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DESIGN OF FOUNDATIONS FOR WIND TURBINES

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Preface

This master thesis was carried out at Ramböll Sweden, Malmö in cooperation with the Divison of Structural Mechanics at LTH, Lund University, from February 2010 to December 2010.

First of all I would like to thank my supervisors Lic. Lars Johansson at the Geotechnical department at Ramböll, and Prof. Per Johan Gustafsson at Structural Mechanics for their valuable support and help.

I would also like to express my sincere gratitude to the whole department for Building Construction at Ramböll for always being there helping out with everything, and for letting me have a workplace with a high quality computer and access to useful computer software. A special thanks to Erik Wahlström at Ramböll for his enormous willingness to help despite of his heavy workload.

Finally I would like to thank my beloved Christina for her constant encouragement and her understanding during the work with this thesis and throughout my whole civil engineering education.

Henrik Svensson
Lund, in December 2010

Abstract

The Swedish government has specified a goal for the Swedish wind power that in 2020 it will generate 30 TWh of energy per year. This should be compared with the present energy produced from wind power of 2.5 TWh / year. To meet these goals, several thousand new wind turbines have to be built.

Today, we build the most land-based wind turbines on strong and stiff soils, but probably in the future wind turbines will have to be built also on soils with less good properties. The ordinary and fairly simple foundation method with a concrete slab with large area, may be abandoned since it can give too large differential settlement.

This thesis is examining the foundations for onshore wind turbines where both the more convential method with a large concrete slab are investigated, but also alternative foundation methods are studied, mainly piled foundations.

Different types of foundations is presented and discussed in which the design procedure consists of both manual calculations and numerical analyses. A case study of an 80 meter high wind turbine with realistic loads is presented. The study includes geotechnical and structural design for three different soil profiles, in which three different foundation methods are used.
The three cases are:

1. Strong and stiff moraine soil in which the most common foundation method with a spread foundation is used.
2. A 20 m thick layer of clay that overlay the strong bedrock in which toe-bearing precast concrete piles are used. In this case only the piles are assumed to bear the load.
3. Clay soil with the bedrock at considerable depth in which precast concrete piles are used as cohesion piles. Both piles and the concrete slab are assumed to bear load in a so-called piled-raft foundation.

For the three cases above, the same foundation slab is used, but for case 2 and 3 the slab is cast on piles.

The results of this study show that all three above-mentioned foundation methods are feasible, but for the third case the differential settlements are significantly big resulting in a horizontal displacement of the tower's top of 155 mm. The first case is the cheapest and easiest to perform, and is preferred if the geotechnical conditions permit that. The second case results in a relative small total pile length of 680 m, while the third case results in 3720 m in total pile length.

The big pile length that the third case results in is an expensive and laborious foundation to construct and such should not be constructed. The design of a foundation of this type has many difficulties. In this thesis the geotechnical design was performed using a two-dimensional model in a finite element program for geotechnical applications. Modeling of piles in two dimensions is difficult to do in a realistic way and a three-dimensional model is preferred. This, together with the difficulty of finding the right stiffness ratio between the piles and the plate can be two sources of possible error in the extremely large pile length found for case 3.

Sammanfattning

Den svenska regeringen har satt upp mål om att den svenska vindkraften 2020 skall generera 30 TWh energi per år, vilket kan jämföras med den idag producerade energin från vindkraft på 2,5 TWh/år. För att uppfylla detta mål måste flera tusen nya vindkraftverk byggas.

Idag byggs de flesta landbaserade vindkraftverken på hållfasta och styva jordar, men i framtiden kommer troligtvis vindkraftverk behöva byggas även på jordar med sämre egenskaper. Den annars ganska enkla grundläggningsmetoden med en betongplatta med stor area måste kanske då överges då den ger stora differentialsättningar.

Detta examensarbete studerar grundläggningar för landbaserade vindkraftverk där såväl den mer konventinella metoden med en stor betongplatta undersöks, men även alternativa grundläggningsmetoder studeras, då främst pålade grundläggningar.

Olika typer av grundläggningar presenteras och diskuteras där dimensioneringsförfarandet innefattar både manuella beräkningar och numeriska analyser. En fallstudie för ett 80 meter högt vindkraftverk med verklighetstrogna laster genomförs. Studien innefattar geoteknisk och strukturell dimensionering för tre olika jordprofiler där tre skilda grundläggningsmetoder tillämpas. De tre olika fallen är:

1. Hållfast och styv moränjord där den vanligaste grundläggningsmetoden med en utsträckt platta tillämpas.
2. 20 m tjockt lertäcke överlagrar hållfast berg där stoppslagna prefabricerade betongpålar används. Endast pålarna antas i detta fall bära last.
3. Lerjord med berg på betydande djup där prefabricerade betongpålar används som kohesionspålar. Både pålar och betongplatta antas bära last i en så kallad samverkansgrundläggning.

För de tre fallen ovan används en likadan grundläggningsplatta, med den skillnaden att för fall 2 och 3 är plattan gjuten på pålar.

Resultatet av studien visar att alla tre ovan nämnda grundläggningsmetoder är genomförbara, men för det tredje fallet blir de differentiella sättningarna betydande vilket medför en horisontell förskjutning av tornets topp på 155 mm. Det första fallet är det billigaste och enklaste och är att föredra om de geotekniska förhållandena tillåter det. Det andra fallet ger en relativt liten total pållängd om 680 m medan det tredje fallet ger hela 3720 m i total pållängd.

Den stora pållängden det tredje fallet resulterar i innebär ett orimligt dyrt och arbetskrävande fundament och ett sådant bör inte utföras. Dimensioneringen av ett fundament av denna typ innehåller många svårigheter. I denna rapport gjordes den geotekniska dimensioneringen med hjälp av en tvådimensionell modell i ett FEM-program för geotekniska tillämpningar. Modellering av pålar i två dimensioner är svår att göra reslistisk och en tredimensionell modell är att föredra. Detta tillsammans med svårigheten att finna rätt styvhetsförhållandet mellan pålar och platta kan vara två felkällor som bidragit till det ganska extrema resultatet för fall 3.

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1. Introduction

1.1 Denotations

In this thesis the following denotations is used. In general it follows the standard denotation in Eurocode, but there are some other denotations in addition. The text in brackets denotes a possible unit.

1.1.1 Latin letters

Letter:	Explanation:
a	Different distances, see specific references [m]
A	Area [m^2]
b	Distance from edge to resultant force of the ground pressure [m]
B	Width [m]
c	Concrete cover [m]; Soil cohesion [Pa]
C	Concrete strength (For example C30/37) [Pa]
d	Effective height of a concrete cross section [m]
	Correction factor regarding foundation depth [-]
D	Diameter [m]
e	Eccentricity [m]
E	Young's modulus of elasticity [Pa]
f	Material strength [Pa]
F	Force [N]
G	Permanent load [N]
h	Height [m]
H	Horizontal force [N]
i	Inclination factor [-]
I	Moment of inertia [m^4]
k	Subgrade reaction [N/m^3]; Different constants, see specific references
l	Length [m]
L	Design life of the structure [year]
m	Relative moment [-]; Slope of the compression modulus [-]; Stress exponent for a Wöhler curve [-]
M	Bending- or Twisting moment [Nm]; Compression modulus of soil [Pa]
N	Bearing capacity factors [-]; Number of loading cycles [-]; Axial force [-]
p	Overburden pressure [Pa]
P	Perimeter length [m]
q	Bearing capacity of ground [Pa]
Q	Variable load [N]
r	Radius [m]
R	Radius [m]; Resultant force [N]
s	Distance between reinforcement bars (centre to centre distance) [m]
	Correction factors regarding shape [-]
t	Thickness [m]
T	Tension force [N]
u	Length of control section regarding concrete punching [m]
v	Shear pressure [Pa]
V	Vertical force (often shear force) [N]
w	Crackwidth
W	Bending resistance [m^3]
x	Distance from reinforcement to neutral layer [m]; Unknown variable in equations

z	The inner lever for a cross section [m]

1.1.2 Greek letters

Letter:	**Explanation:**
α	Angle [^{o}]
β	A coefficient in concrete punching [-]
γ	Partial factor [-]; weight of material [N/m^3]
δ	Settlement [m]
ε	Strain [-]
Δ	Denoting a difference [-]
ζ	Concrete cracking safety factor [-]
σ	Stress [Pa]; Ground pressure [Pa]
ϕ	Diameter of reinforcement bar [m]; Friction angle of soil [^{o}]
ω	Mechanical reinforcement share [-]
ρ	Geometrical reinforcement share [-]
τ	Shear stress/strength [Pa]

1.1.3 Index letters

Letter:	**Explanation:**
0	Initial value
c	Concrete; Compression (cc = compressed concrete)
d	Design value
E	Load effect
f	Force; Load
k	Characteristic value
m	Mean value; Material
R	Resistance
s	Steel; Soil
t	Tension
x	X-direction
y	Yield; Y-direction
z	Z-direction
cr	critical
eff	Effective
equ	Static equilibrium
fat	Fatigue
min	A minimum value
max	A maximum value
res	Resulting value
tot	A total value

Many index letters can be combined such as f_{ct} or $\sigma_{cd,\max}$ where the first one denotes the **t**ension strength of **c**oncrete, and the second one is the **max**imum **d**esign value for **c**oncrete.

1.2 Background

The access to energy is a very important matter in the modern society, but even more important is how the energy is provided. There is almost unlimited ways of how to provide energy and each method has got their own benefits and disadvantages. The method should be efficient, and in addition not affect the environment in a bad manner, where the latter is playing a very important role for the energy production today.

Today we are talking about the importance of having a sustainable development, which means that global changes should be progressive without precluding forthcoming generations to satisfy their needs [1]. One major example of sustainable development in the energy industry is renewable energy, meaning that the source is not consumed but just used once, ready to be used again.

One of the bigger challenges for today's society is the change from non renewable energy sources, such as fossil fuel consumption to renewable sources such as wind power. Today, almost one third of Sweden's energy originates from fossil fuels and less than 1% comes from wind power [2]. The situation is not specific for Sweden, but more of a trend valid for most countries.

The use of wind to produce energy is a very old tradition. As early as 3000 years ago China and Japan built wind mills, and later on in the 13th century wind power were spread to Europe. In the 19th century wind power was one of the largest sources of energy. Wind mills were used in several areas, such as grinding seed, pumping water and operating sawmills etc. [3] Today's situation is different with only a few percent of the total energy production origin from wind power [3]

Wind turbines today involves very advanced technology, but the basic principle is that the wind forcing a rotor, via the rotor blades, to rotate and this rotation creates electricity via a generator. In the chapter 2.2 a little more detailed description of wind turbines is to find.

1.3 Wind power

2007 the Swedish Energy Agency (Energimyndigheten) was commissioned by the government to establish a national plan for how to develop the wind power until year 2020. This resulted in a plan that yields a vast increase of wind power as a source of energy. More specific the plan says that 2020 should approximately 30 TWh/year be produced from wind power, where 20 TWh/year is from land based wind power and the rest from offshore wind power [4]. A comparison with the energy produced from wind power today which is according to [5] approximately 2.5 TWh/year (2009) indicates a forthcoming big boom in the building of wind power turbines. To satisfy this demand they're counting on that approximately 3000 up to 6000 new wind turbines, dependent on the output power of the plants, will have to be built in Sweden until 2020. In the end of 2009 there are 1359 current operating wind power turbines in Sweden, of which approximately 95 percent is land based stations [5].

1.4 Objective

The main objective in this master's thesis is to study and analyze different types of foundation methods for land based wind turbines. As the number of turbines that will be built probably will increase in the near future (see chapter 1.3) it will become necessary to build plants even on less good soils such as clays. In that case the standard foundation method with a large spread foundation may be abandoned for other foundation methods more suitable for these worse soil conditions. This thesis will therefore focus a little extra on foundation methods with piles.

The aim of the thesis is also to create templates for the foundation design. This will be done both for a gravity foundation and piled constructions, where the latter will include a wider range of investigations and analysis such as how and when the method is appropriate and moreover if the method is cost efficient. The templates will concern both geotechnical and structural design and will be carried out according to European standards i.e. Eurocode. A comparison of the foundation types will be done in terms of economy, way of function, execution etc.

This thesis is carried out on behalf of Ramböll Sweden and the intention of this work is primarily to investigate alternative methods for the foundation of wind turbines and see how appropriate and economic they are, and secondary to be able to carry out geotechnical and structural design of wind turbine foundations more efficient thanks to the design templates.

1.5 Audience

To fully comprehend this thesis some knowledge about structural mechanics, structural- and geotechnical design, the finite element method and material science (mainly concrete and steel) is recommended.

1.6 Limitations

The foundation design of offshore wind turbines differs very much from the land based. The way they are anchored in the ground and the forces acting on the foundation is totally different from the land based. In addition is the design of offshore foundations following a special offshore standard. This is why this work limits to only concern land based wind turbines.

This thesis is not dealing with foundations with large drilled piles anchored in the rock. The reason for this is that an investigation is already done [6]. The company that made this analysis, Ruukki, is a company producing and providing steel products, and in this case Ruukki-produced steel pipes were used. The foundation is now constructed and the turbine is operating. This thesis is not an extension of the Ruuki study, but a study aside it. Ruukki’s work is limiting this thesis not to study large drilled steel piles anchored in the rock, but it also functions as an input source.

1.7 Ruukki’s study

Ruukki has done a study of one specific foundation method concerning a wind power foundation with drilled steel pipe piles. In that study eight steel piles with a diameter of 600 mm were drilled down to the bedrock, then smaller steel pipes continuing inside the pipes and anchoring the bedrock. The drilled distance to the bedrock was approximately 10 m and then the smaller pipes were injected in the bedrock for a few meters. Once having put the reinforcement in the pipes, concrete was poured in the pipes. On top of the piles, a reinforced octagon shaped concrete slab was casted to function as a stiff connection between the piles and the tower. This slab was octagon shaped with an edge to edge distance of 12 m, and approximately 2 m thick. [6]

Due to the piles a much smaller concrete slab than for an ordinary gravity foundation was needed. The one in the study used approximately 250 m^3 compared to a regular one of 800 m^3 concrete, which is less than one third of the total amount of concrete. [6]

This specific case resulted in a cost efficient foundation. Compared to an ordinary wind power foundation, typical a spread foundation, this method lowered the costs with approximately 10 % according to [6]. Figure 1.1 shows the foundation during construction stage. The concrete “boxes” on top of the slab contains the joint between the pile and the slab.

Figure 1.1: Ruukki's piled foundation [A]

2. Theory

2.1 General

To comprehend this thesis better some essential theory is presented here. The first subchapter is giving an overview of wind turbines in general. The intention is to explain the different parts of a wind turbine. In the following chapters relevant theory about geotechnical- and structural design is presented. In the end of the chapter standards, regulations and general design criteria is to be found.

2.2 Wind turbines

All wind turbines have in common that they use wind power to produce electric energy. Though the way this is done may differ. Today there are essentially two different types of wind turbines that are functioning in quite different ways. There are turbines with blades rotating about a horizontal axis which is most commonly used, and turbines that use a rotating motion about a vertical axis, see figure 2.1.

Figure 2.1: Left: Horizontal axis wind turbine [B]
Right: Vertical axis wind turbine [C]

Both types using a rotating motion to generate electricity. This thesis focus on the first mentioned, the horizontal axis turbines, and this one gets therefore a little more detailed description. The main parts of a horizontal axis wind turbine are shown in figure 2.2.

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

Figure 2.2: Horizontal axis wind turbine, parts numbered [D]

The foundation's only task is to ensure the stability for the wind turbine, and to do so over its life time. This is done by transferring and spreading the loads acting on the foundation to the ground. The vertical force acting on the foundation is mainly dead load from the tower, the nacelle and the rotor blades, but the wind may also give arise to some vertical force. The most significant loads on the foundation origins from the wind. Due to the big height of the tower, a horizontal force from the wind is giving a considerably big bending moment at the foundation.

The tower usually has got the form of a hollow truncated cone, and is made of high quality steel. The wide tower base connects, the prefabricated tower, to the in situ made foundation via an interface. There are many possible solutions of this interface. One solution is a giant steel pipe with a flange, which is embedded in the concrete foundation. Another one is "a bolt cage", where several long bolts are embedded in the concrete, see figure 2.3.

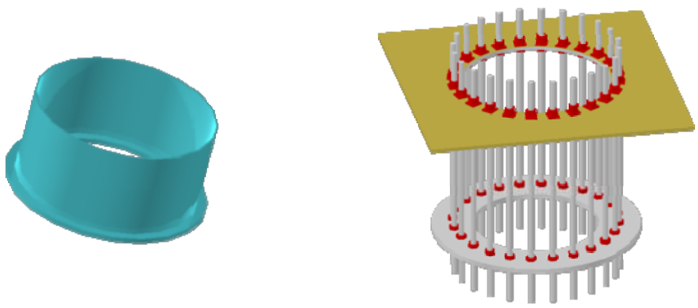


Figure 2.3: Left: Steel "ring" as an interface between tower and foundation
Right: "Bolt cage" as an interface between tower and foundation

The tower height varies very much; from 40 m up to 130 m. The higher the tower is, the greater the wind speed is. Nowadays it is more common to built high towers. [7]

The nacelle holds all the turbine machinery and transforms the rotating energy to electrical energy. The specific manufacturer has its own type of construction, but in general it consists of a gearbox that accelerate the motion to a more suitable speed, a generator that creates the electric energy and a brake system which can force the rotation to stop in case of too strong winds or if another type of failure occur. The nacelle is connected to the tower via bearings, because of the possibility to rotate about the tower axis to tune in the wind direction. [7]

The design of the rotor blades is the reason for the rotating motion. The profile of a rotor blade is similar to an airplane wing and it is the same principle that forces the blade to rotate, i.e the profile creates a pressure difference over the blade and therefore forces the blade to move. Nowadays the three bladed wind turbines are dominating the market, but two- and more then three bladed also exists. The blades are usually made of glass fibre or carbon fibre reinforced plastics. [7]

The hub is the connection between the rotor blades and the rotating bar that goes into the nacelle. The shape of the hub is complicated, why the hub usually is of cast iron. The steel must be very resistant against metal fatigue and therefore a special alloy is used, which after casting undergoes a heat treatment to get the right properties. [7]

The transformer unit is not part of the wind turbine itself, but a unit necessary to transform the wind turbine output power to electric power suitable for the actual environment. [7]

2.3 Design – General aspects

The design of elements and structures aims to find dimensions and types of structures that are suitable and safe for the actual situation and to maintain this during its specified lifetime. Below is listed some of the parameters that should be taken account of when designing structures. Some of them are more important than others. Note that in this thesis, the designing concept only concerns structural and geotechnical design and not design regarding heat, moisture and other building physics related areas.

- The structure must be stable
- The material strength must not be exceeded
- The function of the structure must be maintained
- The structure should be aesthetically pleasing
- The structure has to be resistant against external factors, such as fire, earthquake, flooding, frost, moisture, temperature differences and vermin such as termites and insects and traffic accidents etc.

There are a few different methods to ensure that these conditions are fulfilled with a reasonable margin of safety. The original method is to calculate the probability for failure and then compare this with a reasonable margin of safety. To do this properly one has to know the expected value and the statistical variance for the governing parameters for example the loading and the material strength. This is often very hard and therefore another method is the most used today, namely the partial factor method. This method makes use of factors that are adjusting relevant parameters in the calculation. This is done either by multiplication or division with a partial factor (γ), which is almost always bigger than unity. To increase the safety margin one makes the loading more unfavorable (often bigger) by multiplying with the factor and the material strength weaker by dividing with the factor. Another type of partial factor is controlling the accuracy in the calculation model; the more accurate model there is the nearer unity the factor is. In addition the consequences of a failure are also considered by increases or reducing parameters with a partial factor.

When designing buildings and other civil engineer works one has to consider both the geotechnical aspect and the structural. Chapter 2.4 and 2.5 describes these design areas respectively.

When designing structures one has to make sure that all the requirements are fulfilled. Regarding designing of structures, one is talking about different types of limit states. A limit state is reached when a structure is on the verge to exceed a specific requirement. In general two different limit states are apparent and explained in the forthcoming subchapters.

2.3.1 **Ultimate limit state (ULS)**

This limit state is concerning the safety of human persons and/or the safety of the structure itself. In some cases even the safety of the content in the structure can be seen as ULS. The following designing criteria are related to ULS [29]:

- A lost equilibrium of the structure when it is considered as a rigid body
- Failure due to too big deformations
- If the structure is becoming a mechanism and collapse
- Failure due to too high stress in the material
- Lost overall stability of the structure
- Failure due to fatigue or other time dependent effects

In ULS the partial factors are bigger than for SLS to ensure that the loads really are ultimate, and the material strength is lowered.

2.3.2 **Serviceability limit state (SLS)**

SLS concerns the function of the structure under normal use, the human well-being and the constructions appearance. Distinction shall be made for reversible and irreversible serviceability limit states i.e. temporary inconvenience or permanent inconvenience respectively. The following criterions are related to SLS [29]:

- Deformations affecting:
    - the appearance
    - the users well-being
    - the intended function of the structure, including the function of machines etc.
- Vibrations and oscillations affecting:
    - the users well-being
    - the function of the structure
- Damage that likely will affect the:
    - the appearance
    - the persistence
    - the function of the structure

In SLS the partial factors is often set to unity and the characteristic values for loads and material parameters are used. The reason for this is that SLS should reflect the normal use.

2.3.3 **Design code**

To design structures in a correct manner there are design codes available, which specifies values for safety factors, values for material parameters, designing criterions and other rules and regulations that governs the design. If the code is followed when designing, it is ensured that the design is performed properly and does not violate rules and regulations.

In this thesis the European standards are used i.e. Eurocodes. These designing codes are valid for all countries that are members of the European Committee for Standardization (ECS), which at the moment are all countries members of the European union (27 countries) and in addition Iceland, Norway and Switzerland [30]. Because of each country's specific safety demands each country has got its own national appendix to the eurocodes which specifies values that should be used to fulfill the demands for the country in question. In this thesis the Swedish national appendix is used.

The Swedish version of the codes is issued by the Swedish Standards Institute (SIS) and divided into several subcodes dealing with different areas. The subcodes are the following:

- SS-EN 1990 Eurocode 0 – Basis of structural design
- SS-EN 1991 Eurocode 1 – Actions on structures
- SS-EN 1992 Eurocode 2 – Design of concrete structures
- SS-EN 1993 Eurocode 3 – Design of steel structures
- SS-EN 1994 Eurocode 4 – Design of composite steel and concrete structures
- SS-EN 1995 Eurocode 5 – Design of timber structures
- SS-EN 1996 Eurocode 6 – Design of masonry structures
- SS-EN 1997 Eurocode 7 – Geotechnical design
- SS-EN 1998 Eurocode 8 – Design of structures for earthquake resistance
- SS-EN 1999 Eurocode 9 – Design of aluminum structures

The first subcode is specifying general rules that are not material-specific but general for all design and the second is dealing with actions and loading on structures. The remaining codes are more case specific and the relevant code has to be studied for the current case. In this thesis the first three subcodes are used together with the code for geotechnical design.

2.4 Geotechnical design

The geotechnical design is about the soil or bedrock involved for the building or the civil engineer work. Normally a construction has got some kind of connection to the ground. The most common situation is that the construction is footed on the ground directly, but piles are also commonly used. [8] These connection elements together with the surrounding soil or bedrock are called geostructures, and designing these with respect of deformations and strength of the soil, is what the geotechnical design in this thesis is about.

When designing geostructures one has to consider several possible types of failure. The ones that Eurocode is taken into account are the following [9]:

- Failure due to the geotechnical capacity in the soil (GEO)
- The impact on structural elements, resulting in failure in the soil (STR)
- Overall equilibrium in the geostructure (EQU)
- Lost equilibrium caused by hydraulic uplift or other vertical lifting forces (UPL)
- Bottom relaxation or erosion caused by hydraulic gradient (HYD)

The design code for geotechnical analyses can be found in [9]. The geotechnical design also requires information about soil parameters such as thickness, type and weight (γ) of the soil layers in the ground, the shear strength (τ_f), the ground water level (GWL), the over consolidation ratio (OCR) and the stiffness properties ($M_0, M_L, m, \sigma_c, \sigma_L$). To retrieve information of these parameters a ground investigation might be necessary, where some laboratory testing also may be performed. See [10] for more information about ground investigation and tests.

2.4.1 **Plates**

The following subchapters describes the geotechnical design for foundations with plates.

2.4.1.1 Stability analysis

To ensure that a construction is not turning over, the eccentricity of the load must be within the perimeter of the foundation.

$$e = \frac{M}{V} < B/2 \tag{2.1}$$

Where M is the bending moment acting at the bottom of the structure
V is the vertical load on the structure including the weight of the construction
B is the width of the construction (or diameter if it is a circular construction)

In general a stronger design criterion than equation 2.1 should be used because of the high pressure this will generate on the soil and the big second order moment that will occur due to the rotation of the construction.

2.4.1.2 Bearing capacity

The bearing capacity of the soil, which is a calculation in the ultimate limit state (ULS), can be calculated from the general bearing capacity (equation 2.2), where the strength parameters of the soil (c and φ) first have to be known e.g. from a geotechnical investigation. [11]

$$q_b = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + 0{,}5\gamma' B_{ef} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \tag{2.2}$$

Where q_b is the bearing capacity for the plate
c is the cohesion
q is the surrounding load at the foundation level
γ' is the effective bulk density of the soil
B_{ef} is the effective width of the footing
N_c, N_q, N_γ are bearing capacity factors depending on the friction angle
s_c, s_q, s_γ are correction factors for the shape of the footing
d_c, d_q, d_γ are correction factors for the foundation depth
i_c, i_q, i_γ are correction factors for inclined loading
g_c, g_q, g_γ are correction factors for inclined adjacent ground surface
b_c, b_q, b_γ are correction factors for inclined base area of the footing

The effective foundation area is the area where the ground pressure is assumed equally distributed and is for a square foundation with the moment bending parallel with the side:

$$B_{ef} = B - 2e_B$$

$$L_{ef} = B$$

$$A_{ef} = B_{ef} L_{ef}$$

For a square foundation with the moment bending with an angle of 45 degrees to the side the following effective width- and length:

$$B_{ef} = B - e\sqrt{2}$$
$$L_{ef} = B - e\sqrt{2}$$
$$A_{ef} = B_{ef} L_{ef}$$

Where e is the distance between the actual loading point and the resulting force, calculated with equation 2.1.

The case which gives the smallest bearing capacity is the effective area representation that should be chosen, see figure 2.4.

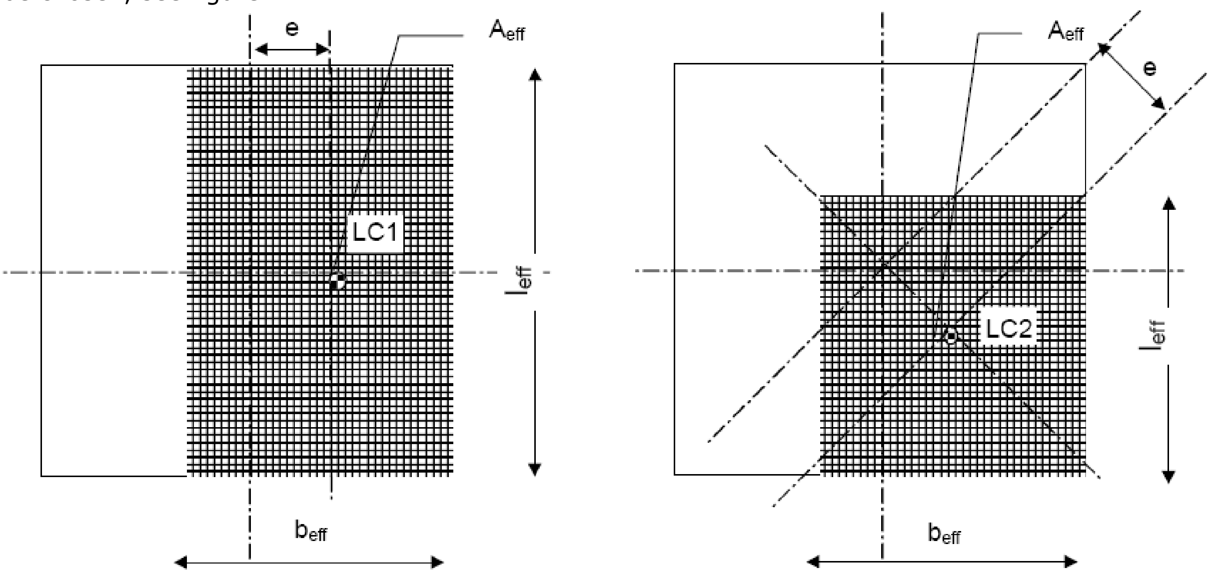


Figure 2.4: Effective area for a square foundation [E]
Left: the moment bending parallel to the side
Right: the moment bending with an angle of 45 degrees to the side

According to [12] the effective area for a circular foundation can be expressed as a rectangular area that origins from an elliptical area see figure 2.5. The effective area can be calculated as:

$$A_{eff} = 2\left[R^2 \arccos\left(\frac{e}{R}\right) - e\sqrt{R^2 - e^2}\right] \tag{2.3}$$

With the elliptic major axes:

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

And the equivalent effective dimensions:

$$L_{eff} = \sqrt{A_{eff}\,\frac{l_e}{b_e}}$$

$$B_{eff} = \frac{L_{eff}}{l_e} b_e$$

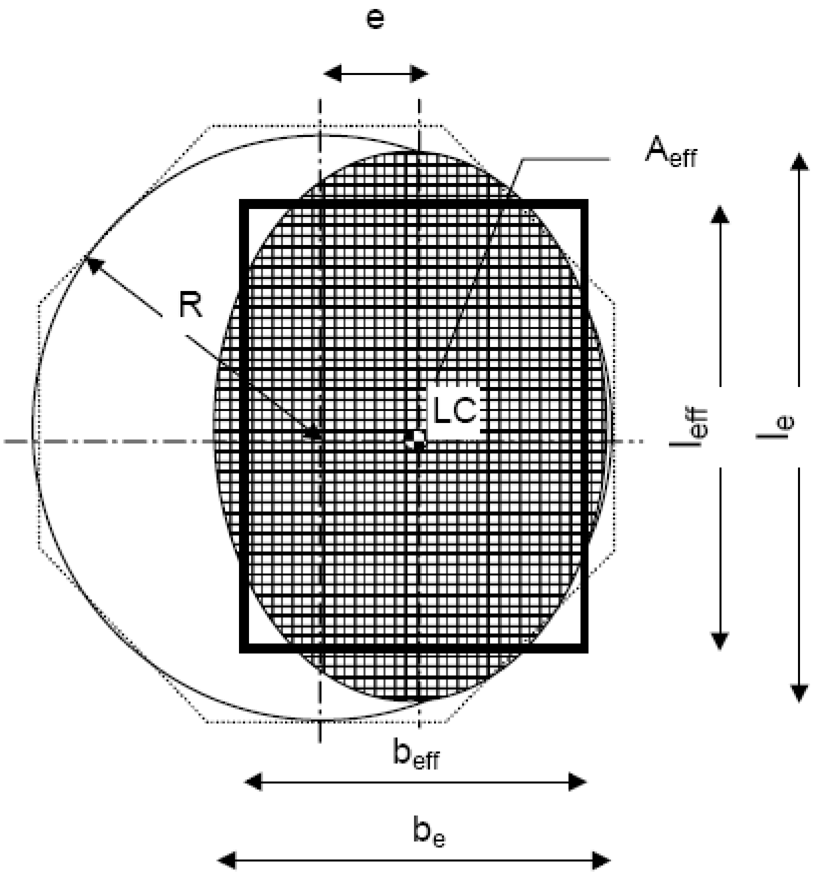


Figure 2.5: Effective area for a circular foundation. [E]

The bearing capacity factors ($N_i \; i = c, q, \gamma$) can be calculated through:

$$N_q = \frac{1+\sin\varphi}{1-\sin\varphi} e^{\pi \tan\varphi}$$

$$\begin{cases} N_c = \pi + 2 & ,\varphi = 0 \\ N_c = \dfrac{N_q - 1}{\tan\varphi} & ,\varphi > 0 \end{cases}$$

$$N_\gamma = F(\varphi)\left[\frac{1+\sin\varphi}{1-\sin\varphi} e^{\left(\frac{3\pi}{2}\tan\varphi\right)} - 1\right]$$

Where $F(\varphi) = 0.08705 + 0.3231\sin(2\varphi) - 0.04836\sin^2(2\varphi)$

The shape factors are defined as:

$$s_c = \begin{cases} 1 + 0.2\dfrac{B_{ef}}{L_{ef}} & ,\varphi = 0 \\ 1 + \dfrac{N_q}{N_c}\dfrac{B_{ef}}{L_{ef}} & ,\varphi > 0 \end{cases}$$

$$s_q = 1 + (\tan\varphi)\frac{B_{ef}}{L_{ef}}$$

$$s_\gamma = 1 - 0.4\frac{B_{ef}}{L_{ef}}$$

The correction factors regarding the foundation depth are defined:

$$d_c = d_q = 1 + 0.35\frac{d}{B_{ef}}\ ;\ d_c, d_q \le 1.7$$

$$d_\gamma = 1$$

The correction factors regarding inclined loading are defined:

$$i_q = \left(1 - \frac{H}{V + B_{ef}L_{ef}c\cot\varphi}\right)^m$$

$$i_c = \begin{cases} 1 - \dfrac{mH}{B_{ef}L_{ef}cN_c} & ,\varphi = 0 \\ i_q - \dfrac{1 - i_q}{N_c\tan\varphi} & ,\varphi > 0 \end{cases}$$

$$i_\gamma = \left(1 - \frac{H}{V + B_{ef}L_{ef}c\cot\varphi}\right)^{m+1}$$

Where H is the horizontal load vector
V is the vertical load vector

$$m = \begin{cases} m_B = \dfrac{2 + \dfrac{B_{ef}}{L_{ef}}}{1 + \dfrac{B_{ef}}{L_{ef}}} \\ m_L = \dfrac{2 + \dfrac{L_{ef}}{B_{ef}}}{1 + \dfrac{L_{ef}}{B_{ef}}} \end{cases}$$

Where the first equation is to be used when H acting in the width direction, and the second when H acting in length direction.

The correction factor regarding inclined adjacent surface are defined:

$$g_c = \begin{cases} 1 - \dfrac{2\beta}{N_c} & ,\varphi = 0 \\ e^{-2\beta\tan\varphi} & ,\varphi > 0 \end{cases}$$

$$g_q = g_\gamma = 1 - \sin 2\beta$$

Where β is the inclination of the ground regarding the horizontal plane.

The correction factors regarding inclined base area of the footing are defined:

$$b_q = b_\gamma = (1 - \alpha\tan\varphi)^2$$

$$b_c = \begin{cases} 1 - \dfrac{2\alpha}{\pi + 2} & ,\varphi = 0 \\ b_q - \dfrac{1 - b_q}{N_c \tan\varphi} & ,\varphi > 0 \end{cases}$$

In case of extremely eccentric loading, i.e. an eccentricity that is bigger than 30 percent of the foundation width, an additional bearing capacity calculation needs to be carried out according to [12]. This capacity corresponds to rupture 2, in figure 2.6 and is a failure in the soil under the foundation, why this capacity doesn't contain the additional capacity from the soil next to the foundation. The bearing capacity is to be taken as the smallest value of the capacity calculated for rapture 1 and rupture 2. The capacity for rupture 2 can be calculated with the following expression:

$$q_b = cN_c s_c d_c i_c g_c b_c (1.05 + \tan^3(\varphi) + \gamma' B_{ef} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \qquad (2.4)$$

Where all the parameters are defined the same as for rupture 1 except the correction factors regarding inclined loading which instead are defined as:

$$i_q = \left(1 + \frac{H}{V + B_{ef} L_{ef} c \cot\varphi}\right)^m$$

$$i_c = \begin{cases} 1 + \dfrac{mH}{B_{ef} L_{ef} c N_c} & ,\varphi = 0 \\ i_q - \dfrac{1 - i_q}{N_c \tan\varphi} & ,\varphi > 0 \end{cases}$$

$$i_\gamma = \left(1 + \frac{H}{V + B_{ef} L_{ef} c \cot\varphi}\right)^{m+1}$$

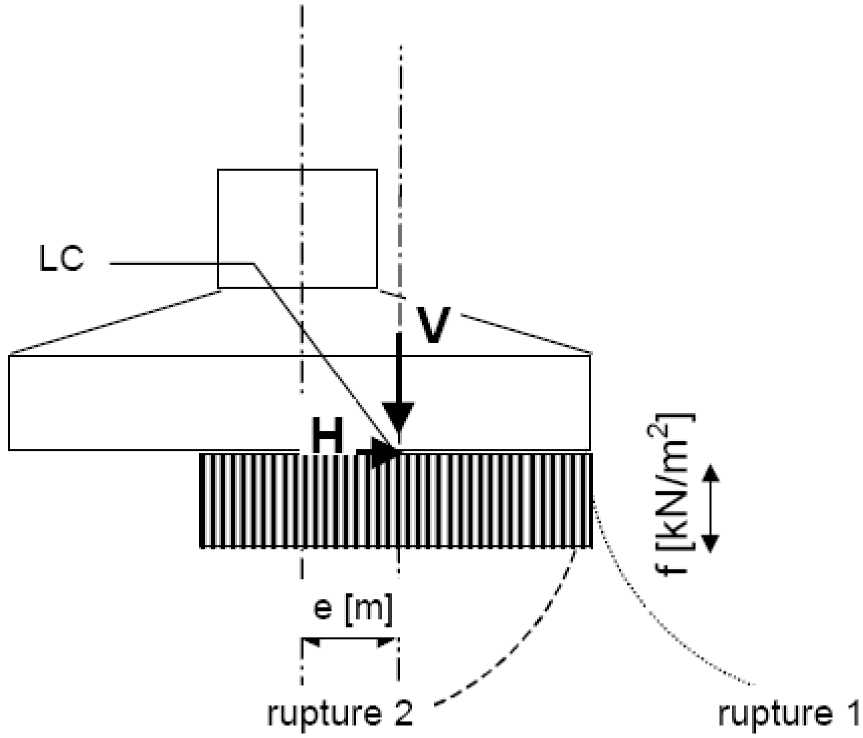


Figure 2.6: Two types of failure exist, rapture 1 and rapture 2 [E]

If a twisting moment (M_z) is acting on the foundation this can be accounted for by replacing the horizontal force H in the calculations with a slightly bigger equivalent horizontal force H´ which according to [12] can be calculated as:

$$H' = \frac{2M_z}{L_{eff}} + \sqrt{\left(\frac{2M_z}{L_{eff}}\right)^2 + H^2} \tag{2.5}$$

Foundations that are subjected to a horizontal loading must also be investigated for sufficient sliding resistance. This condition is, according to [12], fulfilled by the following expression:

$$\frac{A_{eff}\,c + V\tan(\varphi)}{H} > 1 \tag{2.6}$$

In addition to equation 2.6 the following expression must also be fulfilled:

$$\frac{H}{V} < 0.4$$

2.4.1.3 Settlements calculation

In order to calculate settlements under the footing, the stiffness of the soil, i.e. the compression modulus (M) first has to be determined. This can be a problem, because it varies very much between different soils and moreover it depending upon the stress state of the soil.

The stiffness of a fine grained soil is relative constant (M_0) when the soil is overconsolidated, but if the increasing pressure reaches the preconsolidation pressure (σ_c) the modulus is drastically decreased (M_L). If the pressure increases even more the modulus is increasing almost linearly, with the gradient m, see figure 2.7. The curve that defines the stiffness of a fine grained soil can be determined through an oedometertest, nowadays these tests are performed with a constant rate of strain why it's called CRS-test. [13]

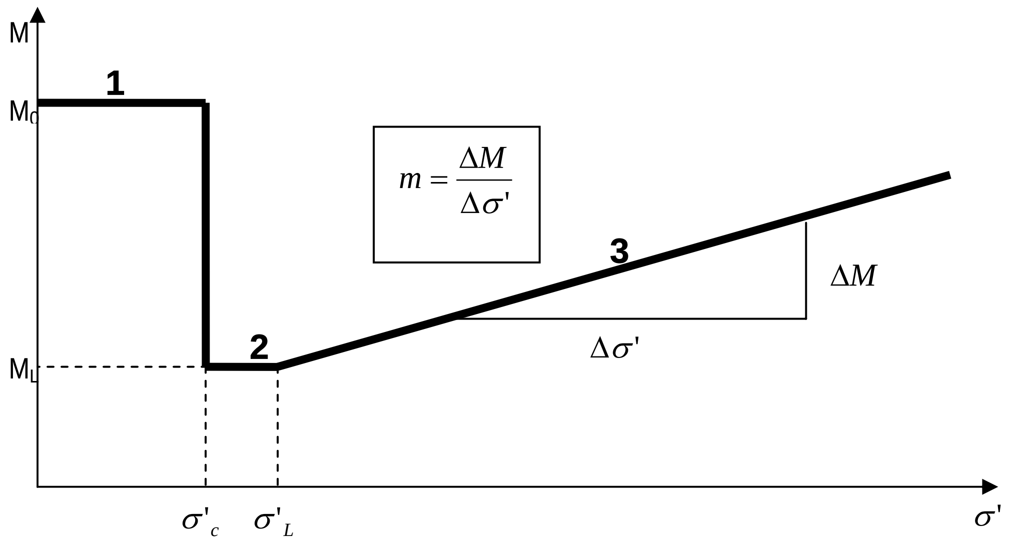


Figure 2.7: Compression modulus as a function of the effective stress and different stress phases indicated with numbers

As the effective stress increases with depth, the modulus (M) will also vary, and therefore settlements is calculated for finite layers in the soil, and after that summarized to a total settlement, see equation 2.7 which is described in [11].

$$\delta = \sum_{i=1}^{n}\left(\frac{\sigma'_c - \sigma'_0}{M_0} + \frac{\sigma'_L - \sigma'_c}{M_L}\frac{1}{m}\ln\frac{(\sigma' - \sigma'_L)m}{M_L} + 1\right)\Delta h_i \qquad (2.7)$$

In equation 2.7 every term corresponds to a specific phase. The first term concerns the over consolidated phase, the second corresponds to the stress interval between the over consolidation stress and the stress where the modulus starts to increase linearly, and the third term concerns the normal consolidation phase where the stiffness increases linearly. For each sublayer calculated should every exceeded stress state be included in the calculation, which for example means that for a sublayer with a current stress state just above the preconsolidation stress both the first- and the second term should be included. See figure 2.7 for the governing equation for the phases.

Phase 1, Over consolidated stress state $0 \le \sigma' \le \sigma'_c$:

$$\delta_i = \frac{\sigma' - \sigma'_0}{M_0}\Delta h_i$$

Phase 2, Intermediate stress state $\sigma'_c \le \sigma' \le \sigma'_L$:

$$\delta_i = \left(\frac{\sigma'_c - \sigma'_0}{M_0} + \frac{\sigma' - \sigma'_c}{M_L}\right)\Delta h_i$$

Phase 3, Normal consolidated stress state $\sigma' \ge \sigma'_L$:

$$\delta_i = \left(\frac{\sigma'_c - \sigma'_0}{M_0} + \frac{\sigma'_L - \sigma'_c}{M_L}\frac{1}{m}\ln\frac{(\sigma' - \sigma'_L)m}{M_L} + 1\right)\Delta h_i$$

For coarse-grained soils the compression modulus can, according to [13], be determined from the expression:

$$M = m\sigma'_r \left(\frac{\sigma'}{\sigma'_r}\right)^{1-\beta} \tag{2.8}$$

Where σ'_r is a reference pressure, usually chosen to be 100kPa [13]

m and β is determined from the actual porosity, see figure 2.8

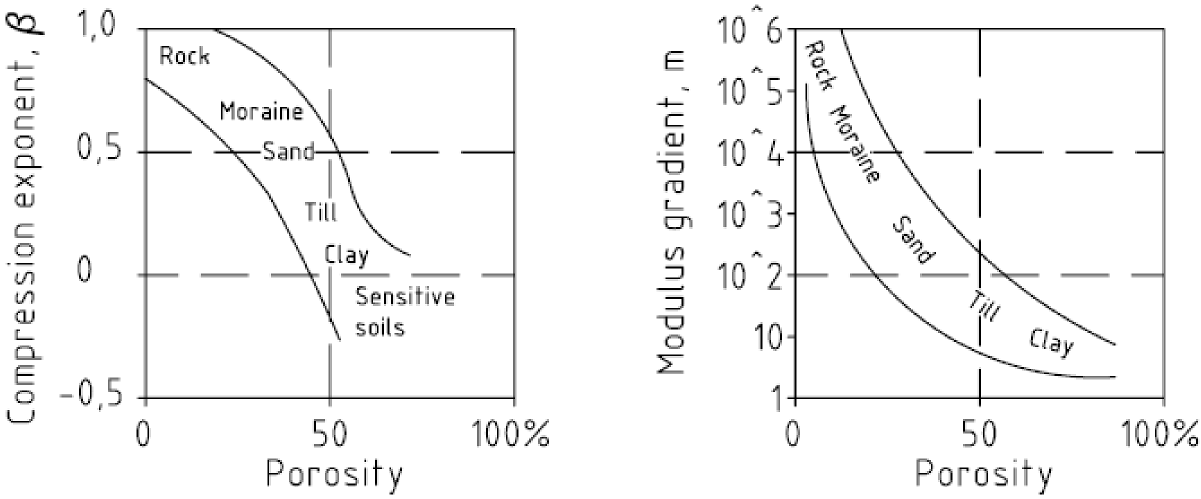


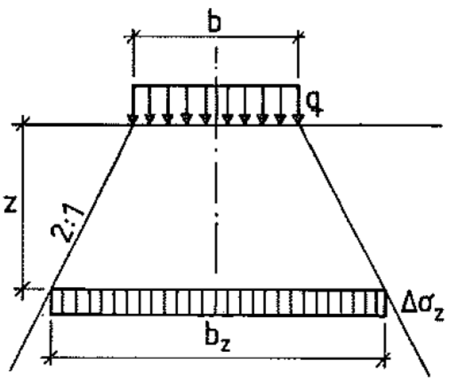
Figure 2.8: Diagrams for determination of the parameters β and m. [F]

The total settlements are calculated as the same principle as for fine-grained soils (see equation 2.7) with a summarization of many finite layers:10[6]

$$\delta = \sum_{i=1}^{n} \left\{ \frac{1}{m\beta} \left[\left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_r} \right)^{\beta} - \left(\frac{\sigma'_0}{\sigma'_r} \right)^{\beta} \right] \right\} \Delta h_i \tag{2.9}$$

According to [11] calculations should be performed until the depth where the pressure that origin from the load is 10% of the current effective stress.

