all wind measurements.

Table 12 - Calculations of the accepted number of load cycles and the fatigue damage inflicted to the concrete, for the first- and last five measurements.

Measurement number	Wind speed (m/s)	$\sigma_{c,max}$ (Mpa)	n_c	$S_{cd,min}$	$S_{cd,max}$	Y	logN1	logN2	N	1/N
1	17,64	3,983	0,72	0,034	0,151	0,571	15,82	23,22	$1{,}65\cdot10^{23}$	$6{,}05\cdot10^{-24}$
2	13,89	2,806	0,75	0,035	0,110	0,575	16,74	28,02	$1{,}05\cdot10^{28}$	$9{,}50\cdot10^{-29}$
3	15,80	3,370	0,73	0,034	0,130	0,572	16,28	25,41	$2{,}59\cdot10^{25}$	$3{,}86\cdot10^{-26}$
4	15,79	3,367	0,73	0,034	0,129	0,572	16,29	25,43	$2{,}70\cdot10^{25}$	$3{,}75\cdot10^{-26}$
5	16,72	3,668	0,73	0,034	0,140	0,571	16,05	24,28	$1{,}92\cdot10^{24}$	$5{,}22\cdot10^{-25}$
...	...	...	...	...	...	...	...	...	...	...
52398	11,79	2,269	0,77	0,036	0,091	0,578	17,24	31,42	$2{,}64\cdot10^{31}$	$3{,}78\cdot10^{-32}$
52399	13,43	2,681	0,75	0,035	0,106	0,575	16,85	28,71	$5{,}16\cdot10^{28}$	$1{,}94\cdot10^{-29}$
52400	12,57	2,458	0,76	0,035	0,098	0,577	17,05	30,08	$1{,}21\cdot10^{30}$	$8{,}30\cdot10^{-31}$
52401	13,16	2,609	0,75	0,035	0,103	0,576	16,92	29,13	$1{,}35\cdot10^{29}$	$7{,}40\cdot10^{-30}$
52402	13,44	2,684	0,75	0,035	0,106	0,575	16,85	28,70	$4{,}98\cdot10^{28}$	$2{,}01\cdot10^{-29}$

By applying the Palmgren-Miner summation (method three) on the results from method two, the total damage from the 52402 measurements can be calculated.

$$\sum_{i=1}^{52402} \frac{1}{N_i} = 0{,}0000000026358$$

Equation 20

Adequate capacity for concrete fatigue can be assumed if the calculated damage is <1. The total damage from all cycles is very small compared to the capacity.

The 52402 calculated cycles will be used as a spectrum for the expected fatigue image. By combining the calculated damages from the 52402 cycles with the expected annual number of cycles in a wind turbine foundation ($5\cdot10^7$ cycles), the annual damage can be calculated. The annual damage is found by the following equation.

$$\text{Total damage from 52402 cycles} \cdot 955 = \text{Annual damage (damage from } 5 \cdot 10^7 \text{ cycles)}$$

Equation 39

The lifetime of the foundation can be calculated by the by the following equation.

$$\frac{1}{Annual\ damage} = Lifetime\ (In\ years)$$

Equation 40

By applying equation 39 and 40, the annual damages and the lifetime of the foundation due to concrete fatigue for compression is found. Table 13 below displays the results from the calculation of the annual damages and lifetime calculations.

Table 13 - Annual damages and lifetime of the foundation for compression.

1 year damage	0,000002517
20 year damage	0,000050344
30 year damage	0,000075516
50 year damage	0,000125859
100 year damage	0,000251719
Lifetime (years)	397268,60

The lifetime of the foundation is calculated to be 397268,6 years for compression when the fatigue spectrum from the 52402 calculated damages is used along with the annual expected number of cycles.

5.2.2 Shear

The equations used for calculating the allowable number of cycles and damage from all wind the measurements for shear force, is equation 21 and 34.

$$\log N = 10(1 - V_{max} / V_{ref})$$

Equation 21

$$V = 565{,}08 \cdot v^2 \text{ (N)}$$

Equation 34

$V_{ref} = V_{rd,c}$ and is the design value for the shear resistance. $V_{rd,c}$ has a value of 1719,8 kN and is calculated in appendix B. In Table 14 below, the calculation of the allowable number of cycles and damage for shear force shown. The calculation was done for all wind measurements.

Table 14 - Allowable cycles and damage from shear force for the first- and last five measurements.

