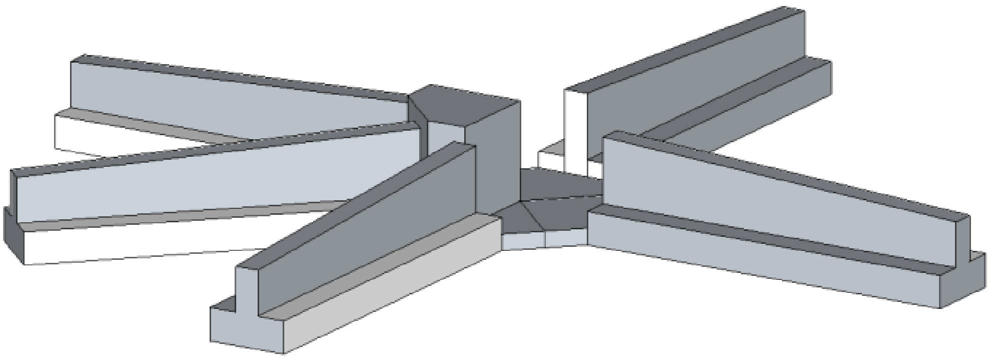

Prefabricated Foundation for Wind Power Plants

A Conceptual Design Study

Master of Science Thesis in the Master's Programme Structural Engineering and Building Technology

EMELIE ENELAND
LINA MÅLLBERG

Department of Civil and Environmental Engineering
Division of Structural Engineering

CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2013
Master's Thesis 2013:139

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Chalmers tekniska högskola 2013:139

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Cover:
A wind power plant foundation is conceptually designed in order to investigate the opportunities of prefabrication. This design is presented in Chapter 6 and Chapter 7.

Department of Civil and Environmental Engineering Göteborg, Sweden 2013

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ABSTRACT

This master thesis project has aimed at investigating the opportunities to prefabricate a gravity foundation of a wind power plant. The foundation should be precast in elements and assembled on site. Collection of existing experience gave the background to a conceptual design study to develop and evaluate different concepts. A general design intention was formulated in order to set up the framework:

The elements of the foundation should be possible to transport and assemble into a structure, where the elements fully interact, with minimum usage of onsite casting. The structure should have a continuous force pattern in the foundation during service. All this must be fulfilled, but not to an excessive cost.

Focus during concept development has been the ability to transport the elements and to achieve a good force pattern in the foundation after assembling by design of the connections between the elements. The goal has been to develop a proposal of a possible design of a prefabricated wind power plant foundation with minimum amount of onsite cast concrete, however the final conclusion is an understanding why most wind power plant foundations are onsite cast.

Regarding the shapes there are some concepts that are promising, with a rather lightweight construction and with a reasonable number of elements. However the methods to connect the elements do not fulfil the demands concerning the structural behaviour and continuous force pattern. Therefore the recommendation is to have the foundation onsite cast shaped with legs and a bottom slab. However, if a wind power plant foundation is to be built prefabricated, the recommendation is to have prefabricated elements cast together with an onsite cast centrepiece for the connection between the tower and the foundation. This is an already proven method in Sjisjka, Gällivare.

Keywords: Prefabricated concrete structure, Wind power plant foundation, Prefabricated foundation, Sjisjka Wind Park, Conceptual design, Prefabricated elements, Connections.

Prefabricering av Vindkraftverksfundament
En konceptuell designstudie
Examensarbete inom Structural Engineering and Building Technology
EMELIE ENELAND
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Institutionen för bygg- och miljöteknik
Avdelningen för konstruktionsteknik
Chalmers tekniska högskola

SAMMANFATTNING

Detta examensarbete har syftat till att undersöka möjligheterna att prefabricera ett gravitationsfundament till ett vindkraftverk. Fundamentet ska vara prefabricerat i element och monteras på plats. Befintlig erfarenhet av tidigare projekt är bakgrund till den konceptuella design studien, där olika koncept har utvecklats och utvärderats. Ett designsyfte med konstruktionen formulerades för att ge ramar för projektet:

Fundamentets element ska gå att transportera och sammanfoga till en konstruktion där elementen till fullo samverkar, med minsta möjliga mängd av platsgjutning. Konstruktionen ska ha ett kontinuerligt kraftspel under användningstiden. Allt detta ska uppfyllas, till en rimlig kostnad.

Under konceptutvecklingen har fokus varit på transport och att möjliggöra ett bra kraftspel i fundamentet efter montering, vilket åstadkoms genom bra fogar mellan elementen. Målet har varit att utveckla ett förslag på en möjlig konstruktion av ett vindkraftverksfundament med minst möjliga platsgjutning, men slutsatsen blir en förståelse för varför dessa vanligtvis är helt plastgjutna.

Vad gäller formen på fundamentet, så har visa former visat sig vara lovande angående vikt och antal element. Men metoderna att sammanfoga dessa element har inte kunnat uppfylla kraven på konstruktionen egenskaper och ett bra kraftspel. Därför är rekommendationen platsgjutning av fundamenten som en konstruktion med tjugo ben och bottenplatta. Är det ändå önskvärt att minska platsgjutningen så rekommenderar vi att ha de yttre elementen prefabricerade och gjuta samman dessa med en platsgjuten mittdel, där tornet ansluts till fundamentet. Detta är en metod som redan använts i Sjisjka, Gällivare.

Nyckelord: Prefabricerad betong, Vindkraftverk fundament, Prefabricerade fundament, Sjisjka Vindpark, konceptuell design, Prefabricerade element, kopplingar

Contents

Preface

In this master’s thesis project the possibilities to prefabricate wind power plant foundations have been investigated. The investigations have been carried out from January 2013 to June 2013 within the Master's Programme Structural Engineering and Building Technology.

The project has been performed at the Division of Structural Engineering, Concrete Structures, Chalmers University of Technology, Sweden. The work is requested by the design company VBK, Gothenburg.

The work with the study and the report has been carried out equally divided, it has been done together in close cooperation.

This master’s thesis project has been carried out with Professor Björn Engström as examiner and Erik Samuelsson at VBK as supervisor. We would like to give a special thank for all help and support they have provided throughout this project.

Our opponents Björn Johansson and Marcus Thyman deserve a credit for all their support during the project.

We would like to thank everyone who has been involved in this project, helped us with information and answering all our questions throughout this project.

Göteborg, June 2013

Emelie Eneland & Lina Mållberg

Notations

A_{soil}	Area of the soil pressure
F_c	Compressive force couple resultant
F_t	Tensile force couple resultant
F_z	Resulting self-weight from the tower
G_k	Self-weight of the foundation and the fill, characteristic value
G_d	Self-weight of the foundation and the fill, design value
H_d	Shear force from the tower
M	Bending moment
M_d	Overturning moment from the tower
N_k	Normal force from the tower, characteristic value
N_d	Normal force from the tower, design value
V	Shear force
cp	Section where the centrepiece starts, defined from the centre of the foundation
e	Eccentricity of the soil pressure
fc	Section where force couple resultant is applied, defined from the centre of the foundation
q_g	Self-weight of the foundation
q_i	Sum of the load on each strip
q_{soil}	Soil pressure resultant
s_i	The lever arm for each strip
sp	Section where soil pressure starts, defined from the centre of the foundation
σ_{Rd}	Strength of soil
σ_{soil}	Soil pressure
$ø_{fc}$	Distance between the force couple resultants
γ	Partial safety factor, favourable or unfavourable depending on load case

1 Introduction

This master's thesis project concerns prefabrication of wind power plant foundations. In the introduction it is presented why it might be beneficial to prefabricate the foundation, some challenges during prefabrication and also the method together with the limitations of the project itself.

1.1 Background

The demand for renewable energy in the world increases with the environmental consciousness. Wind power is one way to produce renewable energy, this is why the expansion rate of wind power plants is increasing in Sweden, as well as in the rest of the world (EON 2013).

1.1.1 Geographic location of new wind power plants

Since the beginning of 2009 a new directive from the European Union promotes the utilisation of renewable energy. This directive declares that Sweden should increase the amount of renewable energy, up to 49%, from today's 40% (Svensk Energi 2012). In order to reach this goal, a national plan is adopted by the Swedish government (Regeringen 2012), to achieve 30 TWh wind power in 2020, whereof 10 TWh offshore and 20 TWh onshore. To achieve this, almost 4000 new wind power plants need to be built onshore (WSP 2009).

Depending on geographic location and local wind variations, different sites have different conditions for producing wind energy. These site conditions are mapped on request from the Swedish government. As a result, a national database is established, with wind conditions at different locations in Sweden, see Figure 1.1 (Vindlov 2013).

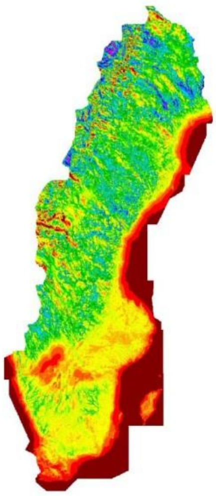

Figure 1.1: Yearly mean wind at the height of 100 m, mapped by Vindlov (2013). Red marks higher wind energy content, while green and blue marks lower wind energy content. (Vindlov 2013).

From this mapping, it is clear that offshore areas are feasible for wind energy production, as well as coastal areas. Also certain areas in northern of Sweden are especially suitable for wind energy production. Many of the areas in the south of

Sweden, with suitable wind conditions, are already used for wind energy production or populated. Generally people do not want wind power plants built in their neighbourhood, due to noise, shadows and ruined views (Vindkraftsnyheter 2011). Therefore it is beneficial to build wind power plants in the sparsely populated parts of Sweden, such as northern of Sweden.

This means that it is desirable to build wind power plants in distant locations. Many of these distant locations are situated in the northern of Sweden, with its benefits and drawbacks.

1.1.2 Challenges during construction of a wind power plant in distant locations

To prevent the wind power plant from tilting and to transfer the wind loads to the ground, a foundation is needed. There are some different types of foundations: rock anchored foundation, where the tower is anchored to firm rock, and gravity foundation, which consists of a large volume of concrete which is usually cast on site in a rectangular or circular shape.

There are several challenges with onsite casting of concrete in distant locations such as the northern part of Sweden. The challenges are concerning logistics, access to fresh concrete, environmental legislation and often a cold climate.

The first issue is the logistics; transportation of machines and commodities, such as fresh concrete, to the construction site. Some interesting areas for wind power plants in the northern of Sweden have a lack of roads and other communications, which means that new roads need to be constructed in order to reach the building site. Existing roads are often in need of strengthening in order to bear the increased loading due to the wind power plant construction (Nilsson 2010).

The needed amount of concrete for onsite casting of the foundation is large, and generally concrete stations are located in populated areas, far away from the construction site. This results in long distance transports to the construction sites when constructing in remote locations. Also special treatment of the concrete is needed, by means of retarders to prohibit the hydration process to start too early (Löfgren 2013-03-08) .

It is common that the biodiversity in the fell area of Sweden is protected by laws and regulations (Miljömål 2012). The protection has different degrees but will affect the possibilities to construct wind power plants. Therefore it is essential that the intrusion in the landscape is limited during construction.

When building in the northern of Sweden, construction time is limited due to the long and cold winter. This may result in an unusually long winter break which is negative in a time and cost perspective (Carlström 2013-02-26). The erection of the tower, nacelle and blades must also be done in lack of snow, ice and low temperatures. Therefore a short construction time is preferable.

1.1.3 Prefabricating wind power plant foundations

In order to outcome all these challenges, prefabrication is one opportunity. The elements of the foundation can be cast independently of the progression on the construction site and then be transported and assembled on site. This will shorten the construction time, decrease the intrusion in the landscape on the site and reduce the transports of fresh concrete to the building site.

The service life of a wind power plant is 20-25 years. This means that the foundation is removed, which is tough work. So if prefabrication could facilitate the demolishment phase this is one additional benefit.

When choosing to build a prefabricated construction, new problems will arise that do not exist when casting on site: transportation of elements, to design more material efficient shapes of the structure and to design joints and connections that make the elements act as a structure. This will result in a more complicated structure, which has high demands on the workmanship in all steps of the process.

Due to the issues with onsite casting, mentioned in the background, it is of interest to investigate the opportunities to prefabricate the foundation. The difficulties with prefabrication can often be overcome, but the question is to what price and if it is economically justifiable when looking at benefits and drawbacks.

1.2 Purpose and objectives

The purpose of this master's thesis project was to investigate the possibilities and limitations with prefabricating the foundation of a wind power plant. During the project, concepts should be developed for the design of the prefabricated elements. The concepts should be designed preliminary in order to develop the concepts and thereafter evaluated in order to have a final concept. This should include an economic, logistic and reality-based point of view.

The objective was to answer these questions:

- What are the different issues to identify and solve? Is it possible to build a prefabricated foundation of a wind power plant to an affordable cost?
- Are there any existing examples, and if there are, what kind of knowledge can that give during development of new ideas and solutions?
- What is a good solution for a prefabricated wind power plant? What is the shape, dimensions and design for this solution?

1.3 Limitations

The master's thesis project should only concern the foundation, not the wind turbine itself. No focus will be on technical parts such as tower, nacelle and hub, other than the sectional forces they transfer to the foundation, which are given as in data from the manufacturer.

The master's thesis project will only concern onshore gravity foundations. The project will only investigate prefabricated concrete foundations.

Focus when developing concepts for prefabrication of wind power plant foundations is the amount of transportation and to minimise the amount of onsite casting of concrete. The reason is to overcome the challenges to construct wind power plant foundations in the north of Sweden.

During development of prefabricated structures and analysis of these concepts, a methodology with some simplifications and approximations is used in design.

However, a more analysing method is presented and the use of the simplifications is motivated.

During analysis of the developed concepts, focus is design in ultimate limit state. However it is important to verify the structural behaviour in serviceability limit state and fatigue limit state. This is performed in more simplified manner, and some details are left to investigate in future projects.

The design of the connection between the tower and the foundation is limiting for the freedom in design of the foundation, no new method for this connection is to be designed.

1.4 Method

There are some earlier master's thesis's written at Chalmers University of Technology in the subject of wind power plant foundations, which have been used as a basis for this thesis. This means that previous knowledge is used as background for the investigations in this master's thesis project.

The master's thesis project consists of the three following steps:

A pre study where existing foundation methods are investigated, a step performed to give better understanding of wind power plants and its foundation. In the pre study also the opportunities of prefabrication are examined. This study gives necessary information concerning important factors and difficulties when prefabricating wind power plant foundations. This includes interviews and mail-conversations with people with experience in the subject.

A conceptual design procedure, used in order to come up with a good concept for prefabricating the foundation concerning all important parameters from the pre study. In order to perform a fair evaluation of the different concepts, the evaluation is performed as a case study for a fictive tower. All calculations are done by hand in the computer program Mathcad in order to gain a good understanding of the task.

Conclusions and recommendations are given by the results from the conceptual design investigations of different concepts. These conclusions and recommendations are meant to be general, and applicable as guidelines for further studies concerning prefabrication of wind power plants.

1.5 Outlines

The disposition of the report is made according to the methodology described above.

The pre study is presented in Chapter 2-4 consisting of an introduction to wind power plants, prefabrication of concrete structures and examples on existing prefabricated foundations for wind power plants.

The conceptual design procedure is presented in Chapter 5-7 consisting of a first a presentation of the methodology for the conceptual design, then implementation, results and conclusions from the design.

The conclusions and recommendations are presented in Chapter 8, also containing criticism on the methodology and further research.

2 Introduction to Wind Power Plants

This chapter is an introduction to wind power plants. The general performance of a wind power plant is presented together with some introductory sections of the loads and sectional forces. Also the performance of the foundation and its connection to the superstructure is described.

2.1 General performance of wind power plants

A wind power plant consists of a tower and a nacelle with a hub, upon which the rotor blades are mounted, see Figure 2.1. The tower is mounted on a foundation by means of prestressing bolts in order to transfer the loads acting on the superstructure.

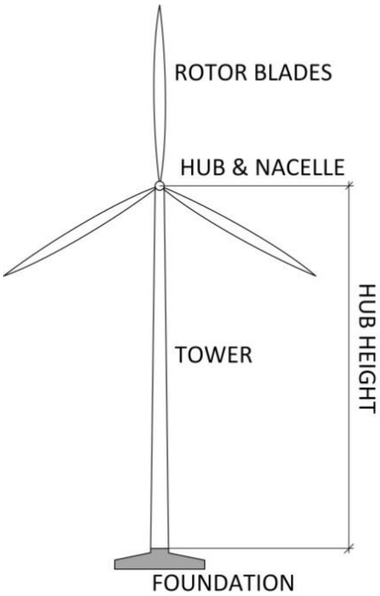

Figure 2.1: Illustration of the different parts of a wind power plant and how they are connected to each other.

In this report, the term wind power plant refers to all parts: such as the tower, the nacelle, the hub, the rotor blades and the foundation. When referring to the parts above the foundation it is called the superstructure. The foundation is also referred to as the concrete structure.

To better grasp the size of a wind power plant, some approximate dimensions are given, the hub height of the wind power plant normally varies between 80 and 130 m and the rotor blades have up to the same diameter. The dimension of a specific wind power plant is dependent on the wind conditions on the specific site.

During the last years the wind power plant industry has gone through a great development, the tower height and the rotor blades are getting larger and larger in order to increase the effect of the wind power plants. As a result of this, the wind power plant and its foundation are exposed to larger stresses and higher demands on foundation than earlier.

2.2 General performance of foundations of wind power plants

As mentioned before, wind power plants can be onshore and offshore structures. The location of the wind power plant enables different types of foundations. For onshore constructions three types of foundations are used: gravity foundations, pile foundations and rock anchored foundation.

The choice of method is mainly depending on the geotechnical conditions. Rock anchored foundations are normally used when the tower can be anchored directly to firm rock. In a gravity foundation, the self-weight of the foundation is a counter weight, preventing the tower from tilting. When the soil resistance is not sufficiently good for gravity foundations, piles are needed to achieve enough bearing capacity. The most common method to found an onshore wind power plant is gravity foundations (Hassanzadeh 2012).

The foundation must transfer the self-weight of the tower to the ground, and distribute the forces so that the soil can resist the forces. The foundation is acting as a counterweight preventing the tower from tilting, due to the overturning moment from the wind load.

Gravity foundations for large wind power plants are approximately 15 to 20 m in diameter and the height 2 to 3 m (Samuelssson 2013-01-28). Usually the foundations are circular, square or octagonal. Circular or octagonal foundations give a more efficient material utilisation, but have more demanding calculation and design procedure due to the geometry. The choice of geometry of the foundation can also depend on the constructor's wishes regarding formwork.

Normally the top face is sloping, to optimize the material usage since the largest stresses occur closest to the tower. The foundation is covered with fill to increase the self-weight of the structure, preventing the tower from tilting.

Bending reinforcement is placed in the top and in the bottom of the foundation, since the overturning moment and the self-weight cause large tensile stresses in these locations. To handle the shear forces, vertical stirrups are normally used, see Figure 2.2.

Figure 2.2. Bending reinforcement is placed in the top and in the bottom of the structure and shear reinforcement is placed as vertical stirrups.

The reinforcement can either be placed in radial direction or parallel to the sides, see Figure 2.3, depending on geometry of the foundation, designer's choice and the design of the connection between the tower and the foundation. If radial bars are used, these are supplemented by circular bars in order to provide the same capacity in all directions and to prevent all bars from passing through one point in the centre of the foundation and get continuity over this point.

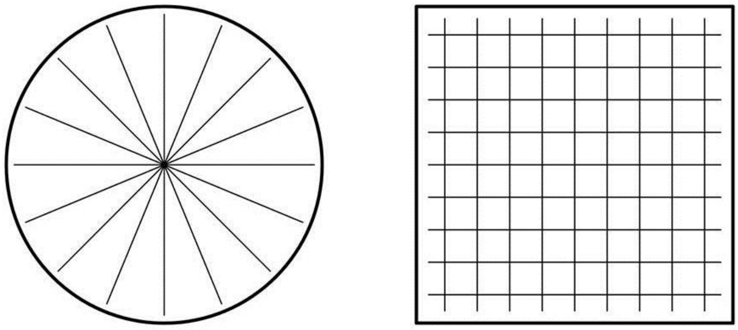

Figure 2.3: Common placement of reinforcement in an onsite cast foundation, a) placed radially in a circular foundation, b) placed parallel to the sides in a square foundation.

2.3 Two dimensional structural analysis of the foundation

The sectional forces in the connection between the tower and the foundation cause stresses that spread in three directions in the foundation. These stresses cause a discontinuity region where beam theory is no longer valid (Landén & Lilljegren 2012). Therefore a three-dimensional strut-and-tie model is a suitable method to design a wind power plant foundation. The strut-and-tie model can take into account the three-dimensional behaviour, and is a lower bound approach in design of cracked reinforced concrete (Engström 2011). This has been investigated in an earlier master's thesis at Chalmers University of Technology.

Designers normally disregard this three-dimensionality in design and instead the foundation is designed considered as a two dimensional beam, where the three-dimensional effects are considered by adaptations of the calculations. This is approximate, however necessary assumptions are made on the safe side in order to ensure the performance of the wind power plant.

2.3.1 Loads and sectional forces acting on the foundation

The wind power plant is exposed to a wind load, which in a simplified manner can be regarded as a distributed triangular load, acting on the whole height of the superstructure, see Figure 2.4.

At the connection between the foundation and the tower, the wind load and the self-weight of superstructure causes sectional forces, see Figure 2.4. These sectional forces are: a horizontal force from the wind, an overturning moment caused by the eccentricity of the wind load, and a normal force from the self-weight of the superstructure (Landén & Lilljegren 2012). The sectional overturning moment is the dominating effect and therefore decisive in ultimate state (Samuelsson 2013-01-28). Another sectional force is a twisting moment due to the variation of the wind load and turbulence (Landén & Lilljegren 2012). This is neglected in this study since the impact on the foundation will be relatively small.

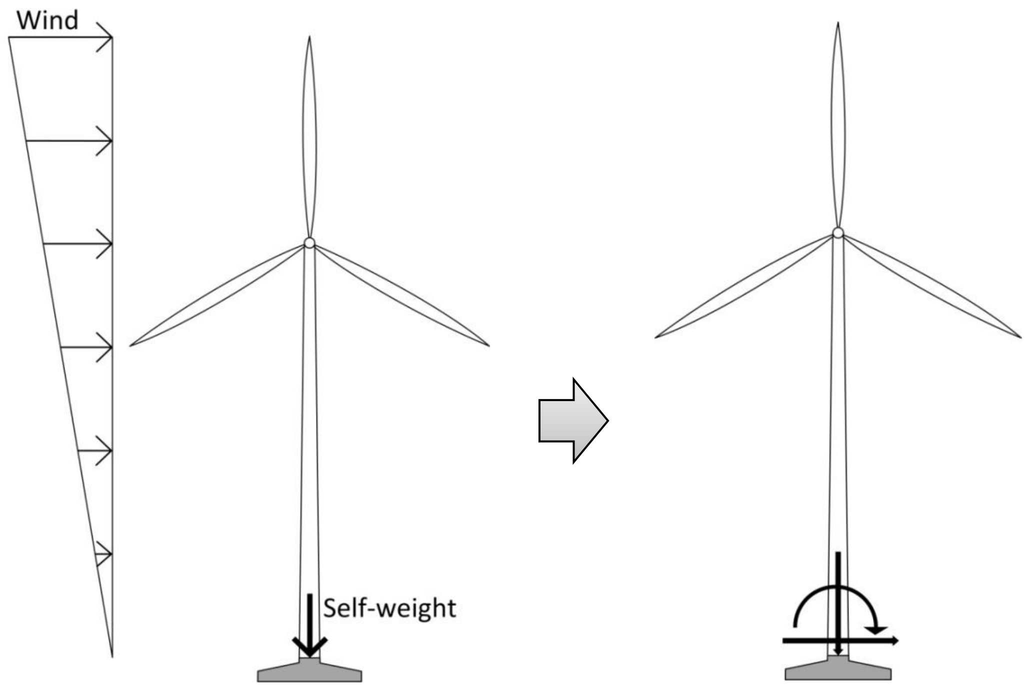

Figure 2.4: The wind load acting on the wind power plant creates sectional forces in the connection between the tower and the foundation, a horizontal force and an overturning moment. The self-weight of the tower creates a vertical sectional force.

The sectional forces are transferred through the foundation to the ground, where they must be resisted by a resulting soil pressure.

The magnitudes of the sectional forces are given by the manufacturer, since the properties of the rotor and the generator have a decisive impact on the design criteria. In the loads specified by the manufacturer, all effects which might increase the load effects are already included (Hassanzadeh 2012).

The wind load is not constant, the foundation is subjected to cyclic loading. This means that besides design in ultimate limit state and serviceability limit state, it is also necessary to consider fatigue. The variation of the fatigue loading is much larger than for other constructions exposed to fatigue loads, such as railway bridges (Samuelsson 2013-01-28). Therefore these effects have a great impact on the sectional forces acting on the foundation, and the design regarding fatigue is of great importance.

2.3.2 The overturning moment as a force couple

As mentioned above, the load effects from the superstructure can be described by sectional forces. The sectional forces result in a stress distribution in the foundation that depends on the material response of the structure.

The overturning moment from the wind causes a stress distribution along the flange of the tower, which in a two dimensional analysis can be regarded as a force couple acting on the foundation, see Figure 2.5. On the windward side the force couple resultant is a tensile force and on the leeward side the force couple resultant is a compressive force.

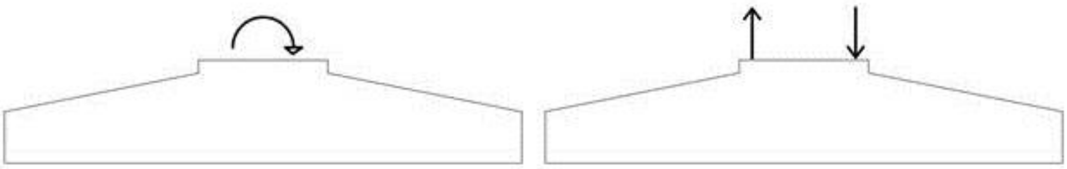

Figure 2.5: *In a two dimensional analysis the stress distribution due to the overturning moment can be described as a force couple, a) overturning moment, b) force couple.*

When considering the force couple resultants of the overturning moment it is important to include the three-dimensional effects in the two dimensional model. To consider the overturning moment as a force couple is a simplification since the overturning moment in reality creates a stress distribution along the circular bottom flange of the tower, see Figure 2.6. The compressive force couple resultant represents the compressive stresses acting along the tower flange on the leeward side, and the tensile force resultant represents the tensile stresses acting along the tower flange on the windward side.