A common method to calculate how the load is spreading downwards in the soil is the 2:1-method which means that the loading is spreading within an area that limits of planes with the inclination 2:1, see figure 2.9.



Figur 2.9: The 2:1-method in one dimension. [G]

The loading contribution according to the 2:1-method is:

$$\Delta\sigma_z = \frac{qb}{b+z} \quad \text{, in one dimension} \tag{2.10}$$

$$\Delta\sigma_z = \frac{qbl}{(b+z)(l+z)} \quad \text{, in two dimensions} \tag{2.11}$$

$$\Delta\sigma_z = \frac{Q}{z^2} \quad \text{, for a point load Q} \tag{2.12}$$

2.4.2 Piles and piling

A pile is an elongated vertical (or almost vertical) thin construction element which main task is to conduct load from soil near the surface with less strength, to deeper soil layers with better strength parameters, or it can be standing on the bedrock [8]. There are many possible solutions for a pile foundation, where the way of function can be different; the piling method can vary, but also the pile material.

The piles can be installed in the soil in different ways, where the most commonly used methods are driven piles and drilled piles. Driven piles are piles driven in the soil by a pile driver machine. This machine makes use of a heavy weight that falls from its highest point on the top of the pile forcing the pile to drive into the ground. The event is repeated over and over until the pile toe reaches the bedrock or a sufficient firm soil layer.

Drilled piles are piles that are installed with a drilling machine. There are a few different types of drilled piles. One type of drilled piles is when the machine drills a hole and installing a steel pipe pile directly in the hole. Another type is when the pile is brought down in an already drilled hole. Piles with very big diameter can be constructed by pouring concrete in already drilled or dug holes, so called bored piles. Before concrete is poured a reinforcement cage can be put in place to improve the pile strength.

There exists many different types of piles, but the most common types is reinforced concrete piles, steel piles and timber piles, where the last mentioned is not frequently used today. In Sweden the use of prefabricated reinforced concrete piles is dominating the market [8]. Piles of steel or iron is generally having a smaller area wherefore they give arise to smaller mass displacement, which can be very significant. Some common profiles of steel piles are hollow-profiles, H-profile, the X-profile, see figure 2.10. Concerning the steel pipes piles they are often filled with concrete, and if the diameter is big a reinforcement cage can be put inside the pipe to increase the strength properties.

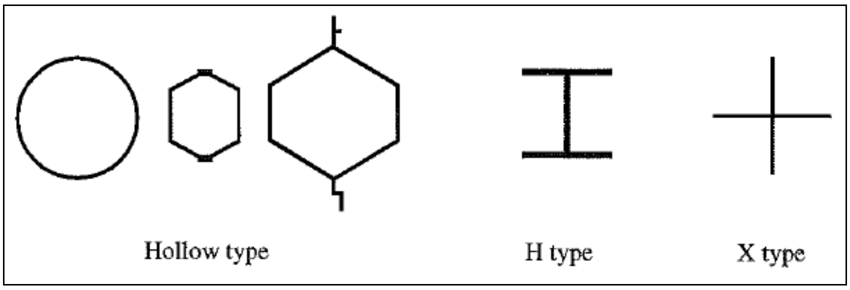


Figure 2.10: Different types of steel piles [H]

2.4.2.1 Bearing capacity - General

The bearing capacity of piles can be divided into two different capacities; the geotechnical capacity and the structural capacity. The one that has got the lowest strength is defining the bearing capacity of the element. At first the geotechnical capacity will be investigated.

2.4.2.2 Geotechnical bearing capacity

When a pile is exposed to a vertical force, shear stresses are developed around the pile and if these stresses exceed the capacity of the soil a ground failure will occur.

There are mainly two ways for a pile to bear load, where the first is the toe capacity, which is the capacity from the lower pile tip, and the second is the shaft bearing capacity, which is the capacity due to friction or cohesion along the perimeter of the pile. The combination of these two also exists and is the bearing capacity of the soil, see equation 2.13.

$$R_{tot} = R_{toe} + R_{shaft} = f_{toe} A_{toe} + f_{shaft} A_{shaft} \tag{2.13}$$

Where R is the resistance

f_{toe} is the compressive strength of the soil at the pile toe level

A_{toe} is the area at the pile section at the toe

f_{shaft} is the average friction strength at the interface between the soil and the shaft

A_{shaft} is the area of the shaft

Friction piles

Piles installed in friction soils are called friction piles. These have bearing capacity both from the toe and the shaft. The toe resistance can according to [14] be expressed in terms of a bearing capacity factor, N_q and the shaft resistance by an empirical factor β. The constants are then multiplied with the actual effective vertical stress, see equations 2.14 and 2.15.

$$f_{toe} = N_q \sigma'_v \tag{2.14}$$

$$f_{shaft} = \beta \overline{\sigma}'_v \tag{2.15}$$

Where σ'_v denoting the effective vertical stress at the pile toe level

$\overline{\sigma}'_v$ denoting the mean effective vertical stress along the pile

β is a product of the pressure coefficient ratio, K and the coefficient of friction between the earth and the soil, $\tan\delta$, see figure 2.11.

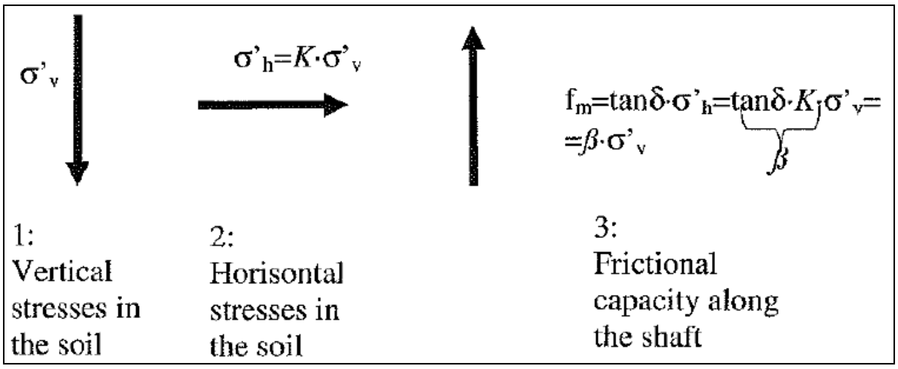


Figure 2.11: Definition of the β -factor [J]

Values of N_q and β can for example being found in [15], which can be seen in figure 2.12.

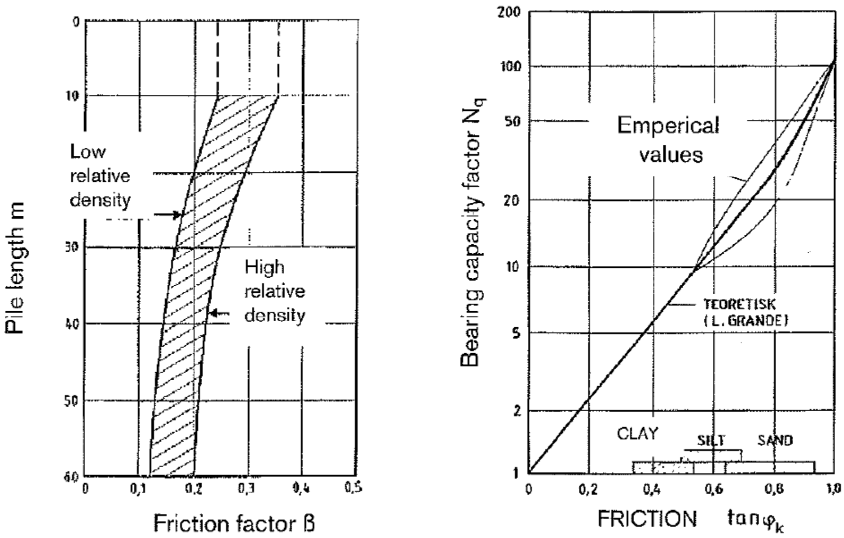


Figure 2.12: Values of β and N_q from [J]

If the soil has got low strength, and the bedrock or a firm soil layer is at a reasonable depth it might be appropriate to drive the piles to the bedrock or a firm soil layer. In that case the geotechnical bearing capacity is determined in terms of the bedrock's or the firm soil layer's bearing capacity. In Sweden the bedrock is in general of good quality and the structural bearing capacity of the pile is then limiting the capacity of the pile [8]. If the pile is standing on a firm soil layer the toe resistance is dominating the bearing capacity, but some resistance from the shaft can be accounted or [8]. The bearing capacity of the bedrock can be verified through a special measurement during the piledriving that measure how much the pole descends from a blow of the piledriving machine.

If a large bending moment is acting on a piled construction some piles might be exposed to tension force which the pile also has to deal with. The bearing capacity for tensional loading can be analyzed almost in the same way, but of course without the resistance from the pile toe. Besides the loss for the toe, the cohesion and the friction from the shaft reduced. A typical reduction is according to [8] and [14] 40-50% for friction piles.

Cohesion piles

In cohesive soils such as clays, the toe resistance is very low and can therefore be neglected [8]. The shaft resistance is here present in terms of cohesion instead of friction. The cohesion is proportional to the shear strength (τ_f) at the actual level (z) in the soil, with a proportional dimensionless constant (α), measuring the relative adhesion. If $\alpha = 1.0$ fully adhesion is met, and if $\alpha = 0.0$ no adhesion is present.

$$f_{shaft} = \alpha \cdot \tau_f(z) \tag{2.16}$$

The shaft bearing resistance, R can then be expressed as:

$$\begin{aligned} dR &= f_{shaft} \cdot P_{shaft} \cdot dz = \alpha \cdot \tau_f(z) \cdot P_{shaft} \cdot dz \\ \Rightarrow R &= \int_{L_P} \alpha \cdot \tau_f(z) \cdot P_{shaft} \cdot dz \end{aligned} \tag{2.17}$$

Where P_s is the perimeter of the pile

L_P the length of the pile.

According to [8] the values in table 2.1 can be used for the adhesion factor α.

Table 2.1: Adhesion factor for different pile materials

Pile material	α
Wooden pile	0.8
Concrete pile	0.7
Steel pile	0.7

In the same way as for friction piles a reduced value for the cohesion should be used when the piles are exposed to tensional force. A typical reduction is according to [8] and [14] 20-30% for cohesion piles.

2.4.2.3 Structural bearing capacity

The structural capacity is the capacity of the pile itself. There are two different types of failure regarding the structural capacity.

- Failure - due to the strength of the material
- Failure – due to instability

Both failure modes have to be considered and the smallest value is determining the structural capacity.

Failure – due to the strength of the material

This mode occurs when the capacity of the cross section is reached. The capacity of the cross section can be calculated with regular theory about structural mechanics. In [9] there are general design rules for structures. The following expression has to be fulfilled.

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1 \tag{2.18}$$

Where N_{Ed}, M_{Ed} denoting sectional forces

N_{Rd}, M_{Rd} denoting the resistance of the material

The actual moment should be calculated with second order theory and thus an additional moment will arise due to deflection from axial force and initial imperfections. The calculation of the deflection should account for the lateral soil resistance. The initial imperfections can be measured after the installation of the piles, but standard values that can be used in calculations can be found in [9].

Failure – due to instability

This mode of failure is apparent when the load forces the pile to buckle. The lateral support of the soil will prevent the pile to buckle, therefore the stronger soil the less risk for buckling. In addition the slenderness of the element is significant. In [16] there is a calculation method for how to calculate the critical load and the critical length, which take into account the lateral support from the soil. The method assumes a sinusoidal deflection curve of both the initial deflection and the deflection from the loading.

$$P_{cr} = 2\sqrt{k_d \cdot d \cdot E_d \cdot I} \qquad (2.19)$$

$$L_{cr} = \pi \sqrt[4]{\frac{E_d \cdot I}{k_d \cdot d}} \qquad (2.20)$$

Where P_{cr} is the critical load (buckling load) for the pile

k_d is the design value for the subgrade reaction

d is the transverse dimension of a pile

E_d is Young’s modulus for the pile material

I is the moment of inertia for the pile

L_{cr} is the critical length (buckling length) of the pile

Suitable values for the lateral stiffness k_d can be found in literature and is dependent on the time the load is acting, and the soils shear strength. For cohesion soils the lateral stiffness is relatively small and constant along the pile, but for friction soils it is much bigger and often said to increase linearly with depth [17].

2.4.2.4 Settlements calculation

The settlements of a pile consists of three different parts; the compression in the pile, the slip between the pile and the ground and the ground settlement. It is generally hard to distinguish clearly limits between the different parts. Nowadays these calculations are often done in terms of numerical analyses with the finite element method (FEM). Such analyses must be performed by computer software to obtain reasonable results. Computer software used for such calculations is PLAXIS, se chapter 4.3 for a more detailed description of the software. In chapter 5 where the design of foundations for wind turbines is done the settlement calculations is performed with PLAXIS.

However, there are non-numerical methods to calculate settlements for piles as well. One method presented in [14] is the neutral plane concept which is based on first determining the level of a neutral plane, where the slip between pile and soil is zero. This is done by setting up an equilibrium equation for the pile, as the pile is at rest. On the one side of the equality there is the action effect, and on the other side is the resistance. The action effect is the sum of the long-time loading and the shaft friction, which is a down drag force above the neutral plane.

$$E = F_{\infty} + \int_{0}^{z} f_{shaft} dA \tag{2.21}$$

In the same way the resistance is a sum of two terms; the toe resistance and the shaft friction below the neutral plane.

$$R = R_{toe} + \int_{z}^{L_p} f_{shaft} dA \tag{2.22}$$

The equilibrium can now be established, where z is the actual level for the neutral plane.

$$E = R \Leftrightarrow F_{\infty} + \int_{0}^{z} f_{shaft} dA = R_{toe} + \int_{z}^{L_p} f_{shaft} dA \tag{2.23}$$

When the neutral plane is determined the settlements can be calculated for the different parts. Due to the relative high stiffness in the pile, the compression in the pile can be neglected and only the deformations in the ground below the neutral plane is considered. The settlements below the neutral plane can be calculated with traditional settlement calculations as done in chapter 2.4.1.3. The loading is then moved to the neutral plane and can be assumed to be distributed in the soil according to the 2:1-method, described in chapter 2.4.1.3.

2.5 Structural design

The structural design is about designing structures that should be built. The main task is to create descriptions and drawings for the construction workers. If a structure is built according to these documents it should serve its purpose. Therefore the purpose and the conditions must be well determined before a designer starts his work. The purpose and the conditions can be stated for example in the following manner:

- The lifetime of the construction
- The loads that will affect the construction
- The climate effects that will interact with the building
- The esthetical aspects

A designer's task is then to assure that these conditions are fulfilled. In this work the structural design focus on retaining the safety of the construction.

In the following subchapters it is described how to design reinforced concrete structures according to Eurocode.

2.5.1 Concrete cover

To prevent reinforcement to corrode it is important to have a sufficient concrete cover around the rebars. The required thickness of the concrete cover depends on several factors:

- The design life length of the structure
- The quality of the concrete
- The exposure to chlorides
- The variation of wet- and dry state
- The exposure to chemical aggressive environment
- If the concrete surfaces are vertical or horizontal
- Whether the construction is above or below frost-free depth.

These factors results in different exposure classes which in turn determine the thickness of the concrete cover. Appendix A lists the choice of exposure classes and appendix B contains a template for how to determine the thickness for the concrete cover from its exposure classes.

2.5.2 **Internal forces**

Forces acting on structures give rise to internal forces in the structure. These forces are of major importance and the design is done according to those. For structures made up of beams there are three types of internal forces:

- Normal force
- Shear forces
- Moments

Normal force is force acting in the axial direction of a bar, beam or another structural element, and the shear forces are the transversal forces. Moments are bending or twisting the structure.

2.5.3 **Design for bending moment**

If a beam section is exposed to a bending moment, the concrete must resist the compression that occurs in the compressed zone and the reinforcement must handle the tension force in the other part of the cross section.

The design in this section refers to Eurocode 2 – Design of concrete Structures chapter 6.1. [18]

For an ultimate limit state analysis the compression zone of a concrete beam can be assumed to be rectangular, and all the tension will be taken in the reinforcement. Figure 2.13 shows the design stress distribution for a concrete beam.

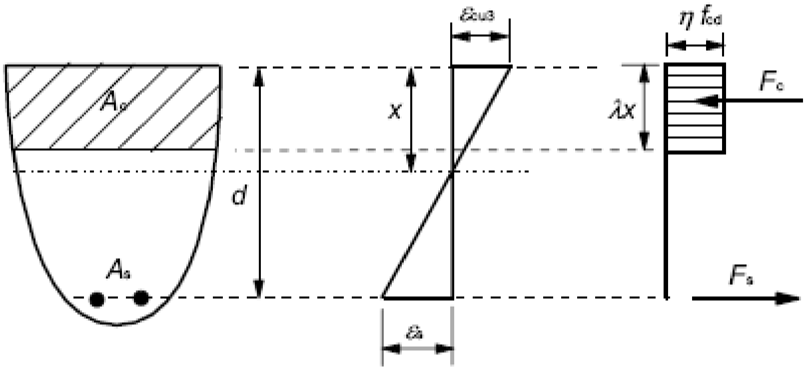


Figure 2.13: The design model for bending moment [K]

The height of the compression zone is set to λx where

$$\lambda = 0.8 \qquad \text{, for } f_{ck} \leq 50\,MPa$$

$$\lambda = 0.8 - \frac{f_{ck} - 50}{400} \qquad \text{, for } 50\,MPa \leq f_{ck} \leq 90\,MPa$$

The compression stress in the concrete is set to ηf_{cd} where

$$\eta = 1.0 \qquad \text{, for } f_{ck} \leq 50\,MPa$$

$$\eta = 1.0 - \frac{f_{ck} - 50}{200} \qquad \text{, for } 50\,MPa \leq f_{ck} \leq 90\,MPa$$

The compression strain in the concrete is set to ε_{cu3} where

$$\varepsilon_{cu3} = 3.5‰ \qquad \text{, for } f_{ck} \leq 50\, MPa$$

Simple equilibrium equations give:

$$(\rightarrow) \qquad F_s = F_c \Leftrightarrow A_s \sigma_s = f_{cd} \lambda x b \tag{2.24}$$

$$(\curvearrowright) \qquad M = \begin{cases} F_s (d - 0{,}5\lambda x) \\ F_c (d - 0{,}5\lambda x) \end{cases} \tag{2.25}$$

According to the rule of uniform triangles the following strain ratio is found:

$$\frac{\varepsilon_s}{d - x} = \frac{\varepsilon_{cu3}}{x} \tag{2.26}$$

It is wise to design the section so that the reinforcement will yield before the concrete to prevent a brittle and suddenly failure. This means that the stress in the reinforcement is set to be the yield stress. ($\sigma_s = f_{yd}$)

It is then important to investigate that the strain in the reinforcement really is bigger than the yield strain.

$$\varepsilon_s > \varepsilon_{sy} = \frac{f_{yd}}{E_s} \tag{2.27}$$

A common approach is to first determine the distance to the neutral layer (x), which can be done by solving equation 2.25 for x and then calculate the reinforcement strain with equation 2.26. The strain should now be compared with the yield strain (equation 2.27) to see whether the failure is brittle or not. Then the required reinforcement area can be calculated from equation 2.24.

If the moment and/or the geometry of the section vary along the beam’s length, calculations should be performed in several sections. This can of course result in different amounts of reinforcement for the sections.

2.5.4 **Design for shear**

Beams that are exposed to shear force must have sufficient resistance against the shear stress that will occur in the section. The concrete itself has got some resistance, and the governing parameters are then the amount of the concrete in the section and the concrete resistance for shear stress. Beside the resistance from the concrete, the bottom- and top reinforcement in the section that are put in for the bending resistance will give shear capacity. If the shear capacity from the concrete and the bending reinforcement isn’t enough extra shear reinforcement is required. Extra shear reinforcement is then apparent in forms of stirrups.

2.5.4.1 Shear capacity without stirrups

The shear capacity without shear reinforcement, but with bending reinforcement, can be calculated as:

$$V_{Rd,c} = C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} bd \tag{2.28}$$

Where $\quad C_{Rd,c} = 0.18 / \gamma_C$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

$$\rho_l = \frac{A_{sl}}{bd} \leq 0.02$$

f_{ck} is the compression strength for the concrete in MPa

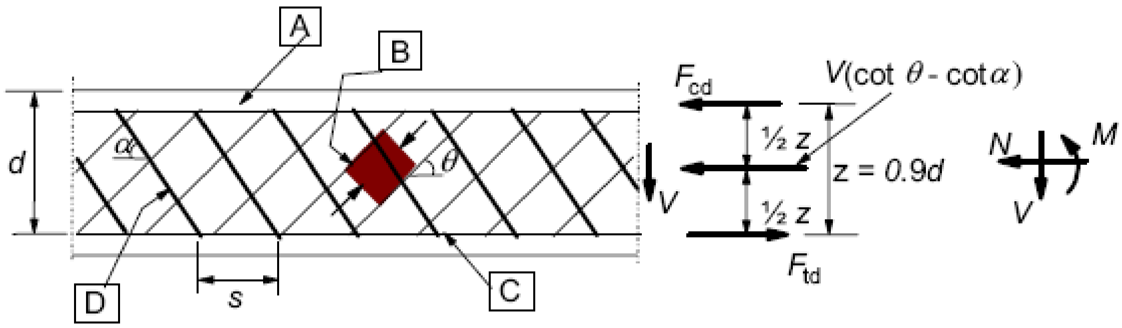
d is the effective height of the beam in mm

The capacity must not be lower than the capacity from only the concrete:

$$V_{Rd,c} \geq 0.035 k^{3/2} f_{ck}^{1/2} bd \qquad (2.29)$$

2.5.4.2 Shear capacity with stirrups

The design of the stirrups is done by a strut- and tie-model which can be seen in figure 2.14.



A – Compressed rebar, B – Strut, C – Tensioned rebar, D – Shear stirrup

Figure 2.14: Strut- and tie model for the design of shear stirrups [K]

For sections with vertical shear stirrups ($\alpha = 90^0$) the shear capacity is the smallest of equation 2.30 and 2.31:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot(\theta) \qquad (2.30)$$

$$V_{Rd,\max} = \frac{\alpha_{cw} b z \nu_1 f_{cd}}{\cot(\theta) + \tan(\theta)} \qquad (2.31)$$

Where $1 \leq \cot(\theta) \leq 2.5$

A_{sw} is the reinforcement area of the stirrups

$\alpha_{cw} = 1$, for structures without pretension

$$\nu_1 = 0.6\left(1 - \frac{f_{ck}}{250}\right) \qquad (2.32)$$

f_{ywd} is the design value for the yield stress in the shear stirrups

f_{cd} is the design value for the compression strength in the concrete

2.5.5 **Design for fatigue loading**

Structures exposed to cyclic loading must be designed for fatigue. Both the concrete and the reinforcement must be designed for fatigue.

2.5.5.1 Control for fatigue in concrete

Sufficient fatigue capacity in the concrete exists if the following condition is met:

$$\frac{\sigma_{c,\max}}{f_{cd,fat}} \leq 0.5 + 0.45\frac{\sigma_{c,\min}}{f_{cd,fat}} \leq 0.9 \text{, for } f_{ck} \leq 50\,MPa$$

$$\frac{\sigma_{c,\max}}{f_{cd,fat}} \leq 0.5 + 0.45\frac{\sigma_{c,\min}}{f_{cd,fat}} \leq 0.8 \text{, for } f_{ck} \geq 50\,MPa \tag{2.33}$$

Where $\sigma_{c,\max}$ is the highest compression stress in the section exposed the cyclic loading

$\sigma_{c,\min}$ is the lowest compression stress in the section exposed to cyclic loading. If the section has got tension stress $\sigma_{c,\min}$ is set to 0.

$$f_{cd,fat} = f_{cd}\left(1 - \frac{f_{ck}}{250}\right) \tag{2.34}$$

σ_c is calculated as:

$$\sigma_c = \frac{M}{0{,}5xbz} \tag{2.35}$$

Where M is the bending moment in ULS
z is the inner lever calculated as:

$$z = d - x/3 \tag{2.36}$$

x is the distance to the neutral layer calculated as:

$$x = d\alpha\rho\left[\sqrt{1 + \frac{2}{\alpha\rho}} - 1\right] \tag{2.37}$$

$$\rho = \frac{A_s}{bd} \tag{2.38}$$

$\alpha = \frac{E_s}{E_{c,eff}}$ is the ratio between the stiffness's of the reinforcement and the concrete.

$$E_{c,eff} = \frac{E_{cm}}{\varphi_{eff} + 1} \tag{2.39}$$

φ_{eff} is the creep factor

2.5.5.2 Control for fatigue in reinforcement

Sufficient fatigue capacity in the reinforcement exists if the following condition is met:

$$\Delta\sigma_s = \sigma_{s,\max} - \sigma_{s,\min} \leq \Delta\sigma_{Rsk} = 70\,MPa \tag{2.40}$$

Where $\Delta\sigma_s$ is the stress width in the reinforcement for the cyclic loading

$\Delta\sigma_{Rsk}$ is the maximum allowed stress width in the reinforcement for the cyclic loading

σ_s is calculated as:

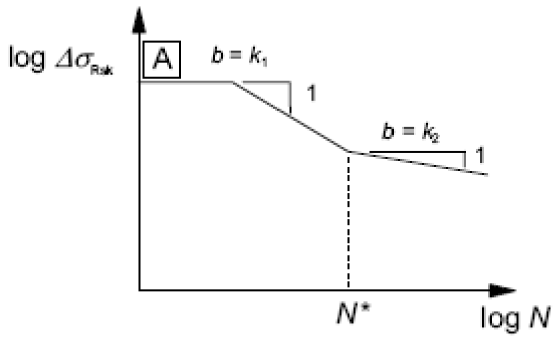
$$\sigma_s = \frac{M}{A_s z} \tag{2.41}$$

Where M is the bending moment for minimum or maximum fatigue load

A_s is the reinforcement area

z is the inner lever defined in equation 2.36

Equation 2.40 is a simplification and an assumption on the safe side. If the stress width is bigger than 70 MPa, a more accurate value for the allowed stress width can be calculated with a simplified Wöhler curve, see figure 2.15. The curve is determined through the stress exponents k_1 and k_2, and one point on the curve $(N^*, \Delta\sigma_{Rsk})$. For this curve a specific fatigue stress width can be determined from the actual number of cycles. Note that the quantities on the Wöhler curve are logarithmic.



A – Yield stress for the reinforcement

Figure 2.15: Simplified Wöhler curve for reinforcement [K]

2.5.6 Control of crack width

Generally, concrete exposed to bending, shear, twisting or tension has cracks. These cracks must not be too big to maintain the structure's function over its lifetime. The acceptable crack width is depending on the environmental exposure of the structure. For example a structure exposed to salt water is more sensitive to big cracks than a structure not exposed to salt water. In table 2.2 the maximum allowed crack width with respect to the exposure class and the design life can be seen. See appendix A for a specification of the exposure classes.

Table 2.2: Acceptable crack width in mm

Exposure class	100 years	50 years	20 years
XC0	-	-	-
XC1	0.45	-	-
XC2	0.40	0.45	-
XC3, XC4	0.30	0.40	-
XS1, XS2, XD1, XD2	0.20	0.30	0.40
XS3, XD3	0.15	0.20	0.30

Characteristic crack width (w_k) can be calculated by first determining a mean strain for the cracks ($\varepsilon_{sm} - \varepsilon_{cm}$) and also a largest distance between the cracks ($s_{r,\max}$):

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{2.42}$$

Where ε_{sm} is the mean strain in the reinforcement

ε_{cm} is the mean strain in the concrete

The mean strain in the cracks can be calculated as:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0.6\frac{\sigma_s}{E_s} \tag{2.43}$$

Where σ_s is the stress in the reinforcement for the cracked section calculated as:

$$\sigma_s = \frac{M}{A_s z} \tag{2.44}$$

Where M is the bending moment in SLS

A_s is the reinforcement area

z is the inner lever calculated with equation 2.36

k_t is a coefficient that depends of the duration of the loading

$k_t = 0.6$ for short time loading

$k_t = 0.4$ for long term loading

$f_{ct,eff}$ is the mean value for the concrete tension strength when the first crack is expected to arise. If loading occurs 28 days or more after the casting then $f_{ct,eff} = f_{ctm}$

$\rho_{p,eff}$ is the effective reinforcement content of the section defined as:

$$\rho_{p,eff} = \frac{A_s}{A_{c,eff}} \tag{2.45}$$

Where A_s is the reinforcement area in the tension zone

$A_{c,eff}$ is the effective concrete area in tension zone defines as the smallest of the following:

$$\begin{cases} A_{c,eff} = 2.5(h-d)b \\ A_{c,eff} = \dfrac{h-x}{3}b \\ A_{c,eff} = \dfrac{h}{2}b \end{cases} \tag{2.46}$$

$\alpha_e = \dfrac{E_s}{E_{c,eff}}$ is the ratio between the stiffness's of the reinforcement and the concrete.

The largest distance between cracks can be determined as:

$$s_{r,\max} = k_3 c + \frac{k_1 k_2 k_4 \phi}{\rho_{p,eff}} \quad (2.47)$$

Where k_1 is a coefficient that depends on the adhesion properties for the reinforcement:

$k_1 = 0.8$ for bars with good adhesion

$k_1 = 1.6$ for bars with smooth surface (cables etc.)

k_2 is a coefficient that depends on the stress distribution:

$k_2 = 0.5$ for section exposed to bending

$k_2 = 1.0$ for section exposed to pure tension

Linear interpolation can be done for sections exposed to both bending and tension.

$k_3 = 7\dfrac{\phi}{c}$ This value is a recommendation in the Swedish national appendix, for general design it can be chosen to be 3.4

$k_4 = 0.425$

c is the concrete cover

θ is the reinforcement diameter

$\rho_{p,eff}$ is as defined in equation 2.45

3. Different types of foundations for wind turbines

There are many types of foundation methods for a wind turbine. In this chapter some of them are presented and analyzed. The methods can be divided into two subgroups; spread foundations and piled foundations. Valid for both types of foundation is that there must be some kind of interface that connects the tower with the foundation. This interface is embedded in concrete and as a consequence of this even the piled foundation has got a concrete plate. The type of interface can in some cases decide the type of foundation method.

3.1 Spread foundation

A spread foundation (or a slab foundation) is a foundation which consists of a big plate that makes use of the big area for spreading the loads to the ground. The geometry is often cylindrical or a square prism and the construction material is almost exclusively reinforced concrete. The bigger bottom area there is the smaller pressure on the ground. This is limiting the area of the foundation so that the ground pressure doesn't exceed the maximum allowed pressure for the soil. Besides the ground pressure, the width of the plate has to be sufficient big to prevent the tower from turning over. The settlements must not be too large, but the most essential is that the differential settlements are kept low to remain the tower vertical.

This type of foundation is suitable for strong and stiff soils that don't give large settlements. That is the reason why this type of foundation mostly is used on friction soils with high friction angle, or moraine and not clays, silty clays, fillings, organic soils or other soils with low modulus of elasticity and/or strength.

As the wind is acting on the tower a huge bending moment occurs in the plate. This moment must be evaluated and those parts that will be exposed to tension stress must be reinforced. This results in reinforcements bars mainly in the bottom of the plate. Shear force is also acting on the plate as a result of the underneath ground pressure. The thickness of the foundation is an essential parameter for the shear strength, and for thick structures the concrete itself can be enough to handle the shear stress. However if the structure is not sufficiently thick reinforcements stirrups must be put in. As the section forces vary along the plate several sections must be analyzed. For every section the apparent section force is compared with the resistance for the specified amount of reinforcement.

The loads from the tower are transferred to the foundation via an interface. Depending on the design of this interface different controls and calculations have to be performed. If the interface consists of a "ring" or "boltcage" as can be seen in chapter 2.2 it is necessary to examine the forces that arise above and underneath the flange in the interface. The bending moment might give arise to both tension- and compression forces above- and under the ring. These forces can result in a need for extra reinforcement both above- and under the ring.

3.1.1 Shallow foundation

It is common that a spread foundation is footed on the ground, or just beneath it, see figure 3.1. The area usually gets quite big to prevent it from overturn. A thick heavy construction will prevent the construction from turning, because the resultant reaction force from the soil is closer to the centre. This type of foundation has the advantage that it is quite easy to build; little excavation- and refilling work.

3.1.2 **Gravity foundation**

This is a spread foundation that is footed some distance down in the soil, meaning that some soil has to be excavated and filling material is replaced above the foundation after it is constructed, see figure 3.1. If strong and stiff soils are overlaid with top layers of soft soils such as clays it can be a good idea to excavate these layers and put the foundation on the better soil. If this is the case the weight of the filling soil above the foundation is preventing the tower from turning over and the area of the plate can be reduced. A benefit of this type of foundation is that it reduces the amount of concrete, but instead requires major excavation- and refilling work.

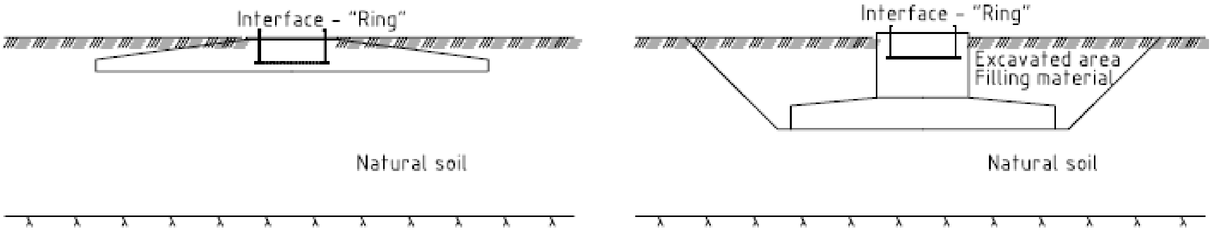


Figure 3.1: Left: Spread foundation on the ground
Right: Gravity foundation

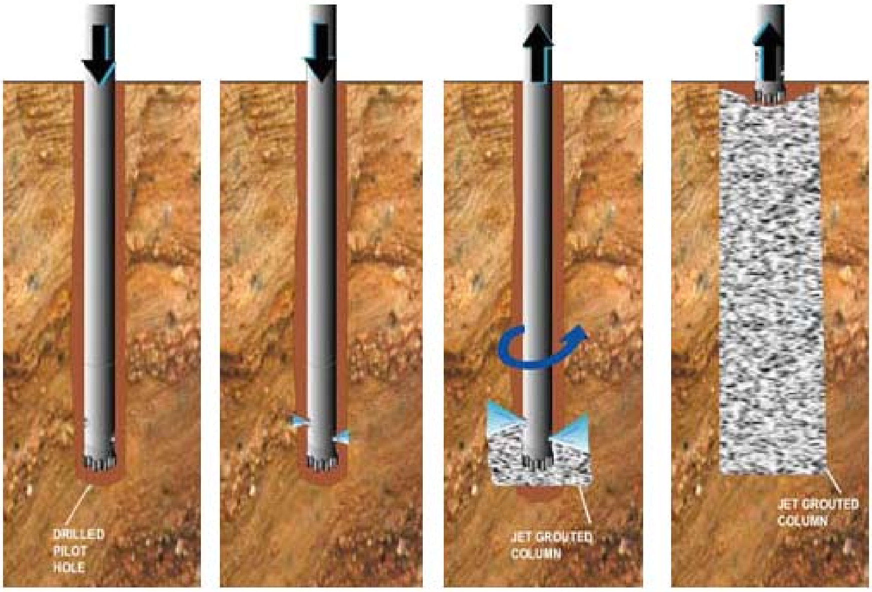
3.2 **Soil stabilization**

If the soil doesn't have the right properties soil improvements can be done to get the right properties. Properties that can be adjusted may be the stiffness, the shear strength, the permeability or the soil homogeneity. There are many different methods for doing this. In [19] the most common methods are presented and divided into compaction/densification methods and methods of soil reinforcement through the introduction of additional material into the ground.

By exposing the soil to preloading and/or compaction, consolidation can be reached which can reduce the settlements. Another method with the same intended result is the vibrating method. By exposing the soil to vibrations, either from the surface in means of releasing heavy weights on the ground or by depth vibration with vibrating machines, the soil will compact.

There are a few methods that use an additional material to reinforce the ground. Among them there are the permeation grouting method, which forcing grout to fill voids in the ground. This method can change the water flow in the ground and/or increase the strength of the soil. Another similar method is the jet-grouting method which uses column-like structures made of soil-grout mixture. These columns is produced when a steel rod first is drilled to the designated depth, after which it is withdrawn and rotating at a controlled rate and at the same time a jet of grout filling the spaces between the soil particles, see figure 3.2.

A closely related method is the lime/cement columns, see figure 3.3. Even these elements are not really columns but columns-like soil improvements. A machine equipped with a mixing tool is driven down in the soil rotating. The mixing tool is then slowly retracted, still rotating and at the same time lime and cement is blown into the soil mixing with the soil. The lime reacts with the water in the undrained soil forming a new product with lower water content, much higher stiffness and stronger. This might be a suitable method if the soil quality's not good and the distance to the bedrock is at too great depth.



Figur 3.2: Jet-grouting technique step by step [L]

Figure 3.3: Lime/Cement columns installed in the ground [L]

Another method which not really is a soil improvement method is simply to exchange the poor soil to a soil with better characteristics. This might be convenient if the poor soil just exist in a shallow and thin soil layer and underlying soil is better. This method requires extensive excavation work, resulting in enormous amount of unused soil which often is of poor quality.

3.3 Piled foundation

If the soil properties are not sufficient to foot the foundation on the ground it can be a good solution to install piles to conduct load to better soil at a greater depth in the ground. Due to the big bending moment from the wind, piles might be exposed to tensional loads which have to be considered. The connection between the piles and the plate is important for the load distribution. The two extreme cases is a clamped connection which does not allow any rotation, and a hinged connection with no rotational stiffness. The clamped case will introduce a big bending moment in the pile top, and the second one will not. The actual case is neither the clamped one, nor the hinged but an intermediate of these two. If the latter one is the one that match the actual connection best the horizontal force acting on the foundation (from the wind load) must be handled in another way. Generally the plate is footed at some depth in the ground having soil surrounding it, and the soil along the perimeter of the foundation can resist the horizontal forces.

Even a piled foundation consists of a concrete plate, but here it serves as a connection between the piles and the tower and the size of the plate can therefore be reduced. But a reduction of the plate width will lead to bigger pile loads. These big loads, especially large tension loads, can be a challenge to design piles for. By increasing the height of the foundation (and so even the weight) one can lower the tensional pile loads at the expense of increased compression for other piles. These factors usually results in fairly big plates even for piled foundations.

3.3.1 Piling to bedrock

If the bedrock is at a reasonable depth and of good quality it can be a good solution to drive piles to the bedrock. In this case the settlement can easily be kept at a low level since it is only the deformation of the piles that contributes to the foundation settlement. In analyses of this case the soil is assumed not to carry any load, which might not be the actual case, but an assumption on the safe side.

When piling to the bedrock it is possible to anchor the piles in the bedrock. This is applied in the Ruukki case (see section 1.7) where very big tensional load where apparent. It is hard to calculate the capacity of an anchored pile and the calculation has big uncertainty. One method is to first calculate the actual pile load, and then after the pile is anchored pull the pile with a machine to verify that it is capable of the necessary tension force.

3.3.2 Piled-raft foundation

This foundation method is a combination of the two methods described above i.e. spread foundation and piled foundation. The way of function is described briefly in [8] and is that the spread foundation ensures load spreading in the upper region in the soil, and the piles transfer load to deeper soil layers. To ensure that both the plate and the piles are load carrying it must not be any gapping between the plate and the ground. This is why the piles can't be standing on the bedrock or a firm soil layer.

A primarily design can be done in terms of equal settlements. That criterion can decide the number of piles and the where they should be installed. The definite design should be done by verification of several limit states. Nowadays this design is preferably done by computer software using the finite element method (FEM). In chapter 4.3 a FEM-software that can be used for this type of calculations is presented.

It is important when creating a piled-raft FEM-model to give the piles, the plate and the soil correct stiffness. The stiffness is governing the load distribution. For example piles with too high stiffness will get too much load at the expense of too little load at the plate.

4.Computer software

In this thesis computer software is used to perform analyses that would be time-consuming, and/or inaccurate if done manually. Generally it can be said that analyses performed with computer software requires that one pay extra attention to the result and its plausibility. The model that is created should be as close to reality as possible. If simplifications are necessary, these should be done wisely, and when evaluating the result one should bear in mind how the simplifications might affect the result.

The advantages of using computer software are that it's time saving and can give a good overview of current situation. A major disadvantage with computer software used for calculations can be that one looses the understanding for the analysis, i.e. one defines the input and then sends it to a black box which just produce the result and what the black box actually is doing one doesn't have to know. If the user has knowledge about the mathematical and physical theories, the governing equations and possible simplifications that the software is using, it is much easier to evaluate the result and perhaps varying input parameters in a valid range.

In this section the computer software used in this thesis is introduced and described briefly.

4.1 **Pile group program (Rymdpålprogram)**

This is Swedish software developed of a Swedish company named Software Engineering, which offers design software for the construction area.

The software computes internal forces, displacements and rotations for piles in a specified pile group for different load cases and loads. The program can handle different types of boundary conditions and also takes the lateral stiffness from the soil into account.

A 3-dimensional model is set up and for each pile the following parameters can be specified:

- Inclination (in 3-dimensions)
- Length
- Maximum pile width
- Cross section type (Circular or square)
- Cross section area
- Moment of inertia
- Torsion constant
- Young's modulus
- Shear modulus
- Lateral stiffness type (Constant or linearly increasing with depth)
- Lateral stiffness (If linearly increasing, this refers to the maximum value)

The loading can be applied both in terms of point- forces and moments and line- forces and moments.

4.2 **FEM-design**

This software is developed of a Swedish company named Structural Design Software in Europe AB (StruSoft) which offering computer software for an industrialized building production process.

This is computer software that computes stresses, strains and displacements for several types of elements. It is specialized on structural design, and the user can specify which standard the design should be carried out after. The user is drawing the structure and defining the element properties in a CAD-like draw space. Supports in form of point-, line- and surface supports are then defined. It is also possible to model the supports with a specified stiffness. The loading can also be defined as point-, line-, or surface forces or moments. Furthermore several load combinations with the specified loads can be determined. When the model is defined with its material parameters, supports, loading and load combinations the calculation can be executed and the result is obtained in terms of section forces, reaction forces, connection forces and displacements.

In this thesis the software is used to calculate ground pressure, pile loads and sectional forces. The software can also be used to design a foundation in terms of reinforcement according to Eurocode, but the software can't perform all the necessary analyses why the structural design in this thesis is performed manually instead.

During the calculation the software make use of the finite element method and the user can specify the types of element that should be used in the analysis. The software got different modules for different types of analyses, where the most important can be found in table 4.1. [20,21]

Table 4.1: Different modules in FEM-design

Name of module	Type of finite elements	Degrees of freedom	Function
Plate	Plate elements with 4 nodes (Thin elements exposed to forces perpendicular to the elements)	3 DOFs per node. Deflection in z-direction, rotations around x- and y-axis.	Used for analysis and design of slabs. Reinforcement can be applied to the model.
Wall	Disk elements with 4 nodes that calculates within the plane stress state (Thin elements exposed to in-plane forces and no stress in the z-direction)	2 DOFs per node. Displacement in x- and y-direction.	Used for analysis and design of load bearing walls and plates. Can also be used for beams with openings.
Plane Strain	Disk elements with 4 nodes that calculates within the plane strain state (Elements where the strain is zero in the z-direction)	2 DOFs per node. Displacement in x- and y-direction.	Used for structures that are invariant-, and extends widely in the z-direction such as retaining walls and tunnels.
3D-structure	Isoparametric thick shell elements with 8 nodes. (A combination of a plate element and a plane stress element)	6 DOFs per node. Displacement in x-, y- and z-direction and rotation around x-, y- and z-axis.	Used for all kinds of structures that falls outside the scope of above mentioned structures.

4.3 Plaxis 2D

This software is developed by a company from the Netherlands named Plaxis (as the product name) and it serves, according to [22], as the most used finite element program for geotechnical analyses today.