Measurement number	Wind speed (m/s)	V (kN)	LogN	N	1/N
1	17,64	175,84	8,978	$9{,}50 \cdot 10^{8}$	$1{,}05 \cdot 10^{-9}$
2	13,89	109,02	9,366	$2{,}32 \cdot 10^{9}$	$4{,}30 \cdot 10^{-10}$
3	15,80	141,07	9,180	$1{,}51 \cdot 10^{9}$	$6{,}61 \cdot 10^{-10}$
4	15,79	140,89	9,181	$1{,}52 \cdot 10^{9}$	$6{,}59 \cdot 10^{-10}$
5	16,72	157,97	9,081	$1{,}21 \cdot 10^{9}$	$8{,}29 \cdot 10^{-10}$
…	…	…	…	…	…
52398	11,79	78,55	9,543	$3{,}49 \cdot 10^{9}$	$2{,}86 \cdot 10^{-10}$
52399	13,43	101,92	9,407	$2{,}55 \cdot 10^{9}$	$3{,}91 \cdot 10^{-10}$
52400	12,57	89,29	9,481	$3{,}03 \cdot 10^{9}$	$3{,}30 \cdot 10^{-10}$
52401	13,16	97,86	9,431	$2{,}70 \cdot 10^{9}$	$3{,}71 \cdot 10^{-10}$
52402	13,44	102,07	9,406	$2{,}55 \cdot 10^{9}$	$3{,}92 \cdot 10^{-10}$

By applying the Palmgren-Miner summation on the results for shear force, the total damage from the 52402 measurements can be calculated.

$$\sum_{i=1}^{52402} \frac{1}{N_i} = 0{,}0000407181038$$

Equation 20

Adequate capacity for concrete fatigue can be assumed if the calculated damage is <1. The foundation has adequate capacity for shear force from 52402 cycles.

By applying equation 39 and 40, the annual damages and the lifetime of the foundation due to concrete fatigue for shear force is found. Table 15 below displays the results from the calculation of the annual damages and lifetime calculations.

Table 15 - Annual damages and lifetime of the foundation for shear force.

1 year damage	0,038885789
20 year damage	0,777715782
30 year damage	1,166573674
50 year damage	1,944289456
100 year damage	3,888578912
Lifetime (years)	25,72

The lifetime of the foundation is calculated to be 25,72 years for shear force when the fatigue spectrum from the 52402 calculated damages is used along with the annual expected number of cycles.

5.3 Verification of concrete fatigue according to DNV-OS-C502

The DNV-OS-C502 has a criterion that a minimum of eight stress blocks should be controlled for concrete fatigue capacity. In the following calculations every measurement will be considered as one stress block, so in total 52402 stress block are being controlled.

5.3.1 Compression

The formula used to calculate the design life of concrete due to fatigue is equation 23, 25 and 36.

$$\log_{10} N = C_1 \left(1 - \sigma_{max}/f_{rd}\right) / \left(1 - \sigma_{min}/f_{rd}\right)$$

Equation 23

$$C_2 = (1 + 0.2 (\log_{10} N - X)) > 1.0$$

Equation 25

$$\sigma_c = \frac{6531500}{7376459{,}551} + \frac{45{,}483 \cdot v^2 \cdot 10^6}{4569171110} \text{ (MPa)}$$

Equation 36

Precalculated values that is used in the calculations is as follows.

$$f_{rd} = f_{cd} = \frac{0{,}85 \cdot 45}{1{,}5} = 25{,}5 \text{ MPa}$$

Equation 7

$$\sigma_{c,min} = \frac{6531500}{7376459{,}551} + \frac{5{,}2578 \cdot 10^6}{4569171110} = 0{,}8866 \text{ MPa}$$

Equation 36

$$X = 12/(1 - \left(\frac{0{,}8866}{25{,}5}\right) + 0{,}1 \cdot 12) = 5{,}5421$$

Equation 24

In Table 16 below, the results of the calculations of the accepted number of load cycles and the fatigue damage inflicted to the concrete from compression stresses, for the first- and last five measurements are displayed. The calculation was done for all wind measurements.

Table 16 - Accepted number of load cycles and the fatigue damage inflicted to the concrete from compression stresses for the first- and last five measurements.

Measurement number	Wind speed (m/s)	$\sigma_{c,max}$ (Mpa)	logN	C2	logN · C2	N	1/N
1	17,64	3,983	10,490	1,990	20,87	$7{,}45\cdot10^{20}$	$1{,}34\cdot10^{-21}$
2	13,89	2,806	11,064	2,104	23,28	$1{,}92\cdot10^{23}$	$5{,}20\cdot10^{-24}$
3	15,80	3,370	10,789	2,049	22,11	$1{,}29\cdot10^{22}$	$7{,}75\cdot10^{-23}$
4	15,79	3,367	10,791	2,050	22,12	$1{,}31\cdot10^{22}$	$7{,}63\cdot10^{-23}$
5	16,72	3,668	10,644	2,020	21,50	$3{,}19\cdot10^{21}$	$3{,}13\cdot10^{-22}$
…	…	…	…	…	…	…	…
52398	11,79	2,269	11,326	2,157	24,43	$2{,}68\cdot10^{24}$	$3{,}74\cdot10^{-25}$
52399	13,43	2,681	11,125	2,117	23,55	$3{,}53\cdot10^{23}$	$2{,}83\cdot10^{-24}$
52400	12,57	2,458	11,234	2,138	24,02	$1{,}05\cdot10^{24}$	$9{,}52\cdot10^{-25}$
52401	13,16	2,609	11,160	2,124	23,70	$5{,}00\cdot10^{23}$	$2{,}00\cdot10^{-24}$
52402	13,44	2,684	11,124	2,116	23,54	$3{,}49\cdot10^{23}$	$2{,}87\cdot10^{-24}$