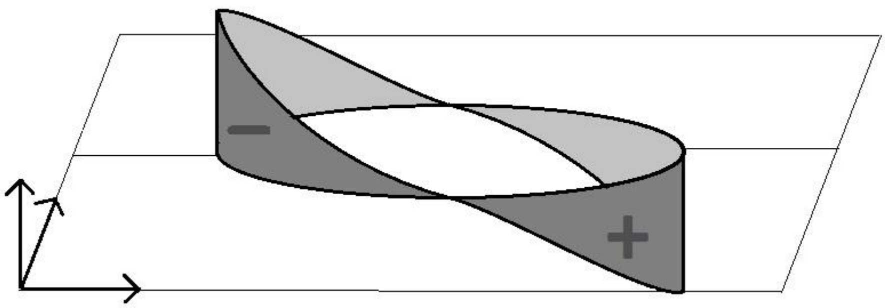

Figure 2.6: *Three-dimensional stress distribution due to the overturning moment.*

The location of the force couple resultants is effected by the stress distribution along the tower flange and the geometry of the flange of the tower. The magnitudes of the force couple resultants represent the sum of all compressive and tensile forces along the bottom flange of the tower.

2.3.3 Soil pressure resisting the loads

In order to make a global analysis of the foundation, the soil pressure must be determined. It is difficult to determine the exact soil pressure distribution and magnitude, why an approximate distribution is assumed.

When only the self-weight of the superstructure is acting on the foundation, the load case when no wind is blowing, the soil pressure will be uniformly distributed under the entire foundation, see Figure 2.7a. When a small wind load is applied, the soil pressure is no longer uniform since it is affected by the overturning moment, see Figure 2.7b. When the wind increases the soil under the foundation starts to plasticise, see Figure 2.7c (Samuelsson 2013-03-18).

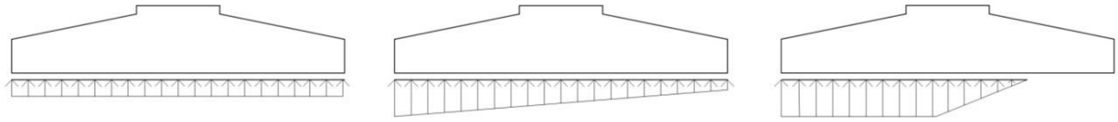

Figure 2.7: The soil pressure under different amount of wind, a) only self-weight, b) small wind load, c) increased wind load - plasticising of the soil.

This variation of the soil pressure, depending on loading, is simplified in design. Different assumptions are made depending on the design situation. In ultimate limit state the soil pressure is uniformly distributed on the leeward side, see Figure 2.8a, an eccentricity is calculated to find the distribution of the soil pressure zone. In fatigue the soil pressure is assumed to act triangular on the leeward side, see Figure 2.8b (Samuelsson 2013-03-18).

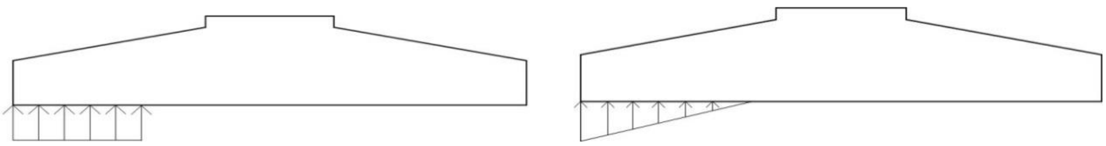

Figure 2.8: The assumptions regarding the soil pressure in the longitudinal direction, a) for calculations in ultimate limit state, b) for fatigue design.

In the transverse direction of the foundation, the soil pressure zone is assumed to be uniformly distributed. In a square foundation the zone would be a rectangle seen from above, and in a circular foundation it would look like a circle sector. This is an effect of the three-dimensional geometry of the structure, which must be included in the analysis (Samuelsson 2013-03-18).

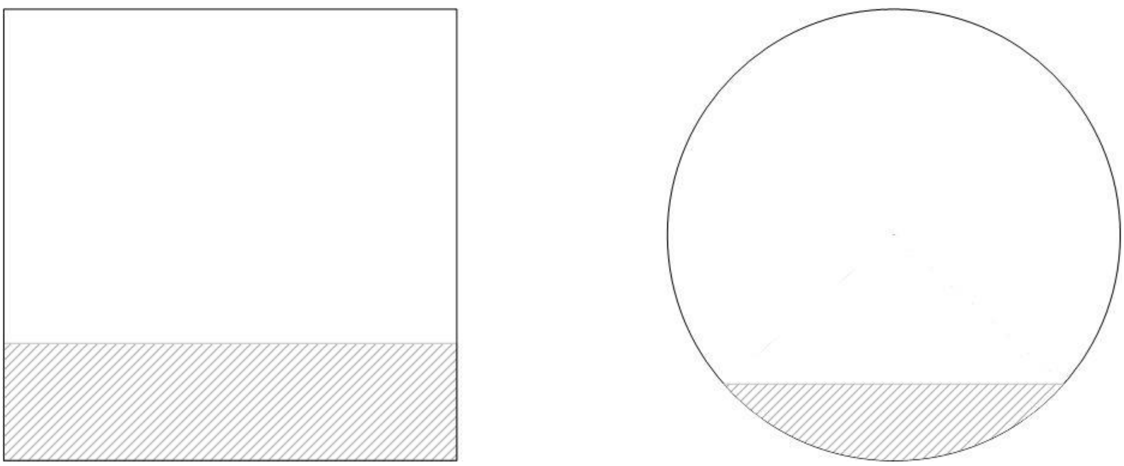

Figure 2.9: The transverse distribution of the soil pressure zone, a) for a square foundation, b) for a circular foundation.

The three-dimensional geometry in Figure 2.9, will affect the distribution of the soil pressure in the two dimensional beam model of the foundation. So even if the distribution is uniform over the area, the distribution might be uneven in the longitudinal direction in the beam model due to the geometrical effects of the area.

2.3.4 Global stability of the two dimensional model

A global stability analysis is performed to verify the global stability of the structure. Globally the soil pressure resultant, q_{soil}, should resist all the forces acting on the foundation and therefore provide sufficient bearing capacity and the dimensions of the structure must be large enough to provide global stability. The forces acting on the foundation are the compressive force couple resultant F_c and the tensile force couple resultant F_t, caused by the wind load. Also the self-weight of the tower, F_z, and the self-weight of the foundation, q_g is acting on the foundation. The two dimensional global model of this system is presented in Figure 2.10. The horizontal force from the wind is resisted by the soil, but these effects are not investigated in this analysis since they are not major.

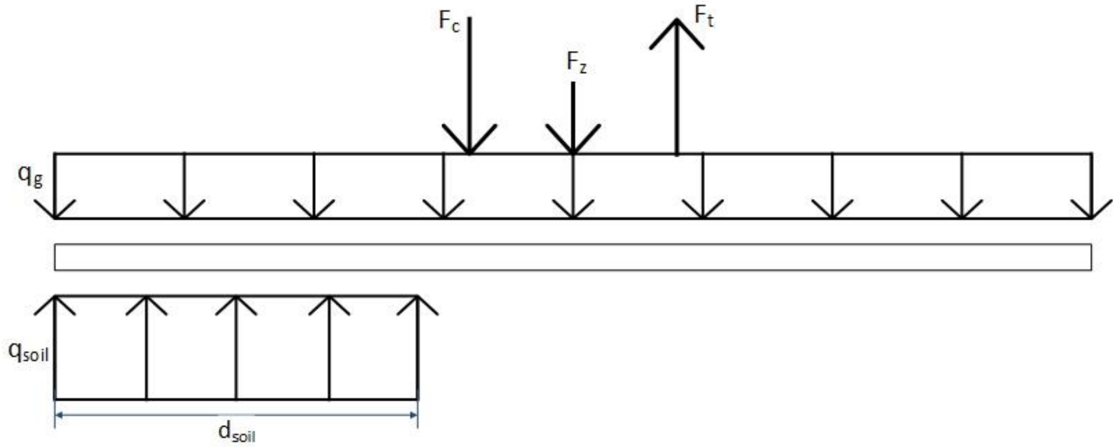

Figure 2.10: The forces acting on the foundation: The force couple resultants caused by the wind load, F_c and F_t, the self-weight of the tower, F_z, and the self-weight of the foundation, q_g. These should be resisted by the soil pressure resultant q_{soil}.

The forces are transferred in the two dimensional model of the foundation by means of a lattice model, see Figure 2.11. This model must be valid in all directions of the foundation, even though the structure is considered as a beam.

In the lattice model the forces are transferred through compressive struts and tensile ties, from the point where the load is applied to the where the soil pressure is acting.

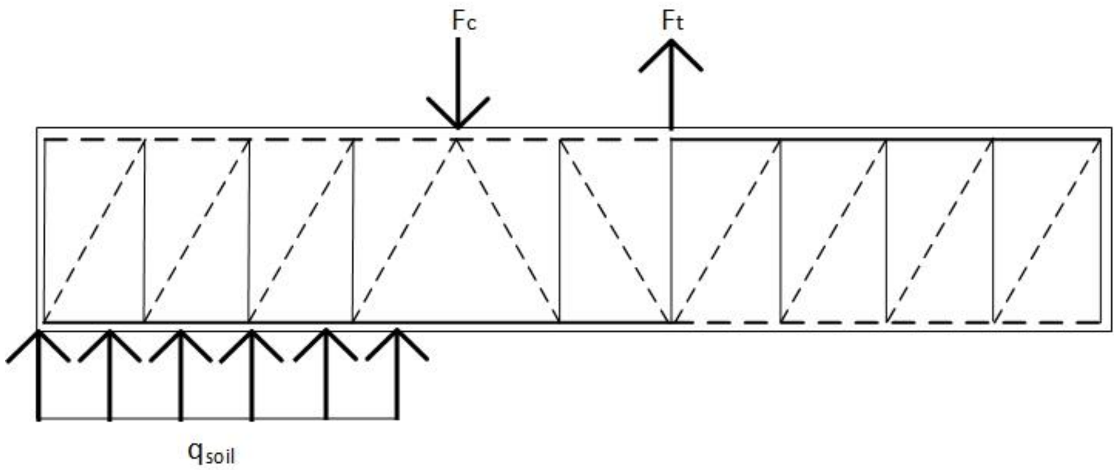

Figure 2.11: Lattice model of the foundation, showing how the force couple is transferred through the foundation when the wind is blowing. The effect of the self-weight of the superstructure is included in the force couple resultants.

When looking on the deformed shape of the beam, the beam will curve upward on the leeward side due to the overturning moment, on the windward side it curves downward due to the self-weight of the foundation. In terms of tension and compression, the upper part on the leeward side will be in compression and the bottom part will be in tension. On the windward side the beam is considered as hanging, cantilevering, and the self-weight results in that the upper part is in tension and the bottom part is in compression, see Figure 2.12.

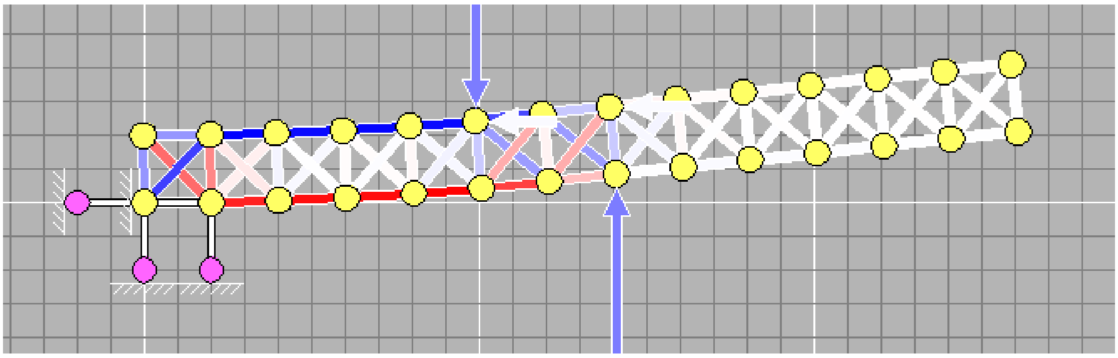

Figure 2.12: The deformed shape of the lattice model of the foundation, including the self-weight. Blue = Compression, Red = Tension. Modelled in Sketch up.

In order to have sufficient tensile capacity, reinforcement is needed in the bottom of the leeward side and in the top of the windward side. Since the wind load acts in all directions, the calculated amount of reinforcement must be placed to resist stresses caused by any wind direction. The tensile stresses on the leeward side are decisive for the bottom reinforcement, whilst the tensile stresses on the windward side are decisive for the top reinforcement. Also stirrups are needed to lift the tensile forces when these are transferred outwards to the soil pressure zone.

2.4 Connection between foundation and tower

An important part of the wind power plant during design, is the connection between the tower and the foundation. The connection must be able to transfer the self-weight of the tower, the overturning moment, the twisting moment and the horizontal force due to the wind, from the tower to the foundation. This function must be fulfilled both in ultimate limit state and serviceability limit state.

The connection has been designed in different ways during the years. A commonly used method has been to use an insert ring cast into the foundation. The insert ring looks like an extension of the tower, a cylinder made of steel, with a bottom flange, see Figure 2.13. Reinforcement bars must be able to pass through the insert ring, therefore holes are made for the reinforcement bars. One problem with this type of connection has been to achieve enough space for the bending reinforcement (Hassanzadeh 2012).

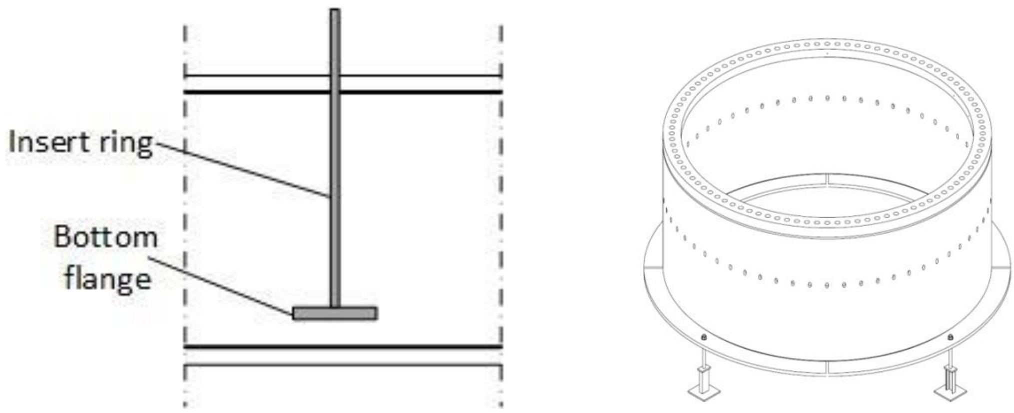

Figure 2.13: Insert ring with the bottom flange placed in the foundation, a) seen from side, b) shown in three dimensions. (Hassanzadeh, M. 2012)

Due to the increase in height of the towers during the years, the stresses in the connection have become very large. High stresses lead to cracking of the concrete, near the insert ring and water can leak into the structure. The consequences of water in the foundation are corrosion of reinforcement and frost damage of the concrete, which affect the bearing capacity of the whole structure (Hassanzadeh 2012).

In order to prevent the problems with the insert ring, another method has been developed which is more suitable. This method is increasingly replacing the method with the insert ring. With the newer method, prestressed anchoring bolts are used to fix the tower to the foundation, see Figure 2.14. The anchoring bolts are fixed in the foundation by an anchor ring. The anchor ring can be placed either in the bottom of the foundation, under the reinforcement, or it can also be placed above the bottom reinforcement. (Hassanzadeh 2012). The anchoring bolts are the method that will be used in this study during design of the prefabricated foundation, so this report will only further explain the function of the prestressed bolts.

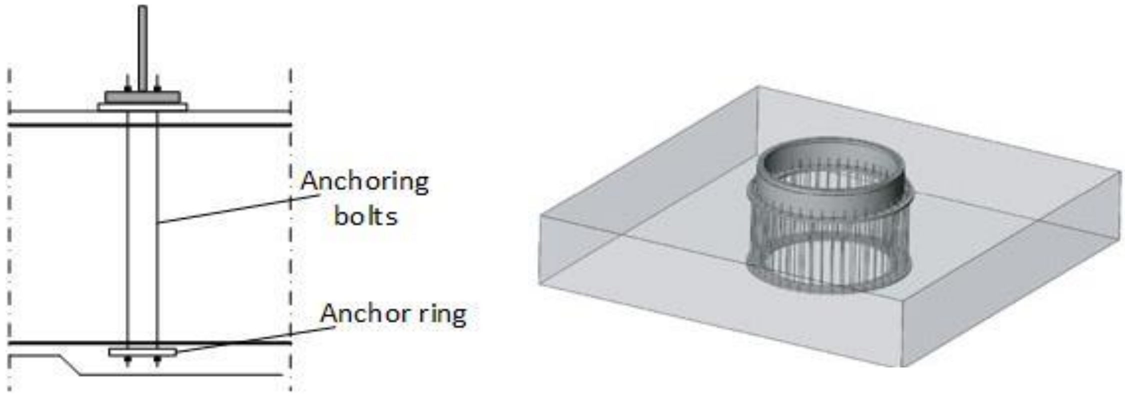

Figure 2.14: Prestressing anchoring bolts and the anchor ring, a) seen from side, b) shown in three dimensions.(Peikko 2013)

When tensioning the bolts, a contact pressure is created in the contact surfaces, both between the bottom flange and the concrete and also between tower and the foundation. Due to the elongation of the bolts, the bolts want to shorten but are prevented by the concrete. This creates a compressive force field in the concrete between the tower and the bottom flange, i.e. along the prestressing bolts, see Figure 2.15.

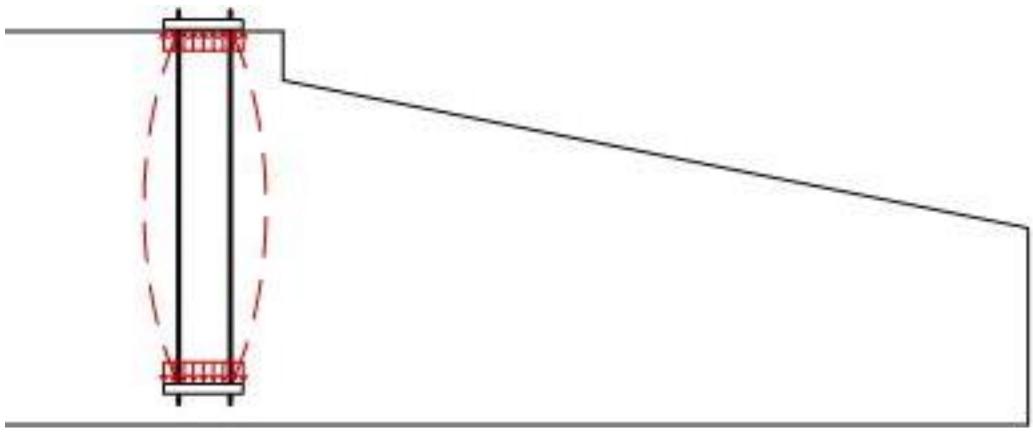

Figure 2.15: The contact pressure caused by the prestressing force and also the compressive force field in the concrete

The initial state, after applying the prestressing force is illustrated in Figure 2.16a. When a compressive force is applied on the compressive stress field, this will increase the upper contact pressure which will be spread according to Figure 2.16b. Correspondingly, if a tensile force is applied, the applied force will decrease the upper contact pressure. If the resultant pressure from the tensile force is equal to the contact pressure, the pressure in the upper part will be zero, while the contact pressure in the bottom part is constant and will be spread according to Figure 2.16c (Engström 2013-03-04).

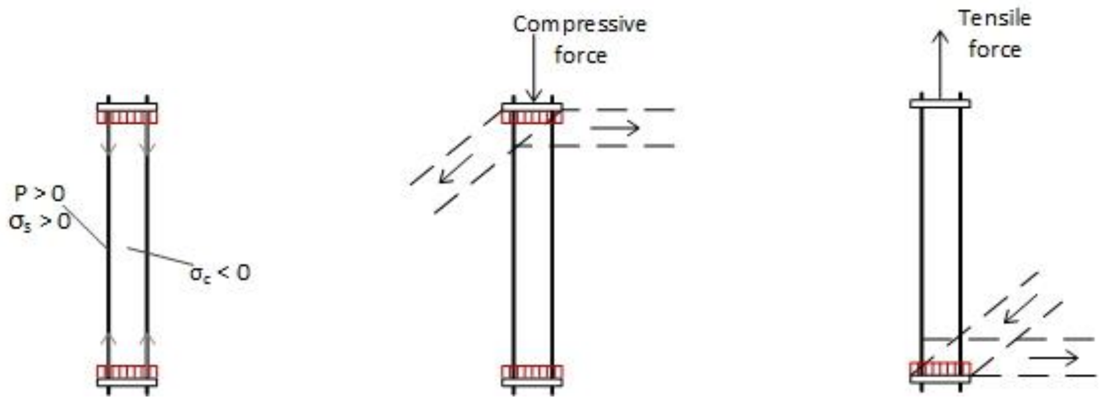

Figure 2.16: Reaction in the concrete due to, a) compressive stress field in the concrete caused by the prestressing bolts, b) when a compressive force is applied upon the prestressing bolts, the stresses are spread from top, c) when a tensile force is applied upon the prestressing bolts, the contact pressure in the upper part decreases, the stresses are spread from the bottom.

When prestressing the bolts in the connection, the fatigue effects on the bolts are positively affected. The bolts are stretched to a certain length, and in order to have relaxation in the steel, the concrete along the bolts must be compressed the same length. Since concrete is a rather stiff material and, in this case, it is a large volume, a very large force is needed to compress the steel to relaxation, which seldom happens. This means that the stress variations in the bolts, due to wind loading, are small and therefore the damage due to fatigue is decreased (Samuelsson 2013-03-11).

3 Prefabrication of Concrete Structures

Prefabrication means that the elements of a structure is produced at a prefabrication plant and transported to the construction site. Prefabrication is a commonly used method when constructing houses, bridges and other structures, and has benefits and drawbacks compared to onsite production. To use this method on a foundation of a wind power plant take a lot of consideration since this is a relatively new area of prefabrication.

3.1 Design of prefabricated elements

To build a prefabricated concrete structure is extensive work; design, manufacturing and construction are steps that must interact with each other and cooperation between the actors is absolutely necessary. This means that during planning it is important to consider all steps of the project. In the initial phase the designer must consider that the design must be suitable concerning production and transportation, if any special devices for lifting and assembling are needed, how to perform joints and connections to make the elements act together as a structure. It is necessary to have everything set before the production of the elements start. For a structure cast on site, the design phase and the construction phase may overlap each other but during prefabrication this is not possible (Bruggeling & Huyghe 1991). Neither can any last-minute changes be made on the construction site, so it is important not to forget any critical details.

> “Prefabrication does not mean to cut an already designed concrete structure into manageable pieces. Prefabrication starts with the first drawing of a project. From the start specialist are needed who are acquainted with all the detail of prefabrication, from manufacturing to finishing of the project.”

(Bruggeling & Huyghe 1991)

By having the production on a prefabrication plant instead of casting onsite will contribute to the industrialisation of the process. At a prefabrication plant, many processes can take place at the same time which makes casting of each element more efficient. If the structure consists of several similar elements, this will make the production more efficient since the same moulds can be used for these castings (Svensk betong 2013). Moulds for precasting can be designed for reusing which is economic if several similar elements should be produced. Therefore it is beneficial to design the elements to be alike. Even though the elements have the same shape, it is also important to consider details in the elements such as ducts for post-tensioning, ducts for installations, protruding reinforcement which will all affect the possibilities of using the same mould.

By prefabricating, the construction work on site is changed from moulding and casting to assembling. It is very important that the connections between the elements are properly performed. It is not only about mounting, but to make all the elements act as a structure where the forces are in equilibrium in an efficient way. Assembling a precast structure is not necessarily a more time efficient process than onsite casting, since it includes much detailed work. However, casting in a prefabrication plant can be done all year and the same applies to mounting precast elements, while onsite casting is governed by the weather and especially the temperature during winter. This means that precasting can extend the construction period especially at cold sites as in the northern of Sweden and accelerate the completion of the structure (Carlström 2013-02-26).

For the quality of the concrete it is favourable to precast in a prefabrication plant. The casting is not affected by the weather conditions in the same way as an onsite cast concrete structure. This makes the concrete more predictable and homogenous, since important parameters such as temperature and relative humidity can be controlled during production. Therefore the concrete and the reinforcement can be efficiently used, and high strength concrete can be used if desired. Also, since the elements can be produced well before delivery they will have enough time to develop their full strength. Compared to onsite casting where the construction must wait for the concrete to gain strength, this is an advantage in construction time on site. However, it is important to consider the detailing and connections when discussing the advantages of the quality of precast concrete elements. Even though the elements themselves are homogenous, it is difficult to make the whole structure homogenous since the connections will be made separately and therefore have different properties. If the connections are made properly and with great consideration, the structure will act properly and transfer the forces in a desired force pattern and the elements will interact.

The existing shapes of the foundation of a wind power plant are all symmetric, which is advantageous when considering producing it in elements since the elements can be similar. It is also common to build several similar wind power plants at a construction site. This means that the advantages with industrialisation of the casting and the opportunities of reusing the moulds at the prefabrication plant can be beneficial.

In a time perspective it is not certain that a prefabricated wind power plant will be faster produced. Not the total construction time and neither the construction time on site are necessarily shorter. For a wind power plant in the northern Sweden, with the drawbacks of that onsite casting is limited to a very short time period due to the winter, prefabrication can still advance the start-up of the energy production. This advance in time is created by the fact that also the winter can be used for production of elements and assembling.

Some other critical issues concerning prefabrication are the vulnerability towards breakage during transport and erection. Also delay in delivery is a critical issue, which might be critical on a construction site with limited construction time. As mentioned above, last-minute changes on the construction site are very difficult. These are all issues that need to be considered already at the initial phase, and actions of how to deal with these risks must be decided and included in the economical calculus.