4.3.1 General

Plaxis 2D is a two dimensional finite element software dealing with geotechnical calculations. The software intended to function as an easy maneuvered program, where advanced calculus such as nonlinear analysis could be handled without being an expert in those areas. Plaxis is divided into subprograms; the input program, the calculation program and the output program. In addition to the output program there is also a program named Plaxis curves, which come in handy when one wants to plot different quantities in specified locations in the soil.

There are two different ways of modeling in Plaxis; a plain strain model and an axisymmetry model. The plain strain condition means that the strain in one direction is zero. This condition is met, or is at least a good approximation, if the structure is long-stretched and has got a uniform cross section area in the direction where the strain is zero. A sheet pile wall or a road embankment is typical examples where the plain strain model is a good approximation [22].

The axisymmetry model can be used to model circular structures with uniform radial cross section, and when forces acting parallel with the symmetry line. With this type of modeling the deformations and the stresses are constant along its radius. In case of axisymmetric modeling the x-coordinate represents the radius, and can therefore of natural reasons not be negative. Furthermore is the y-axis the axial symmetry line.

The input program has got a graphical interface where the user builds up the geometry of the model in 2 dimensions, (x is horizontally, y is the depth). In that step the user can model and specify different soil layers and structural elements.

The program can automatic generate an unstructured mesh of triangular elements, after which the user can make it more or less coarse. Two types of triangular element is to choose from; one second order polynomial element with six nodes, and one fourth order polynomial element with 15 nodes.

Boundary conditions is applied by the user in terms of horizontal or/and vertical fixities. By clicking the standard fixities button the program automatically assigns horizontal fixities both at the left- and right hand side of the model and in addition assigns total fixities at the bottom of the model. This is realistic boundary conditions if the model extends well beyond the studied geometry [25].

4.3.2 Soil models

Plaxis can handle many different soil models, where the most important for this case is the Mohr-Coulomb model (MC). Described here is also the hardening soil model (HS) and the linear elastic element. The MC-model is recommended to use as a first analysis of the problem as the calculation tends to be fast (due to constant stiffness), and then if a more exact solution is needed, the use of a more complex soil model can be of interest, for example the HS-model.

4.3.2.1 Mohr-Coulomb model (MC)

This model is a perfect elastic-plastic model, which means that the behavior of the soil is linear elastic up to a certain stress limit, after which the soil is perfectly plastic meaning that the strains is irreversible after a stress decrease.

This model requires five input parameters for the soil:

E : Young's modulus
v : Poisson's ratio
φ : Friction angle
c : Cohesion
ψ: Dilatancy angle

The value of the stiffness parameter (E), and the cohesion (c) is in general chosen as a representative value that is consistent with the stress level in the soil. In addition one should take into account the increase at greater depth. It is, however, also possible to model both the stiffness and the cohesion with a linear increase with depth.

4.3.2.2 Hardening soil model (HS)

Instead of using a bi-linear stress-strain curve that the MC-model use, this one is using a hyperbolic curve which in a more appropriate way use the current effective stress in the soil to calculate the stiffness modulus. By this means, the model takes the stiffness decrease that occurs when the soil is exposed to high pressure, into account. As for the MC-model this model assumes plasticity above a specific limit.

The model requires some parameters beyond the ones already defined in the MC-model:

E_{50}^{ref} : Secant stiffness in standard drained triaxial tests
E_{oed}^{ref} : Tangent stiffness for primary oedometer loading
E_{ur}^{ref} : Unloading and/or reloading stiffness
m : Power of stress-level dependency of stiffness

The above mentioned stiffness's are reference values and the ones Plaxis actually is using are its adjusted values.

$E_i = E_i(E_i^{ref}, \sigma, \varphi, c, m)$ where $i = 50, oed, ur$

The parameter m is a way to decide the stress dependency for the calculated stiffness, the higher value the bigger dependence. The following equations give the stiffness's that Plaxis is using in this model:

$$E_{50} = E_{50}^{ref}\left(\frac{c\cdot\cos\varphi - \sigma_3^{'}\cdot\sin\varphi}{c\cdot\cos\varphi + p^{ref}\cdot\sin\varphi}\right)^m \tag{4.1}$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cdot\cos\varphi - \sigma_1^{'}\cdot\sin\varphi}{c\cdot\cos\varphi + p^{ref}\cdot\sin\varphi}\right)^m \tag{4.2}$$

$$E_{ur} = E_{ur}^{ref}\left(\frac{c\cdot\cos\varphi - \sigma_3^{'}\cdot\sin\varphi}{c\cdot\cos\varphi + p^{ref}\cdot\sin\varphi}\right)^m \tag{4.3}$$

Where p^{ref} is a reference pressure by default set to 100 kPa

Note that $\sigma_1^{'}$ is denoting the major principal effective stress and $\sigma_3^{'}$ the minor. In a triaxial test $\sigma_3^{'}$ is the confining stress (e.g horizontal stress), and $\sigma_1^{'}$ the compression stress.

4.3.2.3 Linear elastic

This is the simplest of all soil models in Plaxis. The model is a perfect linear elastic model and no plasticity is accounted for.

This model requires only two input parameters:

E : Young's modulus
v : Poisson's ratio

This material model is not a good approximation for soils, but can be used for structures that exist in the soil where the behavior of the structure might affect the soil. Concrete structures, for example foundations, can be modeled as a linear elastic material.

4.3.3 **Elements in Plaxis**

In addition to the soil elements Plaxis has got some types of elements that can come in handy in geotechnical analyses. The most important is described here.

4.3.3.1 Plates

Plates are structural elements that can be used to model slender elements in the soil. Typical examples are sheet piles, thin walls or piles. The parameters that defines a plate is the flexural rigidity (EI) and the axial stiffness (EA). In addition a weight per meter can be designated to the element. In Plaxis a plate element consists of two dimensional FE-beam elements with three degrees of freedom (DOF) in each node. Whereas two of them are translational DOFs and one is a rotational DOF. The elements are based on Mindlin's beam theory, where both bending and shear can result in deflection. In addition an axial deformation can occur if the element is loaded axially.

4.3.3.2 Interface elements

The interface elements are, in contrast to the other elements, not real elements that exist in the soil, but elements used to simulate a more realistic behavior. The elements are thin elements that are primarily used to model the interaction between different materials. For example the interaction between a sheet pile wall and the surrounding soil can be modeled by use of interface elements. The elements will then enables the user to model the smoothness of the sheet pile, or the disturbed soil around the piles that may arise from the pile-driving. The smoothness or/and the disturbed soil can be defined by the user by entering a value for the strength of the bonding between the soil and the plate, where 1 it a perfect bonding, and 0 is no bonding at all.

Another use of interface elements is to prevent stress concentrations around corners in stiff structures. This is achieved by defining interface elements that extends well through the corners of the structure, see figure 4.1.

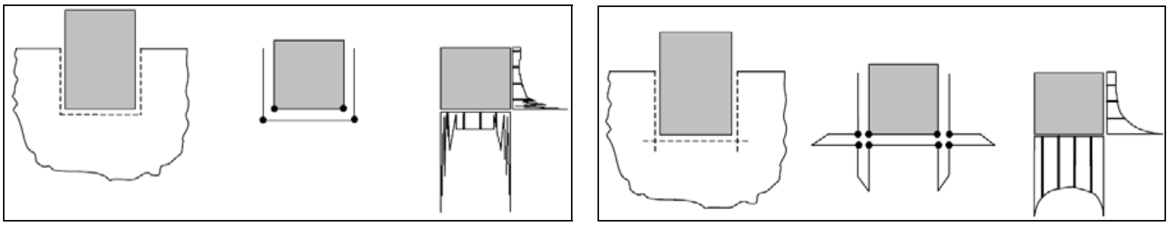


Figure 4.1: The difference of stress distribution when modeling with interface elements [M]
Left: Interface elements not extended through the corners
Right: Interface elements extended through the corners

5.Case study for different geotechnical conditions

For one specific wind power plant with specified load data it can be interesting to compare different type of foundation methods for different geotechnical conditions. The most appropriate methods for each case will be designed. Three different cases are represented; the first one is a moraine soil with good strength and high stiffness, the second and third consists of clayey soil, where the last one has got great depth to the bedrock and the second not great depth. In all three cases are the soil overlaid by a 3 m thick fill material with high permeability (low capillary suction). Furthermore is the groundwater level at three m depth. The soil profiles can be seen in figure 5.1 and the parameter values of the soil can be seen in table 5.1. The values are not from an actual case, but is realistic values for the current soils according to [13, 23]. The values are presented as characteristic values.

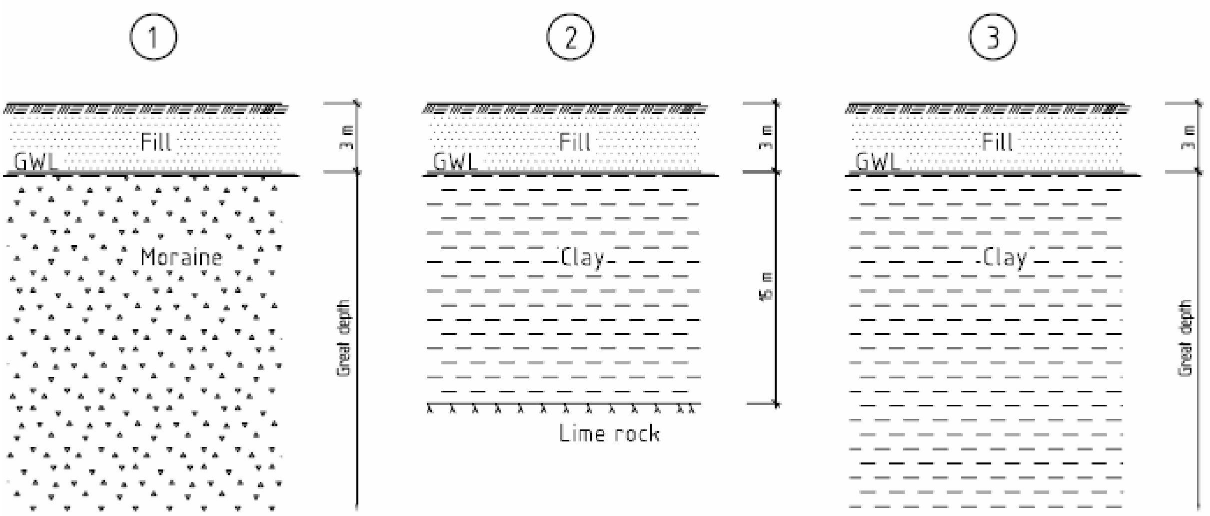


Figure 5.1: The different soil profiles

Table 5.1: Values of the different soil types

Soil	Strength parameters	Stiffness parameters	Weight
Fill material	$c = 0\,kPa,\ \varphi = 38^\circ$ $\psi = 8^\circ$	$E = 35\,MPa$ $\nu = 0.3$	$\gamma' = 12\,kN/m^3$ $\gamma = 19\,kN/m^3$
Moraine	$c = 0\,kPa,\ \varphi = 39^\circ$ $\psi = 9^\circ$	$E = 55\,MPa$ $\nu = 0.3$	$\gamma' = 12\,kN/m^3$ $\gamma = 20\,kN/m^3$
Clay Undrained conditions: Drained conditions:	$c = 40\,kPa,\ \varphi = 0^\circ$ $c = 4\,kPa,\ \varphi = 30^\circ$ $\psi = 0^\circ$	$E = 2\,MPa$ $\nu = 0.3$	$\gamma' = 8\,kN/m^3$ $\gamma = 18\,kN/m^3$
Compacted crushed material	$c = 0\,kPa,\ \varphi = 39^\circ$ $\psi = 9^\circ$	$E = 55\,MPa$ $\nu = 0.3$	$\gamma' = 12\,kN/m^3$ $\gamma = 18\,kN/m^3$

5.1 General conditions

This specific wind turbine will be built outside the city of Halmstad in Sweden. The output effect is 2 MW and the height of the tower is 80 m. The turbine is moreover three-bladed with a blade length of 44 m, which gives a rotating diameter of 90 m (including the central generator) and a wind catching area of 6300 m^2 [24].

The manufacturer of the wind turbine has supplied the actual loads acting on the foundation and they're given in form of two forces, one horizontal and one vertical, one bending moment and one twisting moment [24]. All loads are given at 0.6 m above the top of the foundation, which means that the horizontal force acting on the turbine (that originate from the wind) is moved to the tower base and therefore giving a moment. In addition to that moment some contribution to the moment comes from the own weight moment that will arise when the tower is out of its vertical position. An inclination of 8 mm/m is counted for [24].

Forces will be transferred from the tower to the foundation via an embedded steel ring. This ring will be put in place when the concrete in the foundation is casted, see chapter 2.2.

The power plant should be designed for a life span of 100 years (Class L100) and safety class 3 (Big risk for major injuries in case of failure)

5.1.1 Standards and regulations

To design geotechnical and structural elements it is advantageous to use designing codes. There are several different codes that can be used and often each country has got its own code. Nowadays there is a European standard called Eurocode. Besides this code every country has got its own national appendix where parameter values specific for each country is to be found. In this study Eurocode with the Swedish national appendix is used. This code is issued by the Swedish Standards Institute (SIS) and divided into several subcodes dealing with different areas. The subcodes that are used in this study are Eurocode 2 – Design of concrete structures, and Eurocode 7 – Geotechnical design.

5.1.2 Loads

There are three different sets of loads that are given; ultimate limit state (ULS) loads, serviceability limit state (SLS) loads and the fatigue analysis loads.

Regarding the fatigue loads they are calculated in means of a rain flow count algorithm which often is used in fatigue analysis. This algorithm transforms a spectrum with loads to an equivalent simpler set of loads [31]. From this analysis one get mean values of the loads together with an interval. This interval gives you characteristic values for the forces; one minimum- and one maximum value. The width given is calculated for a stress exponent with a value of 7, which is used in the fatigue design. The number of cycles the loads are given for is 10 millions.

The loading given here is shown in table 5.2 and 5.3 and are typical loads for wind turbines situated on the west coast of Sweden. The definitions of the loads can be seen in figure 5.2. Note that all loads acting 0.6 m above the ground surface and the partial load factors are included.

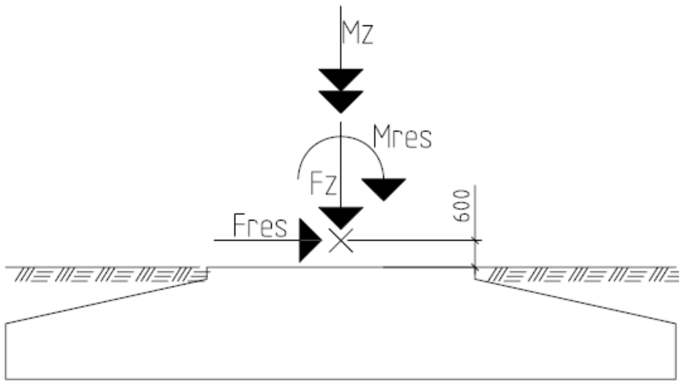


Figure 5.2: Definition of load denotations

Table 5.2: Loads for ULS and SLS

	F_z [kN]	F_{res} [kN]	M_{res} [kNm]	M_z [kNm]
Ultimate limit state	3510	797	63825	1642
Serviceability limit sate	3510	482	35108	303

Table 5.3: Fatigue loads

	Mean load	Interval	Min Load	Max load
F_{res} [kN]	131	241	10	252
M_{res} [kNm]	9143	17451	417	17869

5.2 **Design**

In this chapter analyses that apply to all cases are presented, and in chapter 5.3-5.5 design that are case specific are presented. Chapter 5.6 is comparing the results from the three different cases. The design is performed according to the theory presented in chapter 2.

5.2.1 **Concrete cover**

The following factors are significant in choice of minimum concrete cover for these cases:

- Design life length 100 years, L100
- Concrete class C30/37
- Both Vertical and horizontal surfaces
- Construction is mainly above frost-free depth
- The construction is wet not often dry
- The construction is not exposed to chlorides
- The construction is not exposed to chemical aggressive environment

The above factors classify the structure in structure class S5 and exposure classes XC2, XF1 and XF3 which in turn gives a minimum concrete cover of 30 mm for the worst exposed concrete surface (see appendices A and B). The minimum concrete cover is increased to 50 mm. This is done because of the difficulty for the construction workers to be exact when they are putting the rebars in their correct positions.

5.2.2 **Embedded steel can**

The tower is connected to the foundation via a steel can that are embedded in the foundation. For this case study the embedded steel can is 2,3 m high where 550 mm should protrude above the foundation. In the bottom of the can there is a 330 mm wide and 75 mm high flange. In the upper part of the foundation there are 60 elliptical holes where reinforcement can go through. See figure 5.3 for complete dimensions.

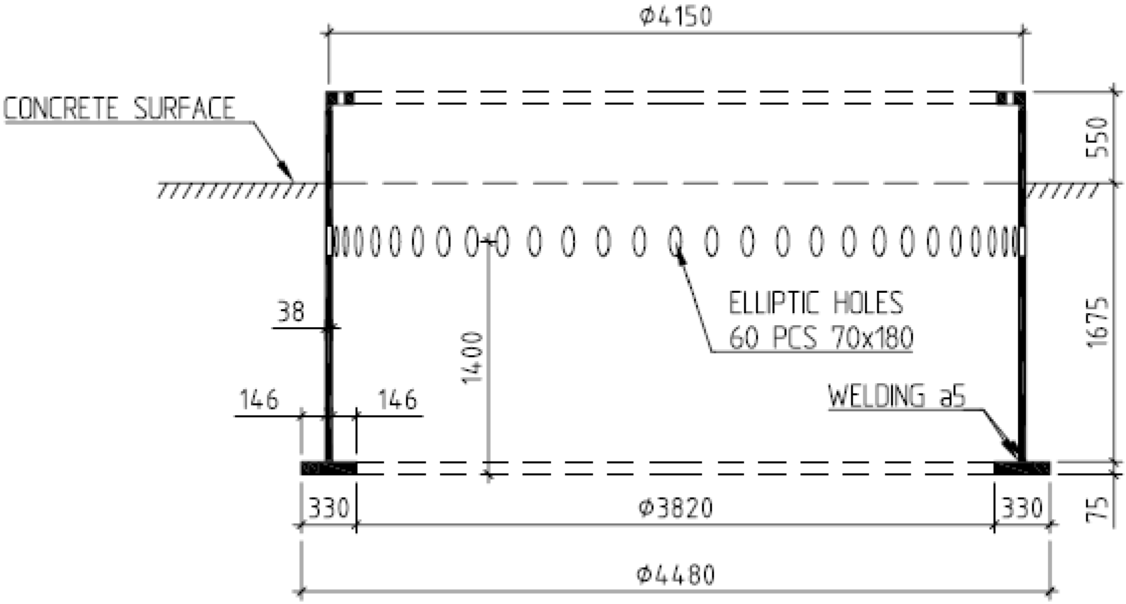


Figure 5.3: The embedded steel can

5.2.3 **Anchor reinforcement**

All the forces from the tower will attack the foundation via the embedded steel can. Because of the bending moment there will be both tension and compression stresses above and under the ring. The compression stress must not be bigger than the strength of the concrete and a sufficient amount of reinforcement is put in for the tensile force.

The maximum stress that occurs in the steel can be calculated as:

$$\sigma_{\max} = -\frac{F_z}{A_{ring}} \pm \frac{M_d}{W} = -\frac{F_z}{\pi D_m s} \pm \frac{4M_d}{\pi D_m^2 s} \qquad (5.1)$$

Where F_z is the vertical force from the tower

$A_{ring} \approx \pi D_m s$ is the area of the flange

Where D_m is the mean diameter of the steel ring

s is the width of the flange

M_d is the design moment at the flange

$W \approx \frac{\pi D_m^2 s}{4}$ is the bending resistance for a cylinder with thin mantle

The minus sign in equation 5.1 gives the maximum compression stress and the plus sign gives the maximum tension stress. The compression strength must not exceed the compression strength of the concrete.

To calculate the required reinforcement, the tension stress is converted to a total tensile force around the perimeter of the steel can by multiplying with A_{ring} :

$$F_T = -\frac{F_z \pi D_m s}{\pi D_m s} \pm \frac{4M_d \pi D_m s}{\pi D_m^2 s} = -F_z + \frac{4M_d}{D_m} \tag{5.2}$$

The reinforcement put in can be calculated though:

$$A_s = \frac{F_T}{f_{yd}} \tag{5.3}$$

Where A_s is the required reinforcement area

f_{yd} is the deign yield stress of the reinforcement

The anchor reinforcement must be designed also for the cyclic loading. Thus the stress width in the reinforcement must not exceed $\Delta\sigma_{Rsd} = 70\,MPa$. The above equations 5.1-5.2 applies also for the fatigue design and the reinforcement required with respect to cyclic loading is obtained as:

$$A_{s.fat} = \frac{\Delta F_T}{\Delta\sigma_{Rsd}} = \frac{F_{T.\max} - F_{T.\min}}{\Delta\sigma_{Rsd}} \tag{5.4}$$

If the minimum tensile force in equation 5.4 is negative, meaning that no tension occurs for the minimum fatigue load, the value must be set to zero.

Finally the required reinforcement area is obtained as the maximum of ULS- and fatigue design.

For the actual loads the maximum compression in the concrete is 15.5 MPa in ULS which should be compared to the design compression strength which for concrete C30/37 is 20 MPa and the total tensile force around the perimeter is 59.8 MN which gives a required reinforcement area of 0.137 m^2.

The fatigue design gave a maximum tensile force around the perimeter of 14.3 MN and a minimum that is below zero i.e. the minimum value is set to zero. This gave a required reinforcement area of 0.204 m^2.

The conclusion is that the fatigue is actually designing the anchor reinforcement. With a reinforcement bar diameter of 32 mm 254 cuts is needed. With U-bows 127 pieces is required, showed in figure 5.4.

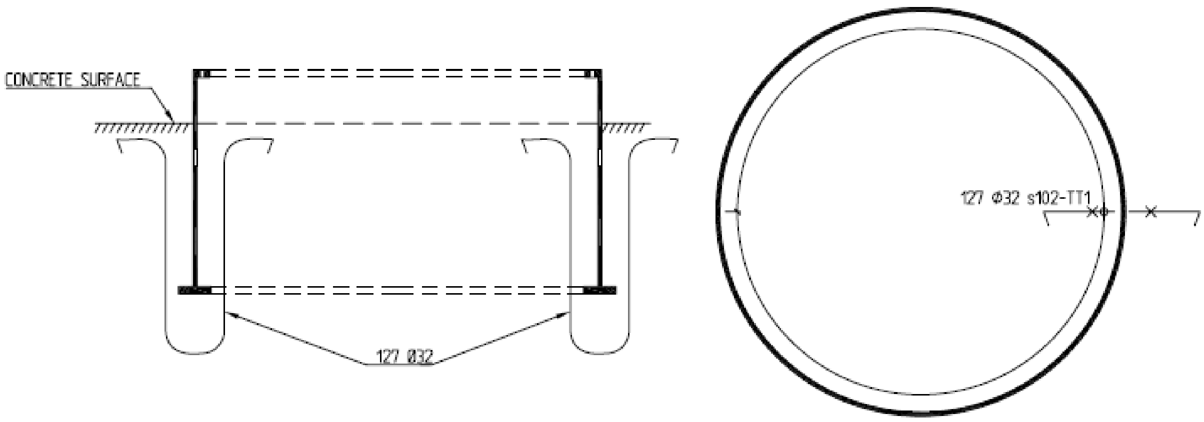


Figure 5.4: Anchor reinforcement.
Left: Section in the middle of the ring
Right: Ring with anchor reinforcement seen from above

The complete calculations can be viewed in appendix C.

5.2.4 Geometry and foundation level

The ground water level is three meters below the ground surface, and to prevent the structure from lifting forces from the groundwater the foundation level should not be lower than the ground water level.

A circular plate is the most efficient geometry and because of the fact that the sectional forces decreasing with the distance from the centre, the height of the foundation should also decrease with the centre distance.

The dimensions of a foundation are of course depending on the foundation method and the loads that are acting on the foundation. In this study the same geometry is chosen for the different soil conditions, but what differs is that for case 2 and 3 piles is added to the geometry. This geometry is described in chapter 5.4 and 5.5 respectively. The proposed geometry for the plate can be seen in figure 5.5.

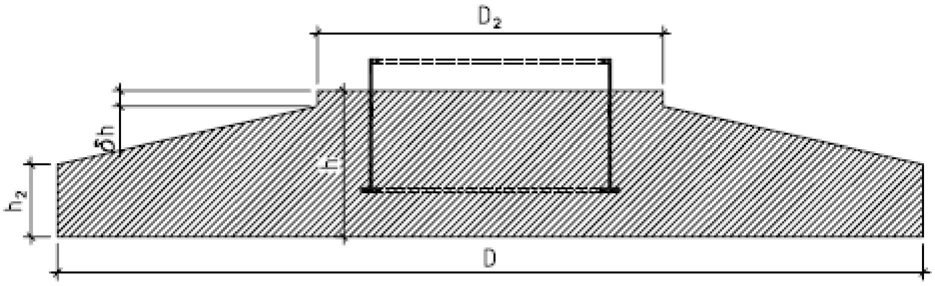


Figure 5.5: Principle geometry of the foundation

The weight of the foundation and the soil above the foundation can now be calculated as:

$$G_c = \left[\frac{\pi D^2}{4} h_2 + \frac{\pi D_2^2}{4}(h - h_2) + \frac{1}{2}\frac{\pi (D^2 - D_2^2)}{4}(h - h_2 - \delta h) \right]\gamma_c \qquad (5.5)$$

$$G_s = \left[\frac{\pi (D^2 - D_2^2)}{4}\delta h + \frac{1}{2}\frac{\pi (D^2 - D_2^2)}{4}(h - h_2 - \delta h) \right]\gamma_s \qquad (5.6)$$

$$G_{tot} = G_c + G_s \qquad (5.7)$$

Where $\gamma_c = 25\, kN/m^3$ is the unit weight for reinforced concrete

$\gamma_s = 18\, kN/m^3$ is the unit weight for the backfilling material

$h, h_2, D, D_2, \delta h$ are dimensions related to those in figure 5.4

The proposed geometry for the plate with actual dimensions can be seen in figure 5.6. The foundation level is set to be the same as the height of the foundation; 2.52m below the ground surface.

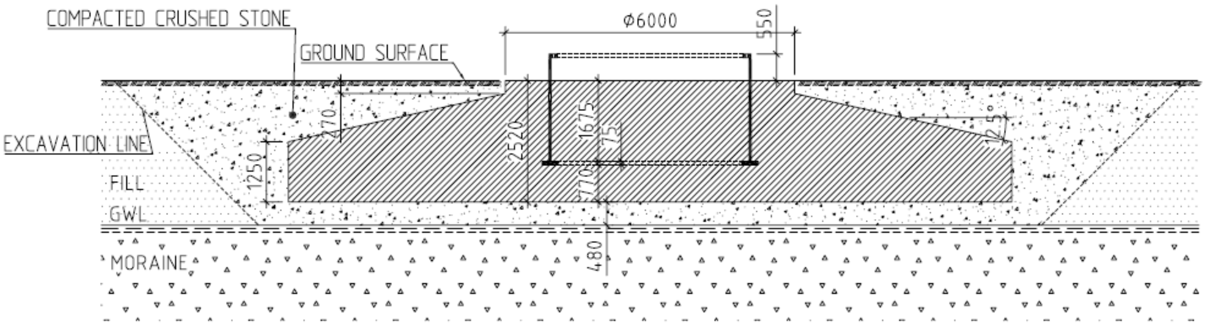


Figure 5.6: Section view of the foundation with its dimensions

The weight of the concrete foundation is calculated with equation 5.5 to be 8276 kN, which corresponds to a volume of 331 m^2 and the weight of the backfilling is (equation 5.6) 2057 kN which gives a total weight of 10333 kN.

5.2.5 Stability analysis

To ensure that the tower's not turning over, the eccentricity of the ultimate load must be inside the perimeter of the foundation. The eccentricity is calculated with equation 2.1 where the design moment is the moment at the base of the foundation and is calculated as the sum of the bending moment and the horizontal force multiplied with the distance to the bottom of the foundation. This eccentricity must be lower than the radius of the foundation, 7.5 m.

$$e = \frac{M_{res} + F_{res}(h+t)}{F_z + G_{tot}}$$

Where h is the height of the foundation
t is the distance from the ground to the point where the loads are defined

The eccentricity is calculated to be 4.79 m which is well below the radius for the foundation; 7.5 m

5.3 Case 1 - Spread foundation on friction soil

The moraine in this profile has got good strength and high stiffness and therefore a standard foundation with a wide spread plate is appropriate.

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

5.3.1 Geotechnical design

5.3.1.1 Bearing capacity

The bearing capacity is calculated in ULS with the general bearing capacity equation in accordance with chapter 2.4.1.2.

For a circular foundation a rectangular effective area that origins from an elliptic effective area is calculated according to section 2.4.1.2:

$$A_{eff} = 43.48 m^2, \; L_{eff} = 9.62 m, \; B_{eff} = 4.52 m.$$

The eccentricity is bigger than 30% of the diameter which means that both rupture 1 and 2 has to be calculated.

The vertical force is the sum of the vertical force from the tower and the weight of the concrete and the soil above the foundation.

$$V = N_d + G_{tot} = 3510\,kN + 10333\,kN = 13843\,kN$$

The horizontal force is corrected for the torque according to equation 2.5 and is:

$$H' = 1208\,kN$$

In the general bearing capacity equation the term that accounts for the cohesion is set to zero because of the lack of cohesion in the soil and the equations becomes:

$$q_b = qN_q s_q d_q i_q g_q b_q + 0{,}5\gamma B_{ef} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \qquad \text{(Rupture 1)}$$

$$q_b = \gamma' B_{ef} N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \qquad \text{(Rupture 2)}$$

The partial coefficient for the friction angle is set to 1.2 and the design value for the friction angle is calculated:

$$\tan(\varphi_d) = \frac{\tan(\varphi')}{\gamma_\varphi} \Rightarrow \varphi_d = \arctan\left(\frac{\tan(\varphi')}{\gamma_\varphi}\right) = \arctan\left(\frac{\tan(39^\circ)}{1{,}2}\right) = 34^\circ$$

The following capacities are calculated as:

$$q_b = 1906\,kPa + 500\,kPa = 2406\,kPa \qquad \text{(Rupture 1)}$$

$$q_b = 1600\,kPa \qquad \text{(Rupture 2)}$$

This means that the bearing capacity for the soil is 1600 kPa (from rupture 2). This capacity should be compared with the actual ground pressure which is:

$$\sigma = \frac{V}{A_{eff}} = \frac{13843}{43.48} = 318\,kPa$$

The bearing capacity is much bigger than the ground pressure.

The sliding resistance is calculated with equation 2.6 and is 9342 kN which should be compared with the horizontal force in ULS corrected for the torque.

$$H' < V\tan(\varphi) \Leftrightarrow 1208\,kN < 9342\,kN$$

In addition to this the following condition must be fulfilled:

$$\frac{H}{V} = \frac{1208}{13843} = 0.09 < 0.4$$

The complete calculations can be seen in appendix D.

5.3.1.2 Settlement calculation

The total settlements and the differential settlements must be held at a reasonable level. The differential settlement is indeed important to hold at a low level because of the big deflection at the tower top this generates. For a tower height of 80 m a differential settlement of 1 mm/m will result in a deflection at the tower top of 8 cm. According to [25] a differential settlement of just 1 mm/m is a reasonable level for foundations of wind turbines.

The theory presented in chapter 2.4.1.3 about manual settlement calculations is not to prefer for a differential settlement calculation. This is because it’s based on an equivalent foundation seen as a plate without a moment and only the total settlements is given as a result. The whole settlement calculation is therefore performed with the computer software Plaxis, described in section 4.3.

The model established in Plaxis is a plain strain. This means that the real geometry has to be transformed into a two dimensional model with no variation within depth. According to [25] a good way to do this is by replacing the circular foundation with an equivalent square foundation. The condition that determines the dimension of the square foundation is that both models should have the same area.

$$A_{circ} = A_{sq} \Leftrightarrow \pi R^2 = B^2 \Leftrightarrow B = \sqrt{\pi}R = \sqrt{\pi}7{,}5 = 13{,}29m$$

For a plain strain analysis in Plaxis a shred with only one meter depth is studied. Figure 5.7 shows the equivalent foundation. By this means equivalent loads has to be calculated. These loads are given by dividing the actual loads with the equivalent width, B.

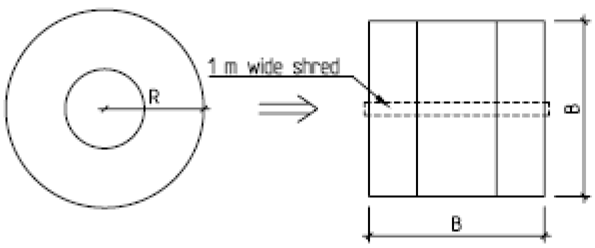


Figure 5.7: Transformation to plain strain model

The differential settlement calculation is performed in SLS and for the corresponding loads given in section 5.1.2. The material parameters that are used are the ones given in table 5.1.

In Plaxis it is not possible to model moment forces, why the moment has to be transformed to point forces instead. This is done by replacing the moment with a force couple with a certain distance, d, between the forces:

$$M_d = F_{eq} d$$

The torque is modeled according to equation 2.5 giving a contribution to the horizontal force. The model established can be seen in figure 5.8.

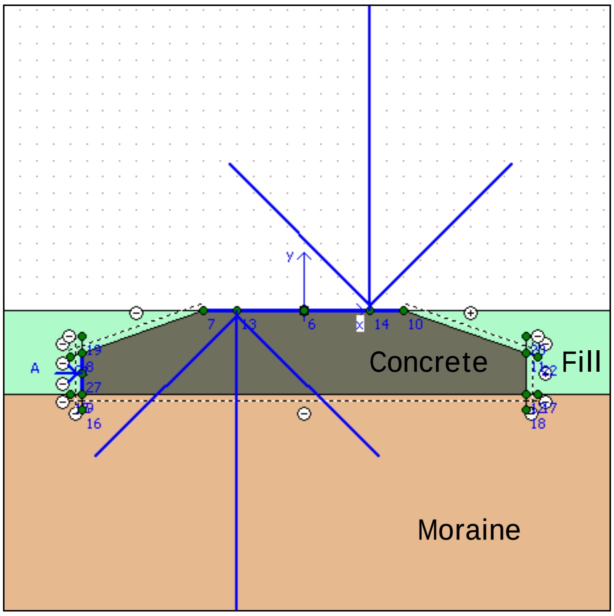


Figure 5.8: Part of the model established in Plaxis

The moraine and the fill are assigned the parameters given in table 5.1 and the concrete is modeled as a linear elastic material with the weight 25 kN/m^3. All soil material is modeled according to the Mohr-Coulomb soil model described in chapter 4.3.

The interaction between the foundation and the soil is modeled with interface elements with a strength value of 0.9 both for the surfaces towards the fill and the moraine. The interface elements are extended through the corners to prevent unrealistic stress concentrations.

After the calculation the deformed mesh is shown in figure 5.9. Note that the deformations are exaggerated, and the figure just showing the principle deformations. Figure 5.10 shows the deformation under the left and the right side of the foundation.

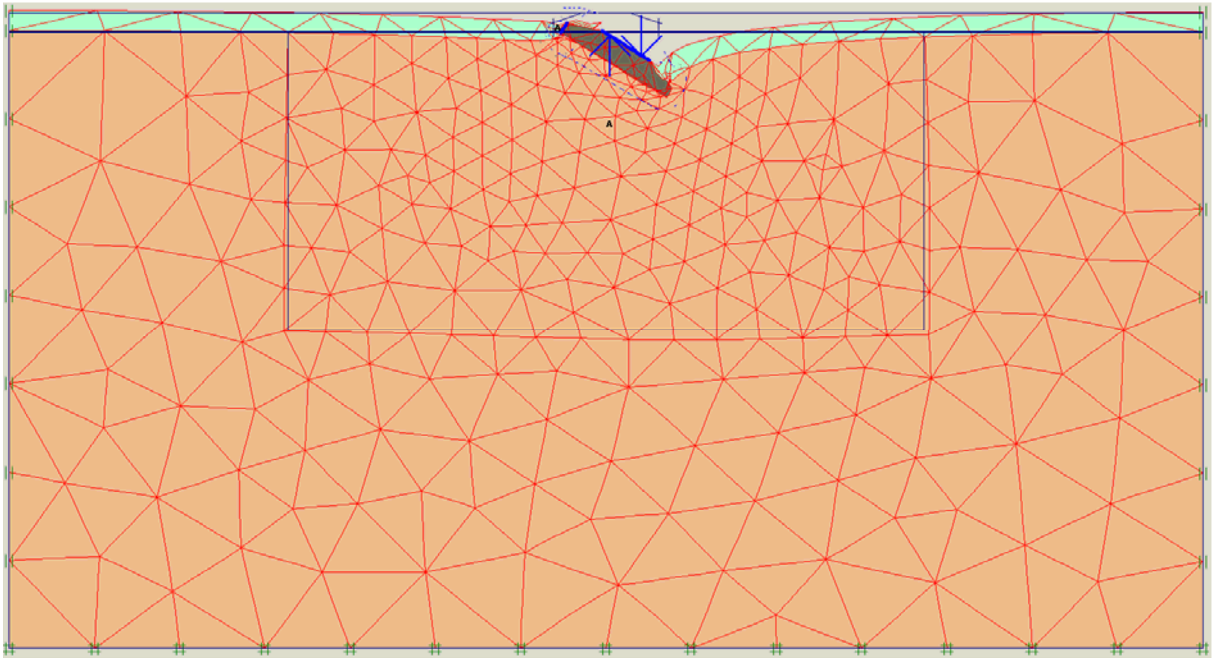


Figure 5.9: The deformed mesh (Displacement exaggerated 840 times)

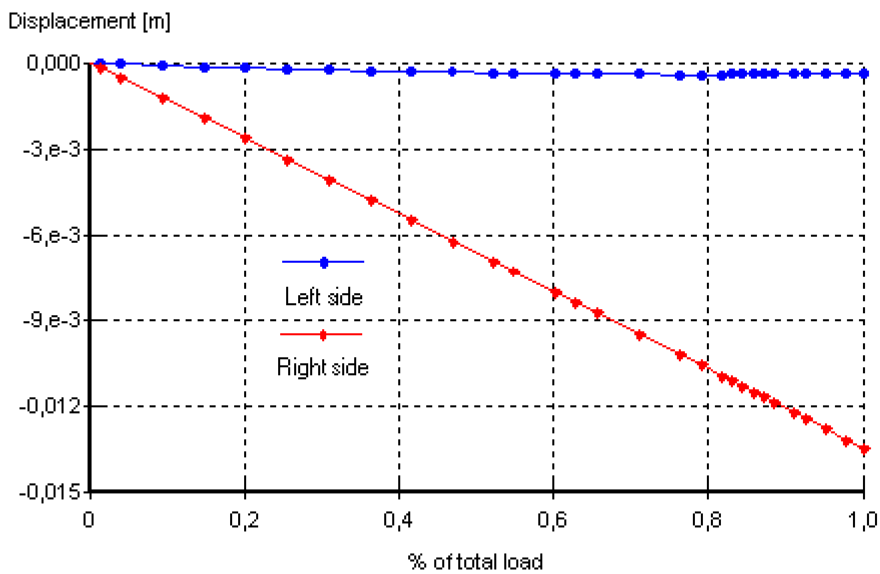


Figure 5.10: Vertical deformation at left- and right side of the foundation

The differential settlement is the difference between the right and the left side:

$$\Delta\delta = \delta_{right} - \delta_{left} = 13.0\,mm - 0.5\,mm = 12.5\,mm$$

These settlements are given for the equivalent width of 13.29 m which gives the differential settlements per m as 12.5 mm/13.29 m = 0.94 mm/m which is just under the limit of 1 mm/m.

5.3.2 **Structural design**

The structural design is performed according to Eurocode – Design of concrete structures.

In table 5.4 the values chosen for the safety factors are listed for the ULS and SLS analyses.

Table 5.4: Values for the safety factors concerning the structural design

Limit state	**Concrete,** γ_C	**Reinforcement,** γ_S	**Long time effect,** $\gamma_{\alpha cc}$	**Fatigue,** γ_{Fat}
ULS	1.50	1.15	1.00	1.00
SLS	1.00	1.00	1.00	1.00

5.3.2.1 Sectional forces

The sectional forces are calculated at a one meter wide shred through the centre of the foundation. The part that extends outside the embedded ring is seen as a cantilever beam on which the sectional forces are calculated. There are two cases to study, where the first one is when the cantilever beam has got load both from the ground (f) and from the weight of the concrete and soil (g). The second case is when only the weight is acting on the beam. The weight (g) is set to have a constant value that is calculated from the total weight from the concrete and the soil equally spread on the total foundation area. This is an approximation on the safe side, because doing this will result in larger bending moments having more weight at a further distance from the clamped support. Figure 5.11 shows the calculation model.

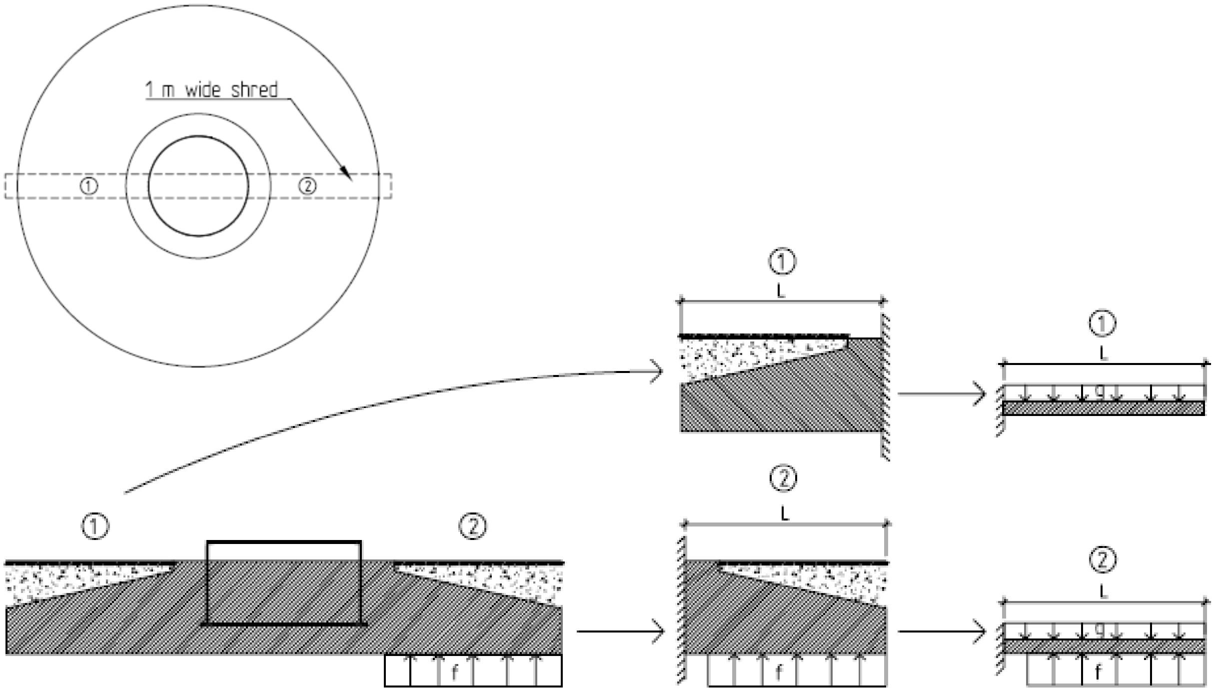


Figure 5.11: Model for calculation of sectional forces

The situation with only the weight acting will give tension in the top of the foundation why this situation later on will be denoted with index Top or just T. The second situation will result of tension in the bottom of the foundation (if f is bigger then g) and this situation will be denoted Bottom or just B further on. The same denotations are valid for the shear forces corresponding to the situations.

The bending moment is calculated as:

$$M_{Top} = -\frac{gL^2}{2} \tag{5.8}$$

$$M_{Bottom} = fB_{ef}\left(L - \frac{B_{ef}}{2}\right) + M_{Top}\,, \quad \text{for } L \geq B_{ef}$$

$$M_{Bottom} = \frac{fL^2}{2} + M_{Top} \quad \text{for } L < B_{ef}\text{, for } L < B_{ef} \tag{5.9}$$

The shear forces are calculated as:

$$V_{Top} = -gL \tag{5.10}$$

$$V_{Bottom} = fB_{ef} + V_{Top} \qquad \text{for } L \geq B_{ef}$$

$$V_{Bottom} = fL + V_{Top} \qquad \text{for } L < B_{ef} \tag{5.11}$$

The sectional moment and sectional shear force is calculated at four different sections equally divided on the beams length, L see figure 5.12. Furthermore the sectional forces are calculated for all sets of loading; ULS, SLS and Fatigue loading, where the latter consists of two separate calculations, one maximum- and one minimum fatigue loading.

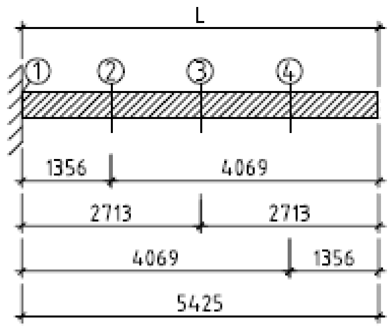


Figure 5.12: Definition of studied sections

The sectional forces for the different cases are listed in table 5.5 and the diagrams for the sectional moments and shear forces can be seen in figure 5.13 and 5.14. For the complete calculations of the sectional forces see appendix E.