For all wind measurements, except two, logN > X. Those two cases did not get extended capacity by the factor C2.

By applying the Palmgren-Miner summation on the results, the total damage from the 52402 measurements can be calculated.

$$\sum_{i=1}^{52402} \frac{1}{N_i} = 0{,}0000078911853$$

Equation 20

Adequate capacity for concrete fatigue can be assumed if the calculated damage is <1. The foundation has adequate capacity for compression for the 52402 cycles.

By applying equation 39 and 40, the annual damages and the lifetime of the foundation is found. Table 17 below displays the results from the calculation of the annual damages and lifetime calculations.

Table 17 - Annual damages and lifetime of the foundation for compression.

1 year damage	0,007536082
20 year damage	0,150721640
30 year damage	0,226082459
50 year damage	0,376804099
100 year damage	0,753608198
Lifetime (years)	132,69

The lifetime of the foundation is calculated to be 132,69 years for compression, when the fatigue spectrum from the 52402 calculated cycles is used in combination with the annual expected number of cycles.

5.3.2 Shear force according to DNV-OS-C502

There are two methods for control of the concrete fatigue shear capacity in DNV-OS-C502. The first method does not account for the shear forces to change signs, while the second method do account for the shear forces to change signs.

5.3.2.1 Shear force without change of signs included.

The equations used for the verification of concrete fatigue for shear force without change of signs included, is equation 26 and 34.

$$\log_{10} N = C_1 \left(1 - V_{max}/V_{cd}\right) / \left(1 - V_{min}/V_{cd}\right)$$

Equation 26

$$V = 565{,}08 \cdot v^2 \text{ (N)}$$

Equation 34

$V_{cd} = V_{rd,c}$ and is the design value for the shear resistance. $V_{rd,c}$ has a value of 1719,8 kN and is calculated in appendix B.

V_{min} is precalculated as follows.

$$V_{min} = 565{,}08 \cdot 0{,}34^2 = 0{,}0653 \ \ (\text{kN})$$

Equation 34

$C_1 = 12$ when the shear force does not change signs.

In Table 18 below, the calculation of the allowable number of cycles and damage for shear force without change of signs included, for the first- and last five wind measurements are shown. The calculation was done for all wind measurements.

Table 18 - Allowable cycles and damage from shear force without change of signs included, for the first- and last five measurements.

Measurement number	Wind speed (m/s)	V (kN)	LogN	N	1/N
1	17,64	175,84	10,774	$5{,}94 \cdot 10^{10}$	$1{,}68 \cdot 10^{-11}$
2	13,89	109,02	11,240	$1{,}74 \cdot 10^{11}$	$5{,}76 \cdot 10^{-12}$
3	15,80	141,07	11,016	$1{,}04 \cdot 10^{11}$	$9{,}64 \cdot 10^{-12}$
4	15,79	140,89	11,017	$1{,}04 \cdot 10^{11}$	$9{,}61 \cdot 10^{-12}$
5	16,72	157,97	10,898	$7{,}91 \cdot 10^{10}$	$1{,}26 \cdot 10^{-11}$
…	…	…	…	…	…
52398	11,79	78,55	9,543	$2{,}83 \cdot 10^{11}$	$3{,}53 \cdot 10^{-12}$
52399	13,43	101,92	9,407	$1{,}95 \cdot 10^{11}$	$5{,}14 \cdot 10^{-12}$
52400	12,57	89,29	9,481	$2{,}38 \cdot 10^{11}$	$4{,}19 \cdot 10^{-12}$
52401	13,16	97,86	9,431	$2{,}08 \cdot 10^{11}$	$4{,}81 \cdot 10^{-12}$
52402	13,44	102,07	9,406	$1{,}94 \cdot 10^{11}$	$5{,}15 \cdot 10^{-12}$

By applying the Palmgren-Miner summation on the results for shear force, the total damage from the 52402 measurements can be calculated.

$$\sum_{i=1}^{52402} \frac{1}{N_i} = 0{,}00000153546$$

Equation 20

Adequate capacity for concrete fatigue can be assumed if the calculated damage is <1. The foundation has adequate capacity for shear force when change of signs is not included for the 52402 cycles.