3.2 Structural connections

In order to obtain interaction between two elements, a connection is needed. Its main purpose is to transfer forces between the elements. The response of the structure is dependent of the connections, their behaviour and their design. The connections enables a global force path and will form an essential part of the structural system (FIB bulletin 43 2008). It is not enough to treat the connections just as a detail in order to assemble the elements at site but to consider the flow of forces.

The structural connection includes the zone where the end regions of two elements are connected to each other, see Figure 3.1. Joint is the denotation of the opening between the two elements, it is normally provided with joint fill.

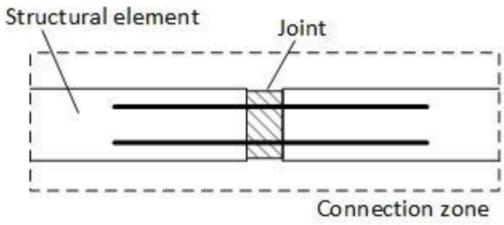

Figure 3.1: Illustration of a structural connection and its parts.

When designing a structural connection the force pattern through the connection must be considered globally including the whole connection and its adjacent structural members. Some design aspects that are of importance during design of connections are: standardisation, simplicity, tensile capacity, ductility, movements and durability (FIB bulletin 43 2008).

During design of the connections between the elements, in order to maintain a good force pattern, these are the main demands that the connection needs to fulfil:

- Transfer tensile and compressive stresses between the elements
- Transfer shear forces between the elements
- Tighten the connection towards aggressive substances in the environment (Marklund & Nilsson 2008)

Performing a connection on a construction site should be simple and not time consuming. Therefore it is not only the strength and design of the connections that are of importance, also the execution is of great importance (Bruggeling & Huyghe 1991).

The connections can be divided into two categories: wet and dry connections. The differences between these are related to the use or lack of concrete in the joint (Bruggeling & Huyghe 1991).

3.2.1 Wet connection

In a wet connection the elements are joined by casting the joint with fresh concrete. Wet connections need time for hardening and curing which effects the construction time on site. These connections are used on a large scale because they are less sensitive for tolerance and more ductile (Bruggeling & Huyghe 1991). Another advantage is that this is a well-known method for the construction worker.

By having protruding reinforcement from the elements it is possible to maintain a sufficient tensile capacity by overlapping of the bars. The elements are mounted with gaps between the elements, the gaps are filled with fresh concrete to achieve interaction between the elements. The protruding reinforcement has tensile capacity and the onsite cast concrete has good compressive capacity in the joint.

One important parameter is the reinforcement anchorage, which must provide enough capacity to fulfil the demands listed in Section 3.2, concerning how to transfer stresses and forces across the connection. In order to achieve a good connection with protruding reinforcement, the reinforcement needs to protrude sufficiently out from the element in order to overlap with the reinforcement from the next element.

Another critical issue with protruding reinforcement is the risk of damage of the reinforcement during transportation and assembling. Also the production of the elements is more complicated, since the moulds must allow the reinforcement to protrude.

3.2.2 Dry connection

A dry connection consists of the joint surface joined by prestressing cables, welds or bolts in order to transfer compressive and tensile stresses. The vertical and horizontal shear forces must be transferred from one side of the joint to the other, in a dry joint, this is often achieved by friction between the elements or overlapping concrete shear keys.

The dry connections between the elements may the bolted. By bolting, the elements are held tightly together, which means that compressive stresses can be transferred across the connection. It is possible to use the bolts as mild reinforcement in order to transfer tensile stresses, which would decrease or even eliminate the need for other mild reinforcement. This means that a bending moment can be transferred across the connection, considered as a force couple with a compressive and a tensile force component. The demands regarding the bolts are high and they must provide sufficient capacity during the service life of the structure.

The connections between the elements may also be pre stressed. By prestressing, the same benefits are achieved as when bolting of the connection is used. But also other beneficial effects are achieved such as crack prevention and a compressive force along the connection. The prestressing force will decrease the tensile stresses since a compressive force is applied.

Prestressing of concrete can be done in three ways: pre tensioning bounded post-tensioning and unbounded post-tensioning. It is not possible to connect two prefabricated elements with pre tensioning, but the two methods using post-tensioning are possible. With post-tensioning, the concrete is cast and hardened before the steel is tensioned, compared to pre tensioning where the steel is tensioned before the concrete is cast. A prestressing force can be applied externally or internally in order to achieve friction.

During tensioning the tendons are elongated. This stretching is free during tensioning, but when the tendons are fastened the steel want to shorten to their original length. The concrete prevents the steel from shorten and thus a compressive stress is created in the concrete.

To use post-tensioning for joining the elements in a prefabricated wind power plant foundation is a method that is investigated in this project. This is a method that is commonly used in bridge design, so called segmental construction. The advantage with this method is that the need for fresh concrete at the construction site is minimized and also a simplification of the demolition process is achieved. It is also possible to use a combination of post-tensioning and cast joint.

One application of post-tensioning is circumferential prestressing of cylinders such as containments, silos and tanks. These consist of precast elements connected to a cylinder by circumferential post-tensioning around the structure. This is an opportunity if the foundation is designed as a solid circular structure.

3.3 Transportation of prefabricated elements

As mentioned earlier, transportation and logistics are very important factors during prefabrication. The precast elements must be delivered to the construction site. The quality of the roads must be sufficiently good due to the heavy transports that will take place. Therefore the roads must be adapted for these transports such as widening, strengthening and removing of obstacles.

When building an onsite cast or precast wind power plant the elements of the superstructure must be transported to the site, often with special transportation. These transports have very large requirements on the quality of the roads. As mentioned earlier in this report, many wind power plants are planned in the northern of Sweden, in areas with sparse traffic, with a total lack of connections sometimes. Existing roads in these areas might be of insufficient quality for these kinds of heavy transports and also there is a risk of damaging the roads. Therefore strengthening, straightening and widening of these roads are often necessary, to cope with the transports without damaging the roads. It might even be necessary to build new roads in order to transport the components to the building site. This has to be investigated in an early phase. The investigation is independently of the choice between precast or onsite cast concrete, since also the turbine and the tower require certain quality of the roads to cope with the transports (Nilsson 2010). However, a precast structure will demand even more heavy transports, but the strengthening of the roads for the superstructure might provide sufficient capacity for these transports, which is beneficial. So the costs for strengthening the roads are already covered.

It is important to optimise the transport of the selected type of precast elements. The size of the different elements is of importance, a very long or heavy element might need special transportation. Economically it is beneficial if the elements are designed to minimise the number of transports. The design must be adapted to the Swedish laws and regulations. Therefore the capacity of a vehicle needs consideration at the design step to optimise the transportation.

3.3.1 Swedish laws and regulations concerning transportation

The public roads of Sweden are divided in bearing capacity classes BK1, BK2 and BK3. The bearing capacity class is decisive for maximum allowed vehicle weight on the road. Roads classed BK1 permit the highest vehicle loads, and applies on 94 per cent of all the public roads (Trafikverket 2012).

The gross weight of a vehicle is the sum of the curb weight of the truck, its trailers and the actual load. The allowed gross weight of a vehicle is dependent on axel distance and what bearing capacity class road the truck shall travel. According to the gross weight table from Trafikverket (2012) for vehicles in goods traffic, the gross weight for a truck must not exceed 60 tons. This is the limit for when special transportation is needed on a road with bearing capacity BK1. Rules also apply for the dimensions of the vehicle, the length and the width, independently on bearing capacity class. The largest permitted width of the truck is 2.6 m and the largest length is 24 m, spread on one or two trailers.

This means that the curb weight is of great importance when it comes to maximum load, which is of interest in this master's thesis project. The curb weight depends on type of truck and trailers, but could be approximately 20 tons, which leaves 40 tons for loading. For a prefabricated element it would be beneficial to transport 2 elements per truck, with a maximum load of 20 ton each and the maximum dimensions 2.5 * 2.5 * 10 m^3.

4 Existing Prefabricated Foundations of Wind Power Plants

Some existing prefabricated wind power plant foundations have already been performed in Sweden as well as in other countries. Some of these projects can be used as inspiration for further development of the idea of prefabricating foundations and therefore presented in this chapter.

4.1 Sjisjka wind park

A wind power plant park is built in Sjisjka in Gällivare municipality. The park consists of 30 wind power plants. The ground conditions made it possible to anchor 6 of these wind power plants to firm rock while the other 24 have gravity foundations, consisting of prefabricated elements. The energy production started in autumn 2012 (Sjisjka vind 2012). These gravity foundations can be a good example for prefabricating wind power plant foundations.

Due to the climate at the construction site casting was only possible during a few months of the year. To shorten the construction a new method to found the tower was developed. The goal was to minimise the amount of onsite casting of the foundation and therefore a precast solution was investigated.

The site at Sjisjka is located in a Natura 2000-area, with a fragile nature. This made the building process demanding and the project included many challenges such as the logistics. Due to fragile surroundings, no permission was given to build roads to the and therefore all transports had to be on the railway, since the only connection to the site was Malmbanan. The concrete elements were transported by truck to Gällivare and then transhipped, from truck to train.

When the planning process started, a lot of different ideas of the design the gravity foundations were under investigation. The elements must be possible to transport to the site, which was decisive for how large the elements can be. In this project the elements are transported both on truck and trains, and the width of the train was decisive for the dimensions of the elements. It was desirable that the elements should have the same geometry and be identical in order to simplify the work at the prefabrication plant and also the assembling work on site.

A difficulty that the designer had to deal with was the centrepiece of the foundation, where the tower is anchored to the foundation. It was desirable to prefabricate this part of the foundation, but it did not become reality in this project. A method with a steel cylinder was investigated, but in the end the centrepiece was cast on site. This led to new challenges concerning connecting the elements to the centrepiece, which was solved by letting reinforcement bars protrude from the element and by casting be integrated in the centrepiece.

Each foundation of the wind power plant consists of 16 prefabricated elements, weighing 18 ton per element. The completed foundation has a diameter of 16 m, a height of 3 m, and the total weight of the foundation was 1350 ton including the weight of the refilled soil (de Frumiere 2012).

Each element consists of a vertical web and a horizontal slab in the bottom, and the elements fit together like pieces of a cake, see Figure 4.1. The centrepiece of the foundation, where the tower is connected to the foundation, was not possible to precast even though this was a wish from the developer. When assembling the structure, the anchoring bolts and the anchor ring was placed, thereafter the elements could be placed around before casting, in Figure 4.1 the assembling is shown.

Figure 4.1: Assembling of the foundation, the elements are attaching to the reinforcement basket.(Sjisjka vind 2013).

To attach the elements to the reinforcement basket, reinforcement is protruding from the elements in order to use for achieving a wet connection. This is inserted into the reinforcement basket before casting. Also the elements were connected to each other with protruding reinforcement and joined by casting, to achieve tensile capacity of the connection. The protruding reinforcement can be seen in Figure 4.2.

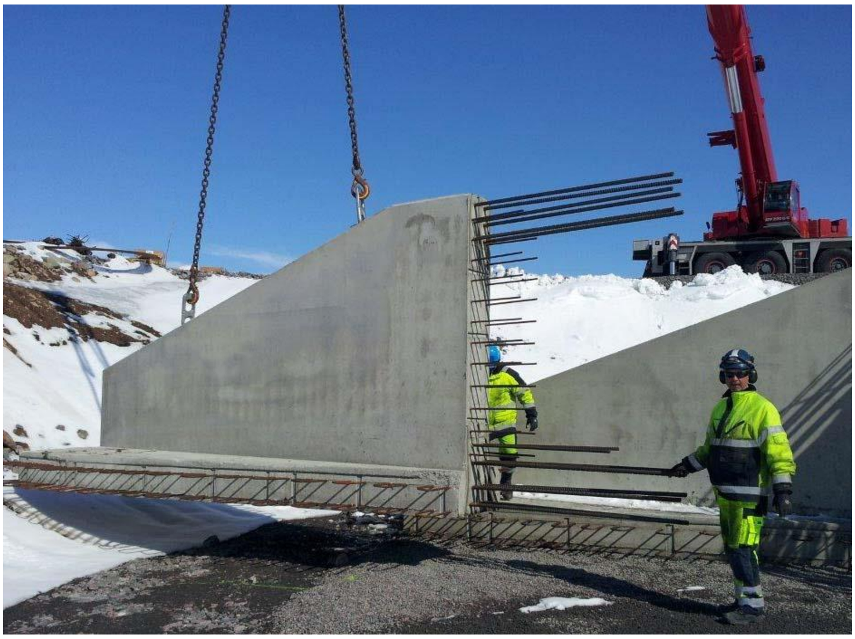

Figure 4.2: One element for the foundation at Sjisjka, which here is lifting on place. Here is also the protruding reinforcement visible. (Sjisjka vind 2013).

The gap between the elements and the centrepiece with the connection was cast on site with fresh concrete from a mobile concrete station. The foundation before casting is shown in Figure 4.3. After casting the onsite concrete, the foundation is covered by the excavated soil which will act as a counterweight for the tower together with the self-weight of the concrete structure.

Figure 4.3: The elements and the moulds, placed at the correct position before casting of fresh concrete can start. (Sjisjka vind 2013).

Skanska estimates that construction time was halved due to this prefabricated solution (de Frumiere 2012).

There are many interesting aspects with this project, there is a clear advantage concerning construction time due to the prefabrication which was a very important parameter at this specific construction site.

With this solution the amount of fresh concrete was decreased, compared to onsite casting, but still quite high. This means that a concrete station was needed at the site and the components of the concrete had to be transported to the site. At this specific site, the transportation of concrete components was especially complicated and expensive. For another site the transportation is not this difficult, therefore this method is not so beneficial on other sites with more advantageous conditions.

The amount of concrete is minimised due to the shape of the elements, which takes advantage of the self-weight of the excavated soil to decrease the need for concrete. This contributes to an optimisation of the material usage which decreases the cost which is beneficial. The moment capacity of the foundation is not affected by this optimisation, since the moment is resisted by the reinforcement and concrete in the elements, while the global stability is achieved by adding the weight of the excavated and refilled soil. This will remove the excess concrete and optimise the use of concrete.

The weakness of this design is that there is still a major part of onsite casting of concrete. This creates additional costs and work. This project has potential to be improved, especially concerning the onsite casting. Therefore it can be an inspiration for concept development in this master's thesis project.

4.2 Star foundation at Bondön, Piteå

At Bondön outside Piteå, a project was performed by the Danish company Global Green (Piteå tidningen 2007). The design of the foundation was performed by Stenger

& Ibsen Construction, whose choice fell upon their product called Star foundation. The reason for this choice was economically, since this method required a smaller amount of concrete. The Star foundation can be produced as prefabricated elements, but in this project they chose to cast the elements on site. This choice was economically, since the company found it too expensive to transport and assemble the elements (Häggström 2013-02-11).

The star foundation consists of twelve triangular sections, supported on a bottom slab, using the excavated soil in order to increase the self-weight, see Figure 4.4 and Figure 4.5.

Figure 4.4: Star foundation at Bondön, onsite cast at this specific site. (Häggström 2013).

Figure 4.5: The star foundations are covered with excavated soil in order to increase the self-weight. (Häggström 2013).

This method is also interesting since it once again shows that the standard method to design a wind power plant foundation can be improved concerning material utilisation. The difference compared to the project in Sjisjka is that at Bondön they chose to cast the elements on site instead of prefabricating, depending on logistic considerations and economy.

So in this project, the star foundation was cast on site, however it is possible to build prefabricated and thereby gain the same advantages as in Sjisjka. However it is not clear how the elements are transported to the site and how the connections are designed. Since they considered this too expensive and difficult at Bondön, perhaps this is not an appropriate solution.

4.3 Modular foundation

In the United States, a system for modular foundation has been developed for smaller and community wind turbines. The system is a precast concept, consisting of square concrete elements that can be assembled on site by prestressing with post-tensioned tendons. According to the manufacturer, the foundation can be delivered and mounted in one day. After the lifetime of the wind power plant, the foundation can easily be dismantled and transported from site (Oldcastle precast 2013).

This method of founding a wind power plant is proven a good method for smaller wind turbines. It might be possible that the system can be enlarged to fulfil the requirements for a larger wind turbine, however it is important to consider the transportation of the elements and the limitations regarding this.

5 Conceptual Design

In order to develop a concept for a prefabricated foundation for wind power plants conceptual design was used. The conceptual design procedure was performed in two steps, an initial phase and an evaluation phase. The working procedure is illustrated in Figure 5.1.

The prestudy which is presented in Chapter 2 to Chapter 4 was used as in-put data for the initial phase which generated promising concepts. The promising concepts were in-put data for the evaluation phase which should generate a final concept and recommendations.

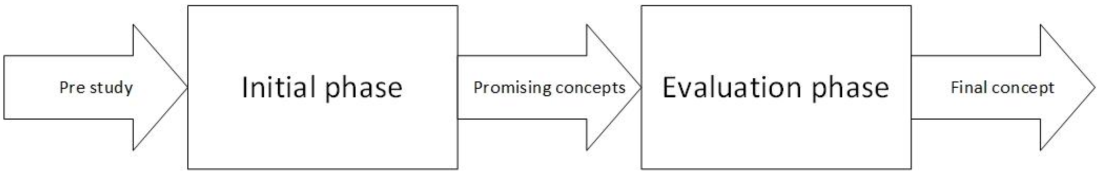

Figure 5.1: Working procedure of the conceptual design process, consisting of two phases.

There are verifications and demands concerning Eurocode that had to be fulfilled during detailed design of a wind power plant foundation. In the initial phase some of these verifications were neglected since this phase was just a preliminary design phase.

5.1 Demands and verifications for design of a foundation

During design of a wind power plant foundation certain design demands must be fulfilled concerning ultimate limit state, serviceability limit state and fatigue. These demands should be fulfilled during the conceptual design procedure in order to design a foundation that provide satisfying resistance to the applied loads.

When designing the foundation of a wind power plant basis in design is Eurocode and especially EC2 (CEN 2004) “Design for concrete structures” is of great importance. Besides Eurocode, an additional code for design of wind power plants must be considered, IEC 61400-1. This is an international standard for designing wind turbines, the loads given by the manufacturer is based on this standard.

For this project the design of the foundation was mainly determined in ultimate limit state but also serviceability limit state and fatigue should be verified in order to have a fully developed concept.

5.1.1 Ultimate limit state

When designing the foundation of a wind power plant the sectional forces acting in the connection of the tower and the foundation must be transferred to the bottom of the foundation in order to have stability. In order to verify this function of the foundation the global equilibrium and the structural resistance must be verified in ultimate limit state.

The ***global equilibrium*** of the whole structure must be verified. The tower must be in global equilibrium in order to avoid tilting of the tower. In this project the global equilibrium was verified during the initial phase, in the global stability calculations.

Concerning the ***structural resistance*** the structure must fulfil the requirements from Eurocode 2 (CEN 2004) concerning compressive and tensile stresses in all sections. The stresses in the compressive struts must not exceed the compressive strength of the concrete and the stresses in the tensile ties must not exceed the tensile strength of the reinforcement. In this project these verifications were performed in the evaluation phase.

Concerning the ***fatigue effects*** it is necessary to verify the long term effects due to the cyclic loading of the wind. The resistance to fatigue of the foundation should be verified. The verification should be performed separately for concrete and steel according to Eurocode 2 (CEN 2004). The fatigue life is influenced by different factors such as load amplitude, number of load cycles, defects and imperfections in the material.

5.1.2 Serviceability limit state

Concerning the serviceability limit state, it is necessary to verify the durability of concrete, crack control and gapping of the foundation. These effects must all be evaluated in the analysis of the critical details of the final concept, and might lead to modifications of the concept.

Stress limitations according to Eurocode 2 (CEN 2004), the compressive stress in the concrete should be limited in order to avoid micro-cracks or high levels of creep. Such cracking may lead to a reduction of durability.

Concerning ***crack control*** the structure should be designed according to Eurocode 2. Cracks occur in all concrete structures that are subjected to bending, shear, twisting or tensioning. This is acceptable and unavoidable, however the cracks must be limited for the structure to maintain its functioning and durability.

Gapping, see Figure 5.2, is restricted by some manufacturers. This is a kind of deflection, where the windward side end is lifted. If this is the case, the maximum lifting of one end of the foundation must not be exceeded.

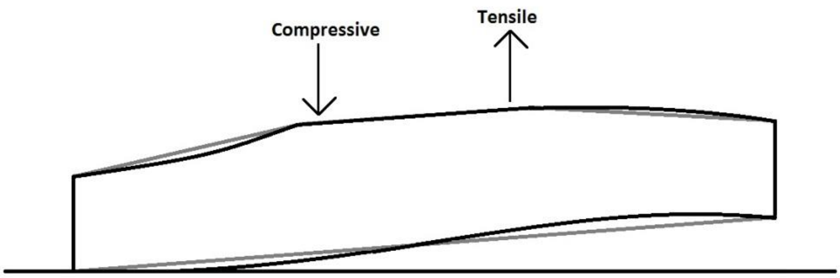

Figure 5.2: Gapping is deflection caused by the overturning moment from the wind.

These verifications are necessary to perform in a detailed designed foundation. However, since this project has focused on concept development in a preliminary design process these effects were not included.

5.2 Working procedure during conceptual design

Before the conceptual design procedure started, a prestudy was performed. The prestudy was started by identifying the task and collecting necessary information from stakeholders. The prestudy resulted in deeper understanding of wind power plants in general, prefabrication in general and experience from former projects. From this a design intention was set up as an in-put to the initial phase of the conceptual design and a specification of goals for the project.

Evaluation criteria were developed based on the design intention. All the essential requirements should be considered by the criteria. When using the criteria they should generate different results when different alternatives were compared.

The design intention and the evaluation criteria were used as background for the concept development and were influencing the evaluation in the initial phase.

The result of the prestudy was the introductory chapters of the report, giving an understanding of the subject. Also the specification of goals and the evaluation criteria were based on the prestudy.

5.2.1 Working procedure of the Initial phase

The information from the prestudy was used as a starting point for the initial phase. In the initial phase a concept development was performed where the purpose was to find a few promising concepts. This was done in an iterative process consisting of generating shapes, global stability calculations, division of the structure into elements and investigation of connections, see Figure 5.3.

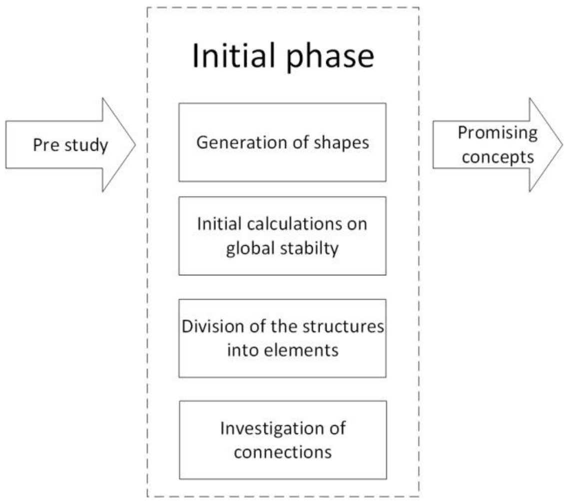

Figure 5.3: The steps in the initial phase, the process was done iteratively. The initial phase resulted in two promising concepts.

The steps in the initial phase were done iteratively. The initial phases was started by generating different shapes of the foundation with inspiration from existing projects and by discussion, see Section 6.2.

All the generated concepts were investigated concerning global stability in order to determine the needed dimensions of the elements, see Section 6.3.

Because of the size and weight of some of the shapes it was necessary to divide them into elements. For a splice between two elements a connection is needed in order to achieve a continuous force pattern. An investigation of which shear force and moment the connections must be resist was done and also some examples of possible connections are given, see Section 6.4.

An evaluation of the concepts was performed by discussion. The discussion was influenced by the design intention and the evaluation criteria but they were not properly used as evaluation criteria in this phase. The evaluation is presented in Section 6.7. From this evaluation two concepts with greatest potential were elected and thereafter further developed in the evaluation phase.

5.2.2 Working procedure of the Evaluation phase

After the initial phase was performed the two most promising concepts were further developed and evaluated. This was done in the evaluation phase where the final concept should be selected and recommendations be made. The working procedure in the evaluation phase is illustrated in Figure 5.4. The primary idea was that the evaluation phase should consist of a further design and analysis of the concepts and an evaluation using the evaluation criteria to choose a suitable final concept. This intention did not fully work, due to lack of possible concepts. However, evaluation and analysis of the concepts were performed iteratively where the results from calculations in each step affected the further calculations.

Figure 5.4: The working procedure in the evaluation phase consisted of analysis and evaluation, which were done iteratively. One final concept should be determined out of the promising concepts.

Preliminary design calculations were already performed for all the initial concepts. Therefore the dimensions of the foundation were already set for all promising concepts. However, they can still needed modifications during the further analysis.

Design calculations were done for the two concepts concerning bending moment and shear force distributions and dimensioning of reinforcement. This analysis is performed in two steps. First what was called a local analysis of legs, one element at a time. Thereafter a global analysis where all legs were included and their effect on the centrepiece was considered.

Different methods to divide and assemble the elements were generated for the promising concepts. The results of the calculations were used to develop the concepts further. The dimensions, the division into elements and the connections were iteratively changed in this phase.

6 Implementation of the Initial Phase

According to the description of the working procedure, the initial phase started with concept development and initial calculations regarding the global stability of the concepts.

6.1 Design intention and evaluation criteria

In order to fulfil the purpose of a prefabricated wind power plant foundation, there are requirements that had to be fulfilled. These requirements were summarised in a design intention for the conceptual design in the master's thesis project:

The elements of the foundation should be possible to transport and assemble into a structure where the elements fully interact, with minimum usage of onsite casting. The structure should have a continuous force pattern in the foundation during service. All this must be fulfilled, but not to an excessive cost.

If the design intention is fulfilled, the designed concept is a good concept. If all demands are fulfilled except for the cost perspective, the concept is possible but not an acceptable solution.

The evaluation criteria were chosen so that the design intention was fulfilled, this is shown in Table 6.1.

Table 6.1: Development of the design intention into evaluation criteria.