Table 5.5: Designing sectional forces at four points

	Section no:	Dist.[m]	L [m]	Mt [kNm]	Mb [kNm]	Vt [kN]	Vb [kN]
ULS	1	2.075	5.425	-860	3693	-317	1121
	2	3.431	4.069	-484	2151	-238	1057
	3	4.788	2.713	-215	956	-159	705
	4	6.144	1.356	-54	239	-79	352
SLS	1	2.075	5.425	-860	1196	-317	441
	2	3.431	4.069	-484	673	-238	331
	3	4.788	2.713	-215	299	-159	220
	4	6.144	1.356	-54	75	-79	110
Fat.Min	1	2.075	5.425	-860	299	-317	110
	2	3.431	4.069	-484	168	-238	83
	3	4.788	2.713	-215	75	-159	55
	4	6.144	1.356	-54	19	-79	28
Fat.Max	1	2.075	5.425	-860	632	-317	233
	2	3.431	4.069	-484	355	-238	175
	3	4.788	2.713	-215	158	-159	116
	4	6.144	1.356	-54	39	-79	58

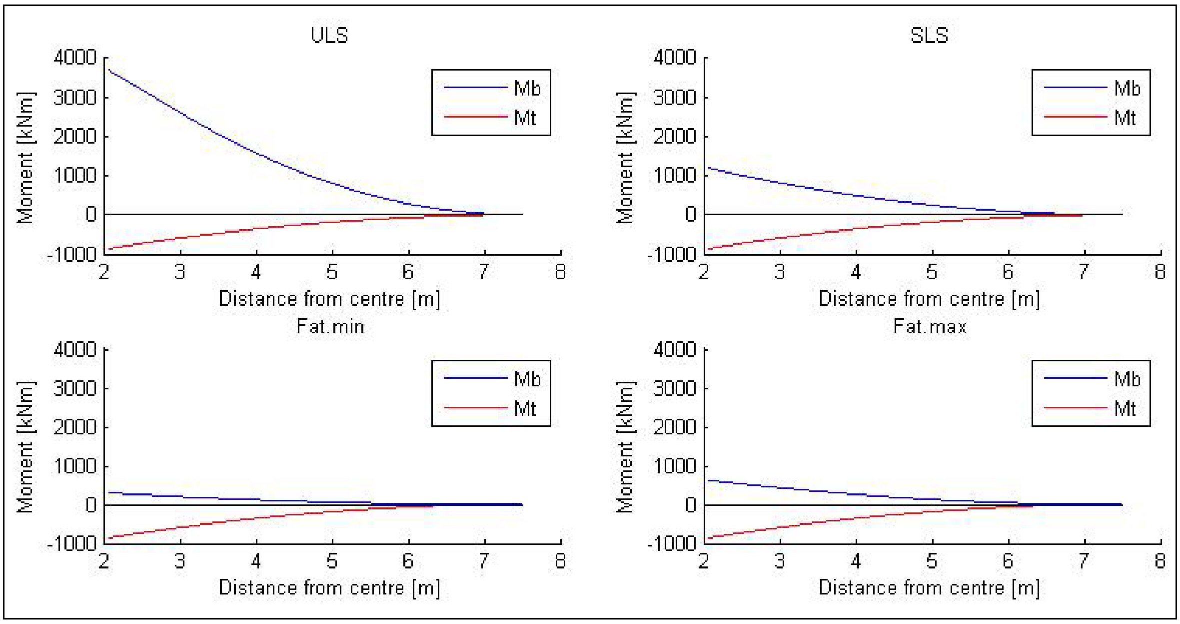


Figure 5.13: Sectional moment

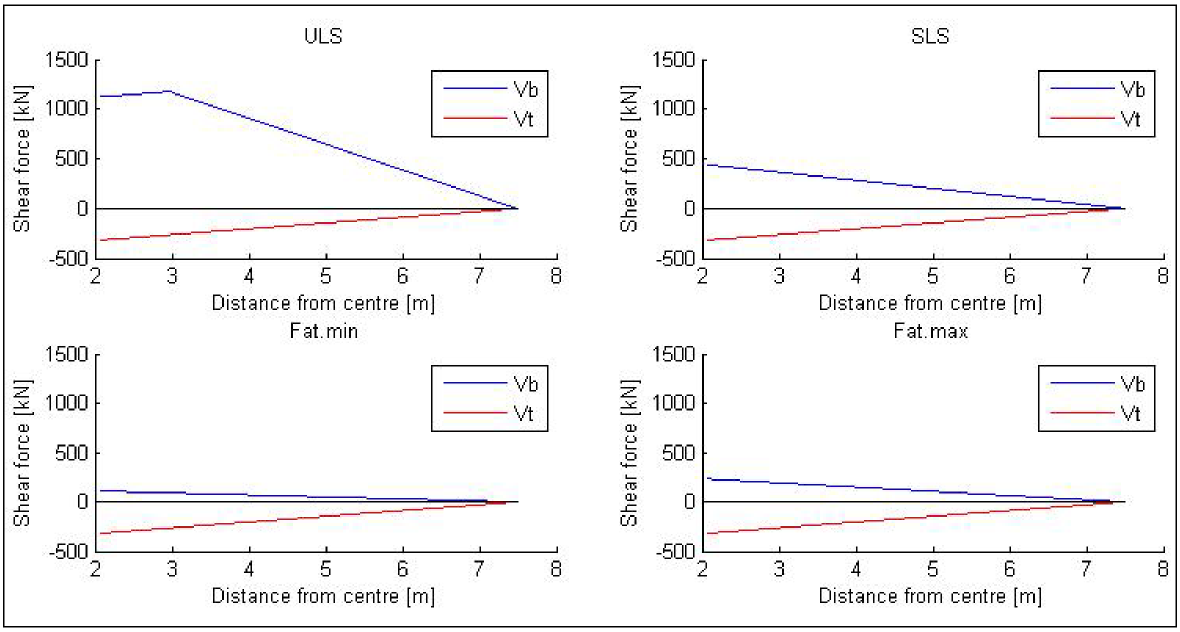


Figure 5.14: Shear force

5.3.2.2 Design for bending moment

The design is performed in ULS and according to Eurocode – Design of concrete structures chapter 6.1 and as described in this thesis in chapter 2.5.3.

The concrete class is C30/37 and has got a compression strength of $f_{ck} = 30MPa$ which means that $\lambda = 0.8$, $\eta = 1.0$ and $\varepsilon_{cu3} = 3.5$ ‰ .

The design is performed in the 4 sections defined in chapter 5.3.2.1 and for the negative moment the top reinforcement area is calculated, and for the positive moment the bottom reinforcement area is calculated. The both cases are performed separately and no double reinforced sections are being considered as an assumption on the safe side.

The calculation order is:

- Calculation of the distance to the neutral layer (x) for the actual bending moment with its correct effective height (d). This is done by solving equation 2.25 with respect to x.
- Calculation of the strain in the reinforcement (ε_s) and compare it with the yield strain for the reinforcement (ε_{sy}). If the strain exceeds the yield strain, the failure will not be brittle. This is done by using equation 2.26 to solve out ε_s and then equation 2.27 to compare the strains.
- The last step is to calculate the reinforcement area required in the section. This is done by solving equation 2.24.

The results are given in table 5.6. The complete calculations can be seen in appendix F.

Table 5.6: Required reinforcement area in top and bottom of the foundation

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$A_{s,Top}$ [mm^2/m]	815	540	281	85
$A_{s,Bottom}$ [mm^2/m]	3917	2852	1477	443

5.3.2.3 Design for shear force

The design is performed in ULS and according to Eurocode – Design of concrete structures chapter 6.2 and as described in this thesis in chapter 2.5.4.

The design is performed in the 4 sections defined in chapter 5.3.2.1 and the calculation order is:

- The shear resistance ($V_{Rd,c}$) is calculated from equation 2.28 after having calculated the constants k and ρ_l. The bending reinforcement both at the top and the bottom of the foundation are included when ρ_l is calculated.
- The minimum shear capacity is calculated from equation 2.29
- The lowest value of the two above calculated values is the shear resistance which is compared with the biggest absolute value of the shear force in the sections (V_{Ed}).
- For those sections (if any) that hasn't got enough shear capacity, a required amount of shear stirrups is calculated with equation 2.30. The shear stirrups are mounted vertically giving $\alpha = 90^\circ$.
- The maximum capacity is calculated according to equation 2.31 and compared with the biggest absolute value of the shear force to see if the struts in the model have got enough capacity.

If there are any sections that requires stirrups, the diameter of the stirrups is set to 25 mm and $\cot(\theta) = 1$ and $\tan(\theta) = 1$. The constant α_{cw} is set to unity for non pre-stressed reinforcement, and the inner lever (z) is calculated as a approximated value of $z = 0.9d$.

The calculation results in enough shear capacity from only the bending reinforcement in section 4 but not for the other sections.

Table 5.7 shows the minimum s-distances required for the stirrups:

Table 5.7: Minimum s-distance for shear stirrups

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
s-distance [mm]	**620**	**589**	**666**	-

As a simplification an s-distance of 550 mm is chosen in all the sections. This is because stirrups are needed for the mounting of the top reinforcement. The diameter of the stirrups in section 4 can be reduced.

The complete calculations can be seen in appendix G.

5.3.2.4 Design for crack width

The design is performed in SLS and according to Eurocode – Design of concrete structures chapter 7.3.1 and 7.3.4 and as described in this thesis in chapter 2.5.6.

The design is performed in the 4 sections defined in chapter 5.3.2.1 and the calculation order is:

- An acceptable crack width is determined according to table 2.2
- The mean strain is calculated according to equation 2.43.
- The largest distance between cracks is calculated with equation 2.47.
- The characteristic crack width is calculated with equation 2.42
- If the crack width is bigger than what is acceptable more reinforcement is added

For the exposure classes determined in section 5.2.1 an acceptable crack width of 0.4 mm is determined.

The crack width, for the applied reinforcement amount from the calculations for bending (section 5.3.2.2), is listed for the 4 sections in table 5.8:

Table 5.8: Crack width for the reinforcement area calculated in section 5.3.2.2

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
Crack width, top [mm]	1.17	1.61	2.82	8.58
Crack width, bottom [mm]	0.18	0.18	0.26	0.65

In table 5.8 it is apparent that the crack width is not acceptable in the top of the foundation for any section, and the same yields for section 4 in the bottom. As a consequence more reinforcement must be put in for these sections. Table 5.9 lists the required reinforcement area to limit the crack width to the acceptable 0.4 mm.

Table 5.9: Required reinforcement

Section	**1**	**2**	**3**	**4**
Distance [m]	2,075	3,431	4,788	6,144
$A_{s,top}$ $[mm^2/m]$	**1540**	**1199**	**818**	**422**
$A_{s,bottom}$ $[mm^2/m]$	3917	2852	1477	**578**

The values in bold in table 5.9 is the reinforcement area that are adjusted to fulfill the crack width.

The complete calculations can be seen in appendix H.

5.3.2.5 Design for fatigue

The fatigue design can be divided into fatigue for the concrete and fatigue for the reinforcement.

The design is performed in ULS and according to Eurocode – Design of concrete structures chapter 6.8 and as described in this thesis in chapter 2.5.5.

Fatigue for compression in concrete

The concrete class is C30/37 and has got a compression strength of $f_{ck} = 30\,MPa$ which means that the first condition in equation 2.33 is used in the design. The design strength of the concrete is calculated according to equation 2.34 and becomes $f_{cd,fat} = 17.6\,MPa$.

The calculations are performed in the 4 sections defined in section 5.3.2.1 and the fatigue criterion is fulfilled in all sections.

Fatigue for tension in reinforcement

The fatigue design is performed according to equation 2.40 with a maximum stress width of 70 MPa.

In the top reinforcement there is no stress difference in the reinforcement between maximum and minimum fatigue load. This is because of the invariance of the bending moment due to the weight of the foundation and the soil above the foundation which isn't a cyclic loading. In the bottom reinforcement where the bending moment is related to the cyclic loading there are stress differences in the reinforcement. The minimum- and maximum stress and the difference can be seen in table 5.10.

Table 5.10: Stresses and stress difference in bottom layer reinforcement

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$\sigma_{s.\max}$ [MPa]	69.17	62.60	62.42	46.97
$\sigma_{s.\min}$ [MPa]	32.73	29.63	29.63	22.88
$\Delta\sigma_s$ [MPa]	36.45	32.98	32.79	24.09

In all sections the stress difference is less than 70 MPa and there is no need to determine a more accurate value of the allowed stress width, see section 2.5.5.2.

The complete calculations can be seen in appendix J.

5.4 **Case 2 - Piled foundation with piles standing on the bedrock**

When the soil hasn't got good strength and stiffness parameters one has to pile to transfer the load to soil with better parameters. In this case the bedrock is at 20 m depth, why piles standing on this are a good solution. This means that the soil strength and the stiffness are not of very big importance, except for the lateral support for the piles which may limit the total capacity of the pile in terms of buckling. The lateral support is described in section 2.4.2.3 and is calculated from the actual value of the modulus of subgrade reaction.

For clay the modulus of subgrade reaction can according to [26] be calculated as:

$$k = 200\frac{c_{uk}}{d} \tag{5.12}$$

Where the above equation is legal for short time loading

c_{uk} is the undrained shear strength of the clay

d is the side length of a quadratic pile or the diameter for a circular pile

In Sweden it is very common to use prefabricated concrete piles with a square cross section in piled foundations [8]. For this reason this type of piles is used in this analysis with a side length of 0.27 m.

There are two options of the connection between the foundation and the piles; a clamped support or a hinged support. Where the latter is the most appropriate in this case. The reason for this is mainly to avoid big bending moments in the piles, but also the difficulty to get the pile to actually be clamped. Also at the bedrock piles are modeled with a hinged support.

The model chosen in this case consists of a plate with the same geometry as for case 1, but with piles placed within a circle with a diameter of 13 m (one meter inside perimeter), and in addition there are piles in the centre parts of the plate within a diameter of 6 m. To prevent that the force will give arise to bending moment at the pile top the outermost piles is placed so that a virtual extension of the piles will meet at the point where horizontal load from the wind origins. For this case with a tower height of 80 m and 20 m long piles the inclination angle for the piles is calculated as follows with the dimensions shown in figure 5.15 :

$$\alpha = \arctan\left(\frac{6.5}{83.12}\right) = 4.77^\circ$$

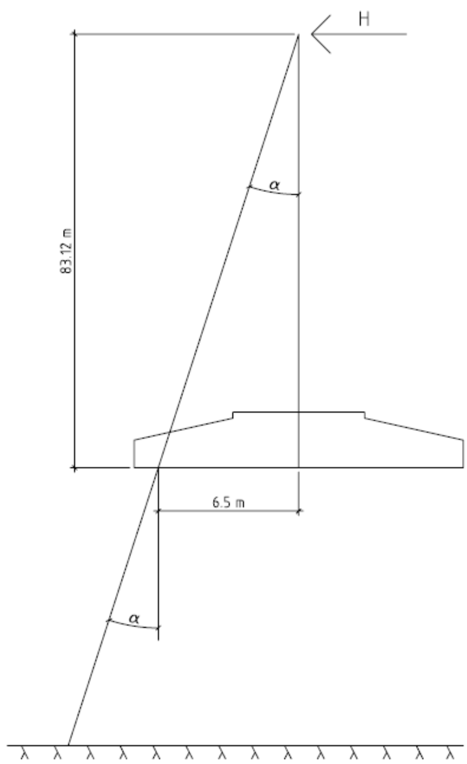


Figure 5.15: Determination of the inclination angle

There are 28 piles equally spread around the foundation with an inclination of 4.77° and 6 vertical piles in the centre. The proposed pile placement can be seen in figure 5.16.

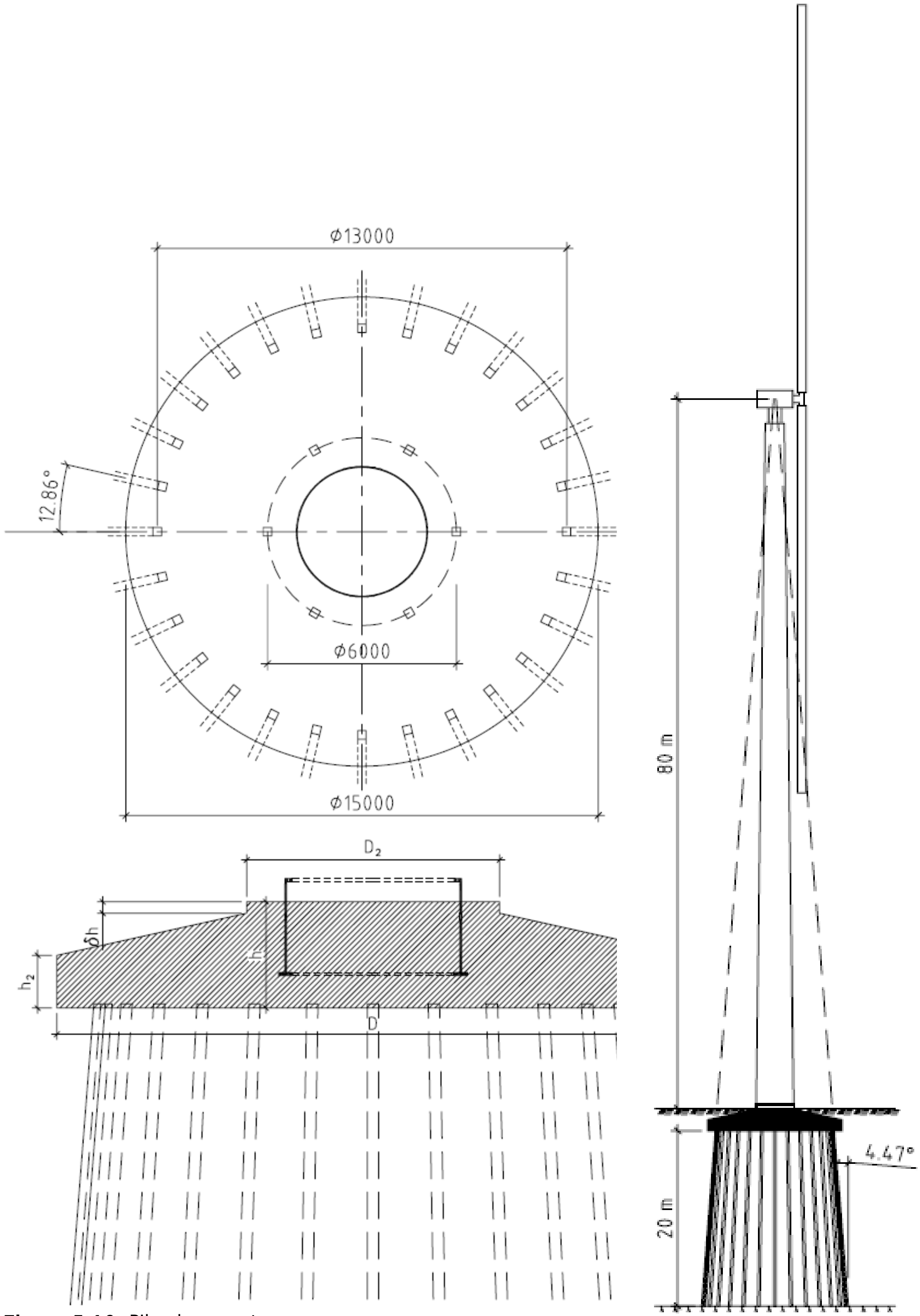


Figure 5.16: Pile placement

To determine the pile forces and the deformations in the piles a model is created in a pile group software "Rymdpålprogram" described in section 4.1. The plate and all the piles in modeled with its right geometry. Furthermore four different load combinations are defined; ULS, SLS, fatigue minimum and fatigue maximum see section 5.1.2. The piles are also defined in the model with the following properties:

- Quadratic cross section with side length, $a = 0.27\ m$
- Moment of inertia with respect to both axes, $I_y = I_z = \dfrac{a^4}{12} = 4.43 \cdot 10^{-4}\ m^4$
- Torsion constant, $K_v = 0.1406 \cdot a^4 = 7.47 \cdot 10^{-4}\ m^4$
- Young's modulus, $E = 33\,GPa$
- Shear modulus, $G = \dfrac{E}{2(1+\nu)} = 13.75\,GPa$

  Given with poisson's ratio, $\nu = 0.2$
- Constant modulus of subgrade reaction, $B = 200 \dfrac{c_{uk}}{d} = 30\ MN/m^3$
- The piles got a hinged support in the top and are elastic lateral supported in the soil

Figure 5.17 showing the model of the foundation in the pile group program.
In appendix K the complete results from the pile group program is shown, including both pile loads and deformations.

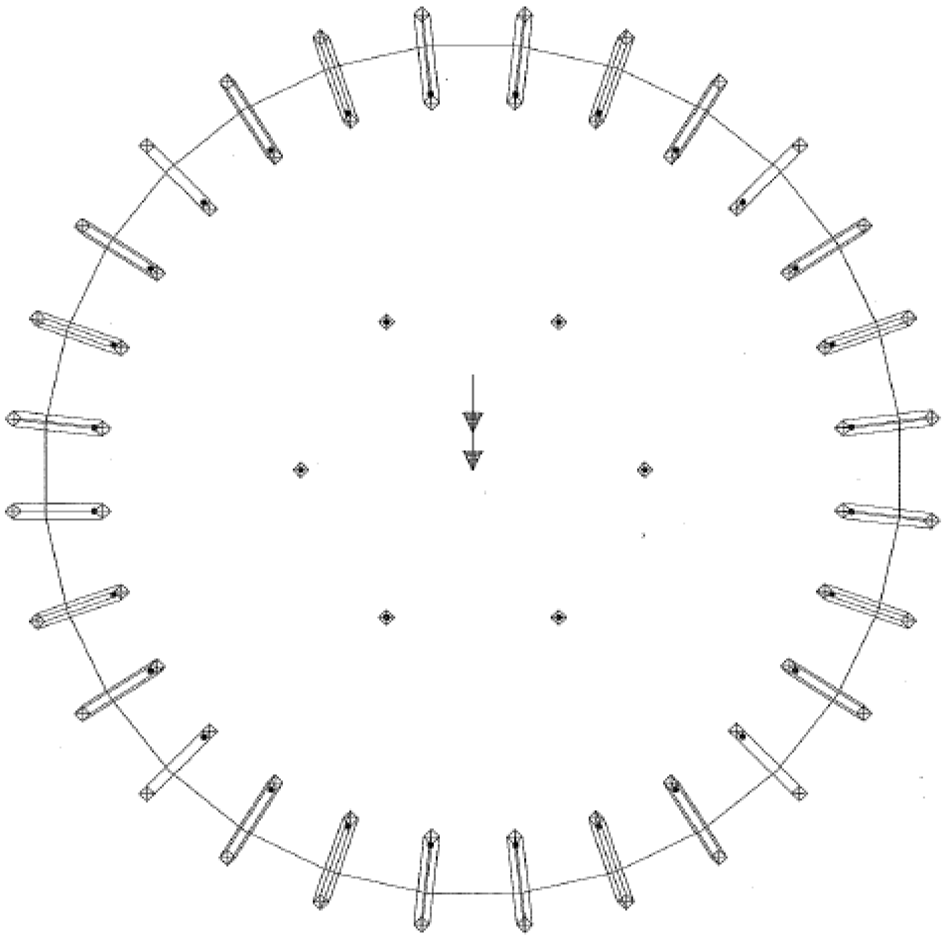


Figure 5.17: The model in pile group program

5.4.1 **Geotechnical design**

For situation 2 the geotechnical design involves only the capacity of the bedrock that the pile is driven to. In this thesis it is assumed that the bedrock is of good quality, and the strength is sufficient for piles. This is a reasonable assumption for high quality rock in Sweden [8].

The capacity of the piles is then only dependent of the structural capacity, which can be calculated according to chapter 2.4.4.3. The structural capacity will decrease if the lateral support of the soil is weak.

5.4.1.1 Pile capacity

The capacity of the piles is depending only of the structural capacity of the pile. The structural capacity is the lowest value of two cases:

- The material will break, due to too high stress
- The pile will buckle, due to the slenderness and the lack of sufficient lateral support in the soil

In [16] there is a model to calculate the load capacity for piles which accounts for the lateral support in the soil. In [27] there are listed load capacities for piles in clays with different shear strengths. For spliced SP2-piles installed in clay, with a design value of 20 kPa, the load capacity is 1220 kN.

The pile loads in ULS obtained from the pile group program is seen in figure 5.18 and the maximum pile load is 1078 kN which is below the capacity of 1220 kN.

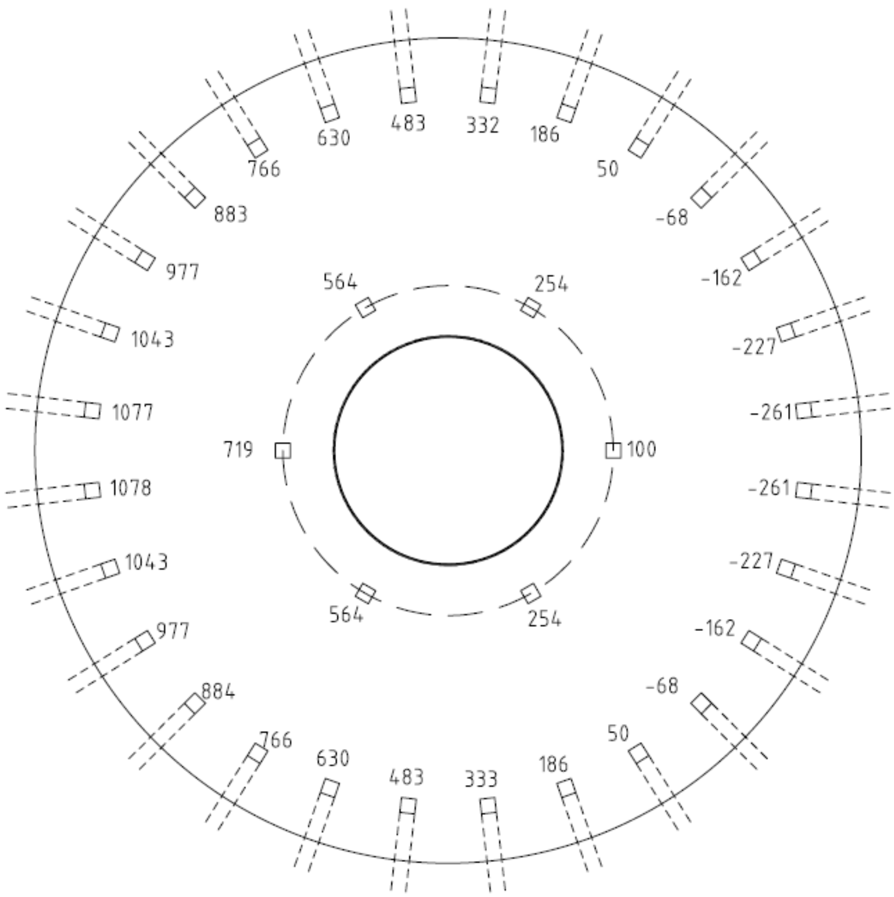


Figure 5.18: Pile loads in kN for ULS

The tensional capacity of the piles must be calculated manually according to the theory in section 2.4.4.2. The calculation is performed both for the drained- and the undrained situation.

Undrained analysis:

Equation 2.17 is used with the following values:

$\alpha = 0.75 * 0.7 = 0.525$, where the first factor is a reduction for the adhesion due to the tension force, and the second is the adhesion factor for concrete piles.

$c_{ud} = \frac{c_{uk}}{\gamma_M} = \frac{40}{1.33} = 30\,kPa$, is the design value for the undrained shear strength of the clay. The partial factor is set to 1.33.

The resistance is now calculated:

$$R = \alpha \cdot c_{ud}(L_{60} - 3) \cdot 4a = 0.525 \cdot 30 \cdot (20 - 3) \cdot 4 \cdot 0.27 = 289\,kN$$

Drained analysis:

The factor β is determined from figure 2.12 and is for 20 m long piles 0.25 and besides that a typical reduction is for friction 0.5 why the factor becomes 0.125. The adhesion factor is set to 0.7 as above, but with no reduction. The resistance is calculated as:

$$R = \alpha \cdot c_d \cdot z \cdot P + \beta \sigma'_{\bar{v}} z \cdot P = (\alpha \cdot c_d + \beta \cdot \sigma'_{\bar{v}}) z \cdot P =$$
$$= (0.7 \cdot 3 + 0.125 \cdot \frac{20+3}{2} 8) \cdot 17 \cdot 4 \cdot 0.27 = 250\,kN$$

The tensional capacity of the piles is then 250 kN. This value must be compared with the maximum tension force that will occur in the piles 261 kN. It is apparent that the tension capacity is not sufficiently big, but the weight of the concrete pile can also be accounted for which gives the maximum pile load [25]:

$$P_{\max} = P - G_{pile} = P - A \cdot L \cdot \gamma_{pile} = 261 - 0.27^2 \cdot 20 \cdot 25 = 225\,kN$$

The capacity is now bigger than the maximum pile load.

5.4.1.2 Settlement calculation

For this case the soil is not bearing any load, and the piles are conducting the load to the bedrock. Therefore any settlements in the soil don’t exist, but some deformation in the piles may be apparent though. These deformations are calculated from the pile group program and the maximum deformation in SLS is 6.45 mm (in the outermost position) and the minimum is 0,33 mm (at the other side) giving a differential deformation of 6.12 mm which can be expressed as 6.12/13 = 0.47 mm/m which is below the acceptable criterion of 1 mm/m. See appendix K for deformations of all piles.

5.4.2 **Structural design**

The structural design is performed according to Eurocode – Design of concrete structures.

In table 5.11 it is listed the value chosen for the safety factors in the different analyses.

Table 5.11: Values for the safety factors concerning the structural design

Limit state	**Concrete,** γ_C	**Reinforcement,** γ_S	**Long time effect,** $\gamma_{\alpha cc}$	**Fatigue,** γ_{Fat}
ULS	1.50	1.15	1.00	1.00
SLS	1.00	1.00	1.00	1.00

5.4.2.1 Sectional forces

In this case the sectional forces are calculated numerically with the finite element software FEM-design, which is described in section 4.2. In the model created in FEM-design all piles is put in its right position and modeled as point supports with an axial stiffness, k calculated with equation 5.13 below:

$$k_{pile} = \frac{EA}{L} = \frac{33 \cdot 10^9 \cdot 0.27^2}{20} = 120.3\, MN/m \tag{5.13}$$

All the loads specified in section 5.1.2 is defined in the model as four different load cases; ULS, SLS; minimum fatigue load and maximum fatigue load. The concrete plate is also modeled with its correct geometry and all its properties. To prevent large stress concentrations the vertical load is spread as a line load along the embedded ring, and the bending moment is modeled as a force couple, see figure 5.19.

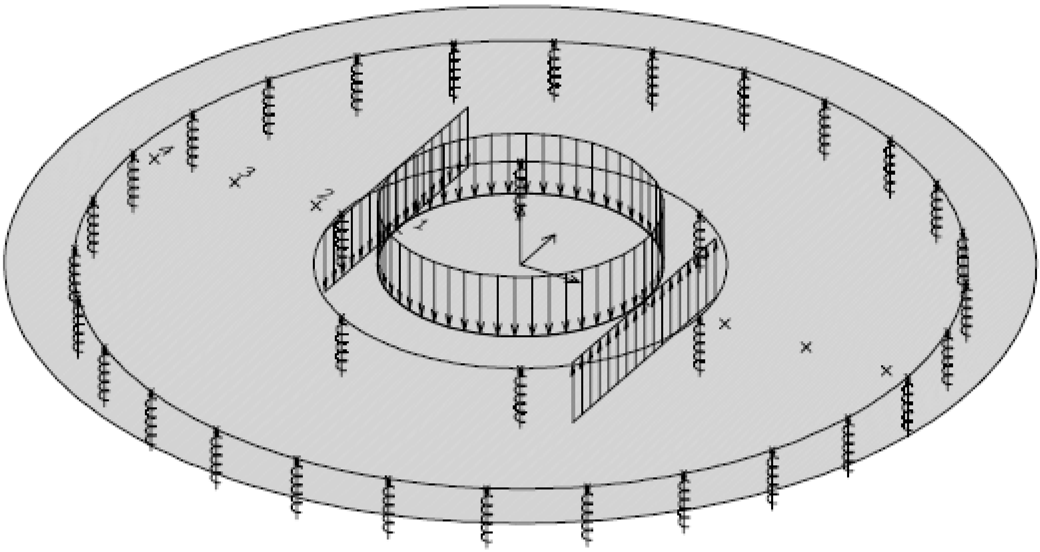


Figure 5.19: The FE-model with the equivalent line loads

The section force distribution can be seen for the different load cases in appendix L.

The designing section forces are read at 4 points (marked in figure 5.19). In the same way as for case 1, one design situation is when only the weight acting on the "beam" giving the same sectional forces here denoted with Top or simply t. The bending moments and shear forces for the other situation is read from the model. The section forces can be seen in table 5.12.

Table 5.12: Designing sectional forces at four points

	Section no:	Dist.[m]	Mt [kNm]	Mb [kNm]	Vt [kN]	Vb [kN]
ULS	1	2.075	-860	3137	-317	1337
	2	3.431	-484	1671	-238	939
	3	4.788	-215	958	-159	791
	4	6.144	-54	103	-79	734
SLS	1	2.075	-860	1986	-317	884
	2	3.431	-484	1301	-238	557
	3	4.788	-215	626	-159	505
	4	6.144	-54	62	-79	505
Fat.Min	1	2.075	-860	603	-317	336
	2	3.431	-484	348	-238	97
	3	4.788	-215	226	-159	174
	4	6.144	-54	14	-79	230
Fat.Max	1	2.075	-860	1295	-317	612
	2	3.431	-484	827	-238	328
	3	4.788	-215	427	-159	333
	4	6.144	-54	38	-79	368

5.4.2.2 Design for bending moment

The design for bending moment is performed in the same manner as for case 1 with the same calculation order, see chapter 5.3.2.2.

The result is presented in table 5.13 as required reinforcement area in bottom and in top of the foundation. The complete calculation can be seen in appendix M.

Table 5.13: Required reinforcement area in top and bottom of the foundation

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$A_{s,top}$ $[mm^2/m]$	815	540	281	85
$A_{s,bottom}$ $[mm^2/m]$	2998	1872	1253	161

5.4.2.3 Design for shear force

The design for shear force is performed in the same manner as for case 1 with the same calculation order, see chapter 5.3.2.3.

The calculation of shear capacity without stirrups, results in not sufficient capacity for any section and therefore stirrups is required. The calculated s-distance of the shear stirrups is presented in table 5.14.

Table 5.14: Minimum s-distance for shear stirrups

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
s-distance [mm]	568	624	629	595

As a simplification an s-distance of 550 mm is chosen in all the sections. This is because stirrups are needed for the mounting of the top reinforcement.

The complete calculations can be seen in appendix N.

5.4.2.4 Design for crack width

The design for crack width is performed in the same manner as for case 1 with the same calculation order, see chapter 5.3.2.4.

For the conditions specified in section 5.2.1 an acceptable crack width of 0.4 mm is determined.

The crack width, with reinforcement according to calculations for bending (section 5.3.2.2), is listed for the 4 sections in table 5.15:

Table 5.15: Crack width for the reinforcement calculated in section 5.3.2.2

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
Crack width, top [mm]	1.17	1.61	2.82	8.58
Crack width, bottom [mm]	0.52	0.74	0.72	3.62

In table 5.15 it is apparent that the crack width is not acceptable for any section. This yields both at the top and the bottom of the foundation. As a consequence more reinforcement must be put in for all sections. Table 5.16 lists the required reinforcement area to limit the crack width to the acceptable 0.4 mm.

Table 5.16: Required reinforcement

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$A_{s,top}$ $[mm^2/m]$	**1540**	**1199**	**818**	**422**
$A_{s,bottom}$ $[mm^2/m]$	**3608**	**2794**	**1800**	**520**

The values in bold in table 5.16 is the reinforcement area that are adjusted to fulfill the crack width.

The complete calculations can be seen in appendix P.

5.4.2.5 Design for fatigue

The design for fatigue is performed in the same manner as for case 1 with the same calculation order, see chapter 5.3.2.5.

<u>Fatigue for compression in concrete</u>

The calculations are performed in the 4 sections defined in section 5.3.2.1 and the fatigue criterion is fulfilled in all sections.

Fatigue for tension in reinforcement

In the top reinforcement there is no stress difference in the reinforcement between maximum and minimum fatigue load. This is because of the invariance of the bending moment due to the weight of the foundation and the soil above the foundation which isn't a cyclic loading. In the bottom reinforcement where the bending moment is related to the cyclic loading there are stress differences in the reinforcement. The minimum- and maximum stress and the difference can be seen in table 5.17.

Table 5.17: Stresses and stress difference in bottom layer reinforcement

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$\sigma_{s.\max}$ $[MPa]$	153.64	148.79	138.83	50.81
$\sigma_{s.\min}$ $[MPa]$	71.54	62.61	73.48	18.72
$\Delta\sigma_s$ $[MPa]$	**82.10**	**86.18**	65.35	32.09

It is apparent that the simplified requirement of the stress width of 70 MPa is exceeded at the bottom reinforcement for the two innermost sections.

One way to reduce the stress width is to put in more reinforcement, but a better possible solution is to more carefully investigate the limit for the stress width which will result in an increase of the allowed stress width.

To do this the right Wöhler curve must be determined. The principle Wöhler curve for reinforcement can be seen in figure 5.20.

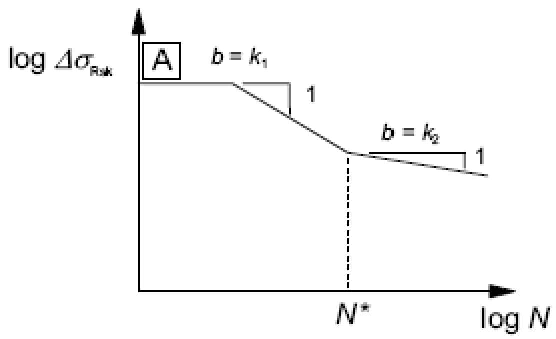


Figure 5.20: Wöhler curve for reinforcement [K]

According to Eurocode one point of the curve is known and has got the following coordinates $\left(N^*, \Delta\sigma_{Rsk}\right) = (10^6 \, cycles\,,\; 162.5\,MPa)$ which becomes (6, 8.21085) in logarithmic values. The actual number of loading cycles is 10 millions and the stress exponent k_2 is has the value 7 (se section 5.1.2). The slope to the right of the known point is then $-1/k_2$ and the curve of interest has the equation:

$$y = -\frac{1}{k_2}(N-6) + \log(162.5 \cdot 10^6) \tag{5.14}$$

$$\Delta\sigma_{Rsk} = 10^y = 10^{-\frac{1}{k_2}(N-6)+\log(162.5\cdot 10^6)} = 162.5 \cdot 10^{-\frac{1}{k_2}(N-6)} = \frac{162.5}{10^{\frac{N-6}{k_2}}} \tag{5.15}$$

This gives the following stress value of interest:

$$\Delta\sigma_{Rsk}(7) = \frac{162.5}{10^{\frac{7-6}{7}}} = 116.9\,MPa$$

It is now apparent that the actual stress width in the reinforcement is within the maximum stress width and there is no need to increase the amount of reinforcement. The complete calculations can be seen in appendix Q.

5.5 Case 3 - Piled foundation with cohesion piles

In situation 3, where the bedrock is at great distance, the piles are functioning as cohesion piles. The shear strength of the soil is then determining the bearing capacity of a pile and the stiffness of the soil and the piles are governing the settlements. This is a foundation where both the piles and the plate are bearing load a so called piled-raft foundation, more detailed described in chapter 3.3.2. One big challenge in the piled-raft foundation method is to determine the stiffness parameters of the piles and the slab. The magnitude of these parameters is of major importance for the load distribution. This is why this model often is associated with large uncertainty.

5.5.1 Geotechnical design

In this case the complete geotechnical design is done in Plaxis. In a clay soil the settlements tend to be rather big [25], why the settlement calculation will determine the piling. The model consists of a plain strain model, and as for case 1 a one meter wide shred of an equivalent quadratic area is modeled instead of the circular, see figure 5.7 (case 1). The piles are modeled with plate elements, where the stiffness parameters are defined per meter depth. Stiffness is added to the piles with an axial stiffness (EA) and a flexural stiffness (EI).

$$EA = Et\text{ , } \left[N/m\right]$$

$$EI = \frac{Et^3}{12}\text{ , } \left[Nm^2/m\right]$$

Where E is Young's modulus for the plate material
t is the thickness of the beam

If the piles are positioned with one meter distance, the stiffness in Plaxis is the actual stiffness of the pile, that is:

$$EA = Eb^2 = 33 \cdot 10^9 \cdot 0.27^2 = 2.4057\, GN/m$$

$$EI = E\frac{b^4}{12} = 33 \cdot 10^9 \frac{0.27^4}{12} = 14.6146\, MNm^2/m$$

If the piles are positioned with half the distance between them, the stiffness parameters above are doubled and so on.

The interaction foundation-soil, and pile-soil are modeled with interface elements with strength parameter 0.9 for surfaces facing friction soil and 0.8 for surfaces facing the clay. The interface elements are extended through corners to avoid stress concentrations around corners, see section 4.3.3.2.

A set up with 56 piles with a length of 60 m at the outmost perimeter and 12 vertical inner piles with a length of 30 m giving reasonable settlements. Figure 5.21 and 5.22 are showing the Plaxis model and figure 5.23 showing the deformed mesh after the calculation. The piles at the outermost perimeter is inclined inwards and outwards alternately both with the inclination angle 4.47 degrees. Inside the rectangle that encloses the foundation (figure 5.21 and 5.23) the mesh is refined causing a more exact result. The standard fixities described in chapter 4.3.1 are applied to the model.