By applying equation 39 and 40, the annual damages and the lifetime of the foundation is found. Table 19 below displays the results from the calculation of the annual damages and lifetime calculations.

Table 19 - Annual damages and lifetime of the foundation for shear force without change of signs included.

1 year damage	0,001466364
20 year damage	0,029327286
30 year damage	0,043990929
50 year damage	0,073318215
100 year damage	0,146636429
Lifetime (years)	681,96

The lifetime of the foundation before it will break is calculated to be 681,96 years for shear force with change of signs not included.

5.3.2.2 Shear force with change of signs included

The equations used for the verification of concrete fatigue for shear force with change signs included, is equation 27 and 34.

$$\log_{10} N = C_1 (1 - V_{max}/V_{cd}) / (1 + V_{min}/V_{cd})$$

Equation 27

$$V = 565{,}08 \cdot v^2 \text{ (N)}$$

Equation 34

$V_{cd} = V_{rd,c}$ and is the design value for the shear resistance. $V_{rd,c}$ has a value of 1719,8 kN and is calculated in appendix B.

$C_1 = 10$ when change of signs is included.

In Table 20 below, the calculation of the allowable number of cycles and damage for shear force with change of signs included, for the first- and last five wind measurements are shown. The calculation was done for all wind measurements.

Table 20 - Allowable cycles and damage from shear force with consideration of changed signs, for the first- and last five measurements.

Measurement number	Wind speed (m/s)	V (kN)	LogN	N	1/N
1	17,64	175,84	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
2	13,89	109,02	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
3	15,80	141,07	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
4	15,79	140,89	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
5	16,72	157,97	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
…	…	…	…	…	…
52398	11,79	78,55	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
52399	13,43	101,92	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
52400	12,57	89,29	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
52401	13,16	97,86	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$
52402	13,44	102,07	10	$1{,}0 \cdot 10^{10}$	$1{,}0 \cdot 10^{-10}$

As the wind can come from any direction, V_{max} and V_{min} will have the same absolute value for all wind speeds when the change of signs is included. This results in the allowable number of cycles to be $1{,}0 \cdot 10^{10}$ for every wind measurement.

By applying the Palmgren-Miner summation on the results for shear force, the total damage from the 52402 measurements can be calculated.

$$\sum_{i=1}^{52402} \frac{1}{N_i} = 0{,}0000052402$$

Equation 20

Adequate capacity for concrete fatigue can be assumed if the calculated damage is <1. The foundation has adequate capacity for shear force when change of signs is included for the 52402 cycles.

By applying equation 39 and 40, the annual damages and the lifetime of the foundation is found. Table 21 below displays the results from the calculation of the annual damages and lifetime calculations.

Table 21 - Annual damages and lifetime of the foundation for shear force with change of signs included.

1 year damage	0,005004295
20 year damage	0,100085910
30 year damage	0,150128865
50 year damage	0,250214775
100 year damage	0,500429550
Lifetime (years)	199,83

The lifetime of the foundation before it will break is calculated to be 199,83 years for shear force with change of signs included.

6 Comparison of calculation methods and discussion

6.1 Compression

Method number 1 in NS-EN 1992-1-1: 2004 is based on controlling the concrete fatigue capacity of a structural part at 10^6 applied load cycles at a specific stress situation. This is the number of cycles a regular wind turbine foundation experiences in just over a week. Therefore, it is difficult to use this method for verification of wind turbine foundations. The only thing that can be confirmed by using this method, is that the foundation will survive for at least a week by controlling the capacity for the highest stress amplitude.

Wind measurements greater than 31 m/s did fail for 10^6 cycles according to this method. This does not mean that the structure does not withstand 10^6 load cycles in total, but 10^6 cycles of the specific stress amplitude. Figure 24 shows that there are very few cases in the spectrum where the wind speed is greater than 31 m/s. However, if the wind speed remained constant at more than 31 m/s, the foundation would have fractured during 10^6 applied cycles.

The method seems well-implemented and are easy to use. It may be a better alternative to use on structures exposed to less than 10^6 cycles during its lifetime.

Method number 2 in NS-EN 1992-1-1:2004 controls a single stress case for adequate capacity, at a constant stress amplitude. The method does not consider the total damage from several stress cycles with varying stress amplitudes. It is thus difficult to apply this method to a wind turbine foundation, as the stresses from the wind vary widely. Adequate capacity was assumed for all measured wind speeds by this method. The method would have been better to use on wind turbine foundations if it had been changed so that overall damage from several different stress amplitudes had been controlled.

Method number 1 in Model Code 2010 requires the controlled structure be exposed to less than 10^8 cycles and thus cannot be used for wind turbine foundations.