Design intention	**Evaluation criteria**
Possible to transport	• Transportation • Production • Material utilisation
Possible to assemble into a structure	• Mounting
The structure should have a continuous force pattern in the foundation during service	• Weak points in the structure
All this must be fulfilled, but not to an excessive cost	

Each evaluation criterion was further developed in order to distinguish them and make them useful in the evaluation. This was important since the criteria should generate different results when different alternatives were compared.

Production

- Possibilities to mass-produce
- Amount of man-hours

To make the production phase more efficient at the precast plant, it is favourable with a concept where all elements have the same shape and are possible to mass-produce.

It is also advantageous if the amount of man-hours during production is decreased, for example if form working and reinforcing are easy to handle. Also the number of critical parts, reuse of moulds and preparation time are factors that will affect this criterion.

Transportation

- Easiness to transport
- Need for special transportation
- Safety

The dimensions and the weight of the element affect the easiness to transport the elements. It is desirable to be able to transport the elements without any need for special transport and therefore it is favourable to avoid these.

Safety is an important parameter in order to prevent the element from damage and protect the construction workers. The shape of the element affects the stability during handling, lifting and transportation.

Mounting

- Amount of onsite casting
- Difficulties with connections
- Amount of man-hours

The amount of onsite cast concrete is considered in the evaluation, since it is costly and time consuming and against the purpose of the design intention. It is also favourable to minimize the amount of concrete transportation to the site.

The difficulty to assemble the structure is an important aspect in order to get a short construction time, especially regarding joints. It is advantageous to minimise the amount of man-hours during mounting.

Material utilisation

It is favourable with a concept that needs minimum amount of material. This is good both in an economical and an environmental point of view. The amount of material will also affect the difficulties in transportation, since a heavy element has higher demands concerning transportation.

Material utilisation is important for concrete due to transportation, but also the amount of reinforcing steel and prestressing steel should be considered.

Weak points in the structure

- Number of splices
- Prestressing steel

The number of critical issues is an important aspect, since these will influence the force pattern which will be interrupted. Therefore it is important to minimise the number of splices.

Critical issues are also the prestressing steel which is more sensitive to fatigue.

6.2 Generation of shapes

From an initial brainstorming process many shapes and designs were generated. Some promising shapes of the foundation were further examined. In order to provide a good attachment for the tower all shapes were designed with a circular centrepiece where the connection between the tower and the foundation is placed.

A traditional way to design foundations for wind power plants is to have a solid structure, either a rectangular, octagonal or a circular structure. The circular and the rectangular foundations, see Figure 6.1, were investigated as prefabricated shapes. The octagonal shape was assumed to have similar properties as the circular shape and therefore not further analysed.

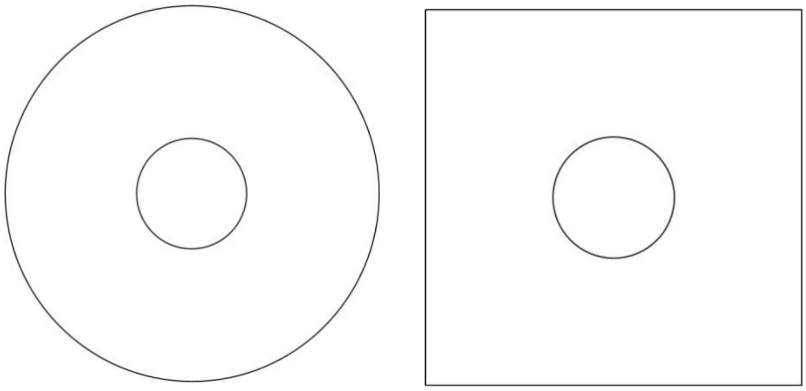

Figure 6.1: Traditional way to design a foundation, a) a solid circular, b) a solid square shape. These shapes was investigated as prefabricated structures in this project.

In order to optimise the material utilisation a shape of the foundation, where legs are placed around the centrepiece, was investigated, see Figure 6.2. The numbers of legs that were investigated were three, four, eight and sixteen legs. It is beneficial to have a symmetric disposition of the legs in order to have similar behaviour in all directions, therefore the shapes with four, eight and sixteen legs were investigated. Also the smallest possible number of legs, three, was investigated due to the interest in seeing the difference compared to the other shapes. Generally it was desirable to have a large variation of the shapes to have better background for conclusions.

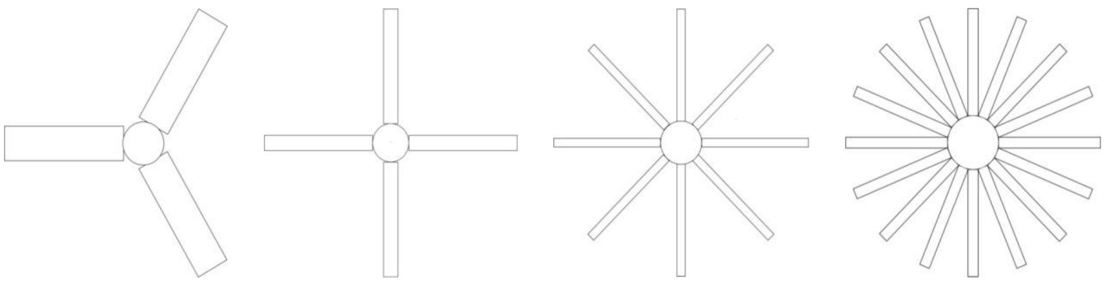

Figure 6.2: The different numbers of legs that were investigated in order to optimise the material utilisation, a) three legs, b) four legs, c) eight legs, d) sixteen legs.

The cross-section of the legs was also varied in order to better compare different structures and to find the most material efficient structure. For the shape with four legs, two variations of the cross-section were examined, one slim structure and one stocky structure, see Figure 6.3. The same variation was done for the shape with eight legs.

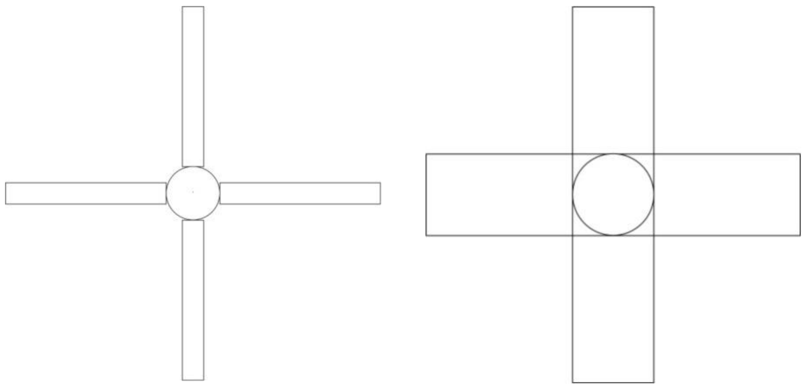

Figure 6.3: The cross-section of the different shapes was varied, a) slim structure, b) stocky structure.

For the shape with eight legs a shape with a t-section was developed, see Figure 6.4. This was done in order to increase the area that can resist the soil pressure and therefore decrease the needed length and the width of the legs. It was found that the shapes with fewer legs needed very large dimensions. Therefore it was decided to only work further on with the shapes with eight legs or more, so the t-section was only assumed for the shape with eight legs.

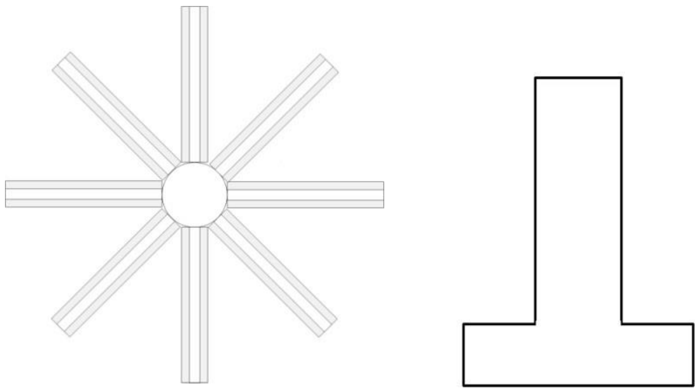

Figure 6.4: A version with a t-section was developed for the shape with eight legs, a) overview of the shape, b) cross-section of the legs.

For the concepts with more than eight legs the flange was instead turned into a bottom slab, since the effective flange width allowed the whole slab to be fully used for resisting the soil pressure. This shape was investigated for sixteen legs, which was then further developed into a shape with twenty legs, see Figure 6.5. This was not done for the concept with eight legs, since the distance between the legs is too large, therfore it was not possible to use the whole slab in order to resist the soil pressure and the t-section was used instead.

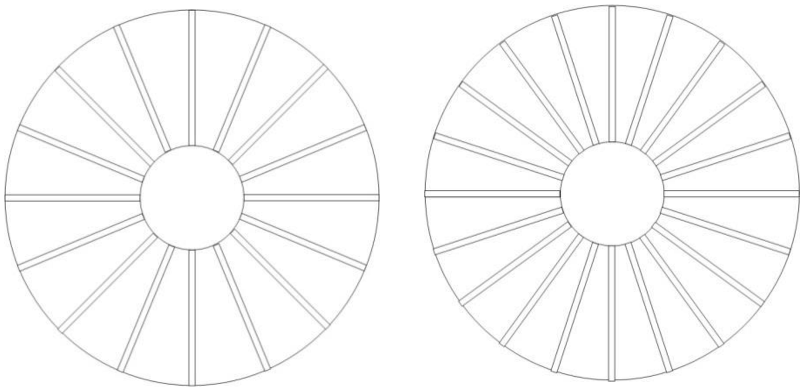

Figure 6.5: *A variant with a bottom slab was developed, a) for sixteen legs, b) for twenty legs.*

Totally eleven shapes were investigated concerning their global stability. The shapes were developed iteratively and some new shapes arose during the process. For example the shape with twenty legs and a bottom slab was a shape that arose during the process.

6.3 Initial calculations on global stability

Initial calculations were performed in order to investigate the shapes concerning the global stability.

6.3.1 Background for calculations on global stability

For the calculations of the global stability the loads were approximated for a fictitious tower. The fictitious tower had a height of 100 m and the sectional forces acting on the foundation were chosen based on experience (Samuelsson 2013-04-02): overturning moment M_d, 100 000 kNm, normal force N_k, 3000 kN and shear force H_d, 1000 kN. The strength of the soil σ_{Rd}, was assumed to be 1000 kPa. It was also assumed that the soil pressure has a uniform distribution in the soil pressure area, see Figure 2.8.

The self-weight of the foundation and the fill G_k was included in the calculations of the global stability. This means that in order to have the correct self-weight, the dimensions of the foundation needed to be assumed and iteratively changed to achieve global stability.

The load effects which are normally given by the manufacturer of the wind turbine are delivered as design values with partial safety factors included, except for the vertical force which has its characteristic value. The reason is that when the self-weight is favourable, on the windward side, the partial safety factor, γ, should be 1.1 and when the wind load is unfavourable, on the leeward side, the partial safety factor should be 0.9 according to IEC 61400-1.

All the developed shapes had the same foundation height as in-put data. The height was decided to be an approximate mean height of 2 m, which was an assumption based on experience. The length and width of the elements for each shape were iteratively changed in order to fulfil global stability.

All developed shapes had a centrepiece that provides an attachment area for the tower of the wind power plant to the foundation. The main purpose of the centrepiece is to provide a good connection between the tower and the foundation, since the loads from the tower must be transferred to the foundation properly. All the shapes consist of a circular solid centrepiece where the connection between the tower and foundation were placed and the method with prestressing bolts was assumed for this connection. The centrepiece was assumed to be 5 m in diameter and have a height of 2.5 m. The total weight of the centrepiece is 117 ton. These dimensions were chosen based on experience from existing examples and from supervision.

In order to simplify the calculations the prestressing forces in the bolted connections were disregarded in the global stability calculations, since they do not affect the global stability. However it should be included in the later structural resistance analysis in order to investigate the risk of crushing of concrete under the tower.

Independently of how the foundations were divided into elements, these were assumed to fully interact as one solid foundation in the global stability calculations.

6.3.2 Global stability methodology

In order to evaluate the shapes in the initial phase calculations were performed for the global equilibrium of each shape. The foundation must prevent the tower from tilting, which was verified by calculations on the global stability from the soil pressure resisting the overturning moment. These calculations are presented in Appendix I and the results give guidelines of whether the shapes are realistic or not.

In order to verify the global stability the soil pressure had to be determined. When calculating in ultimate limit state a uniform soil pressure was assumed, according to Figure 6.6. The overturning moment from the wind load is globally resisted by the soil pressure, σ_{soil}, and its eccentricity, e.

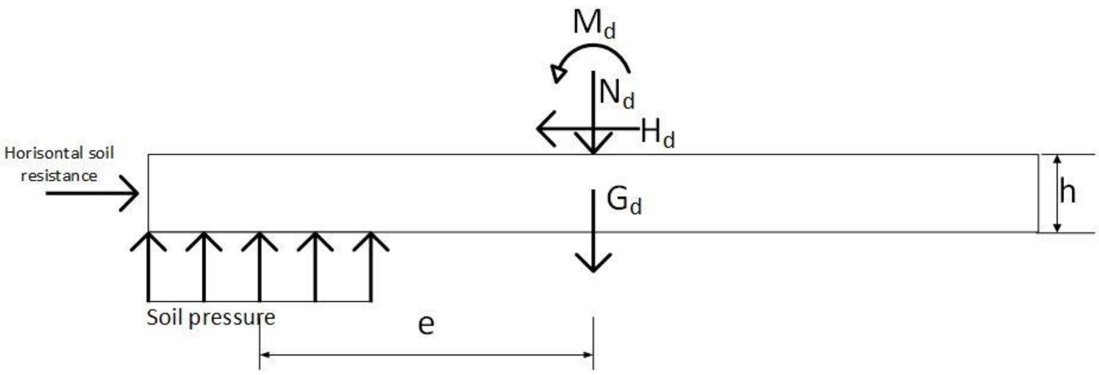

Figure 6.6: Global model to calculate the eccentricity.

The eccentricity was calculated by moment equilibrium around the resultant of the soil pressure, according to Equation 6.1. M_d is the overturning moment due to wind load, H_d is the horizontal force from the wind, N_d is the self-weight from the tower and G_d is the assumed self-weight of the foundation and the fill.

$$e = \frac{M_d + H_d * h}{N_d + G_d} \tag{6.1}$$

From the eccentricity it was possible to calculate the area which is subjected to soil pressure. The eccentricity is defined from the centre of the foundation to the centre of the area that is influenced by soil pressure.

The calculation of the soil pressure area is approximate where the eccentricity determines the shape of the soil pressure area, see Figure 6.7 and Figure 6.8.

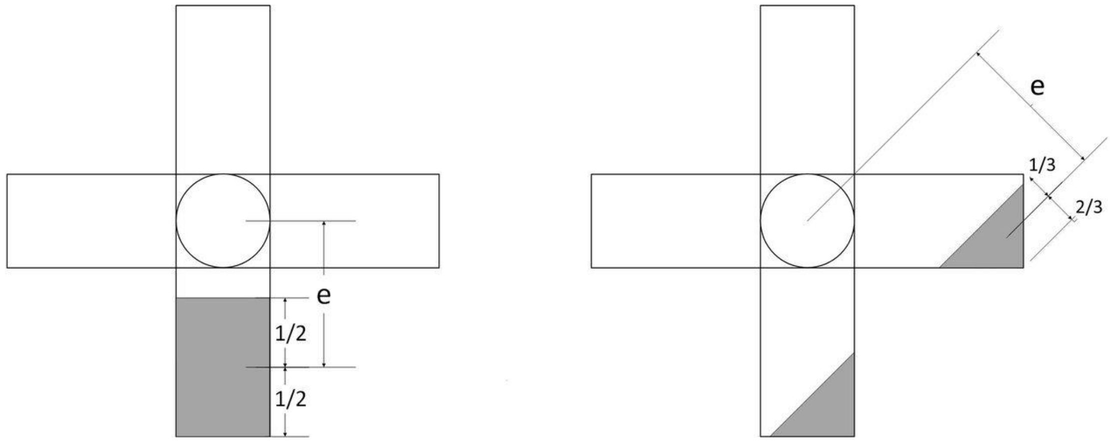

Figure 6.7: All legs that are included in the soil pressure zone, defined by the eccentricity, were assumed to contribute to the soil resistance. The soil pressure area was calculated for two wind directions, a) wind direction 1, b) wind direction 2.

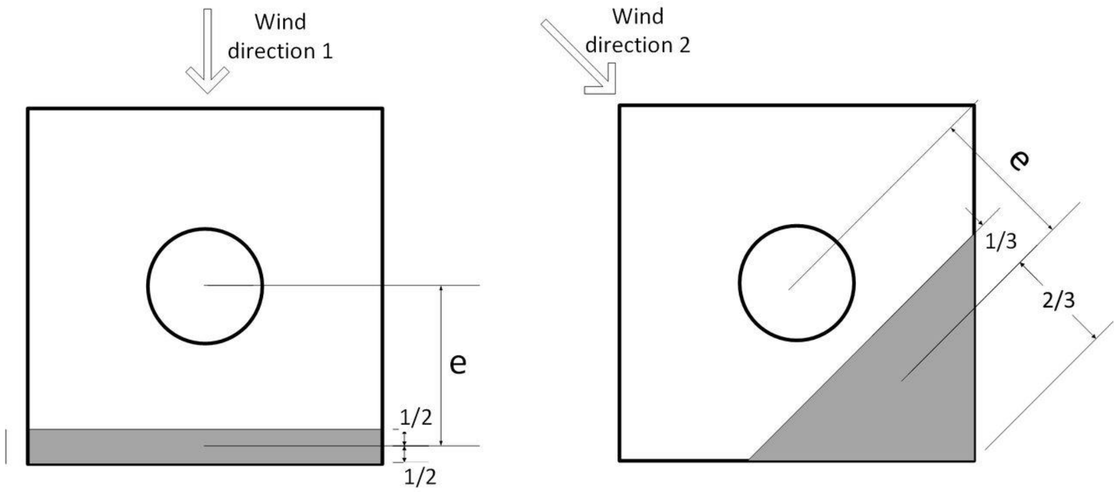

Figure 6.8: For the solid foundation the soil pressure area was defined by the eccentricity. The soil pressure area was calculated for two wind directions, a) wind direction 1, b) wind direction 2.

The area A_{soil}, which is subjected to soil pressure was calculated from the eccentricity and is dependent on the shape of the foundation. The eccentricity should coincide with the centre of the soil pressure area. Thereafter the soil pressure σ_{soil}, was calculated, see Equation 6.2.

$$\sigma_{soil} = \frac{N_d + G_d}{A_{soil}} \tag{6.2}$$

The soil pressure was then compared with the assumed strength of the soil σ_{Rd}, to verify that the soil has enough capacity to resist the pressure from the foundation. If the soil pressure exceeded the capacity of the soil, the dimensions of the foundation had to be changed iteratively to find an acceptable utilisation ratio of the soil resistance.

The global stability was calculated for two wind directions for each shape, see Figure 6.9, in order to capture the resistance of the foundation in the two most extreme wind directions. Exception was made for the solid circular foundation and the shapes with a bottom slab and legs, for which it was only calculated in one wind direction due to symmetry of the foundation.

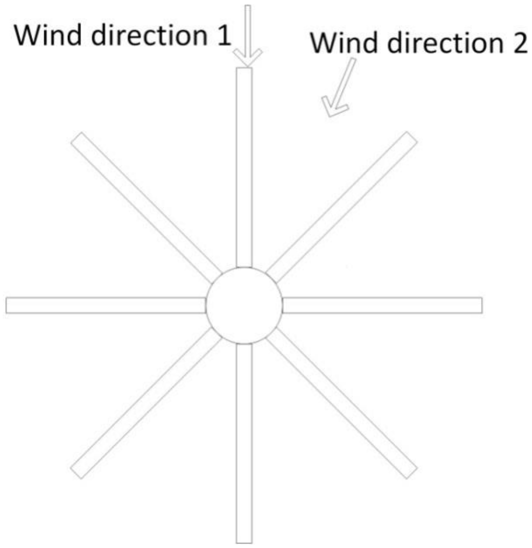

Figure 6.9: Two wind directions were evaluated in the preliminary design of the global stability.

These preliminary calculations resulted in preliminary widths and lengths of the parts of the foundation and the results are presented in Section 6.3.3.

6.3.3 Results from global stability calculations

The results from the global stability calculations are the dimensions and weights of the different shapes. All shapes were able to provide global stability, counteracting the sectional forces from the fictitious tower, but for some shapes the required dimensions were very large.

For the two solid foundations, the square and the circular, the resulting dimensions and weights are presented in Table 6.2.

Table 6.2: The resulting dimensions and weights for the two solid foundations, the circular and square.

	Square foundation	**Circular foundation**
Length of the foundation	15.5 m	16.5 m
Total volume of the concrete	490 m^3	437 m^3
Total weight of the concrete	1250 ton	1115 ton

The needed dimensions, in order to provide global stability for the shapes consisting of a different number of legs around a centrepiece, are presented in Table 6.3.

Table 6.3: The resulting dimensions and weights for the shapes with legs placed around the centrepiece.

	Three legs	**Four legs, stocky**	**Four legs, slim**	**Eight legs, stocky**	**Eight legs, slim**	**Sixteen legs**
Length of leg	14 m	9 m	14.5 m	9.5 m	13.5 m	10 m
Width of leg	4 m	5 m	2 m	2 m	1 m	1 m
Weight of leg	286 ton	229 ton	148 ton	97 ton	69 ton	51 ton
Total volume	390 m^3	410 m^3	280 m^3	350 m^3	260 m^3	370 m^3
Total weight	980 ton	1040 ton	720 ton	900 ton	680 ton	950 ton

For the shapes with a bottom slab or bottom flange interacting with the legs the resulting dimensions, are presented in Table 6.4.

Table 6.4: The resulting dimensions and weights for the shapes with a bottom slab or bottom flange interacting with the legs.

	Eight legs with bottom flange	**Sixteen legs with bottom slab**	**Twenty legs with bottom slab**
Length of leg	10 m	6.5 m	6.5 m
Width of leg	0.8 / 2 m*	0.3 m	0.35 m
Height of slab/flange	0.3 m	0.3 m	0.4 m
Weight of leg	56 ton	10 ton	10 ton
Total volume	200 m^3	170 m^3	190 m^3
Total weight	490 ton	440 ton	470 ton

*Web/Flange

From the initial calculations of the global stability of the foundations conclusion could be drawn that it is possible to achieve global stability for all shapes. However, the dimensions are very large for some of the shapes.

Some shapes were more promising than others due to their geometry:

- A bottom flange or a bottom slab gives smaller dimensions.
- More legs give smaller dimensions.
- Solid foundations give heavier structures.
- Fewer legs give heavier structures.
- Stocky design gives heavier structures.

The best concepts regarding total weight of concrete were the shapes with many legs and specially the shapes with bottom slab and flange. In order to fulfil the demands regarding transportation the shapes need to be divided into elements.

6.4 Division of the structure into elements

Depending on how the structure was divided into elements, different methods of joining them were possible. In order to have properly developed concepts it was necessary to have determined both the division into elements and a well investigated method of how to assemble the elements. The assembling was to be further investigated in the evaluation phase, but the division into elements was examined in the initial phase.

6.4.1 Conditions concerning division

In order to ensure a proper load path from the tower to the foundation it is important to achieve good connections between the elements. This must be fulfilled in all joints in all sections of the structure. However, different sections in the foundation have different demands. Due to the moment and shear force distributions some sections must withstand higher sectional forces. Consequently, some sections are better suited for having a joint. Therefore it was necessary to consider the location of joints during design. For example the intersection between the centrepiece and the outer part of the foundation was a critical section due to the large bending moment in this section. This means that in order to create a good connection in this section it has to be properly designed and properly executed on site.

It is better to have a foundation with few connections. This improves the force pattern in the structure due to the lack of joints between structural elements. However, few joints results in fewer and larger elements. In order to enable transportation of the elements these must not exceed the limitations with regard to their size and weight.

A preliminary division of the outer part was made, based on the limitations regarding transportation, in order to determine a preliminary number of needed elements. The division of the centrepiece was not considered in the initial phase, but these elements were assumed to fully interact as one part. The elements of the centrepiece were later on designed in the evaluation phase

The limiting factors concerning division into elements were weight and dimensions of each element, maximum 20 ton per element and 10*2.5*2.5 m^3. These limitations were based on restrictions given by Transportstyrelsen (2013), the Swedish board of transportation, see Section 3.3.1.

6.4.2 Concepts with solid foundations

The square foundation could be divided into square elements. The circular foundation could be divided as pieces of a cake. Division of concepts with solid foundations are shown in Figure 6.10.

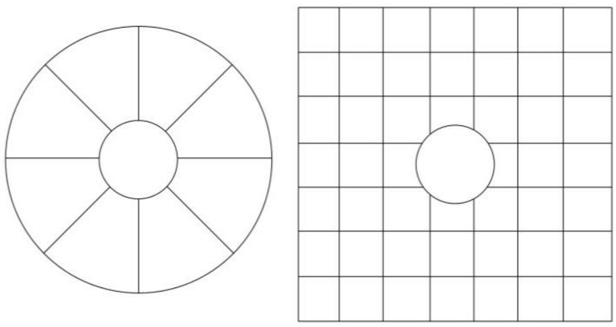

Figure 6.10: Conceptual division of the solid shapes, a) the circular foundation was divided like pieces of a cake, b) the square foundation was divided into square elements.

The resulting dimensions of the elements of the solid circular and square foundations are presented in Table 6.5. The division was executed considering the condition that the elements must not exceed the weight and size limitations given in Section 6.4.1.

Table 6.5: Resulting weights and sizes from the division of solid circular and square foundations.

	Number of elements	**Length**	**Width**	**Weight**	**Evaluation**
Square	64	1.9 m	1.9 m	19.5 ton	OK
Circular	56	6.0 m	0.3- 1.0 m	19.9 ton	OK

The number of elements became large which leads to many joints and a great need for transportation, which are disadvantages with these concepts. This is a consequence of a very high total weight of these concepts, due to the poor material utilisation.

Concerning the square foundation it was impossible to design identical elements and this will lead to extra work during production of the elements. This design issue does not apply on the circular foundation.

Conclusions were drawn that the solid foundations are not suitable for prefabrication and other more material efficient shapes are better.