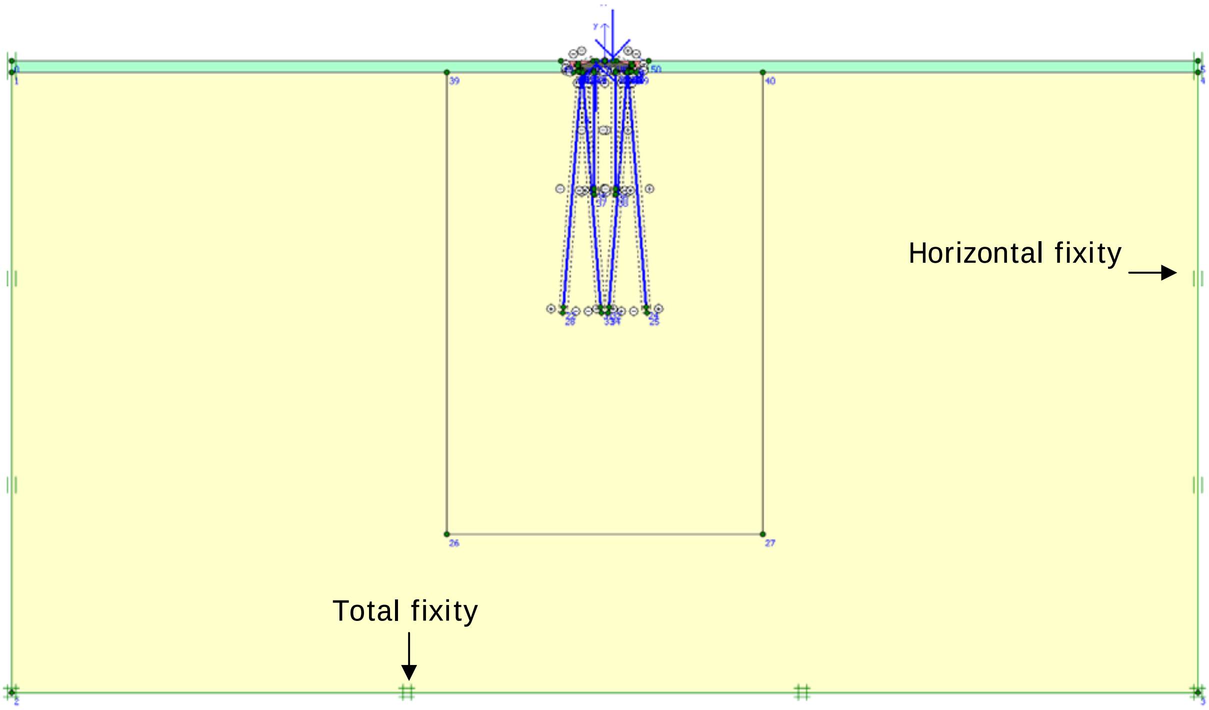


Figure 5.21: The geotechnical model

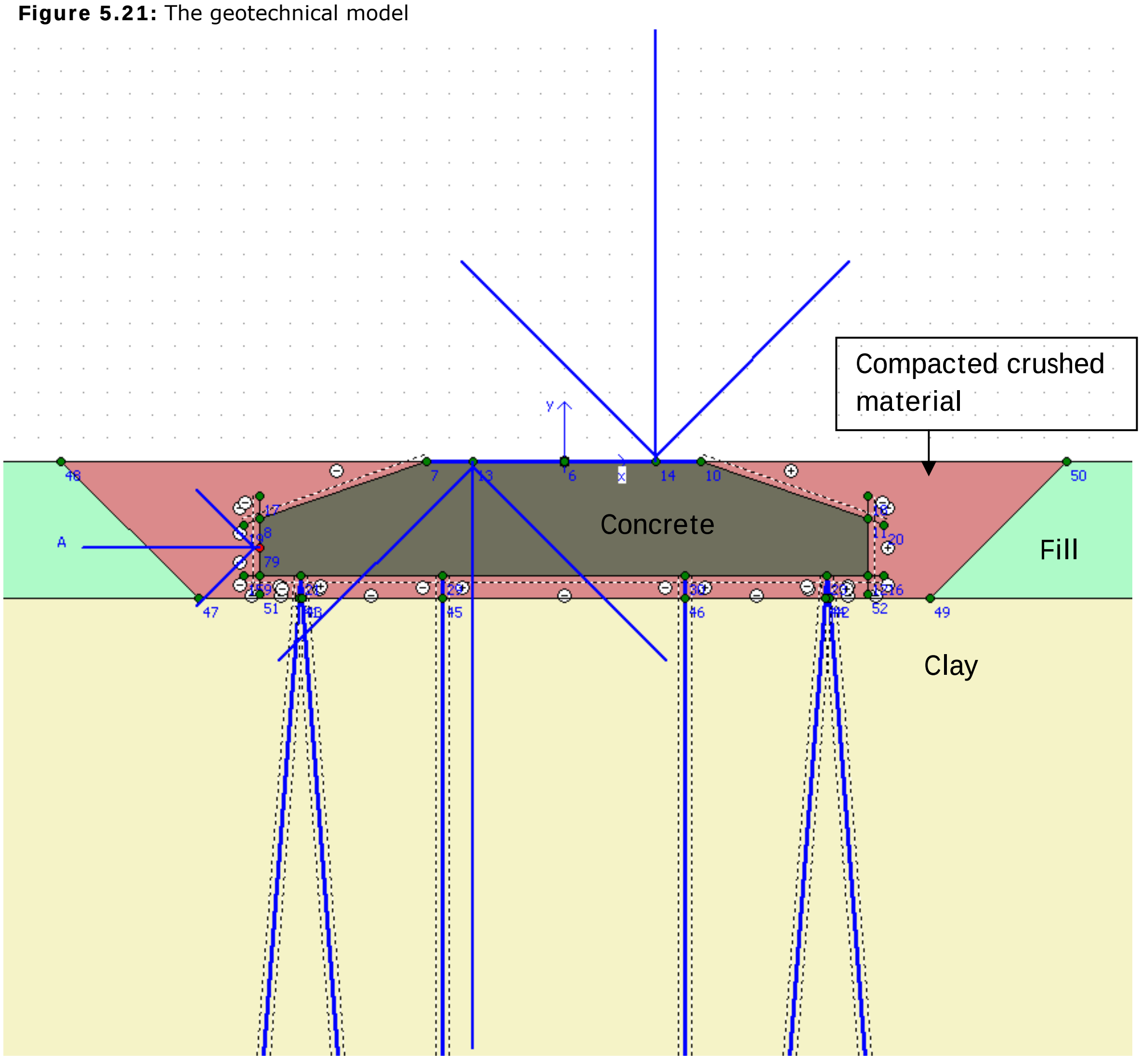


Figure 5.22: Enlarged picture of the foundation

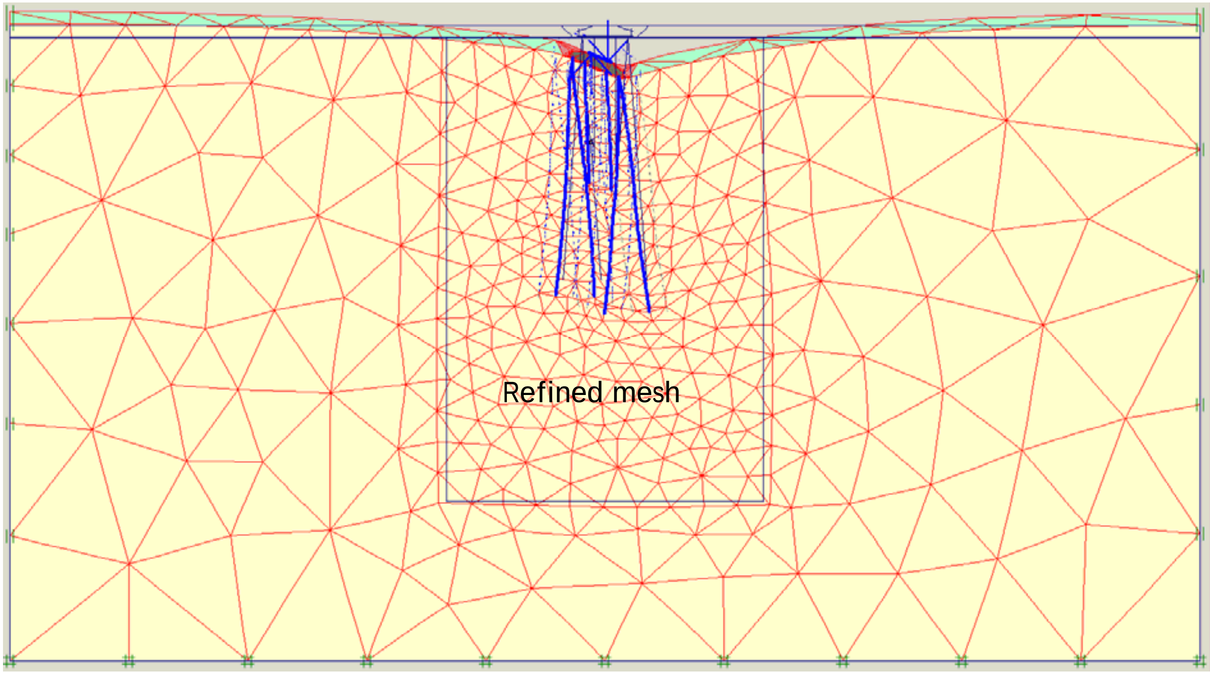


Figure 5.23: The deformed mesh

Figure 5.24 shows the settlements the model is giving under the left- and the right side of the foundation. The settlement under the left side is 55 mm and under the right side 28 mm giving a differential settlement of 27 mm which gives an equivalent settlement of 2.03 mm/m. This settlement might be too big, but a set up with longer piles or with more piles is not decreasing the settlements.

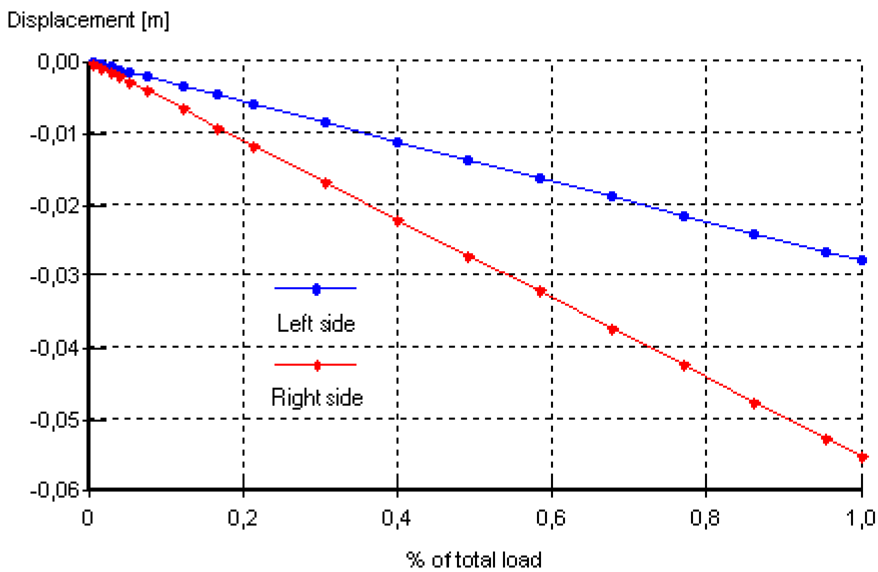


Figure 5.24: Settlements under the left- and the right side of the foundation

The bearing capacity for the piles must also be calculated, and this is done according to section 2.4.4.2. The calculation is performed both for the drained- and the undrained situation and for both the long 60 m piles, and the shorter 30 m long piles. When the tension capacity is calculated a reduction is calculated for.

Undrained analysis:

Equation 2.17 is used with the following values:

$\alpha = 0.7$ which is the adhesion factor for concrete piles

The adhesion factor is reduced to 75 % for the tension capacity. $c_{ud} = \frac{c_{uk}}{\gamma_M} = \frac{40}{1.33} = 30\,kPa$, is the design value for the undrained shear strength of the clay. The partial factor is set to 1.33.

The resistance is now calculated for compression and tension respectively:

Compression:

$$R_{60.C} = \alpha \cdot c_{ud}(L_{60} - 3) \cdot 4a = 0.7 \cdot 30 \cdot (60 - 3) \cdot 4 \cdot 0.27 = 1293\,kN$$

$$R_{30.C} = 0.7 \cdot 30 \cdot (30 - 3) \cdot 4 \cdot 0.27 = 612\,kN$$

Tension:

$$R_{60.T} = \alpha \cdot c_{ud}(L_{60} - 3) \cdot 4a = 0.75 \cdot 0.7 \cdot 30 \cdot (60 - 3) \cdot 4 \cdot 0.27 = 970\,kN$$

$$R_{30.T} = 0.75 \cdot 0.7 \cdot 30 \cdot (30 - 3) \cdot 4 \cdot 0.27 = 459\,kN$$

Drained analysis:

The factor β is determined from figure 2.12 and is for 60 m long piles 0.15 and for 30 m long piles 0.20 and besides that a typical reduction for tension capacity is for friction 0.5. The adhesion factor is set to 0.7 as above, but with no reduction. The resistance is calculated for compression and for tension:

Compression:

$$R_{60.C} = \alpha \cdot c_d \cdot z \cdot P + \beta \sigma'_{\bar{v}}\, z \cdot P = (\alpha \cdot c_d + \beta \cdot \sigma'_{\bar{v}})z \cdot P =$$

$$= (0.7 \cdot 3 + 0.15 \cdot \frac{60+3}{2} 8) \cdot 57 \cdot 4 \cdot 0.27 = 2456\,kN$$

$$R_{30.C} = (0.7 \cdot 3 + 0.20 \cdot \frac{30+3}{2} 8) \cdot 27 \cdot 4 \cdot 0.27 = 831\,kN$$

Tension:

$$R_{60.T} = \alpha \cdot c_d \cdot z \cdot P + \beta \sigma'_{\bar{v}}\, z \cdot P = (\alpha \cdot c_d + \beta \cdot \sigma'_{\bar{v}})z \cdot P =$$

$$= (0.7 \cdot 3 + 0.5 \cdot 0.15 \cdot \frac{60+3}{2} 8) \cdot 57 \cdot 4 \cdot 0.27 = 1293\,kN$$

$$R_{30.T} = (0.7 \cdot 3 + 0.5 \cdot 0.20 \cdot \frac{30+3}{2} 8) \cdot 27 \cdot 4 \cdot 0.27 = 446\,kN$$

The compression capacity and the tension capacity for the long piles are 1293 kN and 970 kN respectively. For the shorter piles the capacities are 612 kN and 446 respectively. These values must be compared with the maximum compression- and tension forces that will occur in the piles, which is calculated in section 5.5.2.1.

5.5.2 **Structural design**

5.5.2.1 Sectional forces and pile loads

In this case the pile loads, the ground pressure and the sectional forces are calculated numerically with the finite element software FEM-design, which is described in section 4.2. In the model created in FEM-design all piles is put in its right position and modeled as point supports with an axial stiffness. The soil is modeled as a surface support also with a stiffness k. The values for the stiffness's are important to model right to provide a realistic load distribution. To obtain the stiffness's, two different analyses is performed in Plaxis; one with a pile loaded axially, and one with a plate loaded vertically. The models can be seen in figure 5.25 and 5.26.

The stiffness is not constant values, but varies with the loading. Therefore several calculations are performed with increasing load. The stiffness is calculated as:

$$k = \frac{P}{\delta} \tag{5.16}$$

Where P is the vertical load

δ is the displacement

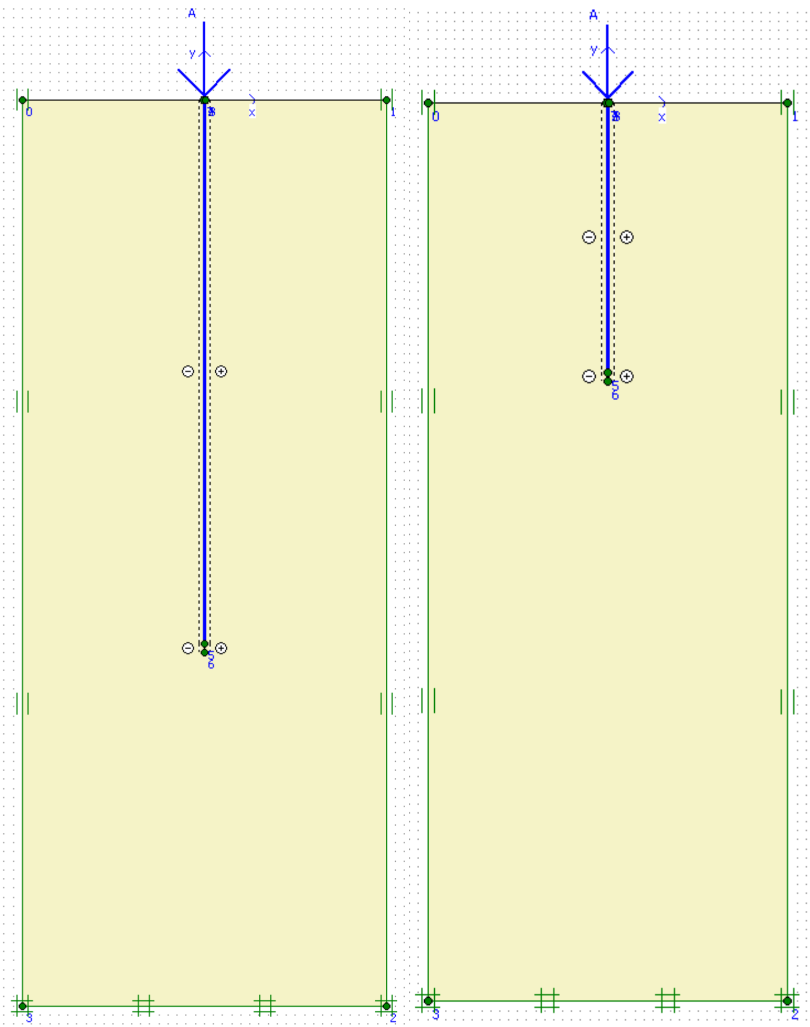


Figure 5.25: Calculation model for determining the pile stiffness as a function of the load.

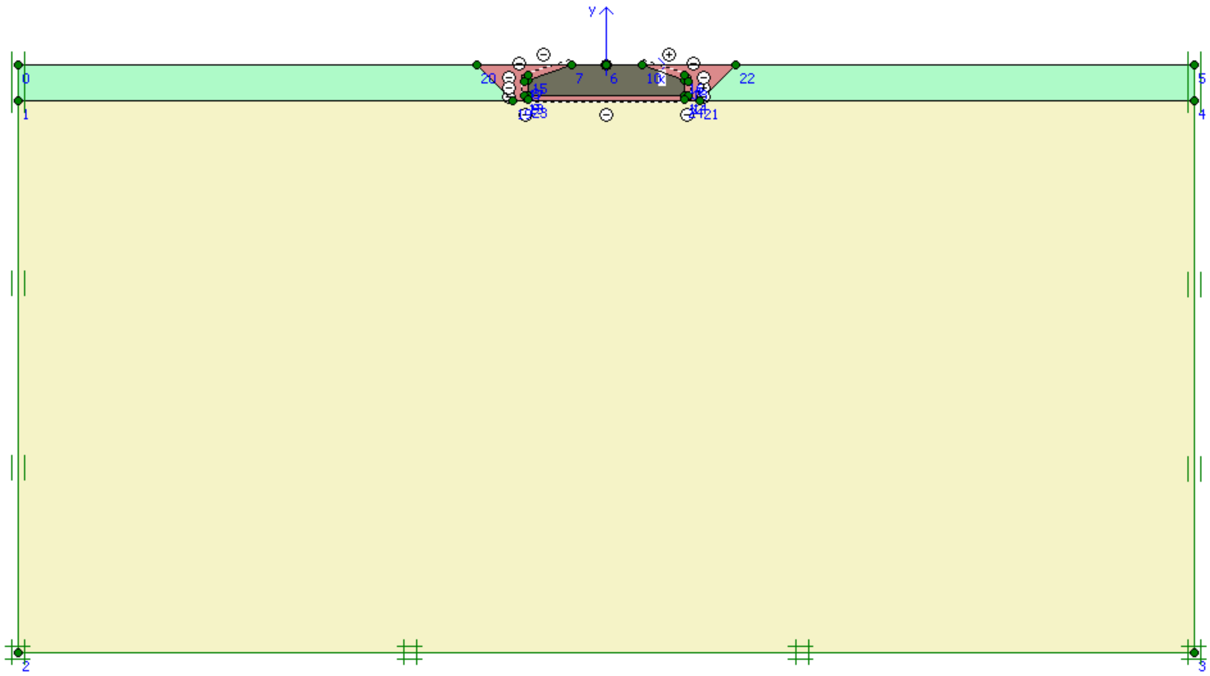


Figure 5.26: Calculation model for determining the plate stiffness as a function of the load.

The stiffness is plotted as a function of the vertical load in figure 5.27 for both the long and the short pile.

The input plate stiffness in Plaxis, is defined per square meter why the plate stiffness in figure 5.28 is showed both as a total stiffness for the whole plate, and a stiffness per square meter for different distributed load.

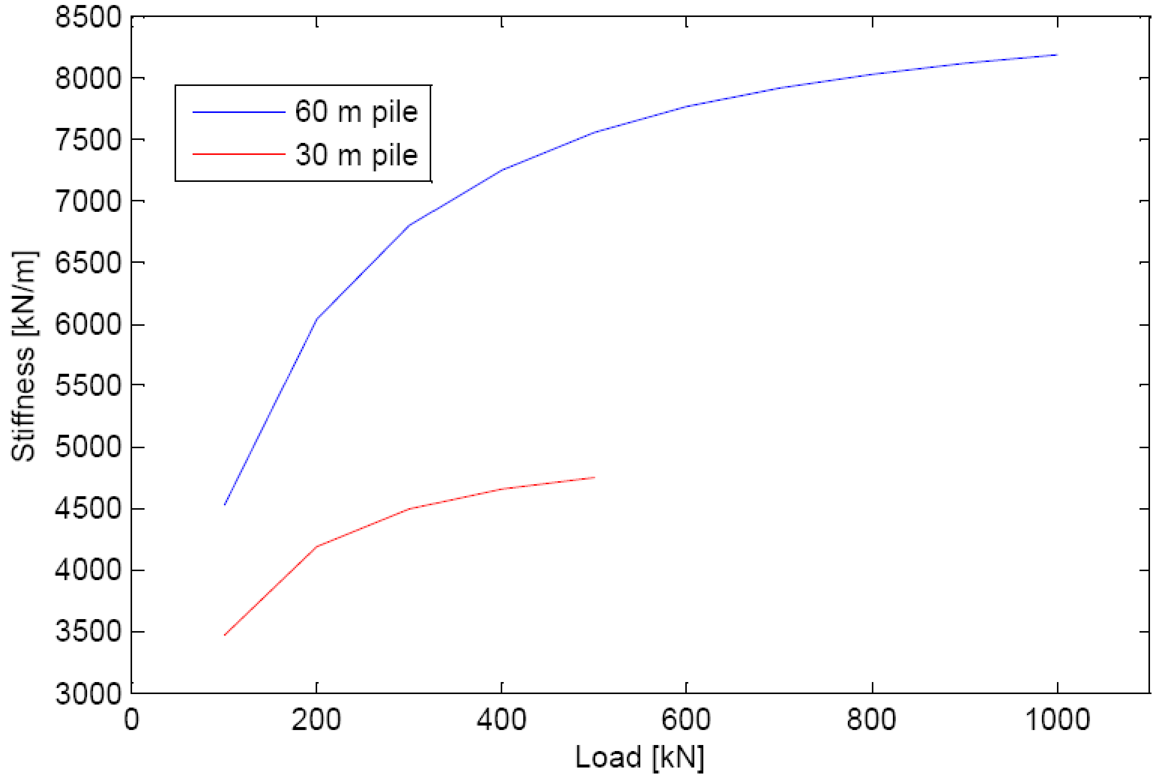


Figure 5.27: Pile stiffness as a function of the pile load

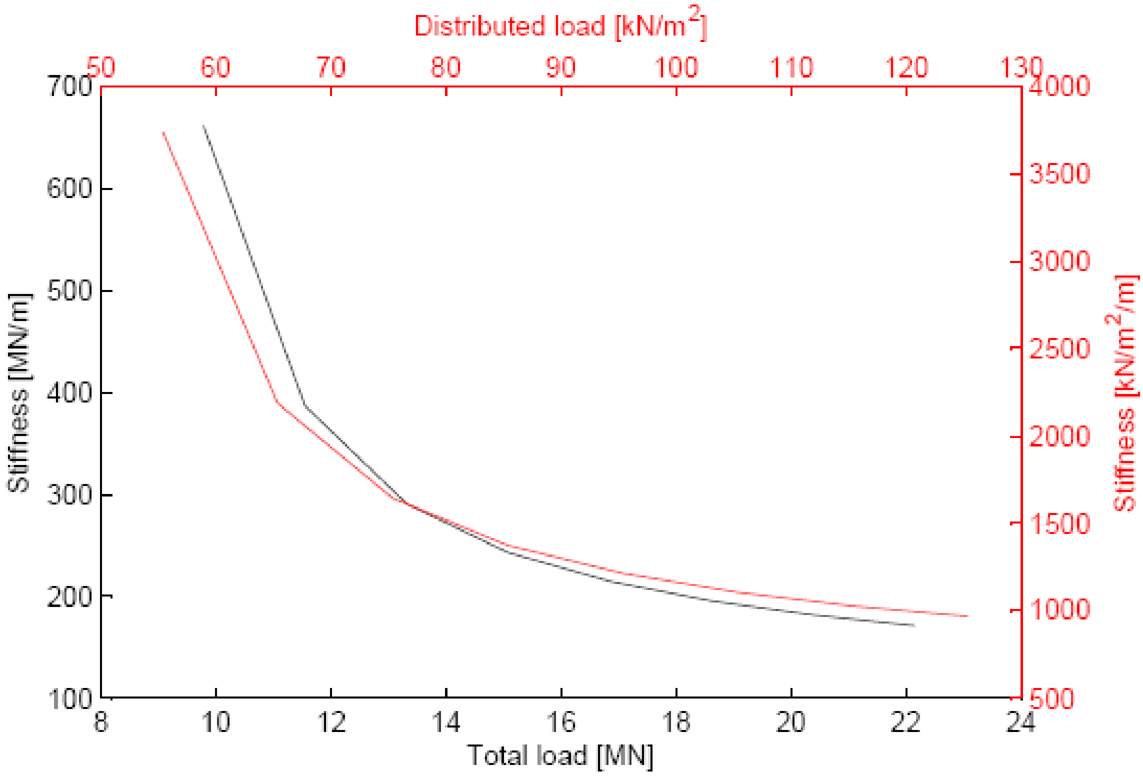


Figure 5.28: Plate stiffness as a function of the load

The stiffness values that are input for Plaxis, and the corresponding loads must match the calculated output pile loads and the distributed load. To find the right stiffness an iterative process is necessary. If the output pile loads are bigger than the corresponding loads for the input stiffness, a lower input stiffness must be used and vice versa.

Table 5.18 shows the input values chosen for the stiffness parameters and the corresponding load, and the output values for the load. The values are chosen after an iteration process customized for the ultimate loads, but is even used for the other loading cases. The output pile loads for the outermost piles exceeding the corresponding load value for the modeled stiffness, but a decrease of the stiffness for the piles will result in an even bigger difference between input and output load.

Table 5.18: Input- and output values for modeling he stiffness in the FEM-model

Input stiffness and the corresponding load						Output loads		
P_{60}	k_{60}	P_{30}	k_{30}	p_{plate}	k_{plate}	P_{60}	P_{30}	p_{plate}
[kN]	[kN/m]	[kN]	[kN/m]	[kN/m^2]	[kN/m^2/m]	[kN]	[kN]	[kN/m^2]
300	6800	150	3830	85	1373	387	145	85

All the loads specified in section 5.1.2 is defined in the model as four different load cases; ULS, SLS; minimum fatigue load and maximum fatigue load. The concrete plate is also modeled with its correct geometry and all its properties. To prevent large stress concentrations the vertical load is spread as a line load along the embedded ring, and the bending moment is modeled as a force couple, see figure 5.29 for a view of the model in FEM-design.

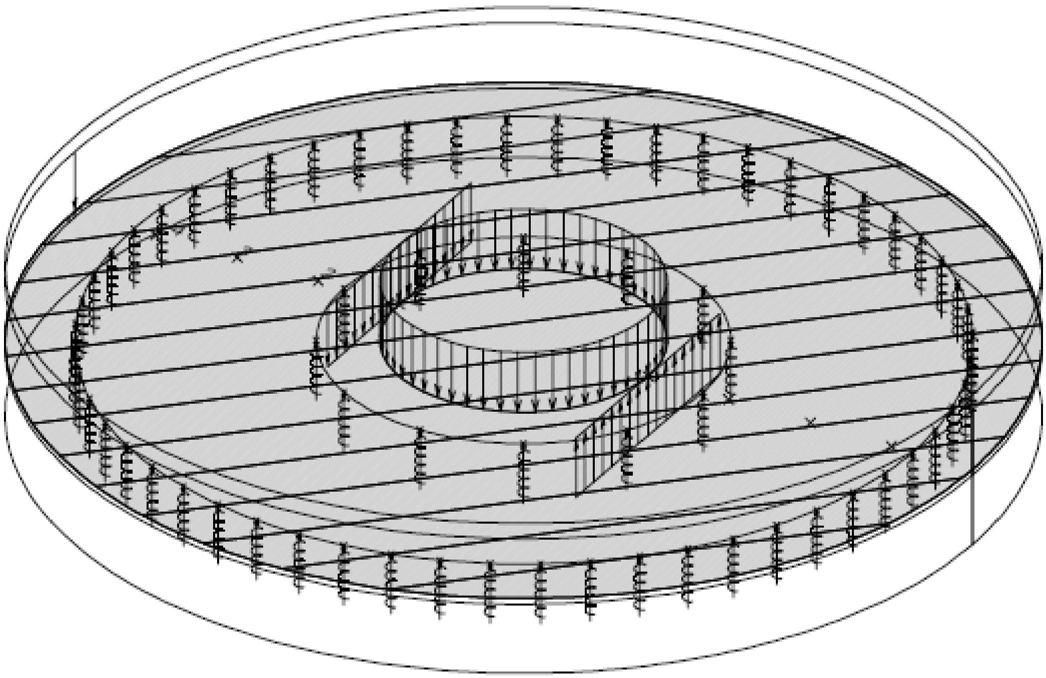


Figure 5.29: The FE-model with the equivalent line loads

The pile loads is calculated for all load combinations and figure 5.30 shows the result for ULS. For piles at one side of the foundation tension forces exists in the piles. Table 5.19 shows the maximum pile loads compared with its capacities.

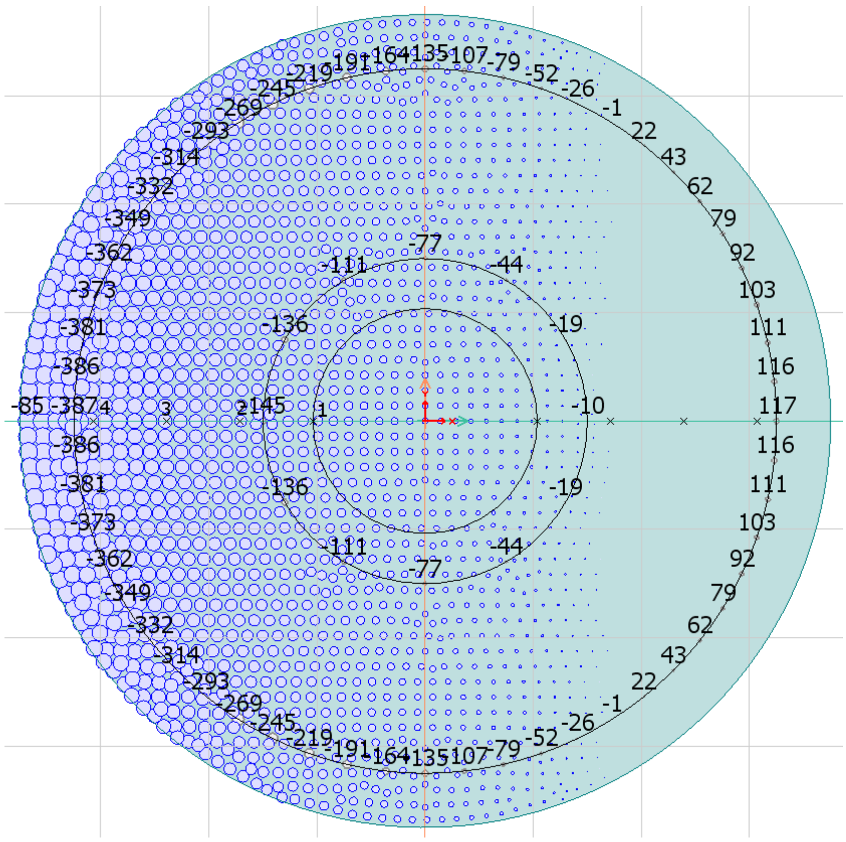


Figure 5.30: The pile loads in kN and the maximal ground pressure in kN/m2 for ULS

Table 5.19: Maximum pile loads and the corresponding capacities

	60 m long piles		**30 m long piles**	
	Compression	Tension	Compression	Tension
Load [kN]	387	117	145	-
Capacity [kN]	1293	970	612	446

The section force distribution can be seen for the different load cases in appendix R.

The designing section forces are read at 4 points (marked in figure 5.29). In the same way as for case 1, one design situation is when only the weight acting on the "beam" giving the same sectional forces here denoted with Top or simply t. The bending moments and shear forces for the other situation is read from the model. The section forces can be seen in table 5.20.

Table 5.20: Designing sectional forces at four points

	Section no:	**Dist.[m]**	**Mt [kNm]**	**Mb [kNm]**	**Vt [kN]**	**Vb [kN]**
ULS	1	2.075	-860	3408	-317	1668
	2	3.431	-484	1605	-238	1008
	3	4.788	-215	865	-159	788
	4	6.144	-54	107	-79	591
SLS	1	2.075	-860	2107	-317	1045
	2	3.431	-484	1001	-238	610
	3	4.788	-215	550	-159	497
	4	6.144	-54	63	-79	395
Fat.Min	1	2.075	-860	560	-317	296
	2	3.431	-484	287	-238	133
	3	4.788	-215	180	-159	153
	4	6.144	-54	12	-79	164
Fat.Max	1	2.075	-860	1335	-317	672
	2	3.431	-484	646	-238	373
	3	4.788	-215	366	-159	324
	4	6.144	-54	38	-79	280

5.5.2.2 Design for bending moment

The design for bending moment is performed in the same manner as for case 1 and 2 with the same calculation order, see chapter 5.3.2.2 and 5.4.2.2.

The result is presented in table 5.21 as required reinforcement area in bottom and in top of the foundation. The complete calculation can be seen in appendix S.

Table 5.22: Required reinforcement area in top and bottom of the foundation

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$A_{s.Top}$ $[mm^2/m]$	815	540	281	85
$A_{s.Bottom}$ $[mm^2/m]$	3261	1798	1131	168

5.5.2.3 Design for shear force

The design for shear force is performed in the same manner as for case 1 and 2 with the same calculation order, see chapter 5.3.2.3 and 5.4.2.3.

The calculation of shear capacity without stirrups, results in not sufficient capacity for any section and therefore stirrups is required. The calculated s-distance of the shear stirrups is presented in table 5.23.

Table 5.23: Minimum s-distance for shear stirrups

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
s-distance [mm]	508	603	630	663

As a simplification an s-distance of 500 mm is chosen in all the sections. This is because stirrups are needed for the mounting of the top reinforcement.

The complete calculations can be seen in appendix T.

5.5.2.4 Design for crack width

The design for crack width is performed in the same manner as for case 1 and 2 with the same calculation order, see chapter 5.3.2.4 and 5.4.2.4.

For the conditions specified in section 5.2.1 an acceptable crack width of 0.4 mm is determined.

The crack width with reinforcement according to calculations for bending (section 5.5.2.2) is listed for the 4 sections in table 5.24:

Table 5.24: Crack width for the reinforcement calculated in section 5.5.2.2

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
Crack width, top [mm]	1.17	1.61	2.82	8.58
Crack width, bottom [mm]	0.50	0.55	0.75	3.41

In table 5.24 it is apparent that the crack width is not acceptable for any section. This yields both at the top and the bottom of the foundation. As a consequence more reinforcement must be put in for all sections. Table 5.25 lists the required reinforcement area to limit the crack width to the acceptable 0.4 mm.

Table 5.25: Required reinforcement

Section	**1**	**2**	**3**	**4**
Distance [m]	2,075	3,431	4,788	6,144
A_s Top [mm^2/m]	**1540**	**1199**	**818**	**422**
A_s Bottom [mm^2/m]	**3818**	**2201**	**1659**	**525**

The values in bold in table 5.25 is the reinforcement area that are adjusted to fulfill the crack width.

The complete calculations can be seen in appendix U.

5.5.2.5 Design for fatigue

The design for fatigue is performed in the same manner as for case 1 with the same calculation order, see chapter 5.3.2.5 and 5.4.2.5.

Fatigue for compression in concrete

The calculations are performed in the 4 sections defined in section 5.3.2.1 and the fatigue criterion is fulfilled in all sections.

Fatigue for tension in reinforcement

In the top reinforcement there is no stress difference in the reinforcement between maximum and minimum fatigue load. This is because of the invariance of the bending moment due to the weight of the foundation and the soil above the foundation which isn't a cyclic loading. In the bottom reinforcement where the bending moment is related to the cyclic loading there are stress differences in the reinforcement. The minimum- and maximum stress and the difference can be seen in table 5.26.

Table 5.26: Stresses and stress difference in bottom layer reinforcement

Section	**1**	**2**	**3**	**4**
Distance [m]	2.075	3.431	4.788	6.144
$\sigma_{s.\max}$ [MPa]	149.84	146.89	128.93	50.33
$\sigma_{s.\min}$ [MPa]	62.86	65.26	63.41	15.89
$\Delta\sigma_s$ [MPa]	**86.99**	**81.63**	65.52	34.44

It is apparent that the simplified requirement of the stress width of 70 MPa is exceeded at the bottom reinforcement for the two innermost sections.

In the same way as for case 2 a more exact value for the allowed stress width in the reinforcement determined from a Wöhler curve is used. This value is calculated in section 5.4.2.5 and is valid even for this case. The limit is $\Delta\sigma_{Rsk} = 116.9\,MPa$ which is bigger than the biggest actual stress width of 86,99 MPa.

The complete calculations can be seen in appendix V.

5.6 Comparison of the three different cases

The geometry of the plate is the same for all three cases and the amount of concrete is therefore the same namely 331 m^3 concrete. For case 2 and 3 piles is used and the number of piles and the total piling length is listed in table 5.27.

Table 5.27: Number of piles and the total length of the piling

	Inclined piles	Pile length	Vertical piles	Pile length	Total no. Of piles	Total pile length
Case 2	28 pcs	20 m	6 pcs	20 m	34 pcs	680 m
Case 3	56 pcs	60 m	12 pcs	30 m	68 pcs	3720 m

The maximum ground pressure and pile loads for the three cases can be seen in table 5.28.

Table 5.28: Maximum ground pressure and pile loads

	Maximum ground pressure	Maximum pile load	Maximum tension pile load
Case 1	318 kPa	-	-
Case 2	-	1078 kN	261 kN
Case 3	85 kPa	387 kN	117 kN

The settlements for the three cases can be seen in table 5.29.

Table 5.29: Settlements and horizontal deflection at top of the tower

	Element (s) giving settlement	Right side [mm]	Left side [mm]	Differential settlements [mm/m]	Horizontal deflection at top of the tower [mm]
Case 1*	Soil	13.0	0.5	0.94	78
Case 2**	Piles	6.45	0.33	0.47	39
Case 3*	Soil & Piles	55	28	2.03	168

* - the right- and left settlements refers to the equivalent width for the calculation model in Plaxis
** - the right- and left settlements refers to the settlement at the outermost piles

The anchor reinforcement is the same for all three cases, because the embedded ring is the same and so are the loads acting on the foundation. Totally 127 U-bows with a diameter of 32 mm are required and placed around the embedded ring.

The required reinforcement in the top- and bottom layer for the different cases is listed in table 5.30 as reinforcement area per m width for the four sections that are studied.

Table 5.30: Required reinforcement

Bottom layer reinforcement, A_s [mm^2/m]				
Section No.	**1**	**2**	**3**	**4**
Case 1	3917	2852	1477	578
Case 2	3608	2794	1800	520
Case 3	3818	2201	1659	525
Bottom layer reinforcement, A_s [mm^2/m]				
Section No.	**1**	**2**	**3**	**4**
Case 1	1540	1199	818	422
Case 2	1540	1199	818	422
Case 3	1540	1199	818	422

For all cases shear stirrups is needed and for case 1 and 2 the distance between the stirrups is 550 mm and for case three a distance of 500 mm is required. The distances are legal for stirrups with a diameter of 25 mm.

6. General aspects in the choice of foundation method

The choice of foundation method is dependent on many parameters, and it is hard to decide when a specific method is the most appropriate. Generally it is the most cost efficient solution that is chosen, given that the safety can be granted. When calculating a budget for the construction of a foundation there are almost unlimited number of costs that have to be considered. Though the factors listed here is of major importance:

- The site availability. Is the site very remote? Is there a road leading to the site? Is it necessary to improve the strength of the road?
- The amount of material required. The volume of the concrete, the amount of reinforcement, the number of piles, the pile material etc.
- The designing work. Are the construction documents time-consuming to create? Do the designers have the knowledge and the tools that are required? Is the design optimized?
- The construction work. Is extensive excavation work necessary? Is the construction easy and fast to construct? Does the construction work affect the environment?

Primary focus of this thesis is to come out with cost effective solutions of the foundation design for different types of soil conditions. When the designer starts his work the site is specified and a geotechnical investigation is already performed. The wind turbine with its embedded ring and the loads for the actual case is obtained from the manufacturer of the turbine. With help of this information it is up to the designers to carry out necessary construction documents for the contractor to build after.

The geotechnical investigation from the site is providing information about the soil layer sequence, the distance to the bedrock, the strength and stiffness parameters of the soil layers, the ground water level and its variation. This information is valuable when deciding the foundation level and if piling is appropriate.

If the soil is strong and stiff and the groundwater level is at great depth a spread foundation is to prefer. This is the simplest foundation method in many respects; it is a well proved foundation that is easy to construct and quite easy to design. It is generally also a cheap method as no piling work is necessary.

If the soil parameters are somewhat worse the sufficient area of a spread foundation is quite big to keep the ground pressure below the soil's capacity. The required amount of concrete is getting very big as the volume increasing with the squared distance, and another foundation method may be more cost efficient. There are several possible alternatives besides the spread foundations then.

One good solution can be to foot the foundation deeper in the ground. The capacity of the ground is better at a greater depth, and in addition the width of the foundation can be decreased thanks to the bigger vertical load from the overlaying soil. This will result in a smaller eccentricity of the load. One disadvantage of this solution is the major excavation work this requires. If the site is remote and the excavated soil cannot be used as refilling material this may be an expensive method.

Another solution is to perform soil improvements by exchanging the soil, compacting the soil or add some strength to the soil with help of lime/cement columns or grout. These methods are expensive, but can yet be realistic if the soil quality is poor.

If the soil layer sequence shows that a strong and stiff layer is at reasonable depth it can be a good solution to drive piles to this layer. The piles are then functioning as toe-bearing piles, and soil above the strong layer is not carrying any load. To ensure that the piles are not buckling, the subgrade modulus of the soil cannot be to low. The same is valid for piles driven to the bedrock. A benefit with the bedrock-driven piles is that the piles can be anchored in the bedrock to handle tension forces, though this requires very solid and strong bedrock.

If the bedrock is at great depth and the soil hasn’t got enough stiffness for a reasonable big spread foundation it can be a good method to install cohesion piles. To reduce the number of piles and the pile length a calculation model which assumes that both the plate and the piles are bearing load, can be of interest. It is then necessary that the designers have highly reliable information about the stiffness parameters of the ground to model this in the right way. Modeling the soil to stiff may result in too big loads in the piles and a failure can occur.

In the cases studied in section 5 three different foundation methods are concerned. The first one, the spread foundation, is the obvious choice for the conditions of the site. The soil is strong and stiff, and the groundwater is at a reasonable depth. If the groundwater level is changing very much it could be appropriate to set the foundation level at the ground level to minimize the risk of lifting forces from a high groundwater level. Though for a more shallow footing it would be necessary to put soil above the foundation to get sufficient weight to keep the eccentricity low.

For case 2, with a weak soil and bad stiffness a spread foundation would result in too big settlements if the soil body wouldn’t collapse first. As the bedrock is at a reasonable depth it is a quite easy piling work. By driving the piles to the bedrock one gives the piles high capacity, because it’s only the structural capacity that has to be considered. This capacity is often higher than the bearing capacity from the soil would be for a pile installed without the toe-bearing. For this site conditions would also the foundation in the Ruukki case, described in section 1.7, be suitable. By anchoring the piles in the bedrock a much smaller plate can be used due to the high tension forces the piles can carry. In the Ruukki case only 8 large piles were used with extremely high capacity. The maximum pile loads became in that case 5,8 MN in compression and 2,5 MN in tension.

The site conditions apparent in case 3 are not very well suited for a foundation with this extreme load condition. Anyhow, there are a few options for foundations for this site, where the one chosen, namely a spread foundation with cohesion piles is one of them. Another method could be to perform soil improvements primarily to get a more stiff soil to reduce the settlements. The methods described in chapter 3.2 could then be applicable. The method chosen for this should with cohesion piles and a wide spread plate is benefited if the calculation model assumes that both the plate and the piles are bearing load. This should in general result in a more cost efficient solution, because of the reduced piling length this should generate.

7. Step by step-design of a wind turbine foundation

As mentioned before the design of a wind turbine foundation is a very iterative process if the foundation should be optimized. The process can be divided into three subcategories:

- Choice of foundation method
- Geotechnical design
- Structural design

The choice of foundation method is sometimes easy and obvious, but can also be very hard where several methods may be applicable. Some aspects that one should bear in mind concerning the choice of foundation method are described in chapter 6. For a proposed method the geotechnical design begins.

7.1 Geotechnical design

The geotechnical design can be described as an iterative loop, where the first step is to propose a foundation weight including possible weight from overlaying soil, the foundation depth and the foundation area. Rough estimates for the ground pressure and the eccentricity will govern the choice of the quantities. The next step in the loop is to calculate the bearing capacity and the settlements, including the differential settlements for the foundation.

If the calculations results in too big settlements or a collapse of the soil body, changes in the first step in the geotechnical loop must be done. For example the foundation area cab be increased, the weight can be increased/decreased and so on.

If the safety margin for the settlements and/or the capacity of the ground is significantly big a more optimized foundation can be proposed, i.e. a smaller foundation area, less weight, a more shallow foundation level.

One possible scenario in the geotechnical design loop is that the quantities becomes unreasonable big or low after the iteration process, and then a new foundation method shall be considered. If so is the case the process will start again. The geotechnical procedure is illustrated in figure 7.1:

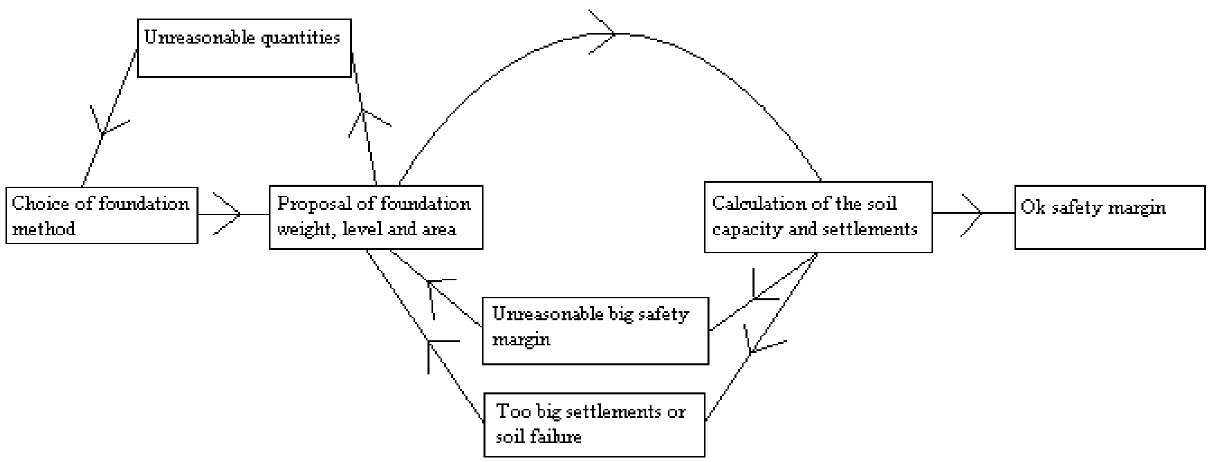


Figure 7.1: Iterative loop for the geotechnical design

7.2 Structural design

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

- The size of the embedded ring and (if specified) the height of the ring above the top of the foundation might give a minimum thickness of the centre part of the foundation.
- The size of the sectional forces will limit the thickness of the foundation outside the embedded ring.
- If the upper part of the foundation having a slope, limit the slope to what's possible for the construction workers to perform when they are casting the concrete.
- Sufficient concrete thickness above piles (if piles exists) due to big shear forces (punching force)
- The concrete quality affecting the amount of reinforcement, the crack width and the life length of the structure
- The reinforcement quality affects the required amount of it
- The diameter of the reinforcement affects the crack width
- Sufficient concrete thickness under- and above the flange of the embedded ring

For the proposed geometry and material qualities sectional forces is computed in several sections between the embedded ring and the outer perimeter of the foundation. The sectional forces should be calculated for ultimate limit state, serviceability limit state and the fatigue loading. The calculation model chosen is important to obtain relevant sectional forces. The reliability of the sectional forces must be investigated regardless of the chosen calculation model.