Method number 2 in Model Code 2010 is well suited for use on wind turbine foundations as it calculates the total number of allowable stress cycles for each stress amplitude. Furthermore, the partial damage caused to the structure for each stress case can be calculated. By using method number 3 (Palmgren-Miner summation) to sum up all the partial damages found in method number 2, the total damage from all inflicted cycles can be calculated. Furthermore, annual damage and lifetime of the foundation can be calculated. The estimated lifetime was found to be 397268.6 years, which seems to be suspiciously high. The method would have been proven to be more trustworthy if the estimated lifetime had been more realistic.

DNV-OS-C502 is well suited for use on wind turbine foundations, as the methods described in the standard considers both varying stress amplitudes and calculates total damage accumulated from all inflicted cycles. The calculation process is developed in the same way as in Model Code 2010, where a total number of allowed cycles for each stress case is calculated. Furthermore, partial damage is calculated, and Palmgren-Miner's summation is used to find the total damage from all inflicted cycles. Annual damage and lifetime of the foundation can also be calculated. The estimated lifetime of 132.69 years deviates remarkably from the calculated lifetime found using Model Code 2010. The lifetime found using DNV-OS-C502 seems to be a more realistic result. In addition, the lowest estimated lifetime is the safest in terms of design.

If method number 2 in NS-EN 1992-1-1:2004 had been changed so that total damage from several stress amplitudes could be calculated from the results, it would be easier to compare the results of this standard with the other two. A standard for calculating total damage and lifetime is important in verification of wind turbine foundations. Thus, the methods described in Model Code 2010 and DNV-OS-C502 will probably be a better option than both methods described in NS-EN 1992-1-1:2004.

The methods in Model Code 2010 and DNV-OS-C502 has very similar calculation process, but very different results. DNV-OS-C502 has a very realistic result, while Model Code 2010 has an unlikely high result. It is conceivable that the method described in DNV-OS-C502 will be the best option as the calculated results seem the most accurate. Such large variations in the results can indicate that concrete fatigue in compression has not been sufficiently investigated and better and more accurate standardization of the field is necessary.

6.2 Shear

The methods for shear force in NS-EN-1992-1-1:2004 are based on the same principle as for method number 2 for compression. They control a single stress case for adequate capacity, at a constant stress amplitude. The methods do not consider overall damage from several stress cycles with varying stress amplitudes. The second method considers that the shear force can change signs, which is important when validating a wind turbine foundation. Thus, method number 2 will always be a better alternative than method number 1 when validating a wind turbine foundation. Method number 2 would have been better to use on wind turbine foundations if it had been modified so that total damage from several different stress amplitudes had been controlled.

The method described in Model Code 2010 was the easiest to use, as it was short, simple and had very few input values. On the other hand, few inputs may lead to a less precise calculation, as the more factors that have been considered in your calculations, the more accurate the result should be. The method does not include for the shear force to change signs either. As the direction of the wind hitting the structure variates, change of signs for the shear force is important to be considered when calculating fatigue damage of a wind turbine foundation. Therefore, this method may be assumed to be unsuitable for use on wind turbine foundations. The calculated lifetime of the structure of 25,72 years seems slightly strict, as regular wind turbine foundations usually have longer lifetime than that.

It may seem that Model Code 2010 has not worked out a detailed method for shear force. It can be assumed that they have made a usable method and made it very rigorous so that there is an option available for shear force control by the standard. If this is the case, the actual lifetime of the foundation will be much higher than calculated, and methods from other standards will most likely provide a more accurate result.

In DNV-OS-C502, method number 2 has a lower calculated lifetime compared to method number 1 as it considers that the shear forces can change signs. Method number 1, like the method described in the Model Code 2010, can be assumed to not be suitable for the control of wind turbine foundations, as the change of signs of the shear forces is not considered. The second method in DNV-OS-C502 considers that the shear forces can change signs like the second method described in NS-EN 1992-1-1:2004. In addition, method number 2 in DNV-OS-C502 calculates total damage from several different stress amplitudes, which in total results in a very well qualified method for validation of wind turbine foundations.

Huge variations in the results is discovered for shear force as well. This may indicate that further development for concrete fatigue for shear force is necessary.

6.3 Overall

DNV seems to stand out as the best standard for both compression and shear force. Still, there are huge variations in the results for both compression and shear force, that may indicate that concrete fatigue has not been sufficiently investigated and needs better and more accurate standardization. Several of the methods seems to have great potential for improvements.

By configuring better and more precise methods, the lifetime of the construction can be extended. This has both positive economic and environmental consequences, as the construction does not need to be replaced as often.

7. Conclusions

The first method described for compression in NS-EN 1992-1-1:2004 is not suitable for calculation of concrete fatigue in a wind turbine foundation as it is only applicable to structures subjected to a significant lower amount of cycles.