6.4.3 Concepts with legs

Concerning the concepts with legs it was preferable to avoid division of the legs and to deliver them in their full lengths. This would create a continuous structure and decrease the need of critical joints.

This was investigated for the shapes with three legs, four legs with stocky cross-section, four legs with slim cross-section, eight legs with stocky cross-section, eight legs with slim cross-section, eight legs with a T-section and sixteen legs. The resulting weights and sizes when no division of the legs were performed is presented in Table 6.6.

Table 6.6: Resulting sizes and weights for the concepts with legs, without division of the legs.

	Number of elements	**Length**	**Width**	**Height**	**Weight**	**Evaluation**
3 legs	3	14.5 m	4 m	2 m	286 ton	Not OK
4 stocky	4	9 m	5 m	2 m	229 ton	Not OK
4 slim	4	14.5 m	2 m	2 m	148 ton	Not OK
8 stocky	8	9.5 m	2 m	2 m	97 ton	Not OK
8 slim	8	13.5 m	1 m	2 m	69 ton	Not OK
8 flange	8	10 m	0.8 / 2 m*	2.3 m	56 ton	Not OK
16 legs	16	10 m	1 m	2 m	51 ton	Not OK

*Web/Flange

No division of the legs resulted in very heavy and large elements, which must be delivered by special transportation vehicles. Therefore the intention to deliver the legs in their full size could not be fulfilled and it is recommended to divide the legs into elements.

Two different methods to divide the legs were investigated, longitudinal division along the legs and transversal division.

Longitudinal division gave several thin and long elements, see Figure 6.11. These were designed in order to investigate whether they can be transported without special transportation. This was done by choosing the dimensions so that the maximum weight was not exceeded and the dimensions were verified regarding the size limitations. The resulting weights and dimensions are presented in Table 6.7.

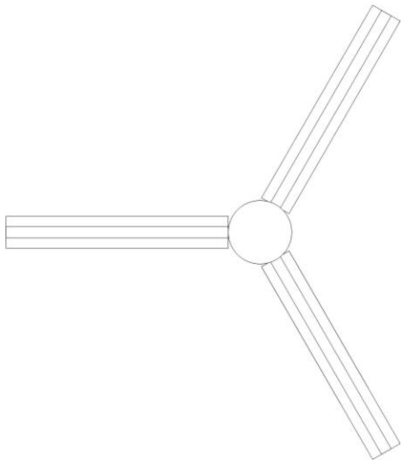

Figure 6.11: Longitudinal division of the legs, illustrated for the concept with three legs.

Table 6.7: Resulting sizes and weights with longitudinal division of the legs.

	Elements per leg	**Number of elements**	**Length**	**Width**	**Height**	**Weight**	**Evaluation**
3	14	42	14 m	0.3 m	2 m	20.4 ton	Not OK
4 stocky	11	44	9 m	0.5 m	2 m	20.7 ton	OK
4 slim	7	28	14.5 m	0.3 m	2 m	21.1 ton	Not OK
8 stocky	5	40	9.5 m	0.4 m	2 m	19.4 ton	OK
8 slim	3	24	13.5 m	0.3 m	2 m	22.9 ton	Not OK
8 flange	3	24	10 m	0.3 m	2 m	18.7 ton	OK
16	3	48	10 m	0.3 m	2 m	17.0 ton	OK

With this method the maximum weight could be fulfilled for all concepts. However the lengths of some concepts were too large.

The elements could also be divided in the transversal direction giving several short and thick elements, see Figure 6.12. In the same manner as for the longitudinal division, the transversal division was made to fulfil the weight limitations. The resulting weight and dimensions are presented in Table 6.8.

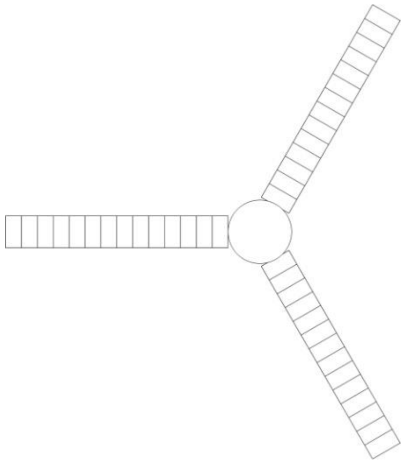

Figure 6.12: Transversal division of the legs, illustrated for the concept with three legs.

Table 6.8: Resulting sizes and weights with transversal division of the legs.

	Elements per leg	**Number of elements**	**Length**	**Width**	**Height**	**Weight**	**Evaluation**
3 legs	14	42	1 m	4 m	2 m	19.9 ton	OK
4 stocky	11	44	0.8 m	5 m	2 m	20.9 ton	OK
4 slim	7	28	2.1 m	2 m	2 m	21.1 ton	OK
8 stocky	5	40	1.9 m	2 m	2 m	19.4 ton	OK
8 slim	3	24	4.5 m	1 m	2 m	22.9 ton	OK
8 flange	3	24	3.3 m	0.8/2m*	2.3 m	18.7 ton	OK
16 legs	3	48	3.3	1 m	2 m	17.0 ton	OK

* Web/Flange

Conclusion were drawn that when the legs were not divided, all concepts consist of elements with weights and dimensions larger than the limitations. This would require special transportation of the elements. On the other hand the lack of joints gives continuity of the legs, which will provide a good resistance with regard to the bending moment and shear forces. However the elements must still be connected to the centrepiece and therefore a joint is needed. So having the legs in their full sizes will not be the whole solution of the connections.

Dividing the elements longitudinally could be beneficial concerning sectional forces, since the leg is continuous in the direction of the bending moment. The reinforcement can easily be placed in this critical direction without interruption or jointing. The need of transversal tensile capacity is small so the need for continuous reinforcement in this direction is less critical.

Dividing the elements transversally makes it difficult to achieve enough bending capacity in the longitudinal direction, since the reinforcement cannot be continuous in the leg in this direction and joints are needed. It is easier to provide transversal bending capacity. However, the need of bending capacity in this direction is smaller than in the longitudinal direction.

In order to divide the legs into elements with a maximum weight of 20 ton, many of the concepts needed to be divided in a large number of elements. This high number of elements also leads to a high number of joints, which is a drawback. The joints must be able to resist bending moment and shear forces. With transversal division the connection must have a high tensile capacity, since the longitudinal bending moment must be resisted in the connections. For the longitudinal division the connections are less critical, since the elements are continuous in the critical direction of the bending moment.

The division of the structure into elements was mainly based on the limitations of the weight. Therefore some element divisions exceeded the limitations regarding their dimensions.

6.4.4 Concepts with webs and a bottom slab

The concepts with webs attached to a bottom slab were divided like pieces of cake so that each element contains a web with a triangular bottom slab, see Figure 6.13. This choice was based on investigation of different methods, see Appendix I.

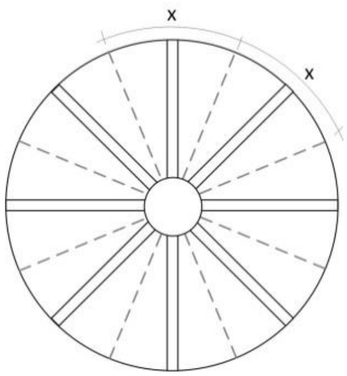

Figure 6.13: The foundation was divided into elements like pieces of a cake.

With this division the joint between the elements will be located in the middle between two webs. If the joint is cast the structure will be continuous, otherwise the splice could be non-cast and the flange considered as cantilevering from the webs. In case of cantilevering the slab must be thicker, but this will result in a decreased number of joints which is beneficial. However, it was considered desirable to have a continuous structure.

For the concept with sixteen webs the elements were too wide and not within the limitations. Therefore a new concept was developed with twenty webs instead.

The resulting weights and dimensions from division of the different concepts with bottom slab are presented in Table 6.9. It was advantageous to have a part of the centrepiece belonging to the legs in order to avoid splices in the maximum moment section. The centrepiece was therefore also divided like pieces of a cake and each centrepiece element assumed to include its corresponding leg. The resulting weights and dimensions for this alternative was also investigated.

Table 6.9: Resulting sizes and weights for the concept with legs and a bottom slab after division into elements, with and without its corresponding centrepiece element.

	Number of elements	Length	Width, x	Weight	Evaluation
16 legs	16	6.5 m	3.5 m	19.7 ton	Not OK
16 + centrepiece	16	9 m	3.5 m	27.5 ton	Not OK
20 legs	20	6.5 m	2.8 m	17.4 ton	OK
20 + centrepiece	20	9 m	2.8 m	23.7 ton	≈ OK

From Table 6.9 conclusion could be drawn that both the sixteen-legged and the twenty-legged concepts were within the limitations of the weight when the centrepiece was not included. When the element also consisted of a corresponding part of the centrepiece, the weight was slightly too high for both concepts, but the concept with twenty legs was assumed to be acceptable.

Considering the dimensions of the elements, neither of the concepts are within the width limitations. However, since this is a preliminary design phase, it was assumed that the concept with twenty legs was acceptable. It was assumed that depending on choice of method to connect the elements the dimensions of the elements might change.

6.4.5 Investigation of the connections

In order to design a good connection in the structure the division of the shapes had to be further investigated and each connection adapted to its location. It was necessary to decide the design of the joint in order to have fully developed concepts.

To design the joints it was necessary to have the moment and shear force distributions in the structure. This was necessary in order to decide the best location for the connection and also to design the joints for the sectional forces they must resist. It was decided that only the two promising concepts, see Section 6.6, should be investigated concerning the connections. Therefore the moment and shear force distributions were developed only for the promising concepts, which was done in the evaluation phase.

6.5 Evaluation by discussion of the initial shapes

Many of the concepts presented were considered possible to produce within the limitations regarding dimensions and weights, but some concepts were more suitable than others. Some of the concepts had to be divided in a very large number of elements to fulfil these limitations. The reason was a poor material utilisation, which gives a heavy structure, for example the solid structures.

The connections between the elements are sensitive details and the more connections the weaker structure, since connections are difficult and expensive to design and construct. An increased self-weight led to an increased number of connections due to the increased number of elements which is unfavourable. An increased number of connections increase the amount of detailed work during assembling. Many connections may also disturb a natural flow of forces in the structure. Furthermore, connections are more sensitive with regard to fatigue. Therefore lightweight structures with a small amount of elements and joints were to prefer. This could be achieved by a more material efficient design of the shape of the foundation. Therefore the concepts with solid foundations were considered not suitable for prefabrication due their heavy structure and many elements.

All shapes with legs without a bottom flange or slab had a rather poor material utilisation compared to the shapes with a bottom flange. Since these shapes had a smaller soil pressure area, they needed larger dimensions to achieve global stability. Larger dimensions give heavier structures. These shapes have a smaller surface area that can utilise the self-weight of the fill to increase the total self-weight. This means that more concrete was needed to achieve sufficient self-weight when no bottom flange or slab was used. Due to the large advantages with a bottom slab or flange it was decided that these shapes are the most promising, why it was chosen to work further on with this kind of shapes.

Based on the arguments concerning weight and number of elements all divisions of the concepts with only three and four legs were eliminated, since these exceed the limitations.

The evaluation also included the wish of not having too similar concepts. The reason was to bring a wider understanding of the structural behaviour of a prefabricated foundation of a wind power plant and to draw conclusions based on different concepts. Therefore the chosen shapes were twenty legs with bottom slab and eight legs with bottom flange, which are further described in Section 6.6.

6.6 Promising concepts

The two promising shapes are presented in their current state after the initial evaluation. The dimensions were set together with the division into elements.

6.6.1 Twenty legs with bottom slab

The first of the promising concepts consisted of twenty elements. Figure 6.14 shows the foundation after assembling of all elements into a structure.

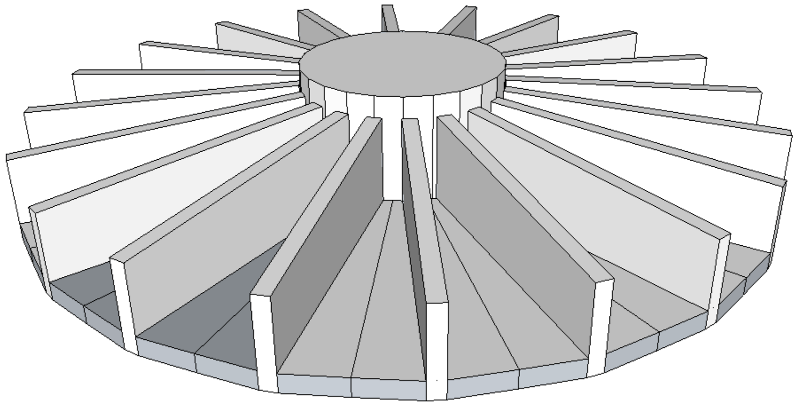

Figure 6.14: Concept with twenty legs and a bottom slab, after assembling.

In this concept the legs did not have to be divided. Each element was assumed to contain a web, a corresponding part of the bottom slab and a corresponding twentieth of the centrepiece, see Figure 6.15. The total number of elements was 20 elements. This division was modified iteratively in the evaluation phase in order to find the best solution.

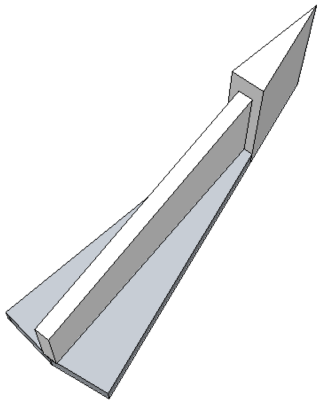

Figure 6.15: Illustration of one element, containing a web, a corresponding part of the bottom slab and a twentieth of the centrepiece.

The preliminary dimensions of the elements were decided in the initial phase and are presented in Figure 6.16. However it was still necessary to modify the concept due to the results in the evaluation phase, since this was an iterative process.

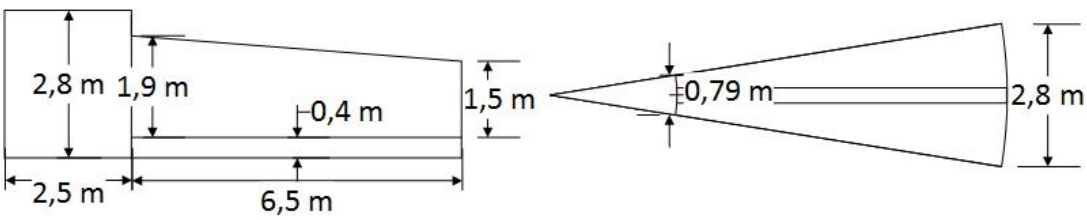

Figure 6.16: The dimensions of the elements for the concept with twenty legs and a bottom slab, a) seen from the side, b) seen from above.

The total weight of one element was 23.7 ton, if the centrepiece and the leg were cast as one element, and 17.4 ton, if the leg was separated from the centrepiece during production and transportation.

6.6.2 Eight legs with bottom flange

The second of the promising concepts consisted of eight legs with an associated bottom flange. Figure 6.17 shows the foundation after assembling. In this concept the legs must be divided into three elements.

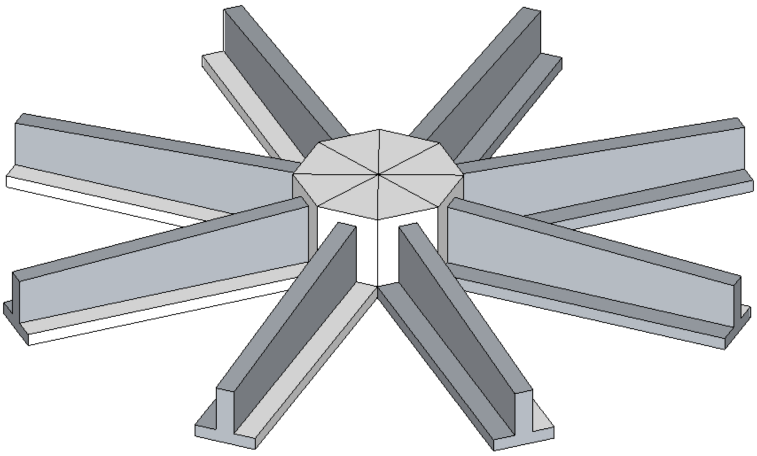

Figure 6.17: Concept with eight legs and bottom flange after assembling.

Each leg with its bottom flange would be too heavy to transport as one element so it had to be transversally divided into three elements. The chosen method for dividing into elements was transversal division, since this was the most appropriate due to the geometry. The inner element contained its corresponding eighth of the centrepiece, see Figure 6.18. The total number of elements was 24 elements.

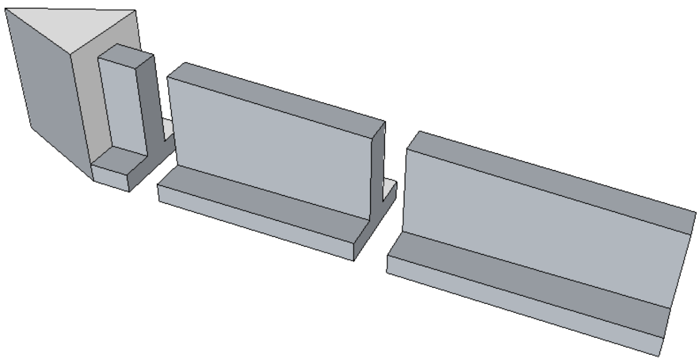

Figure 6.18: Illustration of one leg divided into three elements, each leg containing a web, a bottom flange and an eighth of the centrepiece.

The dimensions of the elements were decided in the initial phase and are presented in Figure 6.19. In the same manner as for the first concept the dimensions and division into elements were iteratively modified in the evaluation phase.

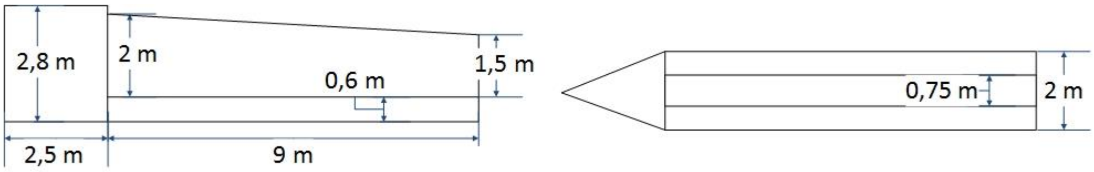

Figure 6.19: The dimensions of the elements in the concept with eight legs and a bottom flange, a) seen from the side, b) seen from above.

The total weight of each element was 18.7 ton when dividing the leg into three elements. If the inner element in the leg contains an eighth of the centrepiece, the weight of each element is in average 23.6 ton. When the centrepiece is attached to the inner element, each element had different length.

7 Implementation of the Evaluation Phase

The promising concepts needed to be further developed and analysed regarding bending moment and shear force distributions before the final evaluation. The analysis was performed in two steps: first a local analysis of each element and thereafter a global analysis of the whole foundation.

7.1 Background for calculations in the evaluation phase

In the evaluation phase the sectional forces were given from the manufacturer Siemens for a wind turbine SWT-2.3-101. The tower has a hub height of 99.5 m, and the sectional forces acting in the connection are: bending moment, M_d, 97 700 kNm, normal force, N_k, 3 600 kN and shear force, H_d, 1 080 kN, see Appendix II.

The sectional forces H_d, N_k and M_d are all applied at the top of the foundation, where the tower is attached to the foundation. Since the task is three-dimensional, but considered as two dimensional during calculation, it was important to modify the in-put data due to geometry. When considering the foundation as a two-dimensional beam, the overturning moment was transformed into one compressive resultant, F_c, and one tensile resultant, F_t, according to Section 2.3.2. The positions of the sectional force resultants were calculated as the centre of gravity of the arcs of each half of the bolt basket (Landén & Lilljegren 2012). This is an approximation, since the actual position of the force resultants should be affected by the stress distribution in Figure 2.6 along the bolt basket. In the approximation the positions of the force resultants are only affected by geometry.

Also the position of the resultant of the self-weight of the tower had to be decided. The self-weight was assumed to be uniformly spread along the bolt basket in reality, but when considering this in two dimensions, it was considered to be acting in three points. Half of the load from the self-weight was assumed to be acting in the middle of the tower and a quarter acting in the same points as each of the sectional force resultants F_c and F_t.

Figure 7.1 shows how half of the self-weight of the superstructure, F_z, acts in the centre of the foundation and how the sectional force resultants, F_c and F_t, act at a distance $\frac{\varnothing_{fc}}{2}$ from the centre of the foundation.

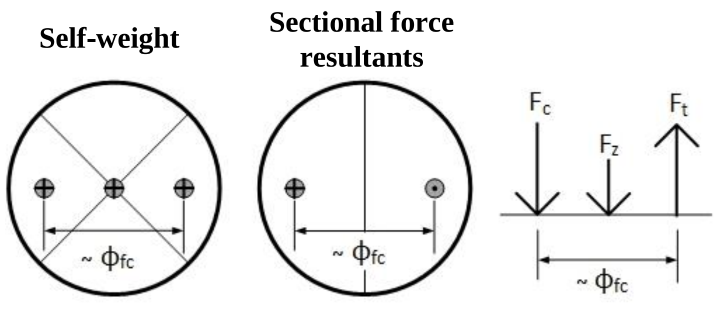

Figure 7.1: The position of the force couple resultants and the self-weight of the tower in a two-dimensional model, a) position of the self-weight seen from above, b) position of the sectional force resultants seen from above, c) the resultants seen from the side

The resulting sectional forces were calculated according to Equations 7.1, 7.2 and 7.3. The parameters in the equations are defined and explained in Section 2.3.1. The normal force was given by the manufacturer as a characteristic value, therefore it is multiplied with the appropriate partial safety factor to achieve the design value for the actual load case.

$$F_c = \frac{M_d}{z} + \frac{N_k \cdot \gamma}{4} \tag{7.1}$$

$$F_z = \frac{N_k \cdot \gamma}{2} \tag{7.2}$$

$$F_t = \frac{-M_d}{z} + \frac{N_k \cdot \gamma}{4} \tag{7.3}$$

The dimensions of the elements were already set in the initial phase and were considered as in-put data for this analysis. Also the model concerning the distribution of the soil pressure was assumed in the initial phase and used as in-put data for the evaluation phase.

In the analysis of the cross-section it was assumed that all elements are fully interacting with each other and the locations of the connections between the elements were not considered. This means that the force pattern was assumed to not be affected by any connections and the whole foundation can be fully used for the flow of forces.

In order to simplify the calculations the prestressing forces in the bolt of the tower-foundation connection were disregarded in the global equilibrium calculation. However it should be included in the structural resistance analysis, in order to investigate the risk of crushing of concrete under the tower.

7.2 Local analysis of the legs

In order to design for structural resistance in the ultimate limit state a local analysis of the outer parts of the structure was performed. In the local analysis one leg was investigated at a time to find the moment, M, and shear force distributions, T, for each such element. The moment and shear force distributions were found and used for determining the needed amount of reinforcement. These calculations are presented in Appendix III and Appendix IV.

The concepts were designed concerning structural resistance such that the requirements in Eurocode 2 (CEN 2004) are fulfilled in all sections. The loads that the foundation is exposed to must not cause any internal flexural or shear failure. The sectional forces were found from the bending moment and shear force distributions.

Any wind direction is possible, why it was necessary to design all legs for the sectional forces from the worst wind direction. The bending moment and shear force distributions were calculated for the most severely affected element, which is the leg opposite the wind, see Figure 7.2.

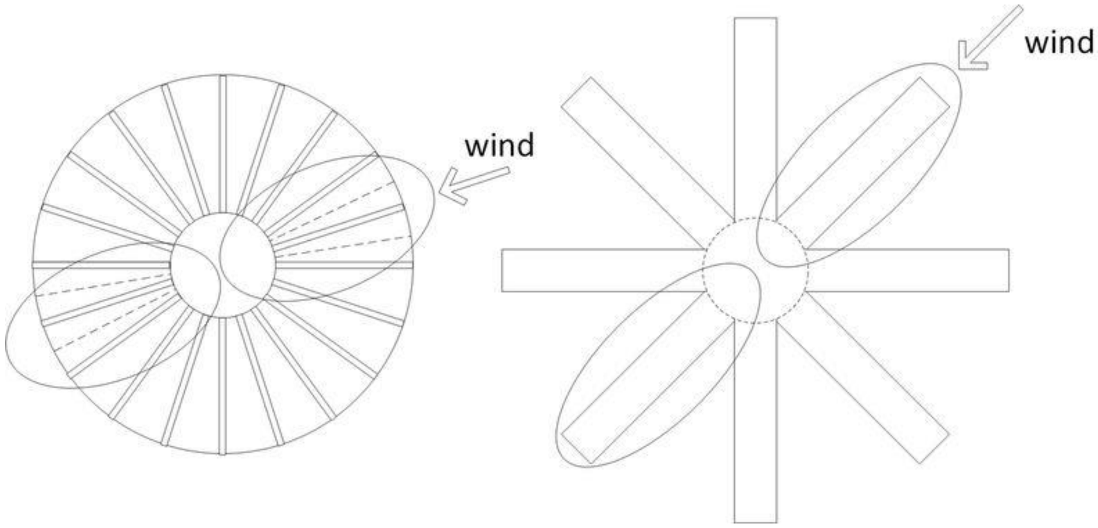

Figure 7.2: *The web element and leg element most severely affected by the wind are decisive for all elements, a) most severely affected element for concept with twenty legs and bottom slab, b) most severely affected element for the concept with eight legs and bottom flange.*

The analysis was performed for the leg and the part of the centrepiece outside the force resultants F_c and F_t, see Figure 7.3. The leg on the leeward side is exposed to a positive bending moment due to the overturning moment and its resisting soil pressure. The leg on the windward side is considered as hanging, exposed to a negative bending moment due to the self-weight of the element itself.