The sectional forces are then input for the reinforcement calculation. As the amount of top- and bottom bending reinforcement matters even for the shear capacity and the crack width calculation it is a good way to start with the design for the bending moment.

The design for shear force is the natural next step in the designing process. Here the concrete itself can give sufficient capacity if the concrete thickness is significant. The bending reinforcement calculated earlier gives contribution for the shear capacity, and if the capacity doesn't exceed the shear force there are some options to proceed:

- The thickness of the foundation can be increased
- The amount of bending reinforcement can be increased
- Shear stirrups can be mounted

The first option requires a new sectional forces calculation which gives a new bending reinforcement area and so on. The second alternative, the increased bending reinforcement area, might be a good solution, but the third option is preferable as stirrups are needed anyhow, for the mounting of the top layer reinforcement. If shear stirrups is chosen a calculation of the shortest distance between the stirrups is necessary, and these stirrups will then serve even as help for mounting the top layer reinforcement.

The next step in the design process is the crack width calculation which often, for structures with large reinforcement diameter and a thick concrete cover, results in too large crack widths. If so is the case, a natural step is to increase the reinforcement area. Note that a calculation for the shear stirrups with the increased amount of reinforcement will probably result in fewer stirrups, why this calculation should be performed again. Another solution is to choose a better quality of the concrete (higher tensile strength). A disadvantage with high quality of the concrete is, except for the fact that it is more expensive, that the concrete shrinkage is much bigger [28].

The fatigue design can be seen as a control against fatigue in the materials, why this control is performed after that the geometry, and the reinforcement amount is determined. The fatigue control comprises fatigue in concrete and fatigue in reinforcement.

If the fatigue verification fails, there are a few options to ensure the fatigue capacity. The governing quantities in the fatigue design are the stress width and the number of load cycles. The latter parameter is not possible to change, then remains to adjust the stress width. This can be done by increasing the weight of the foundation, resulting in a greater proportion of the total load that are non-cyclic which eventually gives a smaller stress width. This scenario with an increased weight means that both the geotechnical design and the structural design have to be performed again.

If the fatigue control of the reinforcement fails, it might be a better solution to increase the amount of reinforcement giving a lower total stress in the reinforcement and also a lower stress width.

8. Discussion

Case 1 above is no doubt the cheapest method as it doesn't involve any pile work. The required reinforcement is a little higher than for the other cases, but the extra amount is however a relative small cost in relation to the pile work.

The design is quite easy and straightforward to perform. Most of the calculation can be made manually, but the settlement calculation is preferred to do numerically as it involves a very big eccentricity of the load which will cause a rotating motion of the foundation in the soil.

A conclusion is that if the ground allows the big pressure that will arise, this type of foundation should be chosen.

Case 2 with gives a relatively little total piling length and is a good method if the distance to the bedrock is reasonable. The amount of reinforcement is quite big as it requires more reinforcement at the parts near the perimeter of the foundation due to the big point loads from the piles. The settlements for this type of foundation are very small and the whole structure is very stiff.

The geotechnical design is very straightforward and the level of uncertainty is fairly low, as it only includes the structural capacity of the piles, given that the bedrock is strong and stiff. This method assumes no movement in the piles and to ensure that this actually is the case, it has to be verified that the piles actually are standing on the bedrock (or the strong soil layer). The pile installation work is then of major importance for this type of foundation.

Case 3 resulted in a very large total piling length, and in reality this foundation would probably not have been constructed as it would result in a too expensive foundation. The question is whether the geotechnical design is performed properly or not. In this thesis a two-dimensional model is created for the geotechnical design with the piles modeled as plate elements. The number of piles is determining the stiffness of the plate elements, but in reality it is the surface area of the piles that are significant; the more piles the bigger surface area and the smaller settlements. This cannot be modeled properly in a 2-dimensional model. A 3-dimensional model would probably have resulted in a more trustworthy design giving lower settlements. As a consequence the amount of piles, and the pile length could be lowered.

Even the structural design in the third case has a big uncertainty because of the difficulties modeling the stiffness ratio in the soil and the piles. The stiffness of the plate and the piles is a function of the load and a more correct result would be obtained if the supports in the FEM-model were modeled with these functions. Though this is not possible in the software, the stiffness used in the model is the one true for the maximum pile loads and the maximum ground pressure. The sectional forces distribution could therefore differ from the actual ones, and with false reinforcement areas as a consequence of this. However, the reinforcement area is not a big issue in this case with such a big total piling length.

The designing aspects considered in this thesis are the most essential ones, but in a real case there are a few more that should be considered. Among them punching and concrete spalling are significant. Punching should be controlled above the piles and over- and under the flange of the embedded ring. Concrete cleavage should be controlled around the piles.

9.Further work

The geotechnical design of the piled-raft foundation that is performed in this thesis is modeled in 2-dimensions giving uncertain results, and it would be of interest to see how the result differs from an analysis performed in 3-dimensions. In addition to this the stiffness parameters could be modeled with functions instead of constant values as in this thesis. However this requires software that enables modeling of (preferably) nonlinear supports.

There are many types of piles that can be used for wind turbine foundations, and in this thesis only prefabricated concrete piles are used. Designs with large diameter steel pipe piles, perhaps with reinforced concrete inside, might give economic foundations, why this method would be of interest.

This thesis is limited to onshore wind turbine foundations, but as even a vast number of offshore turbines will be constructed in the near future, the design of offshore foundation methods is of great interest.

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A - Choice of exposure classes

Klass	Beskrivning av miljö	Exempel, informativa, där exponeringsklassen kan förekomma
1 Ingen risk för korrosion eller angrepp		
XO	För betong utan armering eller ingjuten metall: Alla omgivningsförhållanden utom där frysning/upptining, nötning eller kemiska angrepp förekommer För betong med armering eller ingjuten metall: Mycket torr	Betong inuti byggnader med mycket låg luftfuktighet
2 Korrosion föranledd av karbonatisering		
XC1	Torr eller ständigt våt	Betong inuti byggnader med låg luftfuktighet Betong ständigt stående under vatten
XC2	Våt, sällan torr	Betongytor utsatta för långvarig kontakt med vatten Många grundkonstruktioner
XC3	Måttlig fuktighet	Betong inuti byggnader med måttlig eller hög luftfuktighet Utvändig betong skyddad mot nederbörd
XC4	Cykliskt våt och torr	Betongytor i kontakt med vatten, inte tillhörande exponeringsklass XC2
3 Korrosion orsakad av andra klorider än de från havsvatten		
XD1	Måttlig fuktighet	Betongytor utsatta för luftburna klorider
XD2	Våt, sällan torr	Simbassänger Betong utsatt för industrivatten innehållande klorider
XD3	Cykliskt våt och torr	Brodelar utsatta för stänk innehållande klorider Beläggningar Bjälklag i parkeringsanläggningar
4 Korrosion orsakad av klorider från havsvatten		
XS1	Utsatt för luftburet salt men inte i direkt kontakt med havsvatten	Bärverk nära eller vid kusten
XS2	Ständigt under havsytan	Delar av bärverk belägna i havet
XS3	Tidvatten-, skvalp- och stänkzoner	Delar av bärverk belägna i havet
5 Angrepp av frysning/tining		
XF1	Inte vattenmättad, utan avisningsmedel	Vertikala betongytor utsatta för regn och frysning
XF2	Inte vattenmättad, med avisningsmedel	Vertikala betongytor hos vägbyggnadsbärverk utsatta för frysning och luftburna avisningsmedel
XF3	Nära vattenmättad, utan avisningsmedel	Horisontella betongytor utsatta för regn och frysning
XF4	Nära vattenmättad, med avisningsmedel eller havsvatten	Väg- och brofarbanor utsatta för avisningsmedel Betongytor utsatta för direkt stänk innehållande avisningsmedel och frysning Skvalpzon på bärverk i havet utsatta för frysning

Klass	Beskrivning av miljö	Exempel, informativa, där exponeringsklassen kan förekomma
6 Kemiska angrepp		
XA1	Något aggressiv kemisk miljö enligt SS-EN 206-1, tabell 2	Naturliga jordar och grundvatten
XA2	Måttligt aggressiv kemisk miljö enligt SS-EN 206-1, tabell 2	Naturliga jordar och grundvatten
XA3	Mycket aggressiv kemisk miljö enligt SS-EN 206-1, tabell 2	Naturliga jordar och grundvatten

B - Template for determination of the concrete cover

Tabell 4.3N – Rekommenderad klassindelning av bärverk

Bärverksklass							
Kriterium	Exponeringsklass enligt tabell 4.1						
	XO	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Avsedd livslängd 100 år	öka 2 klasser	öka 2 klasser	öka 2 klasser	öka 2 klasser	öka 2 klasser	öka 2 klasser	öka 2 klasser
Hållfasthetsklass [1) 2)]	≥ C30/37 minska 1 klass	≥ C30/37 minska 1 klass	≥ C35/45 minska 1 klass	≥ C40/50 minska 1 klass	≥ C40/50 minska 1 klass	≥ C40/50 minska 1 klass	≥ C45/55 minska 1 klass
Platt bärverksdel (om armeringens läge inte påverkas under utförandet)	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass
Särskild kvalitets-styrning föreskriven för betongframställningen	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass	minska 1 klass

ANM. till tabell 4.3N

1) Hållfasthetsklass och vattencementtal anses vara relaterade värden. Speciell sammansättning (cementtyp, vattencementtal, finkornig filler) med avsikt att ge låg permeabilitet får beaktas.

2) Gränsen kan minskas med en hållfasthetsklass om luftinblandning överstigande 4% utförs.

Tabell 4.4N – Minsta täckande betongskikt, $c_{min,dur}$, med hänsyn till beständighet för armering enligt EN 10080

Bärverks-klass	Exponeringsklass enligt tabell 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

As default the construction belongs to structure class S4.

C - Design of anchor reinforcement

Loading:		**ULS**	**Fat.max**	**Fat.min**	
	Nd=	3510	3510	3510	kN
	Hd=	797	252	10	kN
	Md=	63825	17869	417	kNm
	Mvd=	1642	0	0	kNm
At anchor ring:	Mda=	65658	18449	440	kNm

Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:			
	Concrete C30/37		
	Compression strength	fck=	30 Mpa
		fcd=αcc*fck/γC=	20 MPa
		fcd.fat=fcd*(1-fck/250)=	17,6 MPa
Reinforcement:	Ribbed bar, B500B		
	Bar diameter	Φ=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	435 MPa
	Maximum stress range	$\Delta\sigma$Rsd=	70 Mpa
Geometry:	Diameter ring	D=	4,15 m
	Ring width	s=	330 mm
	Ring height	hr=	2,3 m

Governing equations:

Stress:	σ=+-Nd/(π*D*s) + 4*Mda/(π*D^2*s)
Force around ring:	F=+-Nd + 4*Mda/D
Extreme reinf. need:	As=F+/fcd
Fatigue reinf need:	As=ΔF/$\Delta\sigma$Rsd
Local pressure:	$\sigma-$ $<$ fcd

	ULS	**Fat.max**	**Fat.min**	
σ-	-15,52	-4,95	-0,91	Mpa
$\sigma+$	13,89	3,32	-0,72	MPa
F-	-66795	-21292	-3934	kN
F+	59775	14272	-3086	kN

Local pressure:		15,52	<	20	Mpa	OK!
Extreme:	As=	137414 mm^2				
Fatigue:	As=	203883 mm^2				
Needed:	As=	**203883** mm^2				
No of reinf. cuts:	nc=	254				
No of U-bows:	nΦ=	**127** pcs				

D - Bearing capacity – Case 1

Case:	**Case 1, Moraine soil**			
Loading:	Ultimate limit state			
	Vertical force (Nd + G)	V=	13843	kN
	Horizontal force, adjusted with respect to torque	H=	1208	kN
Plate:	Effective length	leff=	9,62	m
	Effective width	beff=	4,52	m
Soil:	Friction angle, characteristic	φ=	39	deg
	Partial coefficient	$\gamma\varphi$=	1,2	
	Friction angle, design value	φd=	34,0	deg
	Cohesion, characteristic	c=	0	kPa
	Partial coefficient	γc=	1,75	
	Cohesion, design value	cd=	0	kPa
	Weight, natural condition	γ=	19	kN/m^3
	Weight, effective	γ'=	12	kN/m^3
	Foundation depth	d=	2,52	m
Bearing capacity factors:		Nq=	29,48	
		Nc=	Irrelevant	
		Nγ=	29,02	
		F(φ)=	0,3451	
Shape factors:		sq=	1,3171	
		sc=	Irrelevant	
		sγ=	0,8121	
Foundation depth factors:		dq=	1,1951	
		dc=	Irrelevant	
		dγ=	1	
Inclined loading factors:		iq=	0,8578	
		ic=	Irrelevant	
		iγ=	0,7829	
		m=	1,6803	

Bearing capacity: (Rapture 1)

qd=q*Nq*sq*dq*iq+cd*Nc*sc*dc*ic+0,5*γ*beff*Nγ*sγ*dγ*iγ

1st part (q)	=	1906	kPa
2nd part (c)	=	0	kPa
3rd part (γ)	=	500	kPa
Total bearing capacity	qd=	**2406**	kPa

Bearing capacity: (Rapture 2)

If eccentricity larger than 30% of the diameter rapture 2 is considered

e=4,79m > 0,3*15 = 4,5 m **Rapture 2 considered**

Inclined loading factors:

	ic=	Irrelevant
	$i\gamma$=	1,2514
	m=	1,6803

qd=cd*Nc*sc*dc*ic*(1.05+(tan(φ))^3) + γ'*beff*Nγ*sγ*dγ*iγ

1st part (c)	=	0	kPa
2nd part (γ)	=	1600	kPa
Total bearing capacity	qd=	**1600**	kPa

Sliding:

H < beff*leff*cd + V*tan(φ)

H=	1208	kN
beff*leff*cd + V*tan(f)=	9342	kN
	OK!	

H/V < 0,4 H/V= 0,0873 <0,4

OK!

E - Sectional forces – Case 1

Loading:	**Ultimate limit state**				
	Design load on top of foundation				
	Normal (dead load)		Nd=	3510	kN
	Horizontal force (wind)		Hd=	797	kN
	Bending moment (wind)		Md=	63825	kNm
	Twisting moment (wind)		Mvd=	1642	kNm
	Partial coefficients included	Permanent loads	γfg=	1	
		Wind loads	γfw=	1,35	
	Loading acting above ground level		t=	0,6	m
Geometry:	Plate diameter		D=	15	m
	Pedestal diameter		D2=	6	m
	Embedded ring diameter		Dr=	4,15	m
	Total height		h1=	2,52	m
	Height at outer perimeter		h2=	1,25	m
	Height of pedestal		dh=	0,27	m
Weight:	Reinforced concrete		=	25	kN/m^3
	Compacted fill (above plate)		=	18	kN/m^3
	Plate		=	8085	kN
	Pedestal		=	191	kN
	Soil		=	2057	kN
	Foundation and soil		G=	**10333**	kN
Soil pressure:	Eccentricity, e=(Md + Hd*(h1+t))/(Nd+G)		e=	4,79	m
	Effective area, Aeff				
	Aeff=2*(D^2/4*arccos(2e/D)-e*(D^2/4-e^2)^0.5)				
			Aeff=	43,48	m^2
	Major axes, be=D-2e		be=	5,42	m
	le=D*(1-(1-be/D)^2)^2)^0.5		le=	11,54	m
	Effective length, leff=(Aeff*le/be)^0.5		leff=	9,62	m
	Effective width, beff=leff*be/le		beff=	4,52	m
	Correction for torque, H´ (if torque exists)				
	H´=2*Mvd/leff+(Hd^2+(2*Mvd/leff)^2)^0.5		H´=	1208	kN
	Soil pressure, f=(Nd+G)/Aeff		f=	**318,34**	kPa
Dead load equally spread:	g=4*G/(π*D^2)		g=	**58,47**	kPa

Sectional forces:

Moments:	Top	Mt=-0,5*g*L^2	
	Bottom	Mb=f*b*(L-b/2) + Mt	for b<=L
		Mb=f/2*L^2 + Mt	for b>L
Shear forces:	Top	Vt=-g*L	
	Bottom	Vb=f*b + Vt	for b<=L
		Vb=f*L + Vt	for b>L

Section no:	Dist.[m]	L [m]	Mt	Mb	Vt	Vb
1	2,08	5,43	-860	3693	-317	1121
2	3,43	4,07	-484	2151	-238	1057
3	4,79	2,71	-215	956	-159	705
4	6,14	1,36	-54	239	-79	352

Loading:	**Serviceability limit state (SLS)**			
	Design load on top of foundation			
	Normal (dead load)	Nd=	3510	kN
	Horizontal force (wind)	Hd=	482	kN
	Bending moment (wind)	Md=	35108	kNm
	Twisting moment (wind)	Mvd=	303	kNm
	Partial coefficients included Permanent loads	γfg=	1	
	Wind loads	γfw=	1	
	Loading acting above ground level	t=	0,6	m
Geometry:	Plate diameter	D=	15	m
	Pedestal diameter	D2=	6	m
	Embedded ring diameter	Dr=	4,15	m
	Total height	h1=	2,52	m
	Height at outer perimeter	h2=	1,25	m
	Height of pedestal	dh=	0,27	m
Weight:	Reinforced concrete	=	25	kN/m^3
	Compacted fill (above plate)	=	18	kN/m^3
	Plate	=	8085	kN
	Pedestal	=	191	kN
	Soil	=	2057	kN
	Foundation and soil	G=	**10333**	kN
Soil pressure:	Eccentricity, e=(Md + Hd*(h1+t))/(Nd+G)	e=	2,6448	m
	Effective area, Aeff			
	Aeff=2*(D^2/4*arccos(2e/D)-e*(D^2/4-e^2)^0.5)	Aeff=	99,047	m^2
	Major axes, be=D-2e	be=	9,7104	m
	le=D*(1-(1-be/D)^2)^2)^0.5	le=	14,036	m
	Effective length, leff=(Aeff*le/be)^0.5	leff=	11,965	m
	Effective width, beff=leff*be/le	beff=	8,2777	m
	Correction for torque, H´ (if torque exists)			
	H´=2*Mvd/leff+(Hd^2+(2*Mvd/leff)^2)^0.5	H´=	535,3	kN
	Soil pressure, f=(Nd+G)/Aeff	f=	**139,76**	kPa
Dead load equally spread:	g=4*G/(π*D^2)	g=	**58,47**	kPa

Sectional forces:

Moments:	Top	Mt=-0,5*g*L^2	
	Bottom	Mb=f*b*(L-b/2) + Mt	for b<=L
		Mb=f/2*L^2 + Mt	for b>L
Shear forces:	Top	Vt=-g*L	
	Bottom	Vb=f*b + Vt	for b<=L
		Vb=f*L + Vt	for b>L

Section no:	Dist.[m]	L [m]	Mt	Mb	Vt	Vb
1	2,08	5,43	-860	1196	-317	441
2	3,43	4,07	-484	673	-238	331
3	4,79	2,71	-215	299	-159	220
4	6,14	1,36	-54	75	-79	110

Loading:	**Min fatigue loading**				
	Design load on top of foundation				
	Normal (dead load)		Nd=	3510	kN
	Horizontal force (wind)		Hd=	10	kN
	Bending moment (wind)		Md=	417	kNm
	Twisting moment (wind)		Mvd=	0	kNm
	Partial coefficients included	Permanent loads	γfg=	1	
		Wind loads	γfw=	1	
	Loading acting above ground level		t=	0,6	m
Geometry:	Plate diameter		D=	15	m
	Pedestal diameter		D2=	6	m
	Embedded ring diameter		Dr=	4,15	m
	Total height		h1=	2,52	m
	Height at outer perimeter		h2=	1,25	m
	Height of pedestal		dh=	0,27	m
Weight:	Reinforced concrete		=	25	kN/m^3
	Compacted fill (above plate)		=	18	kN/m^3
	Plate		=	8085	kN
	Pedestal		=	191	kN
	Soil		=	2057	kN
	Foundation and soil		G=	**10333**	kN
Soil pressure:	Eccentricity, e=(Md + Hd*(h1+t))/(Nd+G)		e=	0,0324	m
	Effective area, Aeff				
	Aeff=2*(D^2/4*arccos(2e/D)-e*(D^2/4-e^2)^0.5)				
			Aeff=	175,74	m^2
	Major axes, be=D-2e		be=	14,935	m
	le=D*(1-(1-be/D)^2)^2)^0.5		le=	15	m
	Effective length, leff=(Aeff*le/be)^0.5		leff=	13,285	m
	Effective width, beff=leff*be/le		beff=	13,228	m
	Correction for torque, H´ (if torque exists)				
	H´=2*Mvd/leff+(Hd^2+(2*Mvd/leff)^2)^0.5		H´=	10	kN
	Soil pressure, f=(Nd+G)/Aeff		f=	**78,77**	kPa
Dead load equally spread:	g=4*G/(π*D^2)		g=	**58,47**	kPa

Sectional forces:

Moments: Top $Mt=-0,5*g*L^2$

Bottom $Mb=f*b*(L-b/2) + Mt$ for $b<=L$
$Mb=f/2*L^2 + Mt$ for $b>L$

Shear forces: Top $Vt=-g*L$

Bottom $Vb=f*b + Vt$ for $b<=L$
$Vb=f*L + Vt$ for $b>L$

Section no:	Dist.[m]	L [m]	Mt	Mb	Vt	Vb
1	2,08	5,43	-860	299	-317	110
2	3,43	4,07	-484	168	-238	83
3	4,79	2,71	-215	75	-159	55
4	6,14	1,36	-54	19	-79	28

Loading:	**Max fatigue loading**			
	Design load on top of foundation			
	Normal (dead load)	Nd=	3510	kN
	Horizontal force (wind)	Hd=	252	kN
	Bending moment (wind)	Md=	17869	kNm
	Twisting moment (wind)	Mvd=	0	kNm
	Partial coefficients included — Permanent loads	γfg=	1	
	Wind loads	γfw=	1	
	Loading acting above ground level	t=	0,6	m
Geometry:	Plate diameter	D=	15	m
	Pedestal diameter	D2=	6	m
	Embedded ring diameter	Dr=	4,15	m
	Total height	h1=	2,52	m
	Height at outer perimeter	h2=	1,25	m
	Height of pedestal	dh=	0,27	m
Weight:	Reinforced concrete	=	25	kN/m^3
	Compacted fill (above plate)	=	18	kN/m^3
	Plate	=	8085	kN
	Pedestal	=	191	kN
	Soil	=	2057	kN
	Foundation and soil	G=	**10333**	kN
Soil pressure:	Eccentricity, e=(Md + Hd*(h1+t))/(Nd+G)	e=	1,3476	m
	Effective area, Aeff			
	Aeff=2*(D^2/4*arccos(2e/D)-e*(D^2/4-e^2)^0.5)			
		Aeff=	136,5	m^2
	Major axes, be=D-2e	be=	12,305	m
	le=D*(1-(1-be/D)^2)^2)^0.5	le=	14,756	m
	Effective length, leff=(Aeff*le/be)^0.5	leff=	12,794	m
	Effective width, beff=leff*be/le	beff=	10,669	m
	Correction for torque, H´ (if torque exists)			
	H´=2*Mvd/leff+(Hd^2+(2*Mvd/leff)^2)^0.5	H´=	252	kN
	Soil pressure, f=(Nd+G)/Aeff	f=	**101,41**	kPa
Dead load equally spread:	g=4*G/(π*D^2)	g=	**58,47**	kPa

Sectional forces:

Moments: Top Mt=-0,5*g*L^2

Bottom Mb=f*b*(L-b/2) + Mt for b<=L
Mb=f/2*L^2 + Mt for b>L

Shear forces: Top Vt=-g*L

Bottom Vb=f*b + Vt for b<=L
Vb=f*L + Vt for b>L

Section no:	Dist.[m]	L [m]	Mt	Mb	Vt	Vb
1	2,08	5,43	-860	632	-317	233
2	3,43	4,07	-484	355	-238	175
3	4,79	2,71	-215	158	-159	116
4	6,14	1,36	-54	39	-79	58

F - Design for bending moment – Case 1

Loading:	**Ultimate limit state**		
Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=acc*fck/gC=	20 MPa
		fctd=acc*fctk/gC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
Reinforcement:			
	Ribbed bar, B500B		
	Bar diameter	Top: Φt=	25 mm
		Bottom: Φb=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	435 MPa
	Young's modulus	Es=	200 Gpa
	Yield strain	esy=	0,218 %

Governing equations:

For concrete with fck < 50MPa λ=0,8

η=1.0

M=fcd*0,8*x*b*(d-0,4*x) gives the compressed distance, x

es=ecu*(d-x)/x

es>esy to obtain a non-brittle break

As*fyd=fcd*0,8*x*b gives the reinforcement needed

Top:

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Mt [kNm/m]	-861	-485	-216	-54
x [mm]	22,2	14,7	7,6	2,3
es [%]	38,2	49,1	80,8	223,4
As [mm^2/m]	**815**	**540**	**281**	**85**

Bottom:

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Mb [kNm/m]	4082	2532	1127	282
x [mm]	106,5	77,5	40,1	12,0
es [%]	7,7	9,0	15,1	42,4
As [mm^2/m]	**3917**	**2852**	**1477**	**443**

G - Design for shear force – Case 1

Loading:	**Ultimate limit state**		
Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=acc*fck/gC=	20 MPa
		fctd=acc*fctk/gC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm

Calculation without shear reinforcement:

Governing equations:

Vrd.c=Crd.c*k*(100*ρl*fck)^1/3*bw*d

Vrd.c>0,035*(k^3*fck)^1/2*bw*d=Vrd.c.min

k=1+(200/d)^1/2 < 2,0

ρl=Asl/(bw*d) < 0,02

Crd.c=0,18/γC Crd.c= 0,12

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asl [mm^/m]	4732	3392	1758	528
k [-]	1,28642	1,31067	1,33608	1,3689
ρl [-]	1,9E-03	1,6E-03	9,9E-04	3,6E-04
Vrd.c [kN/m]	677	554	408	247
Vrd.c.min [kN/m]	682	596	524	451
Ved max [kN/m]	1121	1057	705	352
Control	Not OK!	Not OK!	Not OK!	OK!

Shear stirrups needed!

Calculation with shear stirrups:

Governing equations:

Vrd.s=Asw/s^2*z*fywd*cot(Φ)

Vrd.max=αcw*bw*z*v1*fcd/(cot(Φ)+tan(Φ))

v=0,6*(1-fck/250)
z=0.9*d

Reinforcement:	Quality B500B		
	Shear stirrups diameter	Φ=	25 mm
	Yield stress	fywk=	500 Mpa
		fywd=0,8*fywk=	400 MPa
	Reinforcement area, one cut	Asw=	491 mm^2
		cot(Φ)=	1
		tan(Φ)=	1
		αcw=	1
		v=	0,528

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
z [m]	2,1942	1,86498	1,59363	1,3224
Vrd.s [kN/m]	1121	1057	705	352
s [mm]	**620**	**589**	**666**	**859**
Vrd.max [kN/m]	11585	9847	8414	6982

H - Design for crack width – Case 1

Loading: **Serviceability limit state (SLS)**

Safety factors:

Long term effect factor	αc=	1,00
Concrete factor	γC=	1,00
Reinforcement factor	γS=	1,00

Concrete:

Concrete C30/37		
Compression strength	fck=	30 Mpa
Tension strength	fctm=	2,9 MPa
	fcd=αcc*fck/γC=	30 MPa
	fcteff=αc*fctm/γC=	2,9 MPa
Maximum compression strain	ecu=	0,35 %
Covering concrete	c=	50 mm
Young's modulus	Ecm=	33 Gpa
Creep factor, for short time loading	φ=	0
	Eceff=	33 GPa

Reinforcement:

Ribbed bar, B500B			
Bar diameter	Top:	Φt=	25 mm
	Bottom:	Φb=	32 mm
Yield stress		fyk=	500 Mpa
		fyd=fyk/γC=	500 MPa
Young's modulus		Es=	200 Gpa
Yield strain		esy=	0,25 %

Section forces:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Mb [kNm/m]	1196	673	299	75

Governing equations:

wk=(esm-ecm)*srmax

esm-ecm = [σs-kt*fcteff*(1+α*ρpeff)/ρpeff]/Es >= 0,6*σs/Es

srmax = k3c+k1*k2*k4*Φ/ρpeff

σs=M/(As*z)

z=d-x/3

x=d*ρpeff*α*[(1+2/(ρpeff*α))^0.5 - 1]

α=Es/Eceff	α=	6,0606
ρpeff=As/(2.5*c*b)		
kt, duration of loading	kt=	0,6
k1, adhesion	k1=	0,8
k2, stress distribution	k2=	0,5
k3=7Φ/c	k3_top=	3,5
	k3_bottom=	4,48
k4 -	k4=	0,425

Top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_t [mm^2/m]	815	540	281	85
ρpeff [-]	6,52E-03	4,32E-03	2,25E-03	6,80E-04
x [mm]	596	423	269	127
z [m]	2,24	1,93	1,68	1,43
Mt [kNm/m]	860	484	215	54
σs [Mpa]	471,20	464,12	455,17	445,26
$\Delta\varepsilon$ [-]	0,00097	0,00025	-0,00165	-0,01062
$\Delta\varepsilon$max [-]	0,00141	0,00139	0,00137	0,001336
$\Delta\varepsilon$ [-]	0,00141	0,00139	0,00137	0,001336
srmax [mm]	826,84	1158,80	2065,57	6425,00
wk [mm]	1,17	1,61	2,82	8,58
Control	**Not OK!**	**Not OK!**	**Not OK!**	**Not OK!**

Bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_b [mm^2/m]	3917	2852	1477	443
ρpeff [-]	3,13E-02	2,28E-02	1,18E-02	3,54E-03
x [mm]	1109	840	555	275
z [m]	2,07	1,79	1,59	1,38
Mb [kNm/m]	1196	673	299	75
σs [Mpa]	147,63	131,67	127,67	122,88
$\Delta\varepsilon$ [-]	0,00041	0,00022	-0,00015	-0,00189
$\Delta\varepsilon$max [-]	0,00044	0,0004	0,00038	0,000369
$\Delta\varepsilon$ [-]	0,00044	0,0004	0,00038	0,000369
srmax [mm]	397,60	462,43	684,39	1758,99
wk [mm]	0,18	0,18	0,26	0,65
Control	OK!	OK!	OK!	**Not OK!**

Corrected reinforcement area

Loading: **Serviceability limit state (SLS)**

Safety factors:	Long term effect factor	αc=	1,00
	Concrete factor	γC=	1,00
	Reinforcement factor	γS=	1,00
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctm=	2,9 MPa
		fcd=αcc*fck/γC=	30 MPa
		fcteff=αc*fctm/γC=	2,9 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	φ=	0
		Eceff=	33 GPa
Reinforcement:	Ribbed bar, B500B		
	Bar diameter Top:	Φt=	25 mm
	Bottom:	Φb=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	500 MPa
	Young's modulus	Es=	200 Gpa
	Yield strain	esy=	0,25 %

Section forces:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Mb [kNm/m]	1196	673	299	75

Governing equations:

wk=(esm-ecm)*srmax

esm-ecm = [σs-kt*fcteff*(1+α*ρpeff)/ρpeff]/Es >= 0,6*σs/Es

srmax = k3c+k1*k2*k4*Φ/ρpeff

σs=M/(As*z)

z=d-x/3

x=d*ρpeff*α*[(1+2/(ρpeff*α))^0.5 - 1]

α=Es/Eceff	α=	6,0606
ρpeff=As/(2.5*c*b)		
kt, duration of loading	kt=	0,6
k1, adhesion	k1=	0,8
k2, stress distribution	k2=	0,5
k3=7Φ/c	k3_top=	3,5
	k3_bottom=	4,48
k4 -	k4=	0,425

Top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_t [mm^2/m]	**1540**	**1199**	**818**	**422**
ρpeff [-]	1,23E-02	9,59E-03	6,54E-03	3,38E-03
x [mm]	778	596	433	269
z [m]	2,18	1,87	1,63	1,38
Mt [kNm/m]	860	484	215	54
σs [Mpa]	256,30	215,47	161,62	92,74
$\Delta\varepsilon$ [-]	0,00052	0,00012	-0,00057	-0,00217
$\Delta\varepsilon$max [-]	0,00077	0,00065	0,00048	0,000278
$\Delta\varepsilon$ [-]	0,00077	0,00065	0,00048	0,000278
srmax [mm]	519,97	618,08	824,45	1433,89
wk [mm]	0,40	0,40	0,40	0,40
Control	OK!	OK!	OK!	OK!

Bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_b [mm^2/m]	3917	2852	1477	**578**
ρpeff [-]	3,13E-02	2,28E-02	1,18E-02	4,62E-03
x [mm]	1109	840	555	309
z [m]	2,07	1,79	1,59	1,37
Mb [kNm/m]	1196	673	299	75
σs [Mpa]	147,63	131,67	127,67	94,97
$\Delta\varepsilon$ [-]	0,00041	0,00022	-0,00015	-0,00146
$\Delta\varepsilon$max [-]	0,00044	0,0004	0,00038	0,000285
$\Delta\varepsilon$ [-]	0,00044	0,0004	0,00038	0,000285
srmax [mm]	397,60	462,43	684,39	1400,47
wk [mm]	0,18	0,18	0,26	0,40
Control	OK!	OK!	OK!	OK!

Values in bold is corrected reinforcement area

J - Design for fatigue – Case 1

Loading:	**Fatigue**		
Safety factors:	Long term effect factor	$\alpha c=$	1
	Concrete factor	$\gamma C=$	1,5
	Reinforcement factor	$\gamma S=$	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=αcc*fck/γC=	20 MPa
		fcd.fat=fcd*(1-fck/250)=	17,6 MPa
		fctd=αc*fctk/γC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	$\varphi=$	0
		Eceff=	33 GPa
Reinforcement:	Maximum stress range	$\Delta\sigma$Rsd=	70 Mpa
	Young's modulus	Es=	200 Gpa

Section forces:

Max fatigue:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Vt [kN/m]	-317	-238	-159	-79
Mb [kNm/m]	632	355	158	39
Vb [kN/m]	233	175	116	58

Min fatigue:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Vt [kN/m]	-317	-238	-159	-79
Mb [kNm/m]	299	168	75	19
Vb [kN/m]	110	83	55	28

Governing equations:

Fatigue control in concrete, σcmax/fcd.fat < 0.5+0.45*σcmin/fcd.fat<0.9

Fatigue stress in reinforcement, σsmax - σsmin < $\Delta\sigma$Rsd

Fatigue control in concrete:

σc=2*M/(x*z*b)
x=d*ρ*α*[(1+2/(ρ*α))^0.5-1)
z=d-x/3
ρ=As/(b*d)
α=Es/Eceff α= 6,0606

Fatigue top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Ast [mm^2/m]	1540	1199	818	422
ρ [-]	6,3E-04	5,8E-04	4,6E-04	2,9E-04
x [mm]	204	166	128	84
z [m]	2,37	2,02	1,73	1,44
Mtmax [kNm/m]	-860	-484	-215	-54
Mtmin [kNm/m]	-860	-484	-215	-54
σcmax [Mpa]	3,55	2,88	1,95	0,89
σcmin [Mpa]	3,55	2,88	1,95	0,89

Control:

σcmax/fcd.fat	0,20	0,16	0,11	0,05
0.5+0.45*σcmin/fcd.fat	0,59	0,57	0,55	0,52
< 0.9	OK!	OK!	OK!	OK!

Fatigue bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asb [mm^2/m]	3917	2852	1477	578
ρ [-]	1,6E-03	1,4E-03	8,3E-04	3,9E-04
x [mm]	317	251	169	98
z [m]	2,33	1,99	1,71	1,44
Mbmax [kNm/m]	632	355	158	39
Mbmin [kNm/m]	299	168	75	19
σcmax [Mpa]	1,71	1,42	1,09	0,55
σcmin [Mpa]	0,81	0,67	0,52	0,27

Control:

σcmax/fcd.fat	0,10	0,08	0,06	0,03
0.5+0.45*σcmin/fcd.fat	0,52	0,52	0,51	0,51
< 0.9	OK!	OK!	OK!	OK!

Fatigue stress in reinforcement:

σs=M/(As*z)
x=d*ρ*α*[(1+2/(ρ*α))^0.5-1)
z=d-x/3
ρ=As/(b*d)
α=Es/Eceff

α= 6,0606

Fatigue top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Ast [mm^2/m]	1540	1199	818	422
ρ [-]	6,3E-04	5,8E-04	4,6E-04	2,9E-04
x [mm]	204	166	128	84
z [m]	2,37	2,02	1,73	1,44
Mtmax [kNm/m]	-860	-484	-215	-54
Mtmin [kNm/m]	-860	-484	-215	-54
σsmax [Mpa]	235,64	200,16	152,09	88,79
σsmin [Mpa]	235,64	200,16	152,09	88,79

Control:

$\Delta\sigma$ [Mpa]	0,00	0,00	0,00	0,00
$\Delta\sigma$Rsd [Mpa]	70,00	70,00	70,00	70,00
Control	OK!	OK!	OK!	OK!

Fatigue bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asb [mm^2/m]	3917	2852	1477	578
ρ [-]	1,6E-03	1,4E-03	8,3E-04	3,9E-04
x [mm]	317	251	169	98
z [m]	2,33	1,99	1,71	1,44
Mbmax [kNm/m]	632	355	158	39
Mbmin [kNm/m]	299	168	75	19
σsmax [Mpa]	69,17	62,60	62,42	46,97
σsmin [Mpa]	32,73	29,63	29,63	22,88

Control:

$\Delta\sigma$ [Mpa]	36,45	32,98	32,79	24,09
$\Delta\sigma$Rsd [Mpa]	70,00	70,00	70,00	70,00
Control	OK!	OK!	OK!	OK!

K – Results from pile group program – Case 2

```
Case 2 (Clay with piles reaching the bedrock)

Geometry:
Diameter 15 m, Pedestal diameter 6 m
h1=2.52 m, h2=1.25 m, dh=0.27 m,
G=10333kN

Piling:
28 inclined piles at the perimeter
6 vertical piles in the centre
34 piles total
Prefabricated concrete SP2 piles, capacity 1200kN, a=0.27 m, L=20 m

Load combinations:
LC1 ULS
LC2 SLS
LC3 Fatigue Min
LC4 Fatigue Max
```

Indata

Plattjocklek= 0.000 m   Tunghet= 0.000 (kN/m3)

Pålplattans koordinater

nr	y (m)	z (m)	nr	y (m)	z (m)
1	7.450	-0.840	16	-7.080	2.480
2	7.080	-2.480	17	-6.350	3.990
3	6.350	-3.990	18	-5.300	5.300
4	5.300	-5.300	19	-3.990	6.350
5	3.990	-6.350	20	-2.480	7.080
6	2.480	-7.080	21	-0.840	7.450
7	0.840	-7.450	22	0.840	7.450
8	-0.840	-7.450	23	2.480	7.080
9	-2.480	-7.080	24	3.990	6.350
10	-3.990	-6.350	25	5.300	5.300
11	-5.300	-5.300	26	6.350	3.990
12	-6.350	-3.990	27	7.080	2.480
13	-7.080	-2.480	28	7.450	0.840
14	-7.450	-0.840	29	0.000	0.000
15	-7.450	0.840	30	0.000	0.000

Pålgeometri (koordinater för pålhuvud och konstanter)

Nr	x(m)	y(m)	z(m)	alfa(gr)	omega(gr)	kod	ltot(m)	löm(m)
1	0.00	6.46	-0.73	4.47	0.00	4	20.00	0.00
2	0.00	6.14	-2.15	4.47	340.71	4	20.00	0.00
3	0.00	5.50	-3.46	4.47	327.86	4	20.00	0.00
4	0.00	4.60	-4.60	4.47	315.00	4	20.00	0.00
5	0.00	3.46	-5.50	4.47	302.14	4	20.00	0.00
6	0.00	2.15	-6.14	4.47	289.29	4	20.00	0.00
7	0.00	0.73	-6.46	4.47	276.43	4	20.00	0.00
8	0.00	-0.73	-6.46	4.47	263.57	4	20.00	0.00
9	0.00	-2.15	-6.14	4.47	250.71	4	20.00	0.00
10	0.00	-3.46	-5.50	4.47	237.86	4	20.00	0.00
11	0.00	-4.60	-4.60	4.47	225.00	4	20.00	0.00
12	0.00	-5.50	-3.46	4.47	212.14	4	20.00	0.00
13	0.00	-6.14	-2.15	4.47	199.29	4	20.00	0.00
14	0.00	-6.46	-0.73	4.47	186.43	4	20.00	0.00
15	0.00	-6.46	0.73	4.47	173.57	4	20.00	0.00
16	0.00	-6.14	2.15	4.47	160.71	4	20.00	0.00
17	0.00	-5.50	3.46	4.47	147.86	4	20.00	0.00
18	0.00	-4.60	4.60	4.47	135.00	4	20.00	0.00
19	0.00	-3.46	5.50	4.47	122.14	4	20.00	0.00
20	0.00	-2.15	6.14	4.47	109.29	4	20.00	0.00
21	0.00	-0.73	6.46	4.47	96.43	4	20.00	0.00
22	0.00	0.73	6.46	4.47	83.57	4	20.00	0.00
23	0.00	2.15	6.14	4.47	70.71	4	20.00	0.00
24	0.00	3.46	5.50	4.47	57.86	4	20.00	0.00
25	0.00	4.60	4.60	4.47	45.00	4	20.00	0.00
26	0.00	5.50	3.46	4.47	32.14	4	20.00	0.00
27	0.00	6.14	2.15	4.47	19.29	4	20.00	0.00
28	0.00	6.46	0.73	4.47	6.43	4	20.00	0.00
29	0.00	3.00	0.00	0.00	0.00	4	20.00	0.00
30	0.00	1.50	-2.60	0.00	0.00	4	20.00	0.00
31	0.00	-1.50	-2.60	0.00	0.00	4	20.00	0.00
32	0.00	-3.00	0.00	0.00	0.00	4	20.00	0.00
33	0.00	-1.50	2.60	0.00	0.00	4	20.00	0.00
34	0.00	1.50	2.60	0.00	0.00	4	20.00	0.00

Nr	area(dm2)	iy(dm4)	iz(dm4)	kv(dm4)	k*dp(dm)	tol(m)	em(GPa)	gm(GPa)	bm(MPa)
1	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10

Nr	area(dm2)	iy(dm4)	iz(dm4)	kv(dm4)	k*dp(dm)	tol(m)	em(GPa)	gm(GPa)	bm(MPa)
2	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
3	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
4	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
5	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
6	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
7	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
8	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
9	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
10	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
11	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
12	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
13	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
14	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
15	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
16	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
17	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
18	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
19	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
20	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
21	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
22	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
23	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
24	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
25	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
26	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
27	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
28	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
29	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
30	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
31	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
32	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
33	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10
34	7.29	4.43	4.43	7.47	2.70	20.00	33.000	13.750	8.10

Karakteristiska/dimensionerande laster angriper i origo.