The second method described for compression in NS-EN 1992-1-1:2004 is applicable for wind turbine foundations but is not recommended as it do not account for the total accumulated damage from several different load amplitudes and do not provide a calculated lifetime.

The first method for compression described in Model Code 2010 is not suitable for calculation of concrete fatigue in a wind turbine foundation as it is only applicable to structures subjected to a significant lower amount of cycles.

A combination of method number 2 and 3 for compression described in Model Code 2010 is a better option than the methods described in NS-EN 1992-1-1:2004 since the total accumulated damage and lifetime of the foundation is calculated. However, the method is not recommended as it results in an unlikely calculated lifetime of the foundation.

The method described for compression in DNV-OS-C502 is the best suited method for verification of concrete fatigue in a wind turbine foundation, as it calculates partial damage for each stress case. It also calculates a total lifetime of the foundation and it results in the lowest calculated lifetime of the foundation.

The first method described for shear force in NS-EN 1992-1-1:2004 is applicable for wind turbine foundations but is not recommended as it do not account for the shear force to change signs. It does not account for the total accumulated damage from several different load amplitudes and it do not provide a calculated lifetime of the foundation either.

The second method described for shear in NS-EN 1992-1-1:2004 is applicable for wind turbine foundations but is not recommended as it do not account for the total accumulated damage from several different load amplitudes and do not provide a calculated lifetime.

The method described for shear in Model Code 2010 is applicable for wind turbine foundations but is not recommended as it do not account for the shear force to change signs.

The first method described for shear in DNV-OS-C502 is applicable for wind turbine foundations but is not recommended as it do not account for the shear force to change signs.

The second method described for shear in DNV-OS-C502 is the best suited method for wind turbine foundations as it accounts for the shear force to change signs, in addition to accounting the total accumulated damage from several different load amplitudes and lastly it provides a calculated lifetime of the foundation.

Huge deviations in the results between the standards for both compression and shear do indicate that concrete fatigue needs further development and better standardization. There are great potentials for improvements in several methods in the controlled standards which will provide both positive economic and environmental consequences.

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List of appendices

Appendix A: Calculation of A and AR of a turbine blade

Calculation of A

The total area that is exposed to wind load perpendicular to the blade, is equivalent to the blade thickness multiplied with the blade length. As the thickness variates throughout the blade, a function for the blade thickness of the rotor radius is needed for the calculations. Figure 29 below shoes the definition of a blade's cross section and thickness.