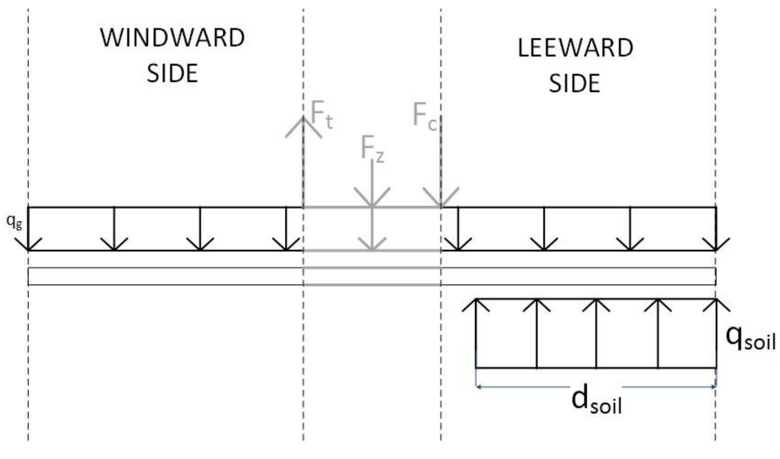

Figure 7.3: *The local analysis considered only the outer sections of the foundation, outside the force resultants F_c and F_t. The middle section was not considered in the local analysis.*

The moment and shear force distributions were calculated along each leg, from the section where F_c or F_t is applied and outwards to the edge of the leg. In order to perform the calculations the leg was divided into transversal strips in order to use vector calculations during the analysis.

The bending moment in each section was calculated as the sum of the load on each strip, q_i, times its lever arm to the section, s_i, according to Equation 7.4. The sum is the bending moment that must be resisted in the section. By compiling the bending moment in all sections into a vector, the bending moment distribution could be plotted.

$$M = \sum q_i * s_i \tag{7.4}$$

The shear force in each section was calculated as the sum of the load, q_i, acting on each strip outside the section, according to Equation 7.5. The sum is the shear force that must be resisted by this section. By compiling the shear force in all sections into a vector, the shear force distribution could be plotted.

$$V = \sum q_i \tag{7.5}$$

From the moment distribution along the element the needed amount of bending reinforcement was estimated. The shear force distribution determines the needed amount of shear reinforcement. The reinforcement calculations were performed in accordance to Eurocode 2 (CEN 2004).

The bending reinforcement was determined dimensioned for the most critical section, where the bending moment is largest. By experience it is known that the most critical section, concerning the bending moment, is the section where the force couple resultants are applied. This was also confirmed by plotting the moment distribution. The bending moment is defined positive on the leeward side and negative on the windward side. The bending moment on the leeward side is decisive for the bottom reinforcement, while the bending moment on the windward side is decisive for the top reinforcement. Since the wind might have any direction, it is necessary to design the bottom reinforcement for the largest positive moment on all sides of the centrepiece and the top reinforcement for the largest negative moment on all sides of the centrepiece.

The maximum moment occurs in Section fc, where the force couple resultants act. The reinforcement was designed for the maximum value. However the reinforcement was arranged with regard to Section cp in order to verify that the reinforcement fits into the web and flange. The sections are defined in Figure 7.4.

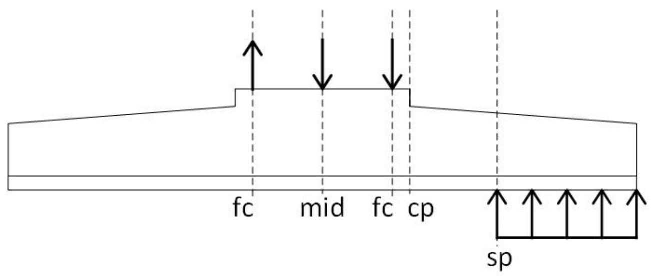

Figure 7.4: Definition of specific sections in the foundation.

Concerning placement of the reinforcement the legs have a constant width. However, the width of the centrepiece element decreases towards the centre, giving a narrower area for placing the bottom reinforcement. It is important that the needed amount of reinforcement fits into all cross-sections of the element, which means that towards the centre of the foundation, it might be too narrow, even though the bending moment decreases.

The resistance of the cross-sections was verified by a sectional analysis in state III with stress block factors for the concrete compressive zone.

The shear force in the leg was verified in two sections: Section fc, where the force couple resultant is acting and in the Section sp, where the soil pressure starts. The most critical section concerning the shear force was found in the section where the soil pressure zone starts.

The maximum shear force was used to decide the needed amount of shear reinforcement in the leg element in accordance to Eurocode 2 (CEN 2004).

7.3 Results from the local analysis

The results from the calculations described in Section 7.2 are presented. The bending moment distribution was calculated for the windward and leeward side separately, one is positive and the other negative and conceptually the distribution will be according to Figure 7.5. The distribution of the bending moment over the centrepiece was not included in the local analysis.

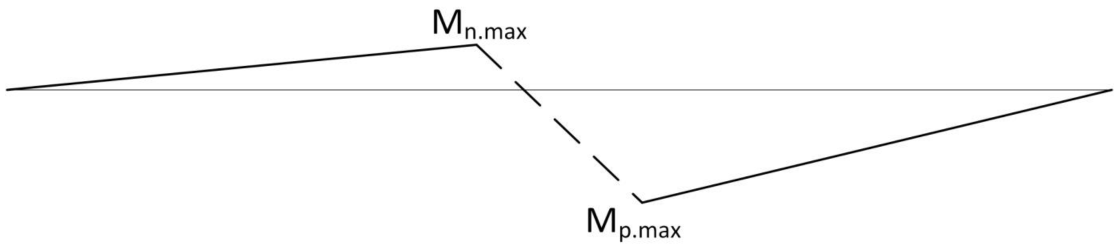

Figure 7.5: Simplified schematic moment distribution. The moment is positive on the leeward side (right) and negative on the windward side (left). The dotted line, in the centrepiece, was not included in the local analysis.

The shear force distribution was calculated in the same way. However, the shear force distributions at the leeward side and the windward side both have the same sign. The distribution of the shear force over the centrepiece was not included in the local analysis. The distribution is conceptually presented in Figure 7.6.

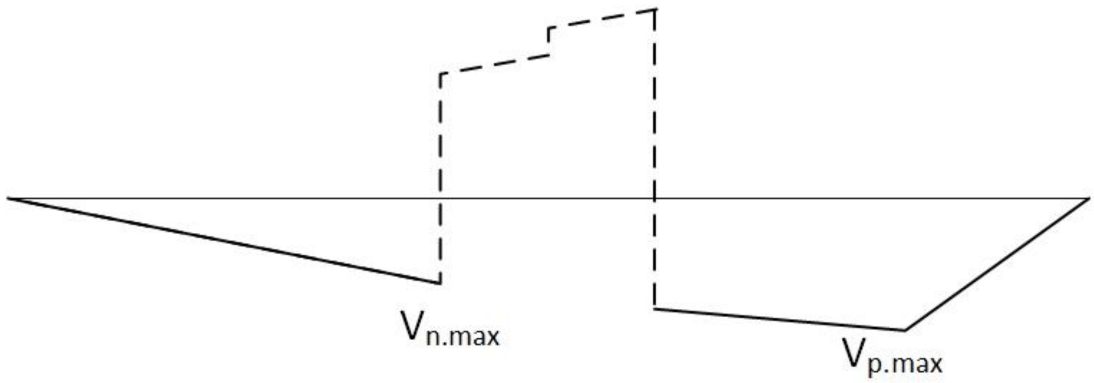

Figure 7.6: Simplified schematic shear force distribution. The shear force have the same sign on the leeward side (right) and the windward side (left). The dotted line, in the centrepiece, is not included in the local analysis.

7.3.1 Twenty legs with bottom slab

When plotting the bending moment distribution it was verified that the critical section concerning the bending moment is Section fc. This section has the largest bending moment in the leg, both concerning the positive and the negative moment. The bending moment approaches zero outwards along the web and towards the centre of the foundation it decreases in order to change sign. Therefore the required moment capacity was calculated for this critical section, since the other sections will have a smaller bending moment.

The results from the analysis of the bending moment distribution are presented in the following. The positive moment on the leeward side is presented in Figure 7.7 and the negative bending moment on the windward side is presented in Figure 7.8.

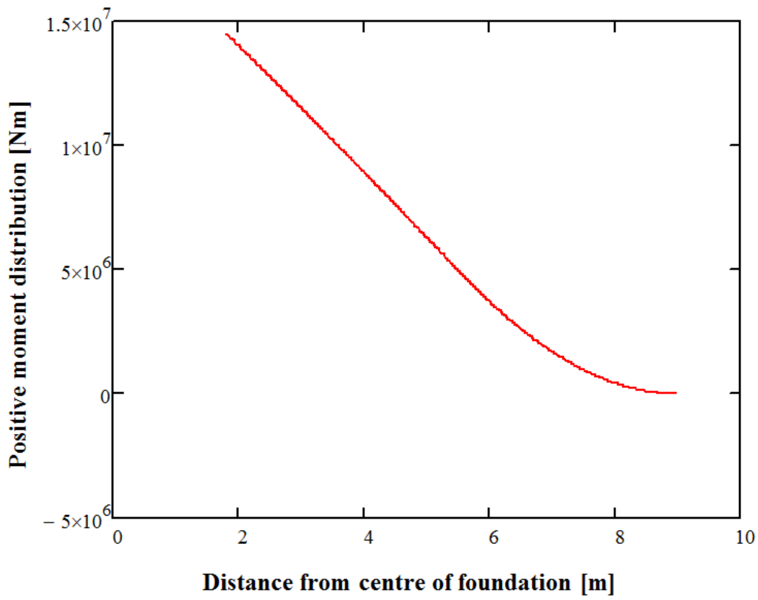

Figure 7.7: Positive moment diagram on the leeward side of the foundation, between Section fc and the edge.

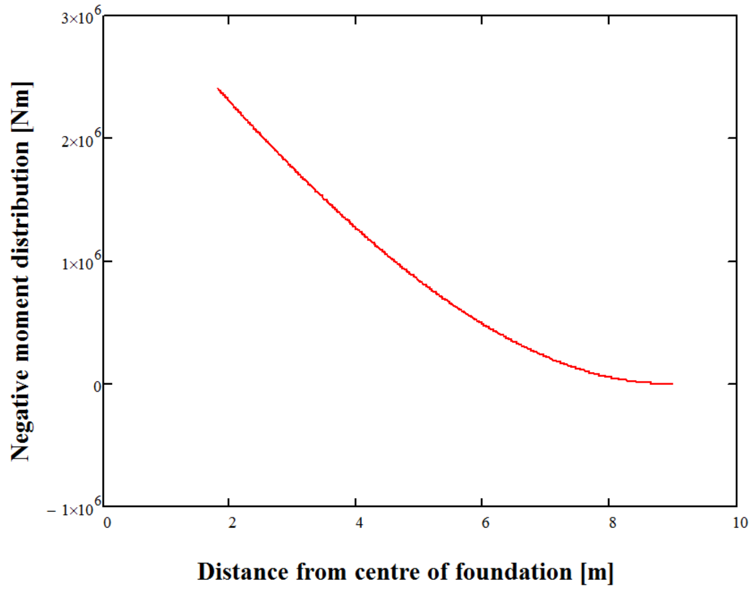

Figure 7.8: Negative moment diagram on the windward side of the foundation, between Section fc and the edge. This figure shows only the magnitude of the moment. It has opposite sign compared to the positive moment.

The positive moment on the leeward side is much larger than the negative moment on the windward side. It is possible to place a larger amount of reinforcement in the bottom flange, which has a cross-section large enough to accommodate more reinforcement than in the top.

The results from the calculations of needed reinforcement are presented in Table 7.1. Calculations are performed for the maximum moment Section fc, but arranged with regard to Section cp.

Table 7.1: Results from the calculations of the required reinforcement with regard to needed moment capacity.

	Bottom reinforcement	**Top reinforcement**
Required moment capacity	14 500 kNm	2400 kNm
Diameter of the bars	25 mm	25 mm
Needed number of bars	34	6
Number of layers	4	2

The chosen arrangement of the reinforcement is presented in Figure 7.9.

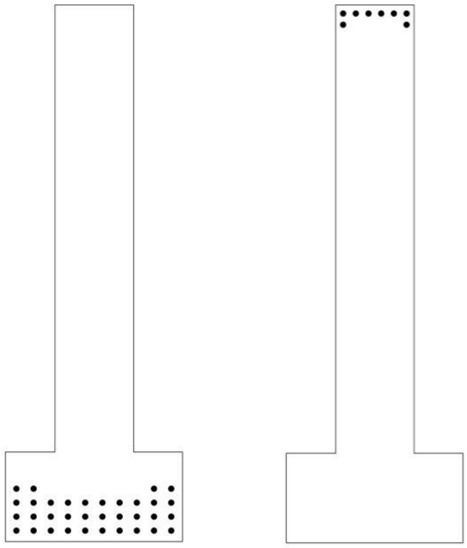

Figure 7.9: Arrangement of the reinforcement in the top and in the bottom of the leg in Section cp, a) bottom reinforcement, b) top reinforcement.

From the shear force distribution it was found that the critical section is Section sp on the leeward side and Section cp on the windward side. The shear force has the same sign on both sides. The shear force distributions in the legs are presented on the leeward side in Figure 7.10 and on the windward side in Figure 7.11.

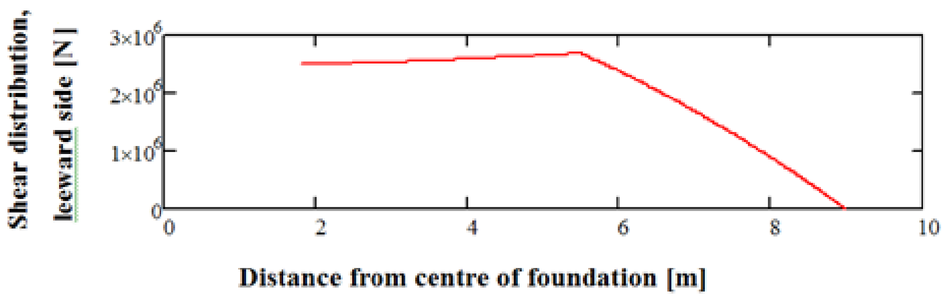

Figure 7.10: Shear force distribution on the leeward side, from Section fc to the edge.

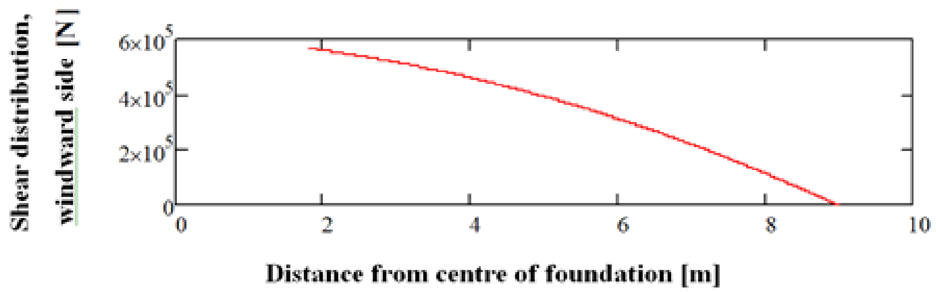

Figure 7.11: Shear force distribution on the windward side, from Section fc to the edge.

The shear reinforcement was determined assuming a strut inclination of 45°. The results from the calculations are presented in Table 7.2. The resulting shear reinforcement is presented as steel area per unit length.

Table 7.2: Results from the calculations of the shear reinforcement, at Section fc and Section sp.

	Section fc	**Section sp**
Required shear force capacity	2 490 kN	2680 kN
Required shear reinforcement area per unit length	2 800 mm^2	3350 mm^2

7.3.2 Eight legs with bottom flange

The same calculations as for the concept with twenty legs was performed for the concept with eight legs. It was verified that also for this concept the maximum moment occurs in Section fc. Therefore the required moment capacity was calculated for this section, since the other sections will have a smaller bending moment.

The results from the analysis of the bending moment distribution are presented in the following. The positive moment on the leeward side is presented in Figure 7.12 and the negative bending moment on the windward side is presented in Figure 7.13.

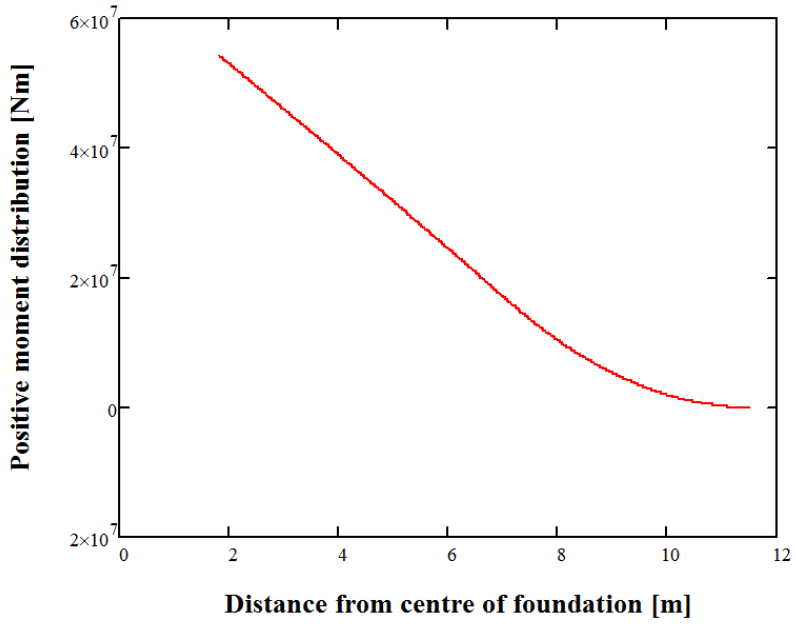

Figure 7.12: Positive moment distribution in the leg elements, on the leeward side, between Section fc and the edge.

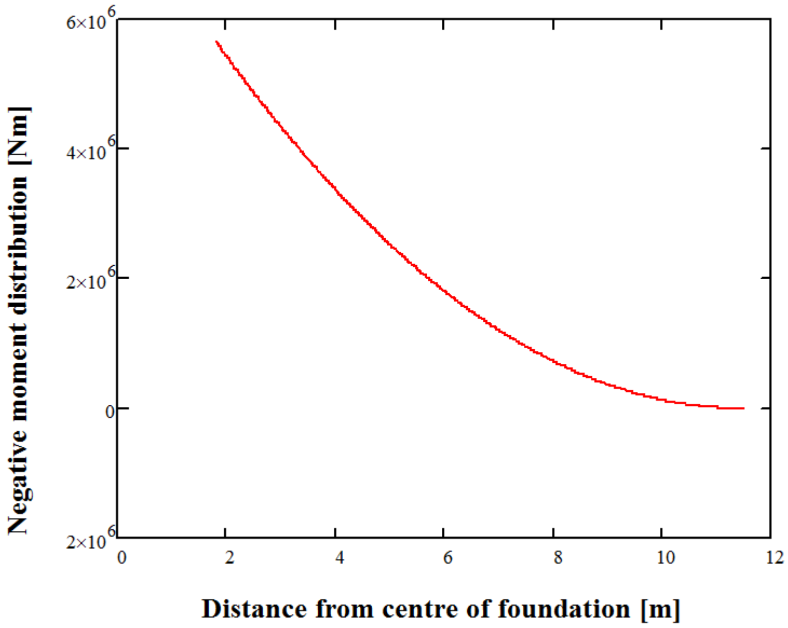

Figure 7.13: Negative moment distribution in the legs, on the windward side, between Section fc and the edge.

As for the concept with twenty legs it is acceptable that the bending moment creating tension in the bottom is much larger than the bending moment creating tension in the top, due to the larger cross-sectional area in the flange.

The results from the reinforcement calculations, based on the moment distribution, are presented in Table 7.3. Calculations were performed for the maximum moment Section fc but arranged with regard to Section cp.

Table 7.3: Results from the calculations of the reinforcement in concept 2

	Bottom reinforcement	**Top reinforcement**
Required moment capacity	54 400 kNm	5 600 kNm
Diameter of the bars	25 mm	25 mm
Number of bars	126	13
Number of layers	5	2

The chosen arrangement of the reinforcement is presented in Figure 7.14. All the needed bottom reinforcement fits in the bottom flange, which is desirable.

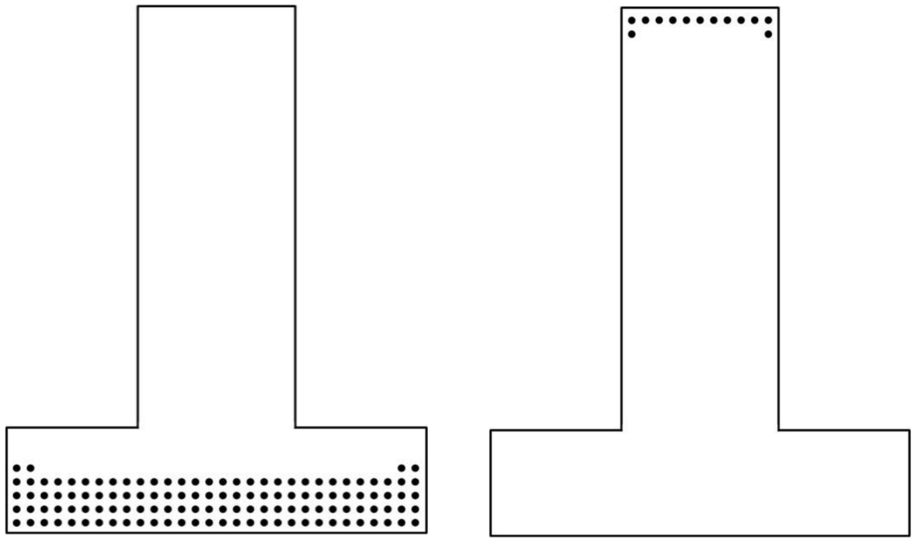

Figure 7.14: Placement of the reinforcement in the top and in the bottom of the leg, in Section cp, a) bottom reinforcement, b) top reinforcement.

From the shear force distribution it was found that the critical section is Section sp on the leeward side and Section cp on the windward side. The shear force has the same sign on both sides. The shear force distribution is plotted in Figure 7.15 on the leeward side and in Figure 7.16 on the windward side.

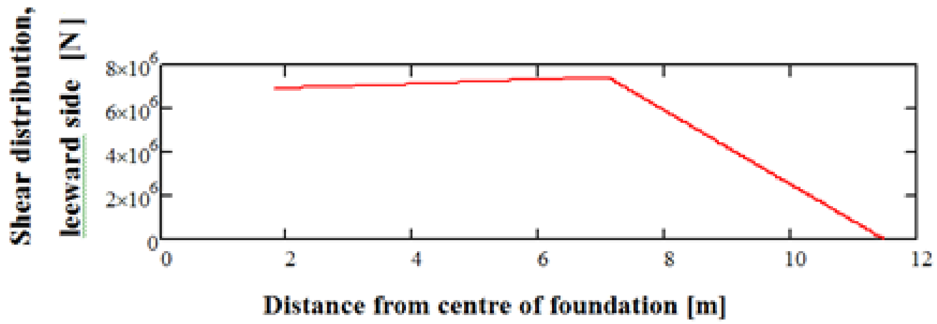

Figure 7.15: Shear force distribution on the leeward side of the tower, between Section fc and the edge.

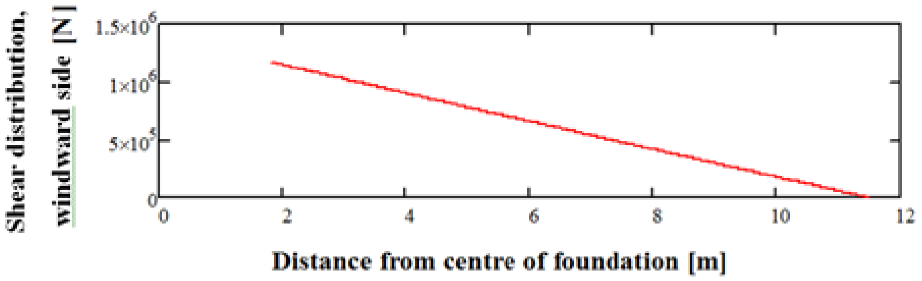

Figure 7.16: Shear force distribution on the windward side of the tower, between Section fc and the edge.

The largest shear force will occur on the leeward side, since this side is influenced by the soil pressure. The shear reinforcement was determined assuming a strut inclination of 45°. The results from the calculations are presented in Table 7.4. The resulting shear reinforcement is presented as steel area per unit length.

Table 7.4: Results from the calculations of the shear reinforcement, at Section fc and Section sp.

	Section fc	**Section sp**
Required shear force capacity	6 920 kN	7 440 kN
Required shear reinforcement area per unit length	7 850 mm^2	9 530 mm^2

7.4 Investigations of the splices and connections

Different methods to connect prefabricated concrete elements are described in Section 3.2, but these solutions should be investigated and verified for the foundation.

The section where the centrepiece and the outer parts of the foundation are attached to each other is a section with very high bending moments. Therefore it is good to avoid having a joint in this section. It is more suitable to have the joints further outwards in the legs.

The maximum positive bending moment is where the force couple resultant F_c is acting. The bending moment decreases inside the centrepiece and changes sign before it reaches the maximum negative bending moment, where the force resultant F_t is acting. This means that very large bending moments must be resisted in the centrepiece and be connected to the applied force couple by load paths in equilibrium. The reinforcement in the maximum moment section must be anchored in the centre region of the foundation where the cross-sectional area of the leg is smaller. Also the shear force is high inside the centrepiece, due to the resultants of the force couple. Therefore it is a difficult task to divide the centrepiece into elements.

A schematic distribution of the global bending moment has a similar shape as the local bending moment in Figure 7.5. The maximum moments are located in the Sections fc, on each side of the tower. The moment distribution right under the tower was approximated to be linear between the largest maximum and the largest minimum bending moments.

There are different ways to divide the centrepiece into elements, for example to divide it like pieces of a cake, see Figure 7.17a, or in slices, see Figure 7.17b. When dividing into pieces of a cake it is natural to divide the centrepiece into the same number of elements as the number of legs. With the division into slices the number of slices is not given by the concept, but can be chosen during design.

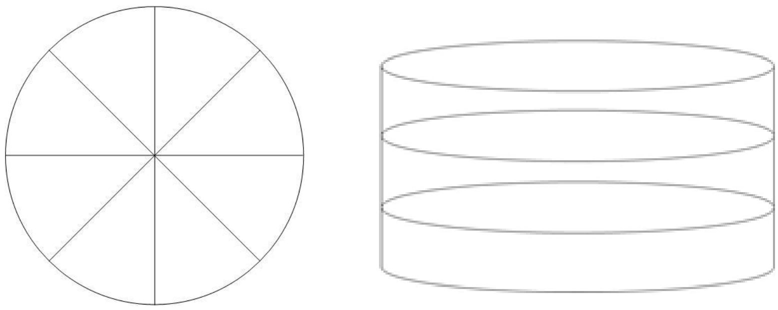

Figure 7.17: Division of the centrepiece, a) like pieces of a cake, b) into slices.