Nr	lk	gamf	px (kN)	py (kN)	pz (kN)	mx (kNm)	my (kNm)	mz (kNm)
1	1	1.00	13843.00	797.00	0.00	1642.00	0.00	-63825.00
2	2	1.00	13843.00	482.00	0.00	303.00	0.00	-35108.00
3	3	1.00	13843.00	10.00	0.00	0.00	0.00	-417.00
4	4	1.00	13843.00	252.00	0.00	0.00	0.00	-17869.00

Max/Min pålnormalkrafter

lkb	påle	max N (kN)	påle	min N (kN)
1	1	1077.61	15	-260.71
2	1	775.98	15	39.90
3	28	412.22	15	403.45
4	28	595.14	15	220.53

Max/Min pålsnittkrafter

Snitt	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)	påle	lkb
max Nx	1077.6	-0.7	8.0	0.1	2.7	0.2	1	1
min Nx	-260.7	-1.9	8.2	0.1	2.8	0.7	15	1
max Vy	254.2	6.3	-2.2	0.1	-0.7	-2.2	31	1
min Vy	39.9	-3.1	1.2	0.0	0.4	1.1	15	2
max Vz	332.5	-1.5	11.3	0.1	3.9	0.5	8	1
min Vz	99.6	3.0	-4.1	0.1	-1.4	-1.0	32	1
max Mx	483.1	-1.5	11.3	0.1	3.8	0.5	7	1
min Mx	386.7	-2.0	-1.8	0.0	-0.6	0.7	21	4
max My	332.5	-1.5	11.3	0.1	3.9	0.5	8	1
min My	99.6	3.0	-4.1	0.1	-1.4	-1.0	32	1
max Mz	39.9	-3.1	1.2	0.0	0.4	1.1	15	2
min Mz	254.2	6.3	-2.2	0.1	-0.7	-2.2	31	1

Pålplattans rotation/förskjutning i origo.

lkb	dx(mm)	dy(mm)	dz(mm)	rx(grad)	ry(grad)	rz(grad)
1	3.40	0.45	-0.04	0.011	0.000	-0.049
2	3.40	0.44	-0.03	0.002	0.000	-0.027
3	3.40	0.02	-0.02	0.000	0.000	0.000
4	3.40	0.25	-0.02	0.000	0.000	-0.014

Pålförskjutningar lastkomb. 1

Påle	dx(mm)	dy(mm)	dz(mm)	rx(grad)	ry(grad)	rz(grad)
1	8.96	-0.10	1.20	0.011	-0.001	-0.049
2	8.67	-0.23	1.36	0.012	0.015	-0.046
3	8.13	-0.23	1.46	0.013	0.025	-0.042
4	7.35	-0.22	1.55	0.014	0.034	-0.035
5	6.37	-0.22	1.62	0.014	0.041	-0.026
6	5.24	-0.22	1.67	0.015	0.045	-0.016
7	4.02	-0.22	1.70	0.015	0.048	-0.005
8	2.76	-0.23	1.71	0.015	0.048	0.006
9	1.54	-0.23	1.69	0.015	0.045	0.016
10	0.41	-0.24	1.66	0.014	0.041	0.026
11	-0.56	-0.25	1.60	0.014	0.034	0.035
12	-1.35	-0.26	1.53	0.013	0.025	0.042
13	-1.89	-0.27	1.44	0.012	0.015	0.046
14	-2.17	-0.28	1.34	0.011	0.005	0.049
15	-2.17	-0.29	1.24	0.011	-0.006	0.049
16	-1.89	-0.30	1.14	0.010	-0.017	0.046
17	-1.35	-0.30	1.04	0.009	-0.027	0.042
18	-0.56	-0.31	0.96	0.008	-0.036	0.035
19	0.41	-0.31	0.89	0.008	-0.042	0.026
20	1.54	-0.31	0.84	0.007	-0.047	0.016
21	2.76	-0.31	0.80	0.007	-0.050	0.005
22	4.02	-0.30	0.80	0.007	-0.050	-0.006
23	5.24	-0.30	0.81	0.007	-0.047	-0.016
24	6.37	-0.29	0.85	0.008	-0.042	-0.026
25	7.34	-0.28	0.90	0.008	-0.035	-0.035
26	8.13	-0.27	0.98	0.009	-0.027	-0.042
27	8.67	-0.26	1.06	0.010	-0.017	-0.046
28	8.95	-0.25	1.16	0.011	-0.006	-0.049
29	5.97	0.45	0.54	0.011	0.000	-0.049
30	4.69	0.96	0.25	0.011	0.000	-0.049
31	2.11	0.96	-0.33	0.011	0.000	-0.049

Påle	dx(mm)	dy(mm)	dz(mm)	rx(grad)	ry(grad)	rz(grad)
32	0.83	0.45	-0.62	0.011	0.000	-0.049
33	2.12	-0.05	-0.33	0.011	0.000	-0.049
34	4.69	-0.05	0.25	0.011	0.000	-0.049

Pålförskjutningar lastkomb. 2

Påle	dx(mm)	dy(mm)	dz(mm)	rx(grad)	ry(grad)	rz(grad)
1	6.45	-0.04	0.18	0.002	0.000	-0.027
2	6.30	-0.06	0.33	0.003	0.009	-0.025
3	6.00	-0.08	0.42	0.003	0.014	-0.023
4	5.57	-0.10	0.50	0.003	0.019	-0.019
5	5.03	-0.13	0.57	0.004	0.023	-0.014
6	4.41	-0.17	0.61	0.004	0.025	-0.009
7	3.74	-0.21	0.64	0.004	0.027	-0.003
8	3.05	-0.26	0.65	0.004	0.027	0.003
9	2.37	-0.31	0.63	0.004	0.025	0.009
10	1.75	-0.35	0.60	0.004	0.023	0.014
11	1.21	-0.39	0.54	0.003	0.019	0.019
12	0.78	-0.42	0.47	0.003	0.014	0.023
13	0.49	-0.45	0.38	0.003	0.009	0.025
14	0.33	-0.46	0.28	0.002	0.003	0.027
15	0.33	-0.47	0.19	0.002	-0.003	0.027
16	0.49	-0.47	0.09	0.001	-0.009	0.025
17	0.78	-0.45	0.00	0.001	-0.014	0.023
18	1.21	-0.43	-0.09	0.000	-0.019	0.019
19	1.75	-0.40	-0.15	0.000	-0.023	0.014
20	2.37	-0.36	-0.20	0.000	-0.025	0.009
21	3.05	-0.32	-0.23	0.000	-0.027	0.003
22	3.73	-0.27	-0.24	0.000	-0.027	-0.003
23	4.41	-0.22	-0.22	0.000	-0.025	-0.009
24	5.03	-0.18	-0.18	0.000	-0.023	-0.014
25	5.57	-0.14	-0.13	0.000	-0.019	-0.019
26	6.00	-0.11	-0.05	0.001	-0.014	-0.023
27	6.30	-0.08	0.03	0.001	-0.009	-0.025
28	6.45	-0.07	0.13	0.002	-0.003	-0.027
29	4.81	0.44	0.07	0.002	0.000	-0.027
30	4.10	0.52	0.02	0.002	0.000	-0.027
31	2.70	0.52	-0.08	0.002	0.000	-0.027
32	1.99	0.44	-0.12	0.002	0.000	-0.027
33	2.70	0.36	-0.08	0.002	0.000	-0.027
34	4.11	0.36	0.02	0.002	0.000	-0.027

Pålförskjutningar lastkomb. 3

Påle	dx(mm)	dy(mm)	dz(mm)	rx(grad)	ry(grad)	rz(grad)
1	3.43	-0.25	-0.03	0.000	0.000	0.000
2	3.43	-0.24	-0.02	0.000	0.000	0.000
3	3.42	-0.24	-0.02	0.000	0.000	0.000
4	3.42	-0.24	-0.01	0.000	0.000	0.000
5	3.41	-0.24	-0.01	0.000	0.000	0.000
6	3.40	-0.24	0.00	0.000	0.000	0.000
7	3.39	-0.25	0.00	0.000	0.000	0.000
8	3.39	-0.25	0.01	0.000	0.000	0.000
9	3.38	-0.26	0.01	0.000	0.000	0.000
10	3.37	-0.26	0.01	0.000	0.000	0.000
11	3.36	-0.27	0.01	0.000	0.000	0.000
12	3.36	-0.27	0.01	0.000	0.000	0.000
13	3.36	-0.28	0.00	0.000	0.000	0.000
14	3.35	-0.28	0.00	0.000	0.000	0.000
15	3.35	-0.29	-0.01	0.000	0.000	0.000
16	3.36	-0.29	-0.01	0.000	0.000	0.000
17	3.36	-0.29	-0.02	0.000	0.000	0.000
18	3.36	-0.29	-0.02	0.000	0.000	0.000
19	3.37	-0.29	-0.03	0.000	0.000	0.000
20	3.38	-0.29	-0.03	0.000	0.000	0.000
21	3.39	-0.28	-0.04	0.000	0.000	0.000
22	3.39	-0.28	-0.04	0.000	0.000	0.000
23	3.40	-0.27	-0.04	0.000	0.000	0.000
24	3.41	-0.27	-0.05	0.000	0.000	0.000
25	3.42	-0.26	-0.04	0.000	0.000	0.000
26	3.42	-0.26	-0.04	0.000	0.000	0.000
27	3.43	-0.25	-0.04	0.000	0.000	0.000
28	3.43	-0.25	-0.03	0.000	0.000	0.000
29	3.42	0.02	-0.02	0.000	0.000	0.000
30	3.41	0.02	-0.02	0.000	0.000	0.000
31	3.39	0.02	-0.01	0.000	0.000	0.000
32	3.38	0.02	-0.01	0.000	0.000	0.000
33	3.39	0.03	-0.01	0.000	0.000	0.000
34	3.41	0.03	-0.02	0.000	0.000	0.000

Påle	dx(mm)	dy(mm)	dz(mm)	rx(grad)	ry(grad)	rz(grad)
1	4.95	-0.13	-0.05	0.000	0.000	-0.014
2	4.87	-0.13	0.04	0.000	0.005	-0.013
3	4.72	-0.14	0.09	0.000	0.007	-0.012
4	4.50	-0.16	0.14	0.001	0.010	-0.010
5	4.22	-0.18	0.18	0.001	0.012	-0.007
6	3.91	-0.20	0.21	0.001	0.013	-0.005
7	3.57	-0.23	0.22	0.001	0.014	-0.002
8	3.22	-0.26	0.23	0.001	0.014	0.002
9	2.87	-0.29	0.22	0.001	0.013	0.005
10	2.56	-0.32	0.20	0.001	0.012	0.007
11	2.28	-0.34	0.17	0.001	0.010	0.010
12	2.06	-0.37	0.13	0.000	0.007	0.012
13	1.91	-0.38	0.08	0.000	0.005	0.013
14	1.83	-0.39	0.02	0.000	0.002	0.014
15	1.83	-0.40	-0.03	0.000	-0.002	0.014
16	1.91	-0.40	-0.09	-0.001	-0.004	0.013
17	2.06	-0.39	-0.14	-0.001	-0.007	0.012
18	2.28	-0.37	-0.19	-0.001	-0.010	0.010
19	2.56	-0.35	-0.23	-0.001	-0.012	0.007
20	2.87	-0.33	-0.26	-0.001	-0.013	0.005
21	3.21	-0.30	-0.28	-0.001	-0.014	0.002
22	3.57	-0.27	-0.28	-0.001	-0.014	-0.002
23	3.91	-0.24	-0.27	-0.001	-0.013	-0.005
24	4.22	-0.21	-0.25	-0.001	-0.012	-0.007
25	4.50	-0.19	-0.22	-0.001	-0.010	-0.010
26	4.72	-0.16	-0.18	-0.001	-0.007	-0.012
27	4.87	-0.15	-0.13	-0.001	-0.004	-0.013
28	4.95	-0.14	-0.08	0.000	-0.001	-0.014
29	4.12	0.25	-0.03	0.000	0.000	-0.014
30	3.76	0.24	-0.03	0.000	0.000	-0.014
31	3.04	0.24	-0.02	0.000	0.000	-0.014
32	2.68	0.25	-0.01	0.000	0.000	-0.014
33	3.04	0.26	-0.02	0.000	0.000	-0.014
34	3.76	0.26	-0.03	0.000	0.000	-0.014

Pålsnittkrafter lastkomb 1

Påle	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
1	1077.61	-0.69	7.99	0.10	2.73	0.23
2	1042.79	-1.55	9.06	0.11	3.09	0.53
3	977.44	-1.51	9.69	0.12	3.31	0.52
4	883.52	-1.49	10.26	0.12	3.50	0.51
5	765.74	-1.47	10.72	0.13	3.66	0.50
6	630.02	-1.47	11.07	0.13	3.78	0.50
7	483.15	-1.49	11.28	0.13	3.85	0.51
8	332.50	-1.51	11.34	0.13	3.87	0.52
9	185.63	-1.55	11.24	0.13	3.84	0.53
10	49.90	-1.61	11.00	0.13	3.75	0.55
11	-67.88	-1.66	10.63	0.12	3.62	0.57
12	-161.81	-1.73	10.13	0.12	3.46	0.59
13	-227.18	-1.79	9.55	0.11	3.26	0.61
14	-260.70	-1.86	8.90	0.10	3.04	0.63
15	-260.71	-1.91	8.23	0.09	2.81	0.65
16	-227.19	-1.96	7.56	0.09	2.58	0.67
17	-161.84	-2.00	6.92	0.08	2.36	0.68
18	-67.91	-2.03	6.36	0.07	2.17	0.69
19	49.86	-2.05	5.89	0.07	2.01	0.70
20	185.58	-2.05	5.54	0.07	1.89	0.70
21	332.45	-2.03	5.34	0.06	1.82	0.69
22	483.10	-2.00	5.28	0.06	1.80	0.68
23	629.97	-1.96	5.37	0.07	1.83	0.67
24	765.70	-1.91	5.61	0.07	1.91	0.65
25	883.48	-1.86	5.99	0.07	2.04	0.63
26	977.41	-1.79	6.48	0.08	2.21	0.61
27	1042.78	-1.73	7.06	0.09	2.41	0.59
28	1076.30	-1.66	7.71	0.09	2.63	0.57
29	718.51	3.02	3.57	0.10	1.22	-1.03
30	563.64	6.34	1.66	0.10	0.57	-2.16
31	254.18	6.34	-2.18	0.10	-0.74	-2.16
32	99.58	3.02	-4.09	0.10	-1.40	-1.03
33	254.45	-0.30	-2.18	0.10	-0.74	0.10
34	563.92	-0.30	1.66	0.10	0.57	0.10

Pålsnittkrafter lastkomb 2

Påle	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
1	775.98	-0.25	1.17	0.02	0.40	0.09
2	757.32	-0.43	2.16	0.02	0.74	0.15
3	721.35	-0.52	2.77	0.03	0.95	0.18
4	669.66	-0.67	3.31	0.03	1.13	0.23
5	604.84	-0.88	3.76	0.03	1.28	0.30

Påle	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
6	530.14	-1.14	4.08	0.03	1.39	0.39
7	449.31	-1.42	4.27	0.03	1.46	0.49
8	366.40	-1.72	4.31	0.03	1.47	0.59
9	285.57	-2.03	4.20	0.03	1.43	0.69
10	210.86	-2.32	3.96	0.03	1.35	0.79
11	146.04	-2.58	3.58	0.03	1.22	0.88
12	94.34	-2.80	3.09	0.03	1.05	0.96
13	58.36	-2.97	2.52	0.02	0.86	1.01
14	39.90	-3.08	1.89	0.02	0.64	1.05
15	39.90	-3.12	1.23	0.01	0.42	1.06
16	58.34	-3.09	0.58	0.01	0.20	1.05
17	94.31	-3.00	-0.03	0.01	-0.01	1.02
18	146.00	-2.84	-0.57	0.00	-0.20	0.97
19	210.82	-2.64	-1.01	0.00	-0.35	0.90
20	285.52	-2.38	-1.34	0.00	-0.46	0.81
21	366.35	-2.10	-1.52	0.00	-0.52	0.72
22	449.26	-1.80	-1.57	0.00	-0.53	0.61
23	530.10	-1.49	-1.46	0.00	-0.50	0.51
24	604.80	-1.20	-1.21	0.00	-0.41	0.41
25	669.62	-0.94	-0.84	0.00	-0.29	0.32
26	721.32	-0.72	-0.35	0.01	-0.12	0.25
27	757.30	-0.55	0.22	0.01	0.08	0.19
28	775.76	-0.44	0.86	0.01	0.29	0.15
29	578.57	2.94	0.45	0.02	0.15	-1.00
30	493.72	3.48	0.13	0.02	0.04	-1.19
31	324.23	3.48	-0.50	0.02	-0.17	-1.19
32	239.59	2.94	-0.82	0.02	-0.28	-1.00
33	324.43	2.39	-0.50	0.02	-0.17	-0.81
34	493.92	2.39	0.13	0.02	0.04	-0.81

Pålsnittkrafter lastkomb 3

Påle	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
1	412.20	-1.64	-0.21	0.00	-0.07	0.56
2	412.01	-1.60	-0.16	0.00	-0.05	0.55
3	411.58	-1.59	-0.12	0.00	-0.04	0.54
4	410.97	-1.59	-0.08	0.00	-0.03	0.54
5	410.20	-1.60	-0.04	0.00	-0.01	0.55
6	409.31	-1.62	0.00	0.00	0.00	0.55
7	408.35	-1.65	0.03	0.00	0.01	0.56
8	407.36	-1.68	0.05	0.00	0.02	0.57
9	406.39	-1.71	0.06	0.00	0.02	0.58
10	405.50	-1.75	0.07	0.00	0.02	0.60
11	404.72	-1.79	0.06	0.00	0.02	0.61
12	404.11	-1.82	0.05	0.00	0.02	0.62
13	403.67	-1.86	0.03	0.00	0.01	0.63
14	403.45	-1.88	0.00	0.00	0.00	0.64
15	403.45	-1.91	-0.03	0.00	-0.01	0.65
16	403.66	-1.92	-0.07	0.00	-0.02	0.66
17	404.09	-1.93	-0.11	0.00	-0.04	0.66
18	404.70	-1.93	-0.15	0.00	-0.05	0.66
19	405.48	-1.92	-0.19	0.00	-0.07	0.65
20	406.37	-1.90	-0.23	0.00	-0.08	0.65
21	407.33	-1.87	-0.26	0.00	-0.09	0.64
22	408.32	-1.84	-0.28	0.00	-0.10	0.63
23	409.28	-1.81	-0.29	0.00	-0.10	0.62
24	410.18	-1.77	-0.30	0.00	-0.10	0.60
25	410.95	-1.73	-0.30	0.00	-0.10	0.59
26	411.57	-1.70	-0.28	0.00	-0.10	0.58
27	412.00	-1.66	-0.26	0.00	-0.09	0.57
28	412.22	-1.64	-0.23	0.00	-0.08	0.56
29	411.03	0.15	-0.15	0.00	-0.05	-0.05
30	410.00	0.11	-0.13	0.00	-0.04	-0.04
31	408.06	0.11	-0.07	0.00	-0.02	-0.04
32	407.14	0.15	-0.05	0.00	-0.02	-0.05
33	408.16	0.20	-0.07	0.00	-0.02	-0.07
34	410.11	0.20	-0.13	0.00	-0.04	-0.07

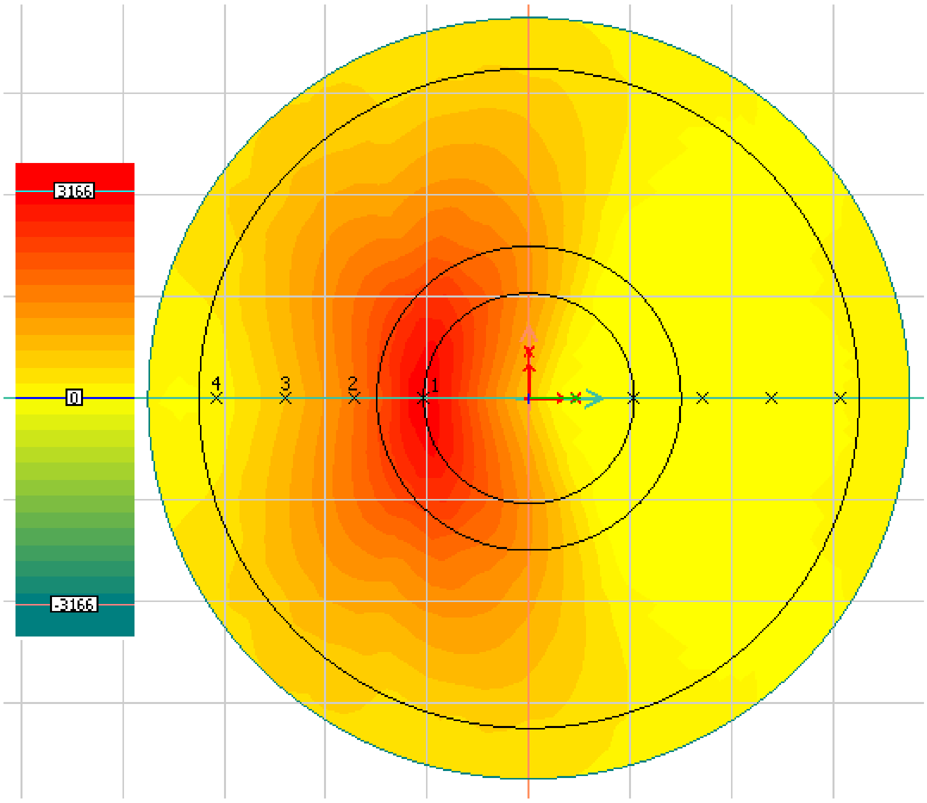
Pålsnittkrafter lastkomb 4

Påle	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
1	595.11	-0.89	-0.31	0.00	-0.11	0.31
2	585.75	-0.88	0.25	0.00	0.09	0.30
3	567.44	-0.94	0.61	0.00	0.21	0.32
4	541.13	-1.04	0.92	0.00	0.32	0.35
5	508.14	-1.17	1.18	0.01	0.40	0.40
6	470.11	-1.33	1.38	0.01	0.47	0.45
7	428.96	-1.52	1.49	0.01	0.51	0.52
8	386.75	-1.71	1.53	0.01	0.52	0.58
9	345.60	-1.91	1.47	0.01	0.50	0.65
10	307.57	-2.11	1.34	0.01	0.46	0.72

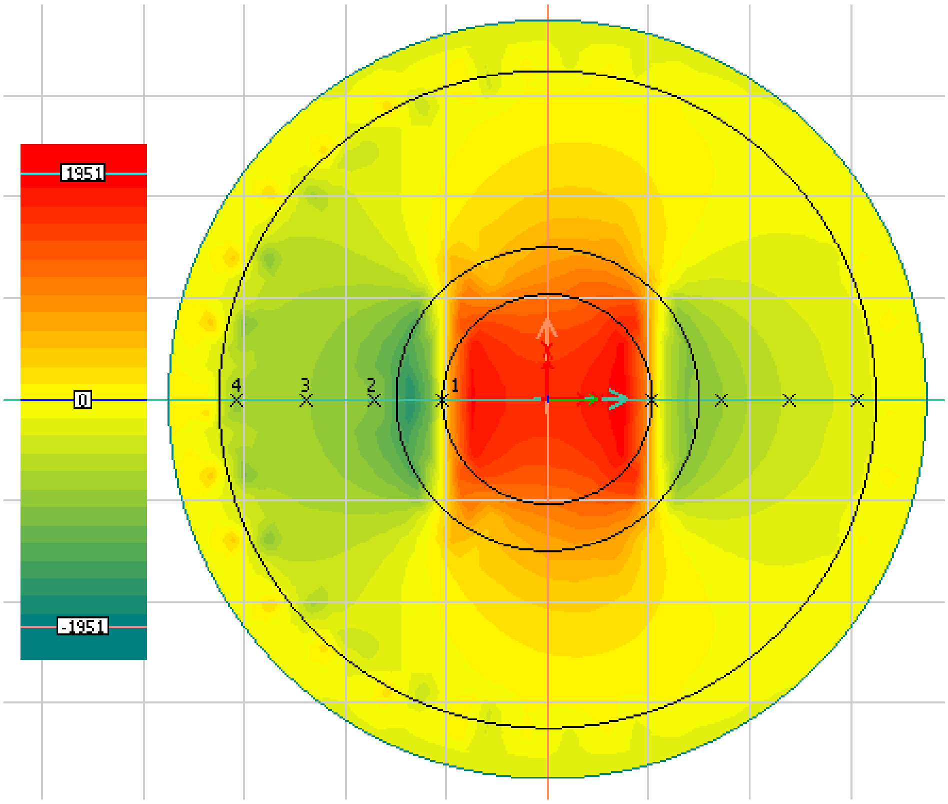
Påle	Nx (kN)	Vy (kN)	Vz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
11	274.57	-2.28	1.13	0.00	0.38	0.78
12	248.25	-2.43	0.85	0.00	0.29	0.83
13	229.94	-2.54	0.53	0.00	0.18	0.87
14	220.54	-2.62	0.16	0.00	0.06	0.89
15	220.53	-2.65	-0.21	0.00	-0.07	0.90
16	229.92	-2.64	-0.59	-0.01	-0.20	0.90
17	248.23	-2.58	-0.94	-0.01	-0.32	0.88
18	274.54	-2.48	-1.26	-0.01	-0.43	0.85
19	307.54	-2.35	-1.52	-0.01	-0.52	0.80
20	345.57	-2.19	-1.71	-0.01	-0.58	0.75
21	386.71	-2.00	-1.83	-0.01	-0.62	0.68
22	428.92	-1.80	-1.86	-0.01	-0.63	0.62
23	470.07	-1.61	-1.81	-0.01	-0.62	0.55
24	508.10	-1.41	-1.67	-0.01	-0.57	0.48
25	541.10	-1.24	-1.46	-0.01	-0.50	0.42
26	567.42	-1.09	-1.19	-0.01	-0.40	0.37
27	585.74	-0.98	-0.86	-0.01	-0.29	0.33
28	595.14	-0.90	-0.50	0.00	-0.17	0.31
29	495.24	1.69	-0.22	0.00	-0.08	-0.58
30	452.09	1.62	-0.18	0.00	-0.06	-0.55
31	365.93	1.62	-0.11	0.00	-0.04	-0.55
32	322.93	1.69	-0.07	0.00	-0.02	-0.58
33	366.08	1.75	-0.11	0.00	-0.04	-0.60
34	452.23	1.75	-0.18	0.00	-0.06	-0.60

L - Sectional forces – Case 2

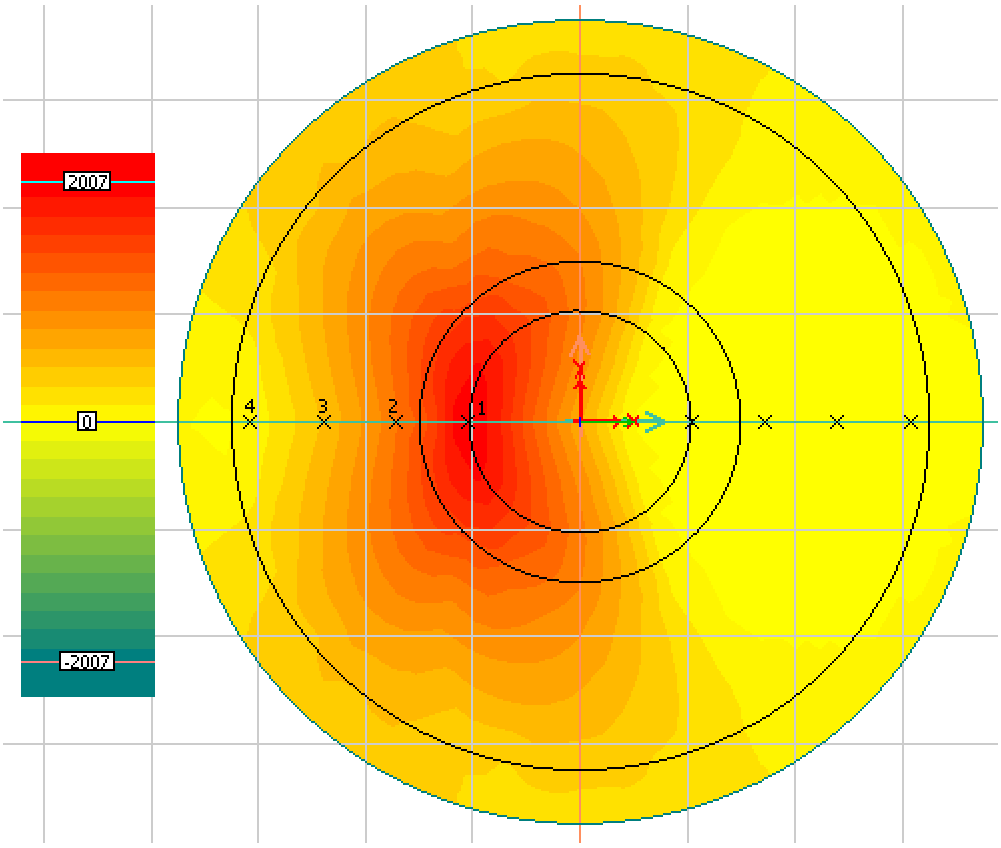
ULS - Bending moment [kNm]



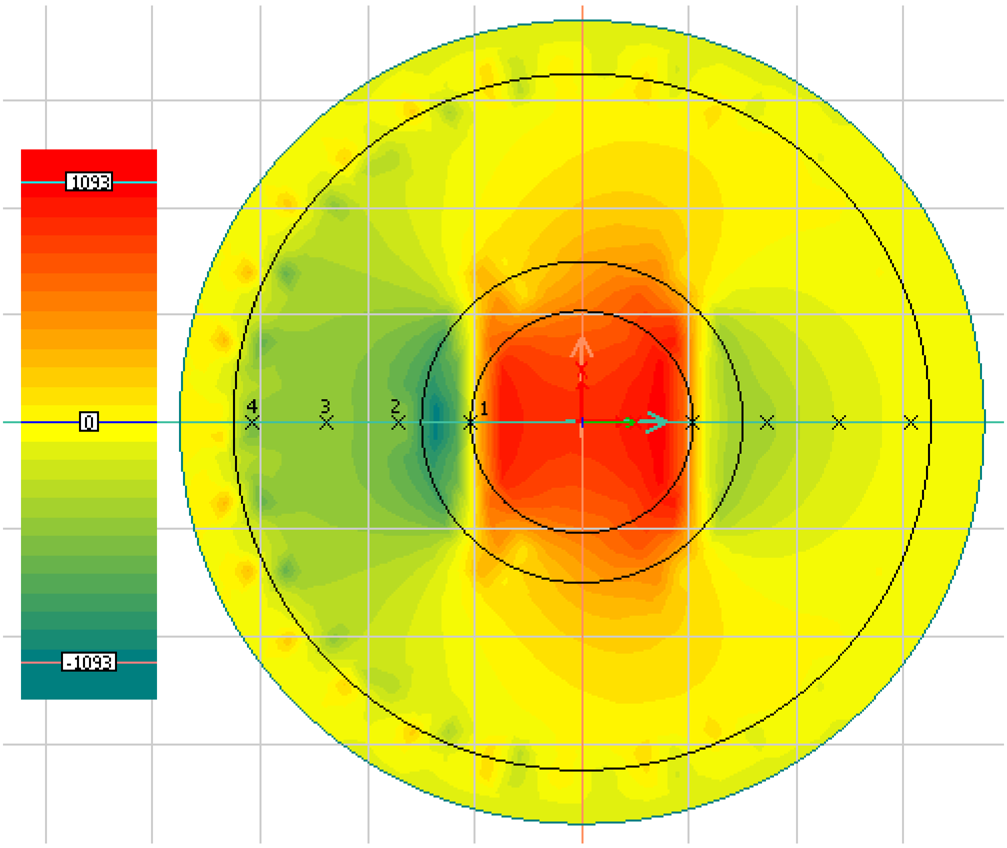
ULS – Shear force [kN]



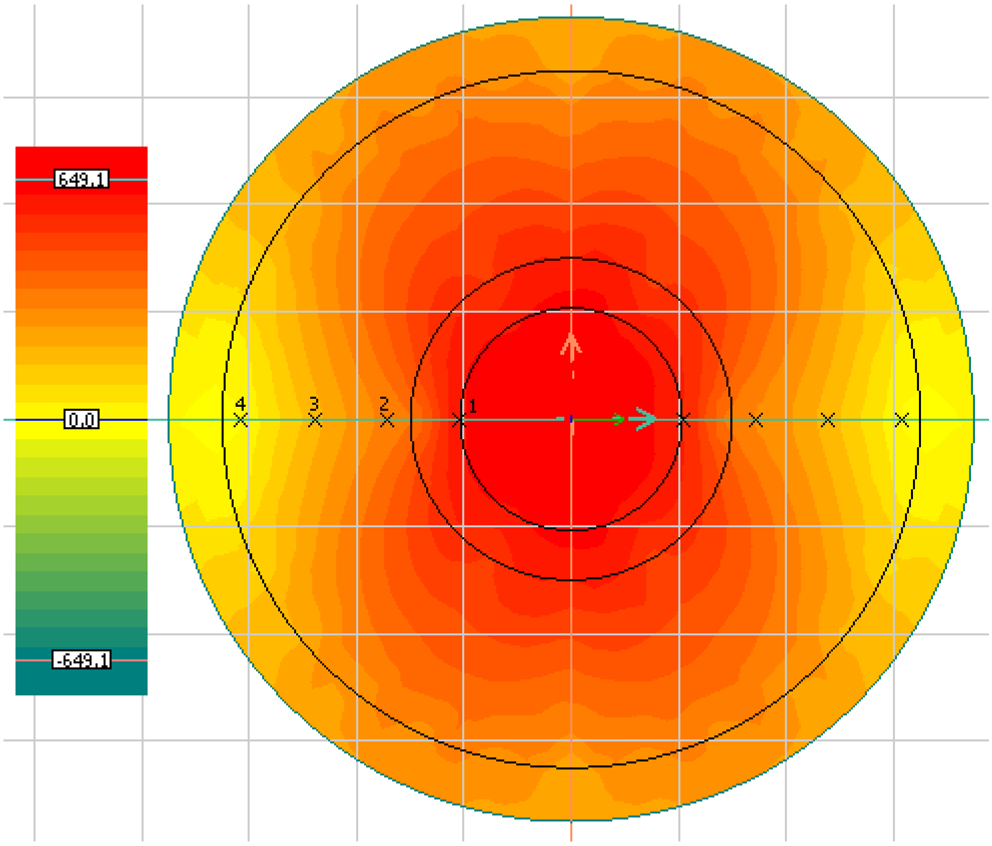
SLS - Bending moment [kNm]



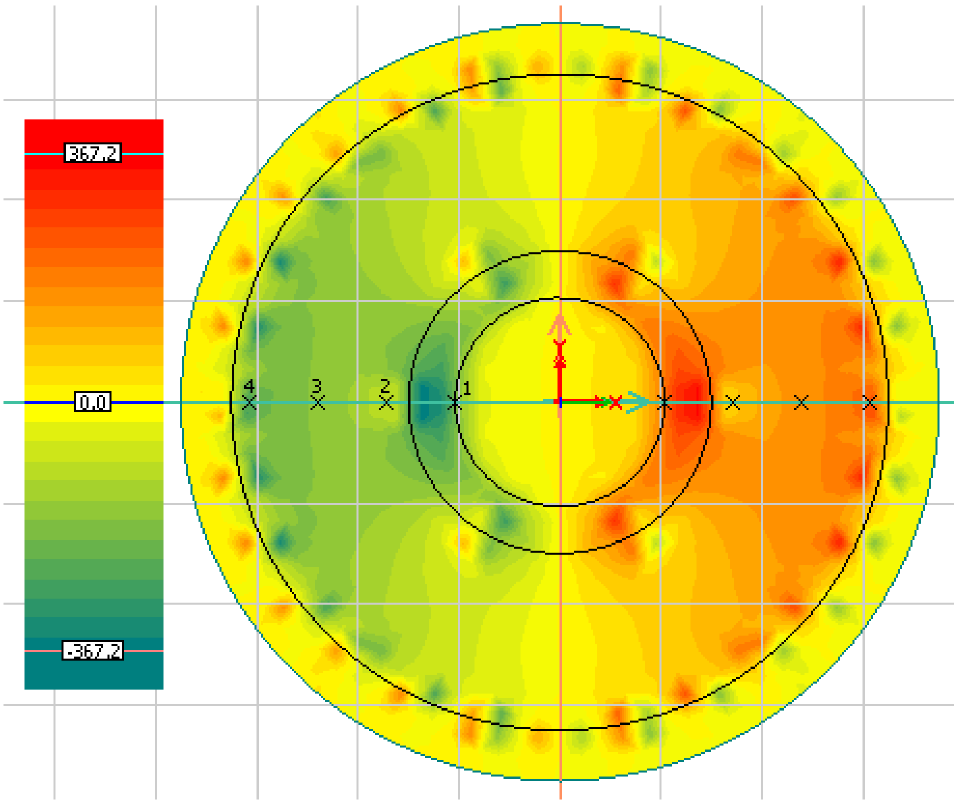
SLS – Shear force [kN]



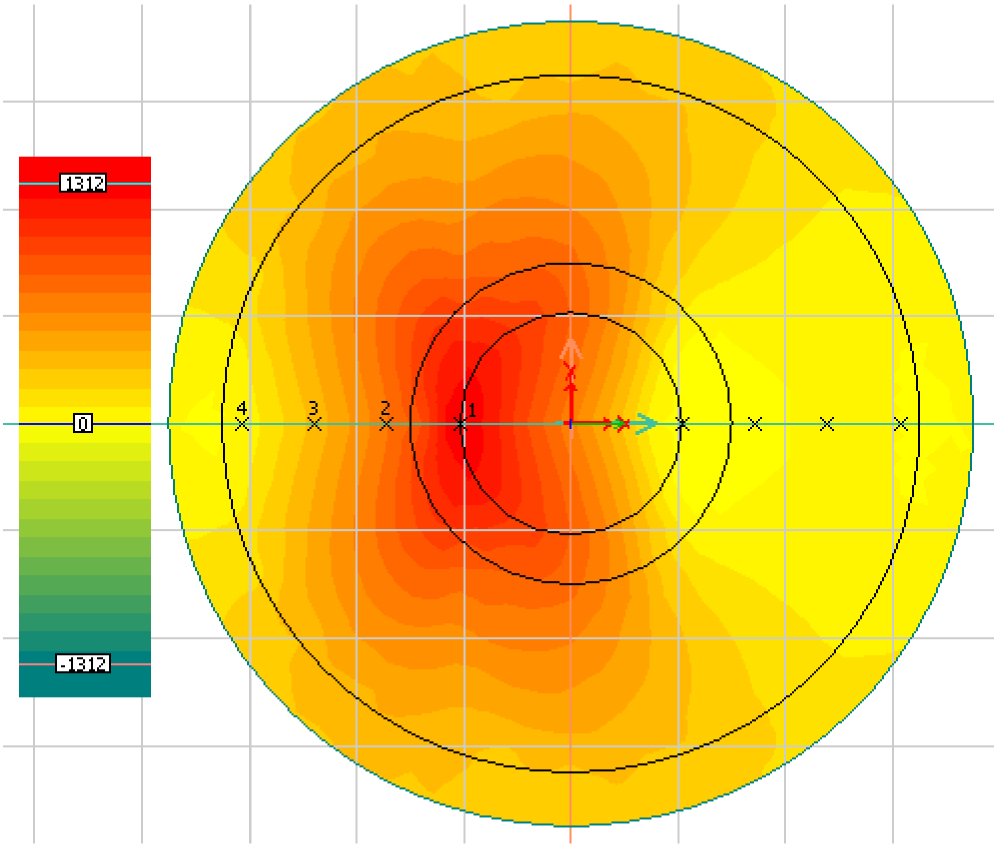
Minimum fatigue load - Bending moment [kNm]



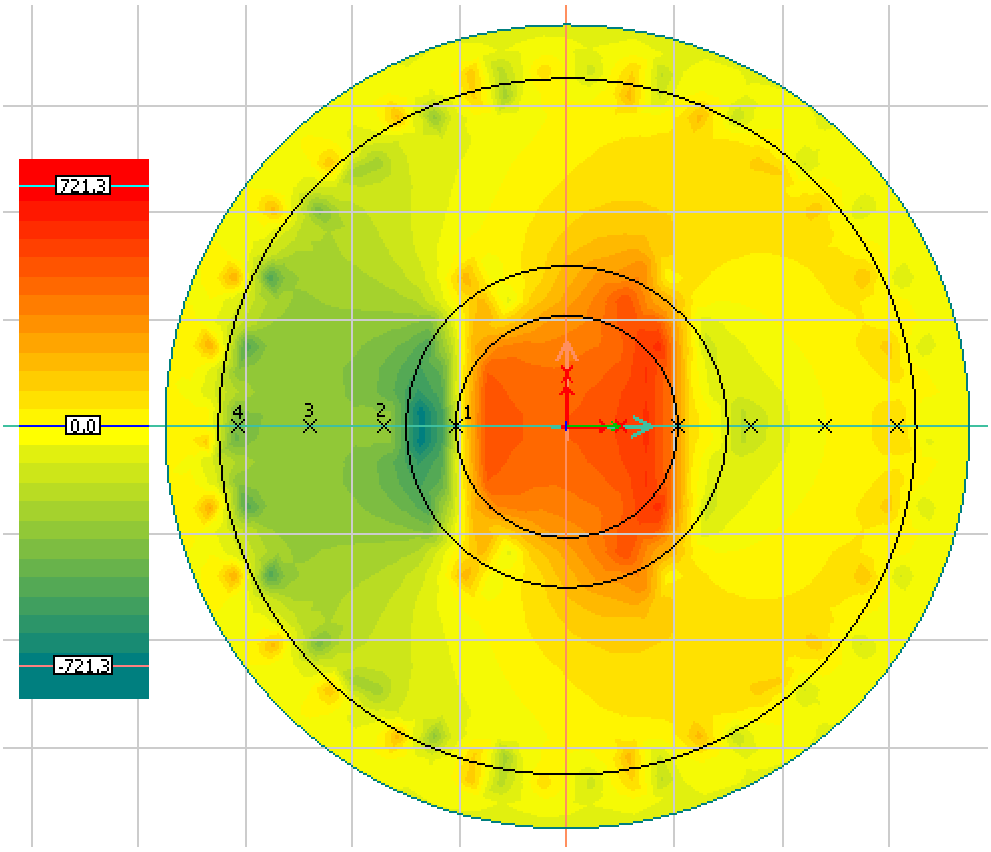
Minimum fatigue load – Shear force [kN]



Maximum fatigue load - Bending moment [kNm]



Maximum fatigue load – Shear force [kN]



M - Design for bending moment – Case 2

Loading:	**Ultimate limit state**		
Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=acc*fck/gC=	20 MPa
		fctd=acc*fctk/gC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
Reinforcement:			
	Ribbed bar, B500B		
	Bar diameter	Top: Φt=	25 mm
		Bottom: Φb=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	435 MPa
	Young's modulus	Es=	200 Gpa
	Yield strain	esy=	0,218 %

Governing equations:

For concrete with fck < 50MPa λ=0,8

η=1.0

M=fcd*0,8*x*b*(d-0,4*x) gives the compressed distance, x

es=ecu*(d-x)/x

es>esy to obtain a non-brittle break

As*fyd=fcd*0,8*x*b gives the reinforcement needed

Top:

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Mt [kNm/m]	-861	-485	-216	-54
x [mm]	22,2	14,7	7,6	2,3
es [%]	38,2	49,1	80,8	223,4
As [mm^2/m]	**815**	**540**	**281**	**85**

Bottom:

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Mb [kNm/m]	3137	1671	958	103
x [mm]	81,5	50,9	34,1	4,4
es [%]	10,1	13,9	17,8	116,9
As [mm^2/m]	**2998**	**1872**	**1253**	**161**

N - Design for shear force – Case 2

Loading:	**Ultimate limit state**		
Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=acc*fck/gC=	20 MPa
		fctd=acc*fctk/gC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm

Calculation without shear reinforcement:

Governing equations:

Vrd.c=Crd.c*k*(100*ρl*fck)^1/3*bw*d

Vrd.c>0,035*(k^3*fck)^1/2*bw*d=Vrd.c.min

k=1+(200/d)^1/2 < 2,0

ρl=Asl/(bw*d) < 0,02

Crd.c=0,18/γC — Crd.c= 0,12

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asl [mm^/m]	3813	2412	1534	246
k [-]	1,28642	1,31067	1,33608	1,3689
ρl [-]	1,6E-03	1,2E-03	8,7E-04	1,7E-04
Vrd.c [kN/m]	630	494	390	192
Vrd.c.min [kN/m]	682	596	524	451
Ved max [kN/m]	1337	939	791	734
Control	Not OK!	Not OK!	Not OK!	Not OK!

Shear stirrups needed!