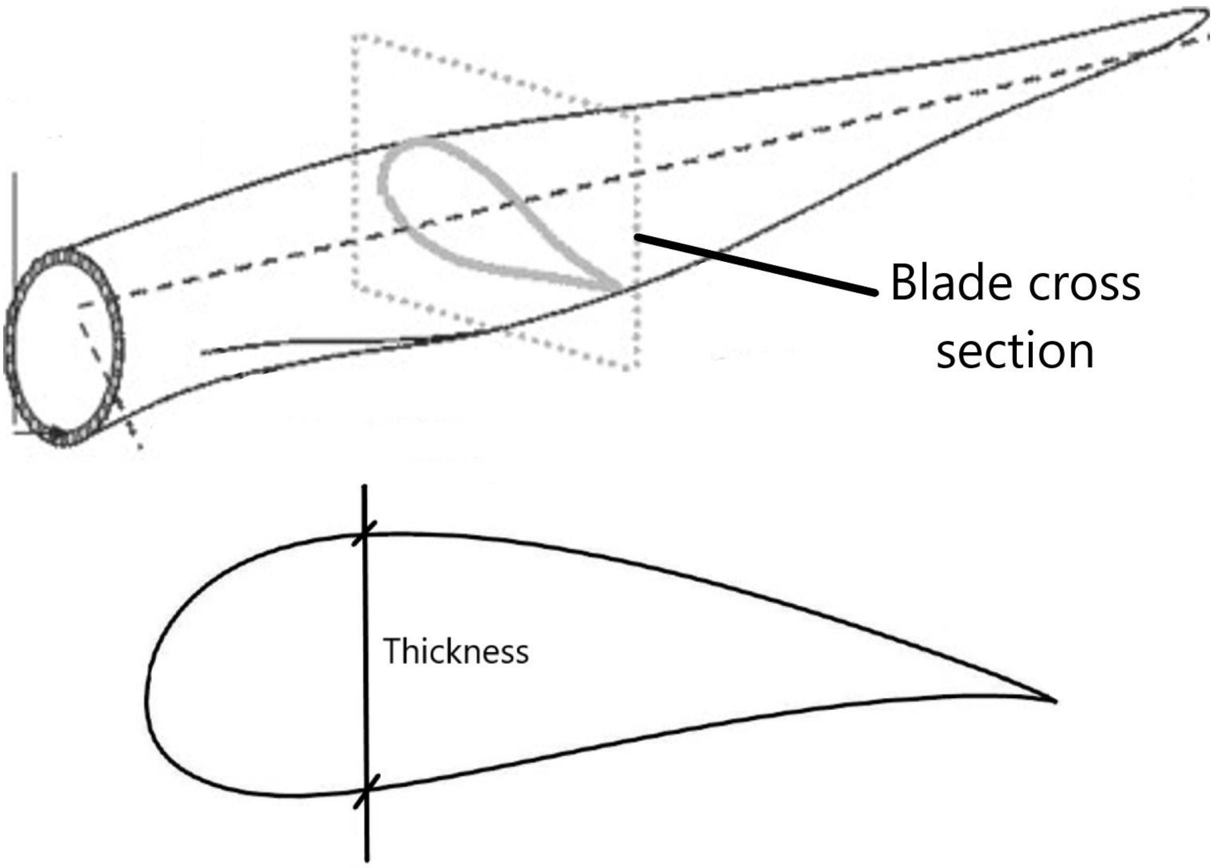

Figure 29 - Definition of a blade's cross section and thickness

Table 22 below can be used to find a selection of wind turbine blade dimensions based on the radius of the rotor, including the thickness. The table is appliable for conventional three bladed wind turbines (Sandia National Laboratories, 2003).

Table 22 - Table for calculating chord length and thickness of a wind turbine blade based on the rotor radius (Sandia National Laboratories, 2003)

Radius Ratio	Chord Ratio	Thickness Ratio
5%	5.2%	100%
15%	7.8%	42%
25%	8.6%	28%
35%	7.6%	24%
45%	6.6%	23%
55%	5.7%	22%
65%	4.9%	21%
75%	4.0%	20%
85%	3.2%	19%
95%	2.4%	18%

Figure 30 below explain the different parameters in Table 22 above which can help understanding the table.

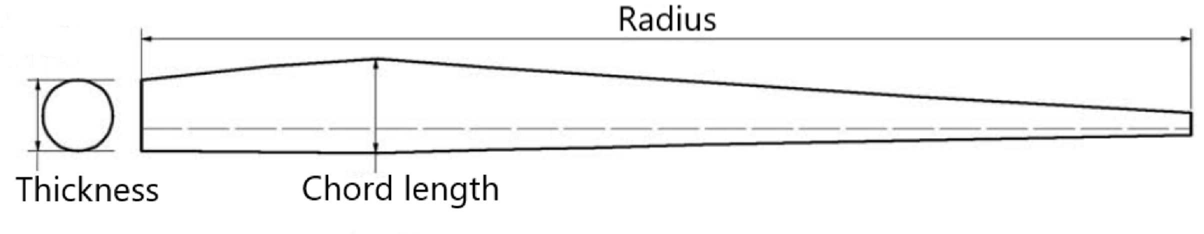

Figure 30 - Thickness, Radius and Chord length displayed for a wind turbine blade

By applying the rotor radius of 79 meter to table, the total area that is exposed to wind load perpendicular to the blade can be calculated. The blade is divided into 11 sections and the thickness in each section is multiplied with the length of the section. Thereafter the total area is found by adding all the section areas together. In Table 23 below the calculation of the total area is shown. The thickness of the blade is assumed to be 6 meters at the hub by examining the thickness of blades of similar length.

Table 23 - Calculation of the area exposed to wind load perpendicular to the blade

Radius Ratio	Radius Value (m)	Thickness Ratio	Thickness Value (m)		Area (m^2)
0,05	3,95	1	6		23,7
0,15	11,85	0,42	2,52		19,908
0,25	19,75	0,28	1,68		13,272
0,35	27,65	0,24	1,44		11,376
0,45	35,55	0,23	1,38		10,902
0,55	43,45	0,22	1,32		10,428
0,65	51,35	0,21	1,26		9,954
0,75	59,25	0,2	1,2		9,48
0,85	67,15	0,19	1,14		9,006
0,95	75,05	0,18	1,08		8,532
1	79	0,18	1,08		4,266
				Sum	**130,824**

Calculation of AR

AR is the aspect ratio of the blade and can be calculated by the following equation.

$$AR = \frac{L}{C}$$

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Where:

L is the length of the blade which is 79 meters

C is the length of a representative chord

Figure 31 is a cross section of a wind turbine blade and explains the definition of the chord length.