In order to divide the centrepiece into pieces of a cake, these must also be appropriately connected to each other. Many ideas were formed of how to perform this connection, either by a wet connection with protruding reinforcement, post-tensioning or with overlapping elements. Depending on method to connect the elements, different sections for the joints were chosen.

7.4.1 Global moment and shear force distribution

The local distributions of the bending moment and shear force, can only be used to design the outer part of the foundation. This means that for the part in the middle of the foundation, the region under the tower, another analysis must be performed. This analysis must include all legs and their effect on the centrepiece. Therefore this analysis is denoted global analysis and global moment and shear distributions were determined for the whole foundation. The global analysis is performed in Appendix III and Appendix IV.

The global analysis is quite similar to the local analysis. The difference is that now all leg elements are regarded at the same time. In a simplified manner the foundation was considered as a two dimensional beam and the loads were handled by concentrating all distributed loads into line loads acting on this beam. Hence the maximum positive moment on the leeward side and the maximum negative moment on the windward side were found and the distribution in the centrepiece was, in a simplified manner, assumed to be linear between these two values.

The needed shear reinforcement is determined with an assumed strut inclination of 45°. The results from the analysis concerning the global bending moment gives the results presented in Table 7.5. The resulting shear reinforcement is presented as steel area per square meter.

Table 7.5: Results from the calculations concerning the global bending moment.

	Maximum positive moment, leeward side	**Maximum negative moment, windward side**
Twenty legs with bottom slab	53 470 kNm	17 630 kNm
Eight legs with bottom flange	63 470 kNm	28 850 kNm

The results from the calculations concerning shear reinforcement is shown in Table 7.6. Then the maximum shear force between the force couple resultants was used to determine the needed amount of shear reinforcement in the centrepiece.

Table 7.6: Results from the calculations of needed shear reinforcement between the force couple.

	Required shear capacity	**Required shear reinforcement area**
Twenty legs with bottom slab	38 070 kN	8 550 mm^2/m^2
Eight legs with bottom flange	35 700 kN	8 101 mm^2/m^2

7.4.2 Wet connections and protruding reinforcement

A well-known method to connect precast elements is to join the elements by overlapping protruding reinforcement bars in joints cast onsite. The elements are placed with a distance corresponding to the lap length of the reinforcement and then the gaps are filled with fresh concrete in order to provide full interaction between the elements. With this method the elements can be assumed to be fully interacting as one unit, due to the connections that turns the elements into one solid structure.

The splice length of the reinforcement bars was calculated as the needed anchorage length of the reinforcement. The calculations were performed according to Eurocode 2 (CEN 2004) for the bars in Section cp, assuming that they reach their yield strength. The resulting splice length was found to be 0.42 m for both concepts. This value was assumed to be the minimum distance between the elements in order to achieve sufficient tensile capacity in all sections.

For the two promising concepts the centrepiece was divided like pieces of a cake, one for each leg. For the concept with eight legs the legs were transversally divided into three elements, whereof the inner element contains the centrepiece. All elements were assumed to be connected by protruding reinforcement in onsite cast concrete. The joints in both concepts that should be onsite cast are shown in Figure 7.18.

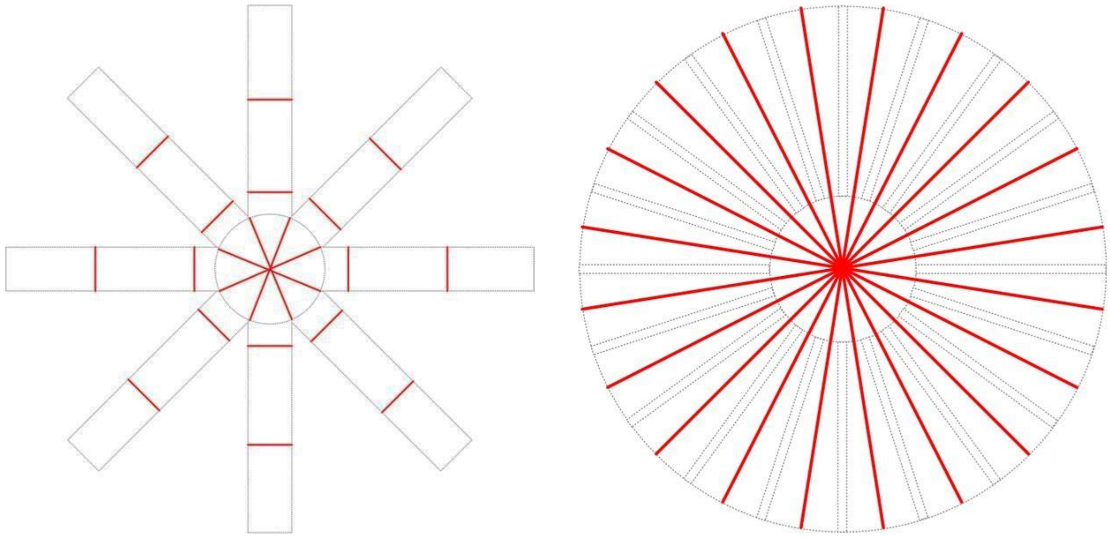

Figure 7.18: The needed joints in the two promising concepts, a) the concept with eight legs and a bottom flange, b) the concept with 20 webs and a bottom slab.

When considering wet connections, it is also a possibility to adopt the concept from Sjisjka, where the centrepiece was cast on site and the surrounding elements were prefabricated. With this method sufficient anchorage length for the reinforcement is possible to achieve in the connection between the legs and the centrepiece. For the concept with eight legs it is still necessary to connect the three elements in each leg.

The needed amount of onsite cast concrete was estimated for the two concepts and also for the two concepts assuming that the centrepiece is cast on site. The results from these calculations are shown in Table 7.7 and the calculations are presented in Appendix V.

Table 7.7: The results from the calculations concerning the needed amount of onsite cast concrete.

		Needed joints	**Amount of onsite cast concrete**
Twenty legs with bottom slab	**Element in the centrepiece**	• 20 joints between slab elements • 20 joints in the centrepiece	70.2 m^3
	Centrepiece onsite cast	• 20 joints between slab elements • The centrepiece	79.3 m^3
Eight legs with bottom flange	**Elements in the centrepiece**	• 2 joints in each leg • 8 joints in the centrepiece	31.2 m^3
	Centrepiece onsite cast	• 2 joints in each leg • The centrepiece	63.0 m^3

In order to make the connections as wet connections, all concepts require a large amount of fresh concrete. For the concept with eight legs it is possible to create the centrepiece by eight elements attached to their respective leg and to join these elements with wet connections. However, for the concept with twenty legs this is not possible, since the needed splice length is larger than the mean value of the width of the centrepiece elements. Therefore it is recommended to have the centrepiece onsite cast for this concept.

The bending moment that must be transferred across the joints, between the elements, was calculated. The calculations are presented in Appendix V for the concept with eight legs according to an assumed distribution of the global bending moment in each element. However, it was hard to decide how to perform these calculations and the results were too vague for drawing reliable conclusion. Also it could not be verified whether the quality of the joints will be enough, since it is hard to achieve continuity of the reinforcement inside the centrepiece. Due to the need for overlapping the amount of reinforcement is very large, this reinforcement is difficult to fit into the narrow space available.

From the investigation of this method some conclusions can be drawn. For the concept with twenty legs it is too narrow to divide the centrepiece into twenty elements with wet connections between the elements. The geometry makes this method impossible, since the width of the elements is narrower than the needed splice length. Therefore it would be necessary to cast the whole centrepiece on site for this concept.

When regarding the needed amount of fresh concrete for the concept with eight legs it would be slightly beneficial to have wet connections between the elements in the centrepiece compared to onsite casting of the whole centrepiece.

Therefore it is recommended to have the whole centrepiece onsite cast instead of using centrepiece elements, since this would result in fewer connections and less advanced construction work. Such a concept will also solve other challenges, such as how to place the reinforcement in order to resist the bending moment between the elements inside the centrepiece. This would give the best result for the concept with twenty legs, since there will only be the connections between the leg elements and the centrepiece. For the concept with eight legs it would still be necessary to have connections in the leg. It is therefore recommended to use the concept with twenty legs, if wet connections are preferred, even though this will give a slightly larger amount of fresh concrete.

7.4.3 Overlapping elements

It was investigated whether it would be possible to combine the elements without additional prestressing or use of fresh concrete. This is a variant of segmental construction, adapted to this specific structure. A concept was developed with a combination of the slice and pieces of a cake divisions.

The elements are connected by vertical post-tensioned prestressing bolts. It was assumed that the ordinary bolts in the connection between the tower and the foundation can be utilised. Some additional prestressing may be needed in order to create vertical compression in the structure, which would enable frictional resistance in the horizontal joints.

This method of joining elements was investigated for the concept with eight legs, in which the centrepiece was divided into two layers of slices. Each layer of slices was divided into four triangular pieces. The triangular piece is denoted centrepiece element. Each of these centrepiece elements is assumed to be continuous with its corresponding leg as one element, see Figure 7.19.

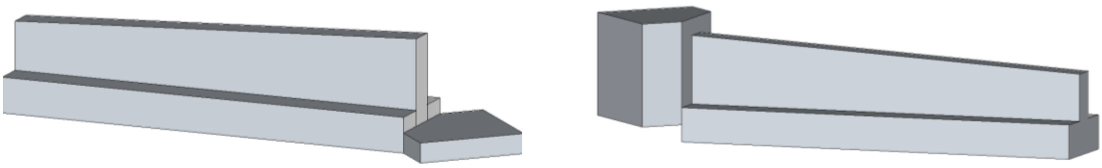

Figure 7.19: Each leg is assumed to be continuous with its corresponding part of the centrepiece, a) containing a quarter of layer one, b) containing a quarter of layer two.

Each layer of the slices contains four legs, see Figure 7.20. The upper layer is placed upon the bottom layer and rotated 45 degrees. The idea is that the slices are overlapping each other, providing overlapping of the reinforcement in the two different layers. Therefore the tensile resultant of the bending moment is taken by the reinforcement, while the compressive part can be taken by the concrete.

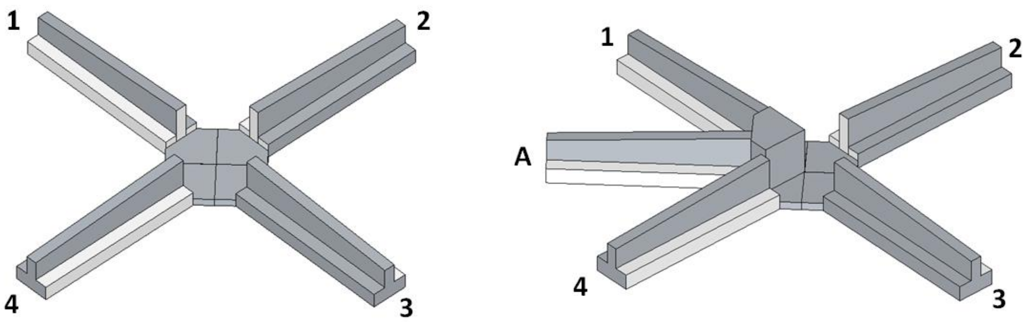

Figure 7.20: The elements are assembled by, a) placing the elements in the first layer in the bottom, b) the first element in layer 2, denoted A, is placed above the first layer.

The idea with the overlapping elements is that each element contains reinforcement, which is designed to overlap due to the overlapping of the elements, see Figure 7.21. The tensile forces from the bending moment must be resisted in the centrepiece by the bending reinforcement, which is placed inside the centrepiece elements. The reinforcement must have sufficient overlapping length in order to maintain the needed capacity in all sections. The overlapping is affected by the fact that the reinforcement has different height levels and that the bars are not placed in the same direction. These effects have not been investigated, why it is hard to verify whether this concept is possible or not. No earlier studies or literature on these effects have been found, why this subject was hard to investigate.

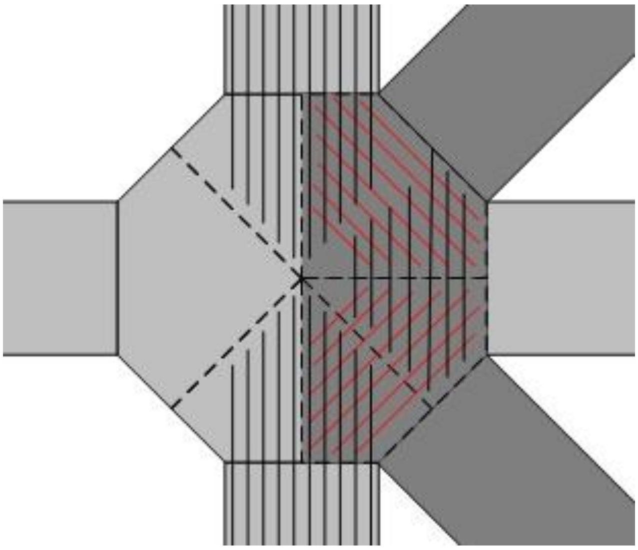

Figure 7.21: The tensile resultant from the bending moment is resisted in the foundation by the reinforcement in the direction of the bending moment. All elements have reinforcement in two directions but not all directions are shown in this figure.

The bending moment is regarded as a force couple, whose tensile resultant is resisted by reinforcement. There must be nodal equilibrium in all sections of the structure and the force pattern in the structure was studied and verified for the tensile forces in this concept. It is difficult to verify this in reality, for example concerning the effects due to the different levels and different directions of the reinforcement.

In order to investigate this method the designed splice length from the local calculations was used. Approximations of the geometry inside the centrepiece were made in order to verify wether this method is geometrically possible assuming that the height difference or the change of direction of the bars have no influence on the needed splice length. Also an rough analysis of the possible nodal equilibrium was performed in order to verify wether the flow of forces is possible.

Nodal equilibrium is necessary in all sections. This is theoretically possible in this concept. The tensile force resultant from the bending moment can be resisted by reinforcement bars across the centrepiece. This force pattern is similar to the circular reinforcement bars in an ordinary solid foundation. However, this is more complicated due to the geometrical effects in this concept. Figure 7.22 shows the nodal equilibrium for one corner of the centrepiece. This nodal equilibrium must be valid in all sections in the centrepiece region.

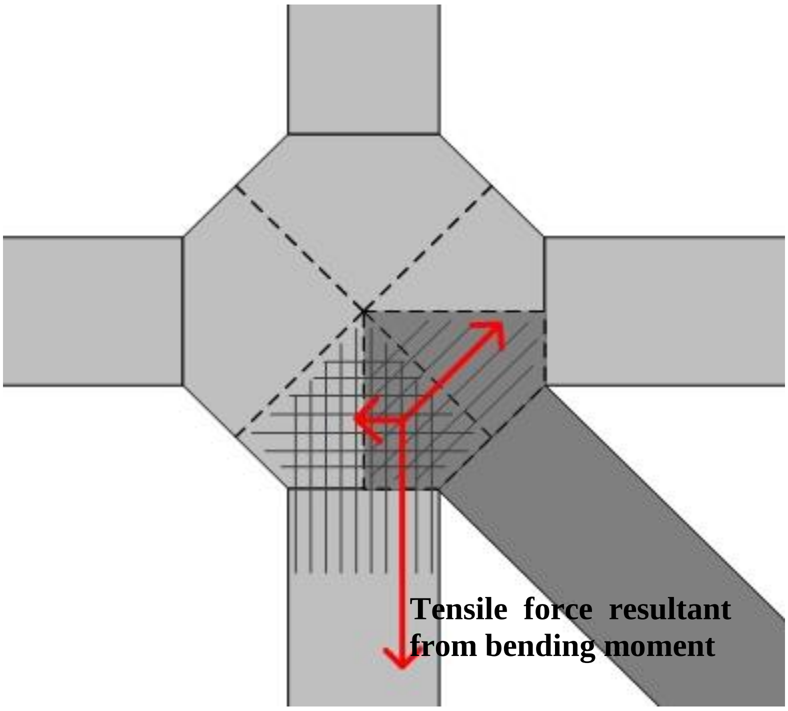

Figure 7.22: Nodal equilibrium is necessary in all sections. This must be verified concerning how the tensile force resultants can be resisted by the tensile capacity of the reinforcement and the compressive strength of the concrete.

The results from these investigations indicate that this method is possible if the effects of the arrangement of the reinforcement are disregarded. However, due to the vagueness of these effects it is impossible to draw correct conclusions. The researh indicates that this method is possible, but it can not be verified based on these investigations.

Also, due to the narrowness in the centrepiece, it is difficult to achieve sufficient lap length. The bars must achieve full anchorage in both edges and have its full strenght in the middle in order to transfer the stresses to the next element. The lap length was calculated for the concept with eight legs. Since this was hard to achieve for the concept with eight legs, it will be even harder to fulfill the requirements for the concept with twenty legs. Therefore this concept was not further analysed concerning this method.

This method was only investigated concerning the connection in the centrepiece. It still leaves the issue with the splices in the leg, for the concept with eight legs, why it will still be necessary with another method for this connection.

7.4.4 Longitudinal prestressing

For the concept with eight legs and a bottom flange, the legs must be divided into three elements in order to avoid exceeding the weight limitation of twenty ton.

It is desirable to connect these elements and the centrepiece elements with one and the same method. Therefore it was natural to investigate the possibility to use longitudinal prestressing as a method to connect the elements. Longitudinal prestressing has the advantage that it will join all elements to each other at one time.

Post-tensioning is the only possible option when it comes to prefabricated elements that should be prestressed together. The prestressing force and a suitable placement of the tendons should be determined within the limitations regarding maximum stresses in steel and concrete. The prestressing force and the tendon placement should be valid for both the positive and the negative bending moments. Most important is to avoid tension in the joints between two elements, in order to prevent the joints from opening.

The prestressing steel can replace the ordinary reinforcement, so if sufficient tensile capacity is achieved in all sections with prestressing steel, no ordinary reinforcing steel is needed as main reinforcement.

The tendon was assumed to be placed longitudinally along the legs, see Figure 7.23. Regarding the narrowness in the structure, the centre of the foundation will be critical, since all prestressing tendons must pass through on single point in this section. Therefore the tendons must be placed on top of each other, which affect the possibility to choose the eccentricity of the tendons. The tendons in the different legs must all have different eccentricities in the middle section. Another critical issue is how the prestressing tendons should pass through the bolt basket, since this is a very narrow space.

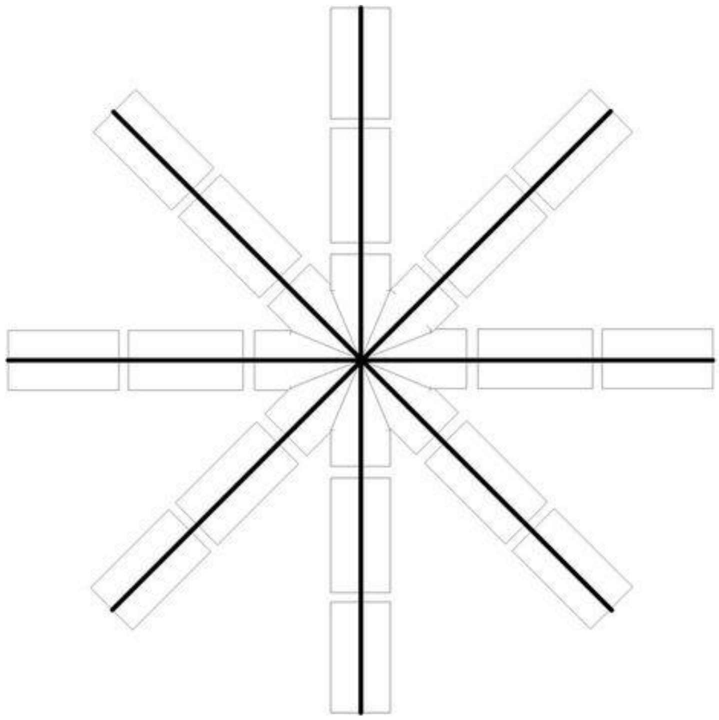

Figure 7.23: The tendons are placed longitudinally along the legs. All the cables must pass through the centre of the foundation.

The method to join the elements by longitudinal prestressing was investigated by calculations that are presented in Appendix V. The necessary prestressing force, in order to avoid tensile stresses, was determined according to Navier's formula for the worst load case. The resulting stresses in the other load cases were calculated and compared with allowable stresses. The eccentricity of the tendon was varied in order to find the most suitable placing of the tendon.

The foundation was analysed as a two-dimensional beam. The bending moments from the local analysis of the legs were used as in-put data to the calculations. From the local analysis, the beam is shown to be exposed to two different load cases: load case 1, on the leeward side, where the soil pressure is the major load and load case 2, on the windward side, where the self-weight is the major load, see Figure 7.24. Several other load cases are possible, but these two are the extreme ones, which are decisive.

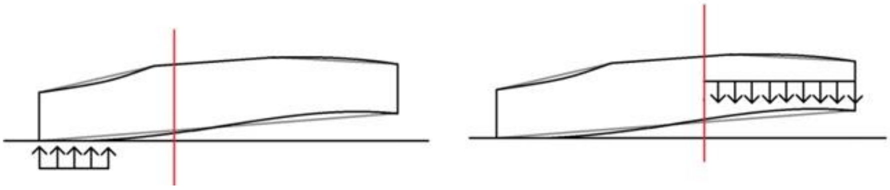

Figure 7.24: The two different load cases investigated, a) load case 1, on the leeward side, b) load case 2, on the windward side.

The two load cases are contradictory since the loads are acting in different directions. The tendon has to be placed in a position where the conditions concerning maximum tensile stresses and maximum compressive stresses are fulfilled for all load cases. This was found by changing the eccentricity iteratively and evaluating the resulting stresses.

The foundation was analysed in five sections: at the edge, in the two joints, in Section cp and Section fc. The investigated sections are shown in Figure 7.25.

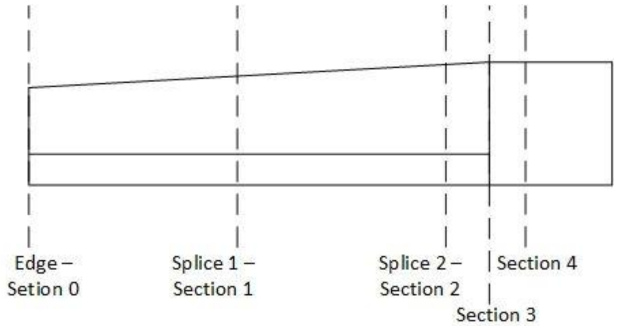

Figure 7.25: The sections that were investigated with regard to prestressing.

Due to the contradiction between the load cases, the prestressing tendon must be placed close to the centre of gravity of the cross-section in order to avoid tensile stresses in any of the load cases. Load case 1 causes tension in the bottom of the flange, while load case 2 causes tension in the top of the web. The needed prestressing force depends on load case 1, since this has the largest bending moment due to the load. The resulting stresses in the web are then checked with regard to the maximum allowable stresses, both for the web in load case 1 and for the web and flange for load case 2.

In order to prevent cracking for the characteristic load in serviceability limit state the long-term tensile concrete stresses must not exceed 0 MPa while the long term compressive concrete stresses must not exceed the allowable compressive stress of concrete $0.45f_{ck}$, 13.5 MPa. For the quasi-permanent load case in serviceability limit state higher stresses can be permitted. The reason for not allowing any tensile stresses is that the splices must never open if the structure should be continuous.

The investigation was performed by first assuming an eccentricity of the tendon. Then the needed tendon force was solved in load case 1, in order to keep all sections in compression. Thereafter the compressive stresses were verified for all load cases and sections.

The needed prestressing force was calculated assuming different eccentricities, in order to find a suitable eccentricity so that all sections will be in compression with the smallest prestressing force possible. The most suitable variation of the eccentricity along the foundation is shown in Figure 7.26. Also placing the tendon in the centre of gravity of the cross-section was investigated.

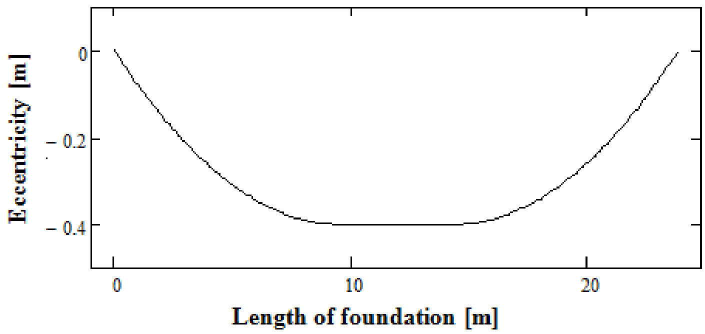

Figure 7.26: Assumed eccentricity for the tendon along the foundation.

The needed total tendon force and the resulting concrete stresses in the different sections for the two different tendon profiles are shown in Table 7.8.

Table 7.8: Resulting concrete stresses in post-tensioned foundations assuming two different eccentricities.

		Section 0	Section 1	Section 2	Section 3	Section 4
Eccentricity according to Figure 7.26 Needed total tendon force: 54.0 MN	Stresses in the top, for load case 1 [MPa]	-24.9	-14.9	-35.0	-40.7	-30.3
	Stresses in the bottom, for load case 2 [MPa]	-24.9	-35.6	-35.1	-33.5	-26.7
	Stresses in the top, for load case 2 [MPa]	-24.9	-3.4	-0.6	-2.1	-3.6
No eccentricity Needed total tendon force: 108.2 MN	Stresses in the top, for load case 1 [MPa]	-49.8	-59.0	-81.8	-85.9	-60.5
	Stresses in the bottom, for load case 2 [MPa]	-49.8	-45.6	-40.8	-39.3	-26.6
	Stresses in the top, for load case 2 [MPa]	-49.8	-47.5	-47.4	-47.4	33.8

When the tendon is placed with eccentricity according to Figure 7.26, the needed total tendon force is 54.0 MN in Section 4, which has the largest bending moment due to the loading for load case 1. The web is in compression in all sections in load case 1. However, the compressive stresses are higher than the allowable stress. Also the stresses in load case 2, both in the web and the flange, are compressive stresses, which exceeds the allowable stress.