Calculation with shear stirrups:

Governing equations:

$Vrd.s=Asw/s*z*fywd*cot(\Phi)$

$Vrd.max=\alpha cw*bw*z*v1*fcd/(cot(\Phi)+tan(\Phi))$

$v=0,6*(1-fck/250)$

$z=0.9*d$

Reinforcement: Quality B500B

Shear stirrups diameter	Φ=	25 mm
Yield stress	fywk=	500 Mpa
	fywd=0,8*fywk=	400 MPa
Reinforcement area, one cut	Asw=	491 mm^2
	cot(Φ)=	1
	tan(Φ)=	1
	αcw=	1
	v=	0,528

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
z [m]	2,1942	1,86498	1,59363	1,3224
Vrd.s [kN/m]	1337	939	791	734
s [mm]	**568**	**624**	**629**	**595**
Vrd.max [kN/m]	11585	9847	8414	6982

P - Design for crack width – Case 2

Design for crack width - Case 2

Loading:	**Serviceability limit state (SLS)**		
Safety factors:	Long term effect factor	αc=	1,00
	Concrete factor	γC=	1,00
	Reinforcement factor	γS=	1,00
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctm=	2,9 MPa
		fcd=αcc*fck/γC=	30 MPa
		fcteff=αc*fctm/γC=	2,9 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	φ=	0
		Eceff=	33 GPa
Reinforcement:	Ribbed bar, B500B		
	Bar diameter	Top: Φt=	25 mm
		Bottom: Φb=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	500 MPa
	Young's modulus	Es=	200 Gpa
	Yield strain	esy=	0,25 %

Section forces:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Mb [kNm/m]	1986	1301	626	62

Governing equations:

wk=(esm-ecm)*srmax

esm-ecm = [σs-kt*fcteff*(1+α*ρpeff)/ρpeff]/Es >= 0,6*σs/Es

srmax = k3c+k1*k2*k4*Φ/ρpeff

σs=M/(As*z)

z=d-x/3

x=d*ρpeff*α*[(1+2/(ρpeff*α))^0.5 - 1]

α=Es/Eceff	α=	6,0606
ρpeff=As/(2.5*c*b)		
kt, duration of loading	kt=	0,6
k1, adhesion	k1=	0,8
k2, stress distribution	k2=	0,5
k3=7Φ/c	k3_top=	3,5
	k3_bottom=	4,48
k4 -	k4=	0,425

Top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_t [mm^2/m]	815	540	281	85
ρpeff [-]	6,52E-03	4,32E-03	2,25E-03	6,80E-04
x [mm]	596	423	269	127
z [m]	2,24	1,93	1,68	1,43
Mt [kNm/m]	860	484	215	54
σs [Mpa]	471,20	464,12	455,17	445,26
Δε [-]	0,00097	0,00025	-0,00165	-0,01062
Δεmax [-]	0,00141	0,00139	0,00137	0,001336
Δε [-]	0,00141	0,00139	0,00137	0,001336
srmax [mm]	826,84	1158,80	2065,57	6425,00
wk [mm]	1,17	1,61	2,82	8,58
Control	**Not OK!**	**Not OK!**	**Not OK!**	**Not OK!**

Bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_b [mm^2/m]	2998	1872	1253	161
ρpeff [-]	2,40E-02	1,50E-02	1,00E-02	1,29E-03
x [mm]	1007	715	519	173
z [m]	2,10	1,83	1,60	1,41
Mb [kNm/m]	1986	1301	626	62
σs [Mpa]	315,10	378,91	312,60	272,19
Δε [-]	0,00116	0,00126	0,00064	-0,00543
Δεmax [-]	0,00095	0,00114	0,00094	0,000817
Δε [-]	0,00116	0,00126	0,00094	0,000817
srmax [mm]	450,81	587,22	766,53	4438,56
wk [mm]	0,52	0,74	0,72	3,62
Control	**Not OK!**	**Not OK!**	**Not OK!**	**Not OK!**

Corrected reinforcement

Loading: **Serviceability limit state (SLS)**

Safety factors:

Long term effect factor	αc=	1,00
Concrete factor	γC=	1,00
Reinforcement factor	γS=	1,00

Concrete:

Concrete C30/37		
Compression strength	fck=	30 Mpa
Tension strength	fctm=	2,9 MPa
	fcd=αcc*fck/γC=	30 MPa
	fcteff=αc*fctm/γC=	2,9 MPa
Maximum compression strain	ecu=	0,35 %
Covering concrete	c=	50 mm
Young's modulus	Ecm=	33 Gpa
Creep factor, for short time loading	φ=	0
	Eceff=	33 GPa

Reinforcement:

Ribbed bar, B500B			
Bar diameter	Top:	Φt=	25 mm
	Bottom:	Φb=	32 mm
Yield stress		fyk=	500 Mpa
		fyd=fyk/γC=	500 MPa
Young's modulus		Es=	200 Gpa
Yield strain		esy=	0,25 %

Section forces:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Mb [kNm/m]	1986	1301	626	62

Governing equations:

wk=(esm-ecm)*srmax

esm-ecm = [σs-kt*fcteff*(1+α*ρpeff)/ρpeff]/Es >= 0,6*σs/Es

srmax = k3c+k1*k2*k4*Φ/ρpeff

σs=M/(As*z)

z=d-x/3

x=d*ρpeff*α*[(1+2/(ρpeff*α))^0.5 - 1]

α=Es/Eceff	α=	6,0606
ρpeff=As/(2.5*c*b)		
kt, duration of loading	kt=	0,6
k1, adhesion	k1=	0,8
k2, stress distribution	k2=	0,5
k3=7Φ/c	k3_top=	3,5
	k3_bottom=	4,48
k4 -	k4=	0,425

Top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_t [mm^2/m]	**1540**	**1199**	**818**	**422**
ρpeff [-]	1,23E-02	9,59E-03	6,54E-03	3,38E-03
x [mm]	778	596	433	269
z [m]	2,18	1,87	1,63	1,38
Mt [kNm/m]	860	484	215	54
σs [Mpa]	256,30	215,47	161,62	92,74
$\Delta\varepsilon$ [-]	0,00052	0,00012	-0,00057	-0,00217
$\Delta\varepsilon$max [-]	0,00077	0,00065	0,00048	0,000278
$\Delta\varepsilon$ [-]	0,00077	0,00065	0,00048	0,000278
srmax [mm]	519,97	618,08	824,45	1433,89
wk [mm]	0,40	0,40	0,40	0,40
Control	OK!	OK!	OK!	OK!

Bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_b [mm^2/m]	**3608**	**2794**	**1800**	**520**
ρpeff [-]	2,89E-02	2,24E-02	1,44E-02	4,16E-03
x [mm]	1077	834	601	295
z [m]	2,08	1,79	1,57	1,37
Mb [kNm/m]	1986	1301	626	62
σs [Mpa]	264,78	259,52	221,47	86,97
$\Delta\varepsilon$ [-]	0,00097	0,00086	0,00045	-0,00171
$\Delta\varepsilon$max [-]	0,00079	0,00078	0,00066	0,000261
$\Delta\varepsilon$ [-]	0,00097	0,00086	0,00066	0,000261
srmax [mm]	412,47	467,38	601,78	1531,69
wk [mm]	0,40	0,40	0,40	0,40
Control	OK!	OK!	OK!	OK!

Values in bold is corrected reinforcement area

Q - Design for fatigue – Case 2

Loading:	**Fatigue**		
Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=αcc*fck/γC=	20 MPa
		fcd.fat=fcd*(1-fck/250)=	17,6 MPa
		fctd=αc*fctk/γC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	φ=	0
		Eceff=	33 GPa
Reinforcement:	Maximum stress range	$\Delta\sigma$Rsd=	70 Mpa
	Young's modulus	Es=	200 Gpa

Section forces:

Max fatigue:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Vt [kN/m]	-317	-238	-159	-79
Mb [kNm/m]	1295	827	427	38
Vb [kN/m]	612	328	333	368

Min fatigue:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Vt [kN/m]	-317	-238	-159	-79
Mb [kNm/m]	603	348	226	14
Vb [kN/m]	336	97	174	230

Governing equations:

Fatigue control in concrete, σcmax/fcd.fat < 0.5+0.45*σcmin/fcd.fat<0.9

Fatigue stress in reinforcement, σsmax - σsmin < $\Delta\sigma$Rsd

Fatigue control in concrete:

$\sigma c=2*M/(x*z*b)$
$x=d*\rho*\alpha*[(1+2/(\rho*\alpha))^{0.5}-1)$
$z=d-x/3$
$\rho=As/(b*d)$
$\alpha=Es/Eceff$ $\alpha=$ 6,0606

Fatigue top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Ast [mm^2/m]	1540	1199	818	422
ρ [-]	6,3E-04	5,8E-04	4,6E-04	2,9E-04
x [mm]	204	166	128	84
z [m]	2,37	2,02	1,73	1,44
Mtmax [kNm/m]	-860	-484	-215	-54
Mtmin [kNm/m]	-860	-484	-215	-54
σcmax [Mpa]	3,55	2,88	1,95	0,89
σcmin [Mpa]	3,55	2,88	1,95	0,89

Control:

σcmax/fcd.fat	0,20	0,16	0,11	0,05
0.5+0.45*σcmin/fcd.fat	0,59	0,57	0,55	0,52
< 0.9	OK!	OK!	OK!	OK!

Fatigue bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asb [mm^2/m]	3608	2794	1800	520
ρ [-]	1,5E-03	1,3E-03	1,0E-03	3,5E-04
x [mm]	305	249	186	93
z [m]	2,34	1,99	1,71	1,44
Mbmax [kNm/m]	1295	827	427	38
Mbmin [kNm/m]	603	348	226	14
σcmax [Mpa]	3,63	3,35	2,69	0,57
σcmin [Mpa]	1,69	1,41	1,42	0,21

Control:

σcmax/fcd.fat	0,21	0,19	0,15	0,03
0.5+0.45*σcmin/fcd.fat	0,54	0,54	0,54	0,51
< 0.9	OK!	OK!	OK!	OK!

Fatigue stress in reinforcement:

$\sigma s = M/(As*z)$
$x = d*\rho*\alpha*[(1+2/(\rho*\alpha))^{0.5}-1)$
$z = d-x/3$
$\rho = As/(b*d)$
$\alpha = Es/Eceff$ $\alpha =$ 6,0606

Fatigue top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Ast [mm^2/m]	1540	1199	818	422
ρ [-]	6,3E-04	5,8E-04	4,6E-04	2,9E-04
x [mm]	204	166	128	84
z [m]	2,37	2,02	1,73	1,44
Mtmax [kNm/m]	-860	-484	-215	-54
Mtmin [kNm/m]	-860	-484	-215	-54
σsmax [Mpa]	235,64	200,16	152,09	88,79
σsmin [Mpa]	235,64	200,16	152,09	88,79

Control:

$\Delta\sigma$ [Mpa]	0,00	0,00	0,00	0,00
$\Delta\sigma$Rsd [Mpa]	70,00	70,00	70,00	70,00
Control	OK!	OK!	OK!	OK!

Fatigue bottom:

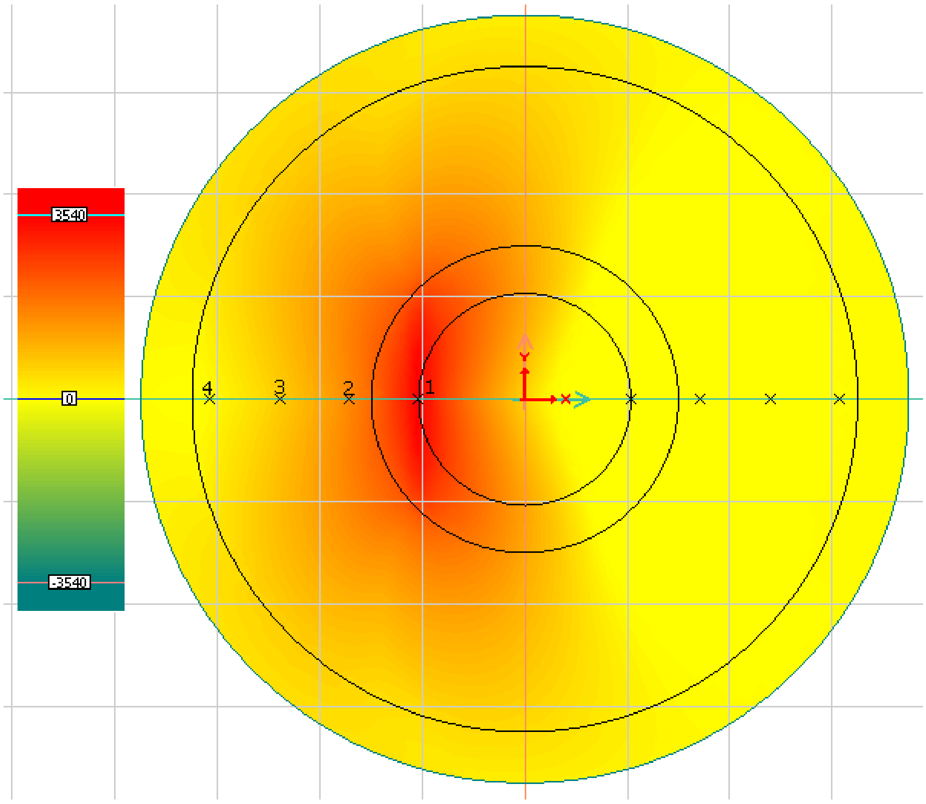
Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asb [mm^2/m]	3608	2794	1800	520
ρ [-]	1,5E-03	1,3E-03	1,0E-03	3,5E-04
x [mm]	305	249	186	93
z [m]	2,34	1,99	1,71	1,44
Mbmax [kNm/m]	1295	827	427	38
Mbmin [kNm/m]	603	348	226	14
σsmax [Mpa]	153,64	148,79	138,83	50,81
σsmin [Mpa]	71,54	62,61	73,48	18,72

Control:

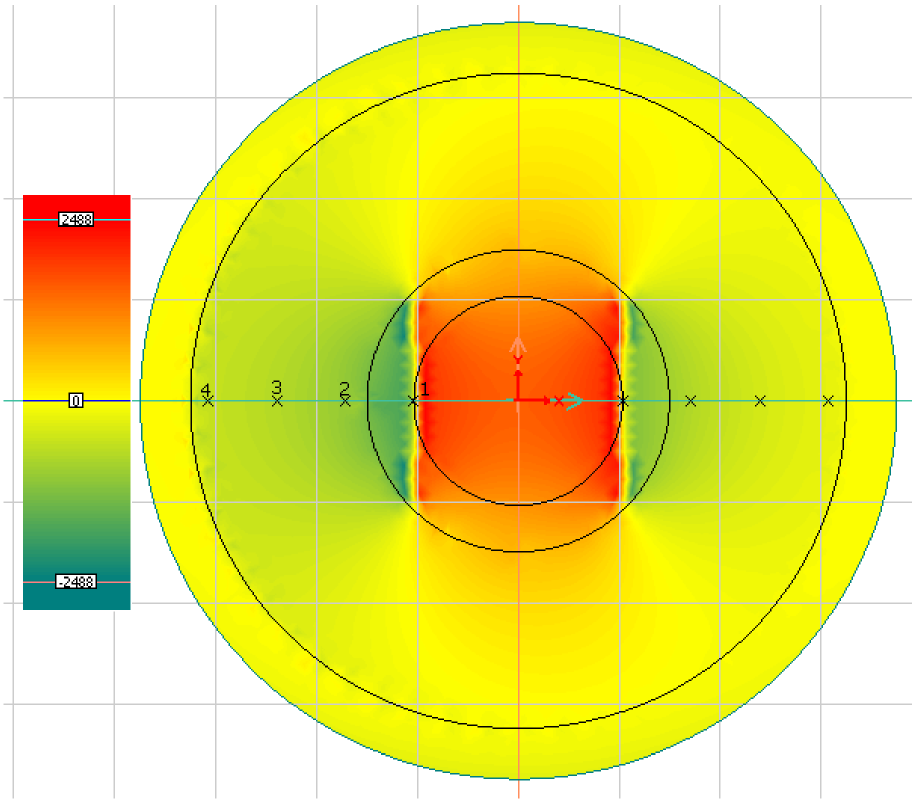
$\Delta\sigma$ [Mpa]	82,10	86,18	65,35	32,09
$\Delta\sigma$Rsd [Mpa]	70,00	70,00	70,00	70,00
Control	Not OK!	Not OK!	OK!	OK!

R - Sectional forces – Case 3

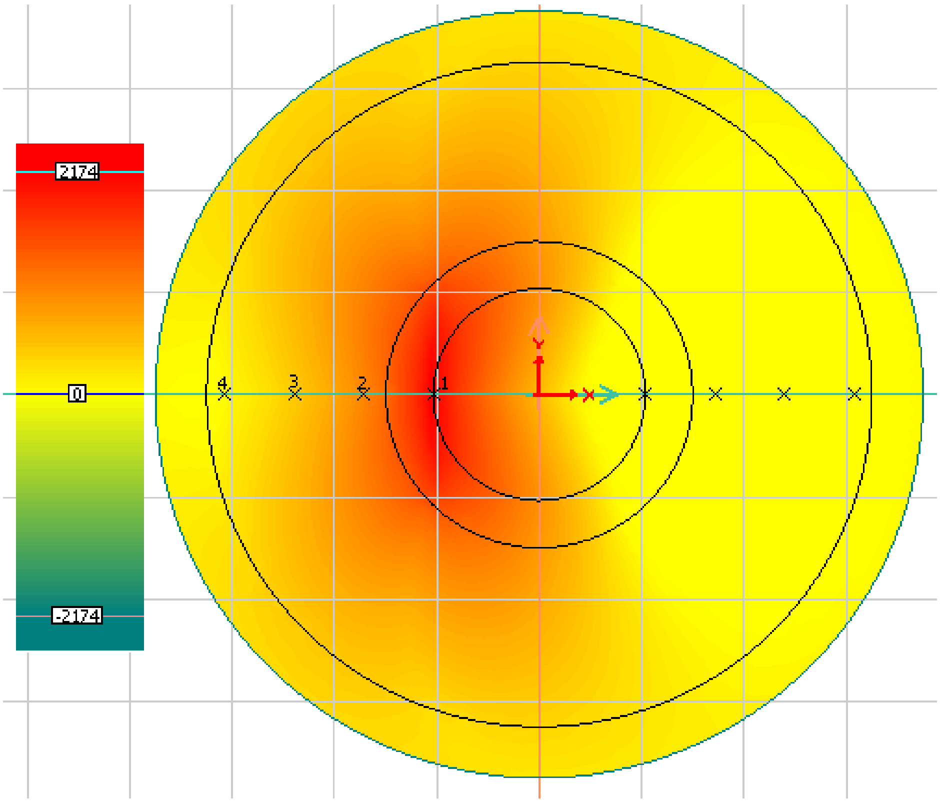
ULS - Bending moment [kNm]



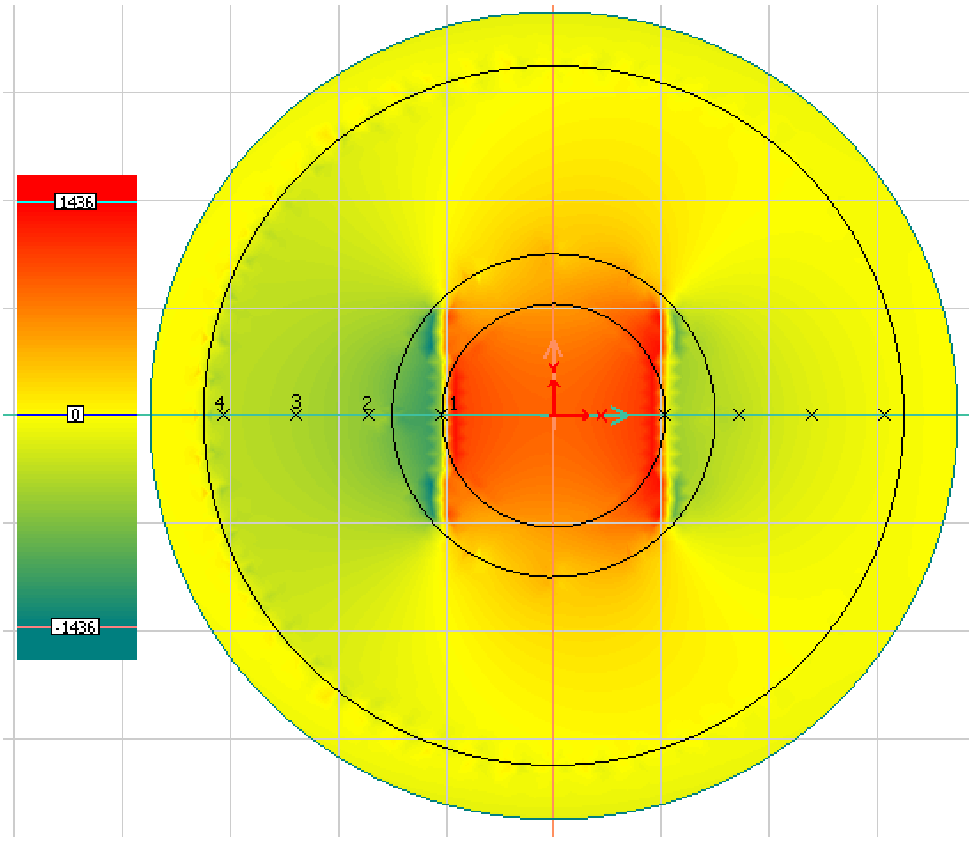
ULS – Shear force [kN]



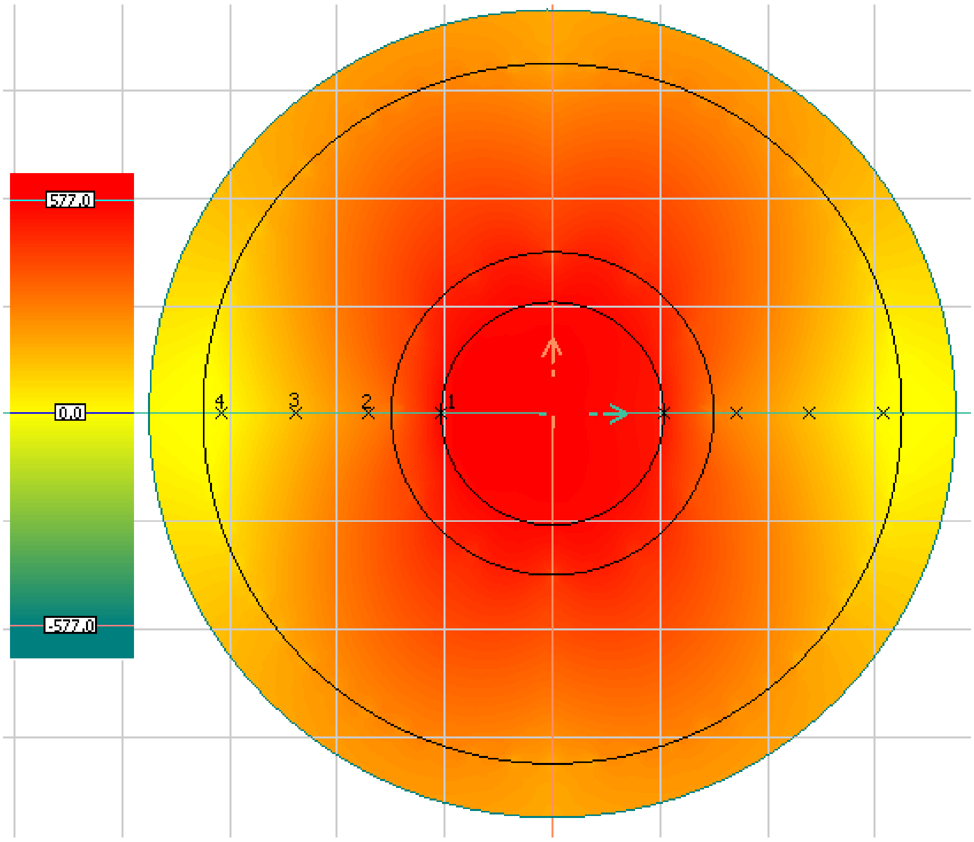
SLS - Bending moment [kNm]



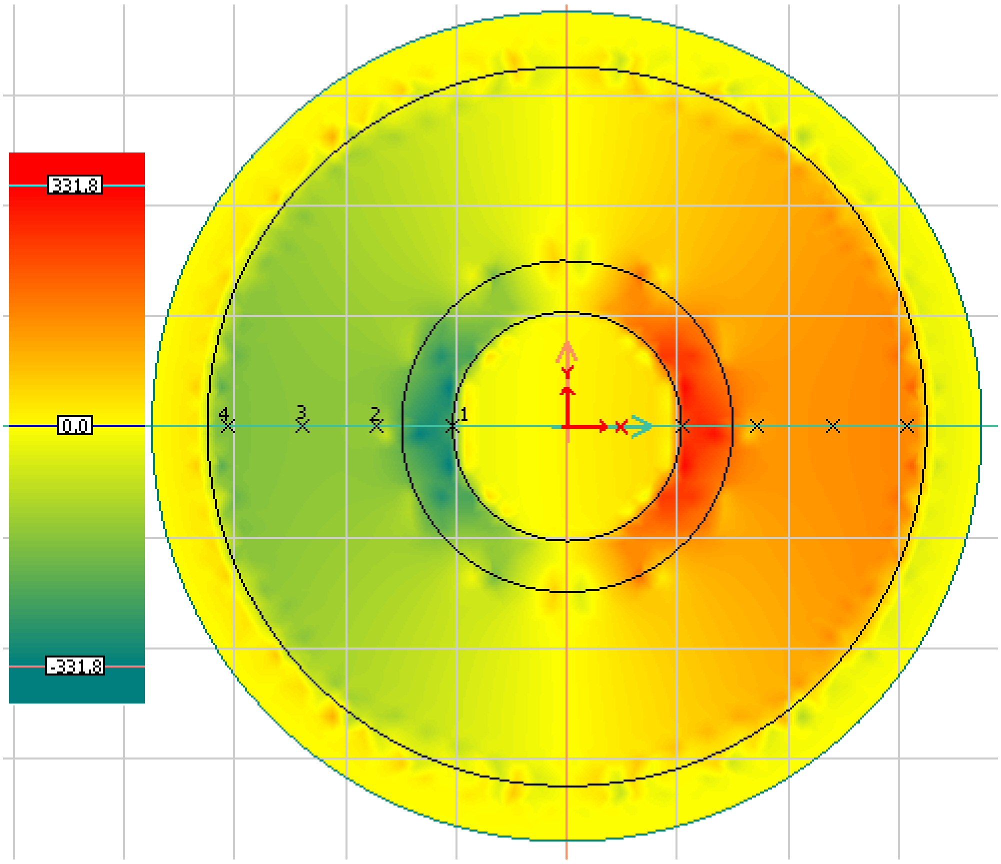
SLS – Shear force [kN]



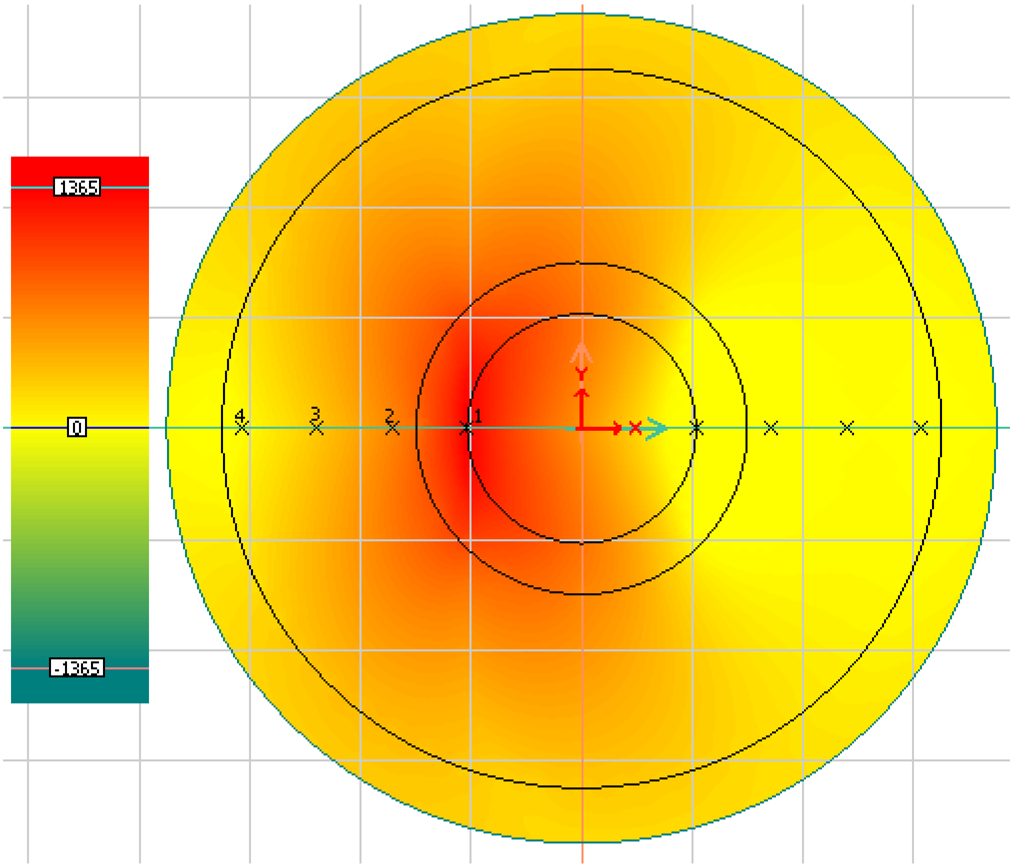
Minimum fatigue load - Bending moment [kNm]



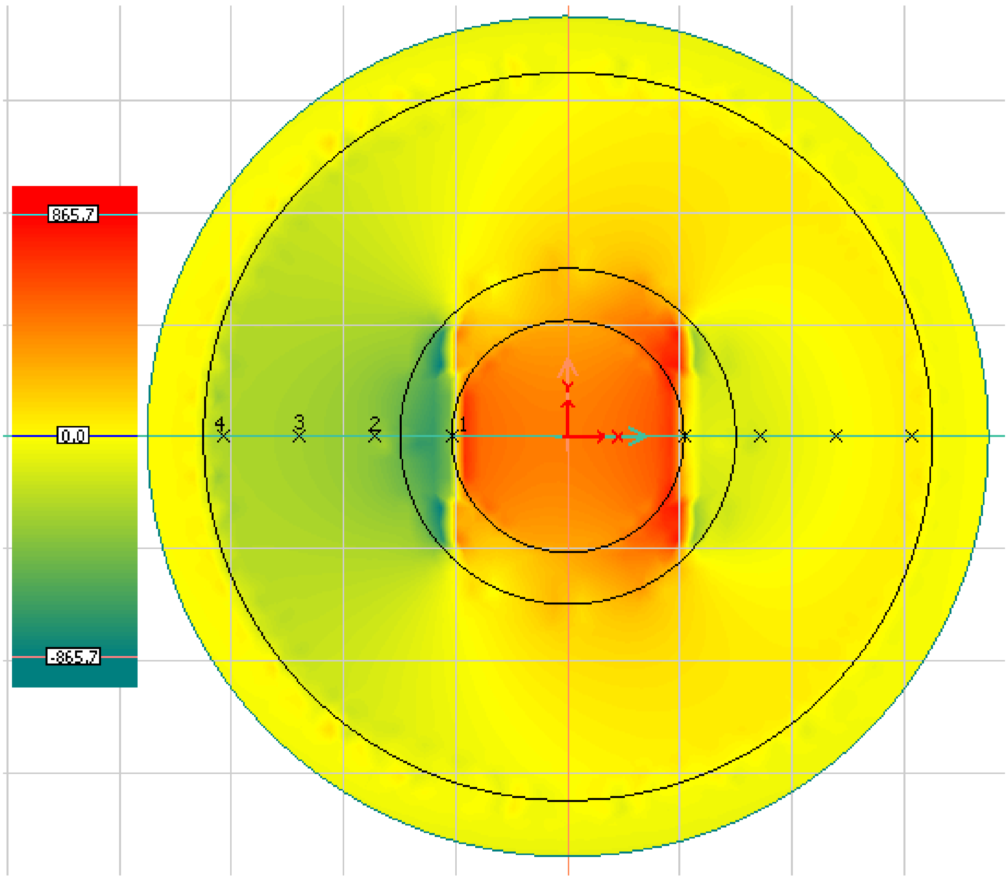
Minimum fatigue load – Shear force [kN]



Maximum fatigue load - Bending moment [kNm]



Maximum fatigue load – Shear force [kN]



S - Design for bending moment – Case 3

Loading: **Ultimate limit state**

Safety factors:

Long term effect factor		αc=	1
Concrete factor		γC=	1,5
Reinforcement factor		γS=	1,15

Concrete:

Concrete C30/37			
Compression strength		fck=	30 Mpa
Tension strength		fctk=	2 MPa
		fcd=acc*fck/gC=	20 MPa
		fctd=acc*fctk/gC=	1,33 MPa
Maximum compression strain		ecu=	0,35 %
Covering concrete		c=	50 mm

Reinforcement:

Ribbed bar, B500B			
Bar diameter	Top:	Φt=	25 mm
	Bottom:	Φb=	32 mm
Yield stress		fyk=	500 Mpa
		fyd=fyk/γC=	435 MPa
Young's modulus		Es=	200 Gpa
Yield strain		esy=	0,218 %

Governing equations:

For concrete with fck < 50MPa λ=0,8

η=1.0

M=fcd*0,8*x*b*(d-0,4*x) gives the compressed distance, x

es=ecu*(d-x)/x

es>esy to obtain a non-brittle break

As*fyd=fcd*0,8*x*b gives the reinforcement needed

Top:

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Mt [kNm/m]	-861	-485	-216	-54
x [mm]	22,2	14,7	7,6	2,3
es [%]	38,2	49,1	80,8	223,4
As [mm^2/m]	**815**	**540**	**281**	**85**

Bottom:

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Mb [kNm/m]	3408	1605	865	107
x [mm]	88,7	48,9	30,7	4,6
es [%]	9,3	14,5	19,8	112,5
As [mm^2/m]	**3261**	**1798**	**1131**	**168**

T - Design for shear force – Case 3

Loading:	**Ultimate limit state**		
Safety factors:	Long term effect factor	αc=	1
	Concrete factor	γC=	1,5
	Reinforcement factor	γS=	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=acc*fck/gC=	20 MPa
		fctd=acc*fctk/gC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm

Calculation without shear reinforcement:

Governing equations:

Vrd.c=Crd.c*k*(100*ρl*fck)^1/3*bw*d

Vrd.c>0,035*(k^3*fck)^1/2*bw*d=Vrd.c.min

k=1+(200/d)^1/2 < 2,0

ρl=Asl/(bw*d) < 0,02

Crd.c=0,18/γC   Crd.c= 0,12

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asl [mm^/m]	4076	2337	1412	252
k [-]	1,28642	1,31067	1,33608	1,3689
ρl [-]	1,7E-03	1,1E-03	8,0E-04	1,7E-04
Vrd.c [kN/m]	644	489	380	193
Vrd.c.min [kN/m]	682	596	524	451
Ved max [kN/m]	1668	1008	788	591
Control	Not OK!	Not OK!	Not OK!	Not OK!

Shear stirrups needed!

Calculation with shear stirrups:

Governing equations:

Vrd.s=Asw/s*z*fywd*cot(Φ)

Vrd.max=αcw*bw*z*v1*fcd/(cot(Φ)+tan(Φ))

v=0,6*(1-fck/250)

z=0.9*d

Reinforcement: Quality B500B

Shear stirrups diameter	Φ=	25	mm
Yield stress	fywk=	500	Mpa
	fywd=0,8*fywk=	400	MPa
Reinforcement area, one cut	Asw=	491	mm^2
	cot(Φ)=	1	
	tan(Φ)=	1	
	αcw=	1	
	v=	0,528	

Section	1	2	3	4
Dist. [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
z [m]	2,1942	1,86498	1,59363	1,3224
Vrd.s [kN/m]	1668	1008	788	591
s [mm]	**508**	**603**	**630**	**663**
Vrd.max [kN/m]	11585	9847	8414	6982

U - Design for crack width – Case 3

Loading:	**Serviceability limit state (SLS)**		
Safety factors:	Long term effect factor	αc=	1,00
	Concrete factor	γC=	1,00
	Reinforcement factor	γS=	1,00
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctm=	2,9 MPa
		fcd=αcc*fck/γC=	30 MPa
		fcteff=αc*fctm/γC=	2,9 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	φ=	0
		Eceff=	33 GPa
Reinforcement:	Ribbed bar, B500B		
	Bar diameter — Top:	Φt=	25 mm
	Bottom:	Φb=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	500 MPa
	Young's modulus	Es=	200 Gpa
	Yield strain	esy=	0,25 %

Section forces:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Mb [kNm/m]	2107	1001	550	63

Governing equations:

wk=(esm-ecm)*srmax

esm-ecm = [σs-kt*fcteff*(1+α*ρpeff)/ρpeff]/Es >= 0,6*σs/Es

srmax = k3c+k1*k2*k4*Φ/ρpeff

σs=M/(As*z)

z=d-x/3

x=d*ρpeff*α*[(1+2/(ρpeff*α))^0.5 - 1]

α=Es/Eceff	α=	6,0606
ρpeff=As/(2.5*c*b)		
kt, duration of loading	kt=	0,6
k1, adhesion	k1=	0,8
k2, stress distribution	k2=	0,5
k3=7Φ/c	k3_top=	3,5
	k3_bottom=	4,48
k4 -	k4=	0,425

Top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_t [mm^2/m]	815	540	281	85
ρpeff [-]	6,52E-03	4,32E-03	2,25E-03	6,80E-04
x [mm]	596	423	269	127
z [m]	2,24	1,93	1,68	1,43
Mt [kNm/m]	860	484	215	54
σs [Mpa]	471,20	464,12	455,17	445,26
$\Delta\varepsilon$ [-]	0,00097	0,00025	-0,00165	-0,01062
$\Delta\varepsilon$max [-]	0,00141	0,00139	0,00137	0,001336
$\Delta\varepsilon$ [-]	0,00141	0,00139	0,00137	0,001336
srmax [mm]	826,84	1158,80	2065,57	6425,00
wk [mm]	1,17	1,61	2,82	8,58
Control	**Not OK!**	**Not OK!**	**Not OK!**	**Not OK!**

Bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_b [mm^2/m]	3261	1798	1131	168
ρpeff [-]	2,61E-02	1,44E-02	9,05E-03	1,34E-03
x [mm]	1039	703	497	176
z [m]	2,09	1,84	1,60	1,41
Mb [kNm/m]	2107	1001	550	63
σs [Mpa]	308,89	302,94	303,00	265,83
$\Delta\varepsilon$ [-]	0,00116	0,00086	0,0005	-0,0052
$\Delta\varepsilon$max [-]	0,00093	0,00091	0,00091	0,000798
$\Delta\varepsilon$ [-]	0,00116	0,00091	0,00091	0,000798
srmax [mm]	432,52	602,20	825,24	4271,62
wk [mm]	0,50	0,55	0,75	3,41
Control	**Not OK!**	**Not OK!**	**Not OK!**	**Not OK!**

Corrected reonforcement area

Loading:	**Serviceability limit state (SLS)**		
Safety factors:	Long term effect factor	αc=	1,00
	Concrete factor	γC=	1,00
	Reinforcement factor	γS=	1,00
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctm=	2,9 MPa
		fcd=αcc*fck/γC=	30 MPa
		fcteff=αc*fctm/γC=	2,9 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	φ=	0
		Eceff=	33 GPa
Reinforcement:	Ribbed bar, B500B		
	Bar diameter	Top: Φt=	25 mm
		Bottom: Φb=	32 mm
	Yield stress	fyk=	500 Mpa
		fyd=fyk/γC=	500 MPa
	Young's modulus	Es=	200 Gpa
	Yield strain	esy=	0,25 %

Section forces:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Mb [kNm/m]	2107	1001	550	63

Governing equations:

wk=(esm-ecm)*srmax

esm-ecm = [σs-kt*fcteff*(1+α*ρpeff)/ρpeff]/Es >= 0,6*σs/Es

srmax = k3c+k1*k2*k4*Φ/ρpeff

σs=M/(As*z)

z=d-x/3

x=d*ρpeff*α*[(1+2/(ρpeff*α))^0.5 - 1]

α=Es/Eceff	α=	6,0606
ρpeff=As/(2.5*c*b)		
kt, duration of loading	kt=	0,6
k1, adhesion	k1=	0,8
k2, stress distribution	k2=	0,5
k3=7Φ/c	k3_top=	3,5
	k3_bottom=	4,48
k4 -	k4=	0,425

Top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_t [mm^2/m]	**1540**	**1199**	**818**	**422**
ρpeff [-]	1,23E-02	9,59E-03	6,54E-03	3,38E-03
x [mm]	778	596	433	269
z [m]	2,18	1,87	1,63	1,38
Mt [kNm/m]	860	484	215	54
σs [Mpa]	256,30	215,47	161,62	92,74
$\Delta\varepsilon$ [-]	0,00052	0,00012	-0,00057	-0,00217
$\Delta\varepsilon$max [-]	0,00077	0,00065	0,00048	0,000278
$\Delta\varepsilon$ [-]	0,00077	0,00065	0,00048	0,000278
srmax [mm]	519,97	618,08	824,45	1433,89
wk [mm]	0,40	0,40	0,40	0,40
Control	OK!	OK!	OK!	OK!

Bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
As_b [mm^2/m]	**3818**	**2201**	**1659**	**525**
ρpeff [-]	3,05E-02	1,76E-02	1,33E-02	4,20E-03
x [mm]	1099	761	582	296
z [m]	2,07	1,82	1,58	1,37
Mb [kNm/m]	2107	1001	550	63
σs [Mpa]	266,40	250,11	210,26	87,56
$\Delta\varepsilon$ [-]	0,00099	0,0007	0,00034	-0,00169
$\Delta\varepsilon$max [-]	0,0008	0,00075	0,00063	0,000263
$\Delta\varepsilon$ [-]	0,00099	0,00075	0,00063	0,000263
srmax [mm]	402,10	532,95	633,89	1519,24
wk [mm]	0,40	0,40	0,40	0,40
Control	OK!	OK!	OK!	OK!

Values in bold is corrected reinforcement area

V - Design for fatigue – Case 3

Loading:	**Fatigue**		
Safety factors:	Long term effect factor	$\alpha c=$	1
	Concrete factor	$\gamma C=$	1,5
	Reinforcement factor	$\gamma S=$	1,15
Concrete:	Concrete C30/37		
	Compression strength	fck=	30 Mpa
	Tension strength	fctk=	2 MPa
		fcd=αcc*fck/γC=	20 MPa
		fcd.fat=fcd*(1-fck/250)=	17,6 MPa
		fctd=αc*fctk/γC=	1,33 MPa
	Maximum compression strain	ecu=	0,35 %
	Covering concrete	c=	50 mm
	Young's modulus	Ecm=	33 Gpa
	Creep factor, for short time loading	$\varphi=$	0
		Eceff=	33 GPa
Reinforcement:	Maximum stress range	$\Delta\sigma$Rsd=	70 Mpa
	Young's modulus	Es=	200 Gpa

Section forces:

Max fatigue:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Vt [kN/m]	-317	-238	-159	-79
Mb [kNm/m]	1335	646	366	38
Vb [kN/m]	672	373	324	280

Min fatigue:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
Mt [kNm/m]	-860	-484	-215	-54
Vt [kN/m]	-317	-238	-159	-79
Mb [kNm/m]	560	287	180	12
Vb [kN/m]	296	133	153	164

Governing equations:

Fatigue control in concrete, σcmax/fcd.fat < 0.5+0.45*σcmin/fcd.fat<0.9

Fatigue stress in reinforcement, σsmax - σsmin < $\Delta\sigma$Rsd

Fatigue control in concrete:

$\sigma c=2*M/(x*z*b)$
$x=d*\rho*\alpha*[(1+2/(\rho*\alpha))\hat{}0.5-1)$
$z=d-x/3$
$\rho=As/(b*d)$
$\alpha=Es/Eceff$ $\alpha=$ 6,0606

Fatigue top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Ast [mm^2/m]	1540	1199	818	422
ρ [-]	6,3E-04	5,8E-04	4,6E-04	2,9E-04
x [mm]	204	166	128	84
z [m]	2,37	2,02	1,73	1,44
Mtmax [kNm/m]	-860	-484	-215	-54
Mtmin [kNm/m]	-860	-484	-215	-54
σcmax [Mpa]	3,55	2,88	1,95	0,89
σcmin [Mpa]	3,55	2,88	1,95	0,89

Control:

σcmax/fcd.fat	0,20	0,16	0,11	0,05
0.5+0.45*σcmin/fcd.fat	0,59	0,57	0,55	0,52
< 0.9	OK!	OK!	OK!	OK!

Fatigue bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asb [mm^2/m]	3818	2201	1659	525
ρ [-]	1,6E-03	1,1E-03	9,4E-04	3,6E-04
x [mm]	314	222	179	94
z [m]	2,33	2,00	1,71	1,44
Mbmax [kNm/m]	1335	646	366	38
Mbmin [kNm/m]	560	287	180	12
σcmax [Mpa]	3,65	2,91	2,39	0,56
σcmin [Mpa]	1,53	1,29	1,18	0,18

Control:

σcmax/fcd.fat	0,21	0,17	0,14	0,03
0.5+0.45*σcmin/fcd.fat	0,54	0,53	0,53	0,50
< 0.9	OK!	OK!	OK!	OK!

Fatigue stress in reinforcement:

σs=M/(As*z)
x=d*ρ*α*[(1+2/(ρ*α))^0.5-1)
z=d-x/3
ρ=As/(b*d)
α=Es/Eceff

α= 6,0606

Fatigue top:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Ast [mm^2/m]	1540	1199	818	422
ρ [-]	6,3E-04	5,8E-04	4,6E-04	2,9E-04
x [mm]	204	166	128	84
z [m]	2,37	2,02	1,73	1,44
Mtmax [kNm/m]	-860	-484	-215	-54
Mtmin [kNm/m]	-860	-484	-215	-54
σsmax [Mpa]	235,64	200,16	152,09	88,79
σsmin [Mpa]	235,64	200,16	152,09	88,79

Control:

$\Delta\sigma$ [Mpa]	0,00	0,00	0,00	0,00
$\Delta\sigma$Rsd [Mpa]	70,00	70,00	70,00	70,00
Control	OK!	OK!	OK!	OK!

Fatigue bottom:

Section	1	2	3	4
Distance [m]	2,08	3,43	4,79	6,14
d [m]	2,44	2,07	1,77	1,47
Asb [mm^2/m]	3818	2201	1659	525
ρ [-]	1,6E-03	1,1E-03	9,4E-04	3,6E-04
x [mm]	314	222	179	94
z [m]	2,33	2,00	1,71	1,44
Mbmax [kNm/m]	1335	646	366	38
Mbmin [kNm/m]	560	287	180	12
σsmax [Mpa]	149,84	146,89	128,93	50,33
σsmin [Mpa]	62,86	65,26	63,41	15,89

Control:

$\Delta\sigma$ [Mpa]	86,99	81,63	65,52	34,44
$\Delta\sigma$Rsd [Mpa]	70,00	70,00	70,00	70,00
Control	Not OK!	Not OK!	OK!	OK!