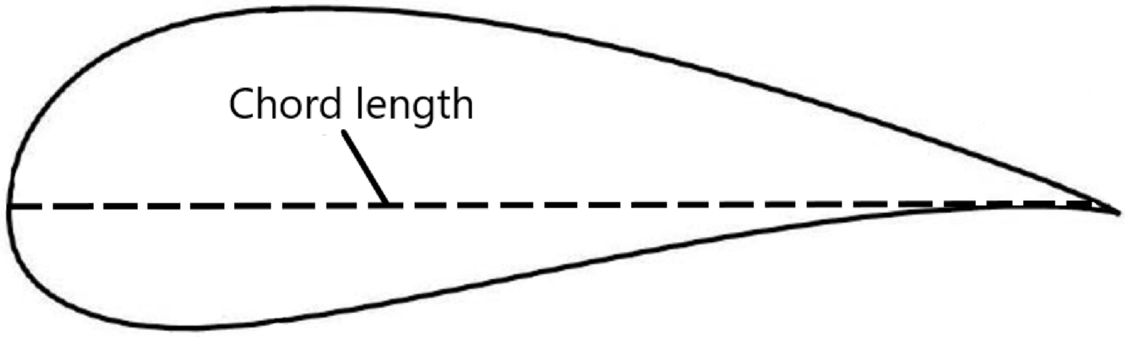

Figure 31 - The definition of the chord length of a turbine blade

As the chord length of a wind turbine blade variates with the length of the blade, Table 22 above is used to calculate AR. First the blade is divided into 11 sections, and the chord length of each section is calculated. The calculation is shown in Table 24 below.

Table 24 - Calculation of the chord length at the 11 different sections of the turbine blade

Radius Ratio	Radius Value (m)	Chord Ratio	Chord length (m)
5 %	3,95	5,2 %	4,108
15 %	11,85	7,8 %	6,162
25 %	19,75	8,6 %	6,794
35 %	27,65	7,6 %	6,004
45 %	35,55	6,6 %	5,214
55 %	43,45	5,7 %	4,503
65 %	51,35	4,9 %	3,871
75 %	59,25	4,0 %	3,16
85 %	67,15	3,2 %	2,528
95 %	75,05	2,4 %	1,896
100 %	79	2,4 %	1,896

These results are than used to calculate the chord length at 20 places along the blade length with equal distance between each other. Thereafter, the average chord length for the blade is found by finding the average value of 20 calculated chord lengths. Table 25 below shows the calculation of the average chord length.

Table 25 - Calculation of average chord length of the blade

Radius ratio	Radius Value (m)	Chord length (m)
5 %	3,95	4,108
10 %	7,9	4,108
15 %	11,85	6,162
20 %	15,8	6,162
25 %	19,75	6,794
30 %	23,7	6,794
35 %	27,65	6,004
40 %	31,6	6,004
45 %	35,55	5,214
50 %	39,5	5,214
55 %	43,45	4,503
60 %	47,4	4,503
65 %	51,35	3,871
70 %	55,3	3,871
75 %	59,25	3,16
80 %	63,2	3,16
85 %	67,15	2,528
90 %	71,1	2,528
95 %	75,05	1,896
100 %	79	1,896
Average chord length:		**4,424**

When the value of C is calculated, AR can be calculated as follows.

$$AR = \frac{L}{C} = \frac{79}{4,424} = 17,86$$

Appendix B: Calculation of $V_{Rd,c}$

The calculation of $V_{Rd,c}$ is done according to section 6.2.2 (1) in NS-EN 1992-2-2:2004 (Standard Norge, 2018, ss. 83-84). By calculation of $V_{Rd,c}$ the formula for the minimum value have been used as this is a safer option in terms of design (formula (6.2.b)).

$$V_{Rd,c} = (v_{min} + k_1 \sigma_{cp})\, b_w d$$

Where:

K_1 has a value of 0,15 for compression

b_w is the smallest width of the cross-section which is equal to the width of the anchorage plate which is 800mm

d is the distance from the top of the surface to the center of the tensile reinforcement in the lower section of the foundation. The value of d is set to 2900 mm for insurance.

$$\sigma_{cp} = N_{Ed}/A_c < 0{,}2\, f_{cd} \quad [\text{MPa}]$$

Where:

N_{Ed} is the axial force in Newton which is 6531500 N.

Ac is the area of the concrete cross section in mm^2 which is 3000mm ·800mm.

$$\sigma cp = \frac{6531500}{3000 \cdot 800} = 2{,}7215 < 0{,}2 \cdot fcd = 5{,}1$$

$$v_{min} = 0{,}035\, k^{3/2} \cdot f_{ck}^{\ 1/2}$$

Where:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \text{ with } d \text{ in mm}$$

$$k = 1 + \sqrt{\frac{200}{2900}} = 1{,}2626 < 2{,}0$$

$$Vmin = 0{,}035 \cdot 1{,}2626^{1{,}5} \cdot 45^{0{,}5} = 0{,}3331$$

$$V_{Rd,c} = (0{,}3331 + 0{,}15 \cdot 2{,}7215) \cdot 800 \cdot 2900 = 1719874\, N = 1719{,}8\, kN$$