For the situation where the tendon is placed in the centre of gravity of the cross-section, i.e. with no eccentricity, the needed total tendon force is 108.2 MN. All the sections along the foundation are in compression. However, the stresses exceeds the allowable stress in all sections for both load cases.

From this results the conclusions are that it is possible to find a prestressing force with a certain eccentricity, where the foundation is in compression in all sections for both load cases. However, all compressive stresses exceed the allowable compressive stress. With the given geometry it was impossible to find a combination of eccentricity and prestressing force for which the allowable stresses for both tension and compression were not exceeded.

The reason could be insufficient cross-sectional area of the concrete. This concept is very interesting. However, it seems impossible to keep the joints in compression without exceeding the allowable concrete compressive stress. One solution to avoid the very high compressive stresses could be to increase the cross-sectional area. However, this would also increase the total weight of the structure. It is also possible to choose a higher concrete strength class in order to permit higher compressive stresses. Another consideration is that the limitations concerning maximum stresses could be eased and larger compressive stresses allowed. The consequences of this choice must then be investigated.

Another difficulty in design is the centre of the foundation where all tendons from the different legs should pass through one section. Hence, the chosen eccentricity will only be possible for one of the legs due to the narrowness of the centrepiece. The other tendons must be placed with other eccentricities. Thus, all the tendons from the different legs must be designed with different eccentricities in the centre of the foundation, but still be sufficient for all load cases.

If all tendons should fit in the centre section, these must be placed within the maximum possible eccentricity, 0.4 m, and minimum eccentricity, 0 m. This is very narrow for the prestressing tendons. The case where the tendon is placed in the centre of gravity of the cross-section is worst affected with regard to high compressive stresses.

The method with prestressing has been analysed for the concept with eight legs and found not suitable. Concerning the concept with twenty legs and bottom slab, the legs are produced as one element and it is only necessary to attach them to the centrepiece elements. Even though the dimensioning bending moment is smaller, the cross-section decreases which means that the compressive stresses will be too high also for this concept. Also a larger number of tendons must pass through the centre. Therefore prestressing is not the whole solution for neither of the concepts with eight legs or twenty legs.

7.5 Evaluation and conclusions

From the investigation of how to connect the elements in the two promising concepts, it can be concluded that it is hard to find a good solution with minimum use of onsite cast concrete.

The purpose of the evaluation phase was to find a number of promising concepts and perform an evaluation of these concepts in order to select a winning concept. However, only one concept is possible, so an evaluation between different concepts could not be performed.

Conclusions on each concept are presented in the relevant section together with the analysis of that concept. It was found that the only reasonable solution was the concept with twenty legs and a bottom slab, where the whole centrepiece is cast onsite. The concept with prestressing seems impossible due to too large compressive stresses in the concrete. From the calculations on the overlapping elements it is not proven that the concept is impossible. However, the calculations indicate that it is not a good solution and the cost in time and money in order to enable this solution will probably not be worth the effort. The concept with eight legs and wet connections seems possible, but due to the large amount of onsite casting and the large number of joints this does not seem like a good solution.

Hence, the only reasonable solution was the concept with twenty legs and a bottom slab and with an onsite cast centrepiece. Here the connections between the flanges of the elements are realised by splicing protruding reinforcement in joint with onsite casting of concrete. The other methods were found not to be reasonable, as mentioned above, either because they showed to be impossible or it was not possible to verify that critical issues seem possible to solve.

Below the concept with twenty legs and an onsite cast centrepiece is evaluated with regard to the evaluation criteria. Due to the wish to have alike elements it might be possible to mass-produce the elements, which is one advantage with this concept. However, the protruding reinforcement is troublesome during production, since the moulds need to be adapted. The shape of the foundation is rather material efficient due the bottom slab and the legs and the use of fill as self-weight. The concept has few weak points compared to the other solutions, since it has a well proven connection method, which gives the connection a similar behaviour as the rest of the structure.

Considering the transportation quite a high amount of onsite cast concrete is used for this concept. Fewer elements need to be transported to the site, but instead a higher amount of fresh concrete must be delivered and used. The elements are small enough to avoid special transportation. However, the elements have protruding reinforcement, which is sensitive to damage during transportation.

With regard to the evaluation criterion mounting the time onsite will be quite long, since it takes time to build the moulds, place the reinforcement in the centrepiece and cast the concrete and the concrete needs also time for curing. The connection method is well-known, so the work with the connections is not so difficult.

Even if the concept works, it does not fulfil the entire design intention, since it fails on the minimum usage of onsite cast concrete. The use of both onsite and prefabricated concrete will result in an increased risk of excessive costs and labour, since both methods will be performed.

8 Conclusions and Recommendations

Some conclusions and recommendations are summarising the master's thesis project. Also some criticism and suggestions for further research are presented.

8.1 Conclusions from the conceptual design

The most important issues to solve in order to design a prefabricated wind power plant foundation are material efficient shape, how to divide the structure into elements and how to connect the elements into a structure. The global stability of the wind power plant, and resistance of the flow of forces in all sections must be verified during design. Especially the centrepiece, where the tower is attached to the foundation, is a critical region, which must be verified.

The general intention with this master's thesis project was to investigate the possibilities to design a prefabricated wind power plant. A general design intention was formulated in order to set up the framework and the design process aimed to fulfil this intention.

Regarding the shapes some promising concepts were found, with a rather lightweight structure and with a reasonable number of elements. However, the methods to connect the elements do not fulfil the demands concerning the structural behaviour and force pattern. The connections are rather complex and there are uncertainties in execution and it is therefore difficult to verify the behaviour.

8.1.1 Reflections on shape

In order to resist the overturning moment the foundation should have a large self-weight in order to prevent the tower from tilting. Since the foundation needs a high weight, a high amount of concrete is needed. In order to design the foundation material efficient, it is beneficial to reuse the excavated fill above the foundation in order to increase the dead-weight, but decrease the amount of concrete. The concepts with a bottom flange or slab, which can take advantage of the self-weight of the fill, will need a smaller amount of concrete than a traditional onsite cast solid foundation. These concepts are therefore more suitable for prefabrication due to the improved properties concerning weight and thereby transportation. The bottom flange or slab will also distribute the load that is resisted by the soil on a larger area, which is beneficial in order to decrease the soil pressure.

However, the amount of concrete is still high. This means that the foundation must be divided in a large number of elements, due to the limitations in weight regarding transportation. This results in many connections that must be designed in order to ensure a sufficient force pattern in the structure.

8.1.2 Reflections on connections

Joints should preferably be placed in sections with smaller bending moments and shear forces. The intersection between leg and centrepiece is an example of a section with a high bending moment, which should be avoided.

During the investigation of the connections, different methods were investigated: wet connections with protruding reinforcement, connection with overlapping elements and longitudinal prestressing. None of these methods showed to be very good.

For the **wet connections** it would be necessary to have large joints in order to achieve sufficient splice length of the reinforcement. This would increase the need for fresh concrete, which counteracts the purpose to minimise the need for onsite casting. However, the resulting structure is likely to work as one continuous structure. Due to the lack of space in the centrepiece for arranging and splicing the reinforcement, the concept where the whole centrepiece is cast onsite is better than casting joints between centrepiece elements.

For the method with **overlapping elements** there are still some major investigations to perform in order to verify this concept. How the forces are transferred between the reinforcement bars at different levels and placed in angle towards each other are issues that were hard to verify within this master's thesis project. However, due to geometry it seems like the narrowness in the centrepiece will cause problems to place the reinforcement in a satisfactory manner.

For the **longitudinal prestressing** the different load cases that a wind power plant is exposed to are so contradictory that the only possible eccentricity of the prestressing force is in the centre of gravity of the cross-section. However, this would induce such large compressive stresses, in order to prevent the joints to open due to tensile stresses, that the concrete would suffer too high compressive stresses. The placement of all tendons in the centre of the foundation will also cause problems, since all tendons should pass through the centre of the foundation.

8.2 Recommendations

Due to the research performed within this master's thesis project the only concept, out of those investigated, that seems possible in a satisfactory way is the concept with twenty legs and a bottom slab where the centrepiece is cast onsite. However, this concept conflicts with the design intention to decrease the use of fresh concrete, which means that our recommendation will be to onsite cast the whole structure. Prefabrication is not recommended. If the circumstances for production are normal, then we would recommend onsite casting of a structure with legs and a bottom slab.

We think that the developed shape will work well also as an onsite cast structure. It will decrease the use of material for a foundation for the specific tower. Compared to a traditional square solid foundation, the total amount of concrete will decrease with 50 % for the concept with eight legs and 60 % for the concept with twenty legs. This will give some extra formwork, but we think that the extra amount of work will pay off due to the saving in material.

From the investigations it is possible to say that none of the methods to connect the elements is recommended. It was hard to design the joints with a method where its capacity could be verified in a satisfactory manner.

We do not consider that this master's thesis project ensures whether a prefabricated foundation can be built to an affordable cost. The company behind Sjisjka wind park do not think that their method is economically defendable on a site less extreme than theirs. Since the concept with greatest potential in this project is similar to the Sjisjka foundation, it is questionable that this solution can be made to an affordable cost.

8.3 Critical review

We think that the project has answered the formulated research questions satisfactory. One of the research questions was to find a good solution for a prefabricated wind power plant foundation. However, we were aware that there was a possibility that the result might be that prefabrication is less favourable. We have not proven that it is a bad solution, but we are now well aware of the many difficulties with prefabrication.

The concepts in this analysis are not fully developed, but preliminary designed to a certain point where it is possible to judge them conceptually. This is an initial study in order to investigate whether prefabrication of wind power plant foundations is possible or not. The aim with this master thesis was not to have a fully designed foundation. Therefore it is possible that results in a later phase will change the design of the foundation. Fatigue design is one such important aspect. Fatigue has large effects on design for normal wind power plants, for example the amount of tensile reinforcement in the top is mainly decided by fatigue. Therefore this must be verified. Also the compressive stresses in the concrete must be verified, this is especially important due to the small cross-section in the top of the leg for this shape.

During the design process is important to think about prefabrication from the start with regard to joints and connections. In some ways we failed on this intention during the concept development, which means that we might have missed some ideas or concepts that would have been better suited. However, in terms of the whole project, we are satisfied with the process and the results.

The limitations of this master's thesis project might also be the key to a better solution. In this project some of the limitations have been that it should be a gravity foundation made of concrete and the method for connecting the tower to the foundation should be a bolted connection. Perhaps the limitations have hidden some opportunities in this project. If someone continues on this project, it would be possible to investigate other types of foundations, other materials or other types of connections between the tower and the foundation. These entrances might give more promising results and recommendations.

Also it would be interesting to further develop the concept with overlapping elements and to decide whether a possible force pattern can be achieved in the structure despite the different levels of the reinforcement and the different directions of the reinforcement.

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Appendices

Appendix I
Initial calculations of global equilibrium

Appendix II
Preliminary foundation loads SWT-2.3-101

Appendix III
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Appendix IV
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Appendix V
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Appendix I
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Table of contents

1. Site specific indata

Loads from the tower

$\gamma_{fav} := 0.9$ — Partial safety factors for favourable loads

Design loads assumed for a tower of approximate 100 m high, assumed loads are inclusive partial safety factors (except for the self-weight of the tower)

$M_d := 100000kN \cdot m$ — Design load on top of the foundation; overturning moment

$H_d := 1000kN$ — Design load on top of the foundation; transverse load

$N_k := 3000kN$ — Characteristic load on top of the foundation; dead load

$N_d := N_k \cdot \gamma_{fav} = 2.7 \times 10^3 \cdot kN$ — Calculated design load for the dead load

Properties of materials

$\rho_c := 25 \frac{kN}{m^3}$ — Density of concrete

$\rho_{fill} := 1600 \frac{kg}{m^3} \cdot g = 15.691 \cdot \frac{kN}{m^3}$ — Density of fill

$\sigma_{Rv} := 1000kPa$ — Assumed soil resistance based on experience

Correction of units

$ton := 1000kg$

Geometry of the connection between tower and foundation

$d_{centrepiece} := 5m$ — Diameter of the centerpiece

$A_{centrepiece} := \frac{\pi \cdot d_{centrepiece}^2}{4} = 19.635\,m^2$ — Area of the top of the centerpiece

$h_{centrepiece} := 2.5m$ — Height of the centerpiece, defined from the top to the bottom of the foundation

$V_{centrepiece} := h_{centrepiece} \cdot A_{centrepiece} = 49.087 \cdot m^3$ — Volume of the centerpiece

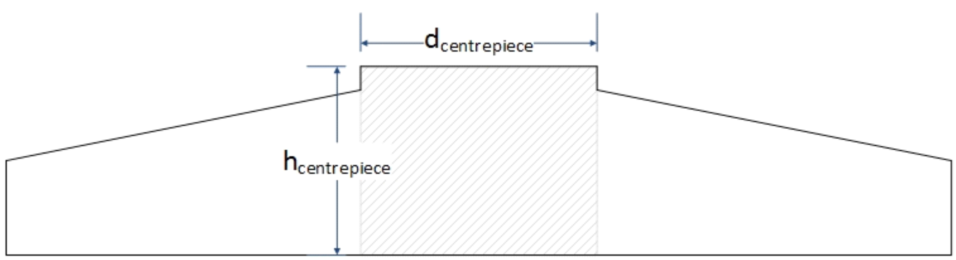


Figure 1: The definition of the dimension of the centerpiece

2. Four-legged stocky structure

Geometry of the foundation

$d_{leg.4.st} := 5m$ — Width of the leg

$l_{leg.4.st} := 9m$ — Length of the leg

$h_{leg.4.st} := 2m$ — Height of the leg

$V_{leg.4.st} := d_{leg.4.st} \cdot l_{leg.4.st} \cdot h_{leg.4.st} = 90 \cdot m^3$ — Volume of one leg

$h_{fill.4.st} := 0.5m$ — Height of the fill above the legs

$V_{fill.4.st} := 4d_{leg.4.st} \cdot l_{leg.4.st} \cdot h_{fill.4.st} = 90 \cdot m^3$ — Volume of the fill above the legs

Figure 2: Four-legged stocky structure. Calculations are performed for two different wind directions

In this shape it is not possible to have a circular centrepiece due to the geometry of the legs, therfore the gap between the circular centrepiece and the legs is filled with concrete. The same applies for the other shapes but consequences will be smaller.

Self-weight of the foundation and the fill

$$G_{k.4.st} := \left[\left[4V_{leg.4.st} + d_{centrepiece}^{2} \cdot h_{leg.4.st} + A_{centrepiece} \cdot \left(h_{centrepiece} - h_{leg.4.st} \right) \right] \cdot \rho_c \ldots \atop + V_{fill.4.st} \cdot \rho_{fill} \right] = 11.908 \cdot MN$$

$G_{d.4.st} := G_{k.4.st} \cdot \gamma_{fav} = 10.717 \cdot MN$ — Design load for the self-weight, self-weight is a favourable load when calculating the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated.

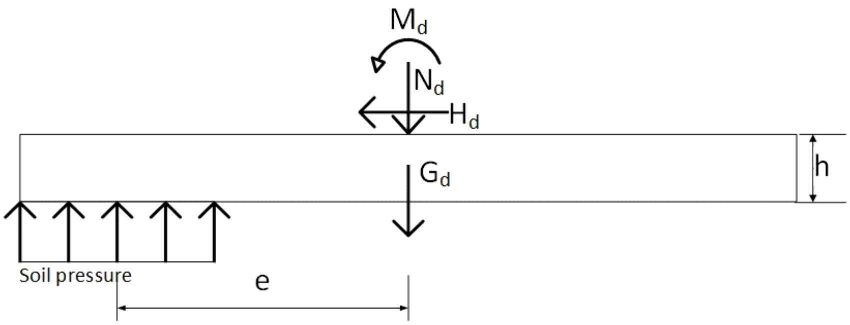


Figure 3: A conceptual sketch of the loads acting on the foundation and definition of the eccentricity, e.

$$e_{4.st} := \frac{M_d + H_d \cdot h_{centrepiece}}{N_d + G_{d.4.st}} = 7.64\,\text{m}$$

Eccentricity of the soil pressure resultant

Wind direction 1

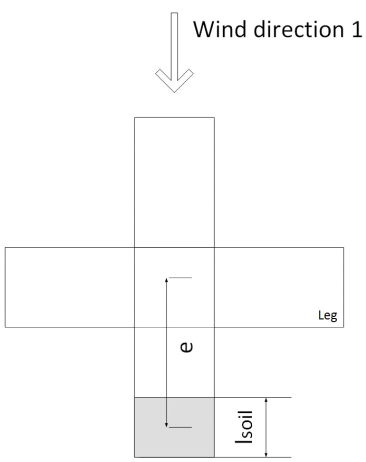


Figure 4: Wind direction 1. The grey area is the area that resist the overturning moment

Check if the eccentricity fits within the length of the leg

$$l_{wind1.4.st} := l_{leg.4.st} + \frac{d_{centrepiece}}{2} = 11.5\,\text{m}$$

Length of one leg defined from center of the foundation

$$\text{Check1} := \begin{cases} \text{"OK! Eccentricity is inside the length of the leg"} & \text{if } e_{4.st} \le l_{wind1.4.st} \\ \text{"Not OK! Eccentricity is too big"} & \text{otherwise} \end{cases}$$

Check1 = "OK! Eccentricity is inside the length of the leg"

Soil pressure

$$l_{soil.4.st} := 2\left[\left(l_{leg.4.st} + \frac{d_{centrepiece}}{2}\right) - e_{4.st}\right] = 7.721\,m$$

Length of the soil pressure zone

$$A_{soil.4.st} := l_{soil.4.st} \cdot d_{leg.4.st} = 38.603\,m^2$$

Area of the soil that resist the overturning moment

$$\sigma_{soil.4.st} := \frac{N_d + G_{d.4.st}}{A_{soil.4.st}} = 347.555 \cdot kPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.4.st}}{\sigma_{Rv}} = 0.348$$

Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.4.st}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

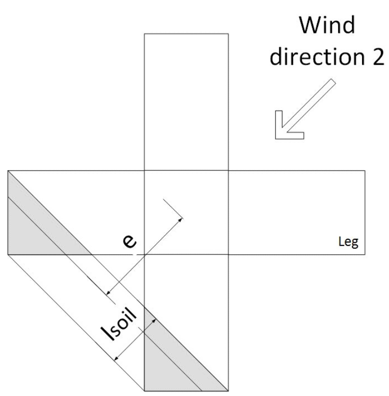


Figure 5: Wind direction 2. The two grey areas area the resultant area that resist the overturning moment.

Check if the eccentricity fits within the length of the leg

$$l_{wind2.4.st} := \frac{\sqrt{2l_{leg.4.st}^{2}}}{2} + \frac{\sqrt{2d_{leg.4.st}^{2}}}{2} = 9.899\,m$$

Length of the projection of the legs in the direction of the wind, defined from the centre of the foundation

$$Check1_{wind2} := \begin{cases} \text{"Eccentricity is inside the length of the leg"} & \text{if } e_{4.st} \le l_{wind2.4.st} \\ \text{"Eccentricity is too big"} & \text{otherwise} \end{cases}$$

$Check1_{wind2}$ = "Eccentricity is inside the length of the leg"

Soil pressure

$l_{soil.wind2.4.st} := \left(l_{wind2.4.st} - e_{4.st}\right)\cdot(1.5) = 3.39\,m$ Length of the soil pressure zone

$A_{soil.wind2.4.st} := 2\left(l_{soil.wind2.4.st}\right)^2 = 22.981\,m^2$ Area of the soil that resist the pressure, two legs are active and resist the overturning moment

$\sigma_{soil.wind2.4.st} := \dfrac{N_d + G_{d.4.st}}{A_{soil.wind2.4.st}} = 583.825\cdot kPa$ Soil pressure

Check if the resistance of the soil is sufficient

$\dfrac{\sigma_{soil.wind2.4.st}}{\sigma_{Rv}} = 0.584$ Utilisation

$$Check2_{wind2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.wind2.4.st}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

$Check2_{wind2} = \text{"OK! Soil resistance is sufficent"}$

Summary of the shape four-legged stocky

$V_{tot.4.stocky} := 4\cdot V_{leg.4.st} + V_{centrepiece} = 409.087\cdot m^3$ Total volume of the concrete

$m_{tot.4.stocky} := V_{tot.4.stocky}\cdot\dfrac{\rho_c}{g} = 1.043\times 10^3\cdot ton$ Total weight of the concrete

$m_{leg.4.stocky} := V_{leg.4.st}\cdot\dfrac{\rho_c}{g} = 229.436\cdot ton$ Weight of one leg

$m_{centrepiece.4.stocky} := V_{centrepiece}\cdot\dfrac{\rho_c}{g} = 125.138\cdot ton$ Weight of the centerpiece

$l_{foundation.4.stocky} := l_{leg.4.st}\cdot 2 + d_{centrepiece} = 23\,m$ Total length of the foundation

3. Four-legged slim structure

Geometry of the foundation

$d_{leg.4.sl} := 2m$ — Width of the leg

$l_{leg.4.sl} := 14.5m$ — Length of the leg

$h_{leg.4.sl} := 2m$ — Height of the leg

$V_{leg.4.sl} := d_{leg.4.sl} \cdot l_{leg.4.sl} \cdot h_{leg.4.sl} = 58 \cdot m^3$ — Volume of one leg

$h_{fill.4.sl} := 0.5m$ — Height of fill above the legs

$V_{fill.4.sl} := 4 l_{leg.4.sl} \cdot d_{leg.4.sl} \cdot h_{fill.4.sl} = 58 \cdot m^3$ — Volume of the fill above the legs

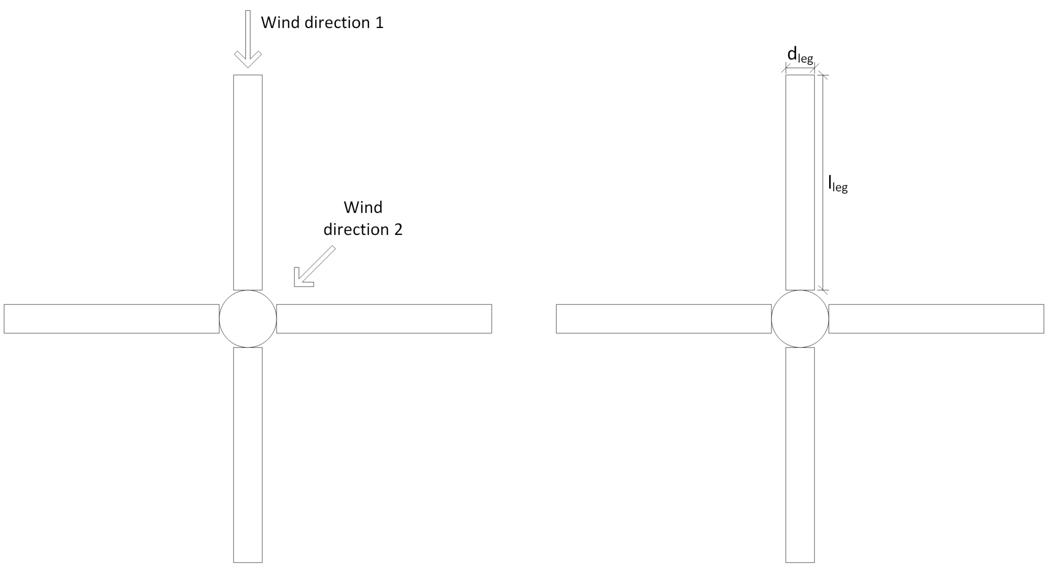


Figure 6: Four-legged slim structure. Calculations are performed for two different wind directions

Self-weight of the foundation and the soil

$G_{k.4.sl} := \left[\left(4V_{leg.4.sl} + V_{centrepiece}\right) \cdot \rho_c + V_{fill.4.sl} \cdot \rho_{fill}\right] = 7.937 \cdot MN$ — Characteristic load for the self-weight

$G_{d.4.sl} := G_{k.4.sl} \cdot \gamma_{fav} = 7.144 \cdot MN$ — Design load for the self-weight, self-weight is a favourable load when calculating the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Fiure 3.

$$e_{4.sl} := \frac{M_d + H_d \cdot h_{centrepiece}}{N_d + G_{d.4.sl}} = 10.413\, m$$

Eccentricity for the global stability of the soil pressure resultant

Wind direction 1

Wind direction 1 is defined in same way as for the concept with four stocky legs, see Figure 2

Check if the eccentricity fits within the length of the leg

$$l_{wind1.4.sl} := l_{leg.4.sl} + \frac{d_{centrepiece}}{2} = 17\,m$$

Length of one leg defined from the center of the foundation

$$Check1 := \begin{cases} \text{"OK! Eccentricity is inside the length of the leg"} & \text{if } e_{4.sl} \le l_{leg.4.sl} \\ \text{"Not OK! Eccentricity is too big"} & \text{otherwise} \end{cases}$$

$Check1 =$ "OK! Eccentricity is inside the length of the leg"

Soil pressure

$$l_{soil.4.sl} := 2\left[\left(l_{leg.4.sl} + \frac{d_{centrepiece}}{2}\right) - e_{4.sl}\right] = 13.174\,m$$

Length of the soil pressure zone

$$A_{soil.4.sl} := l_{soil.4.sl} \cdot d_{leg.4.sl} = 26.348\,m^2$$

Area of the soil pressure zone

$$\sigma_{soil.4.sl} := \frac{G_{d.4.sl} + N_d}{A_{soil.4.sl}} = 373.593 \cdot kPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.4.sl}}{\sigma_{Rv}} = 0.374$$

Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.4.sl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

$Check2 =$ "OK! Soil resistance is sufficent"

Wind direction 2

Check that the eccentricity fits within the length of the leg

$$l_{wind2.4.sl} := \frac{\sqrt{\left(l_{leg.4.sl} + \frac{d_{centrepiece}}{2}\right)^2 \cdot 2}}{2} = 12.021\,m$$

Length of the projection of the legs in the direction of the wind, defined from the centre of the foundation

$$Check1_{wind2} := \begin{cases} \text{"Eccentricity is inside the length of the leg"} & \text{if } e_{4.sl} \le l_{wind2.4.sl} \\ \text{"Eccentricity is too big"} & \text{otherwise} \end{cases}$$

$Check1_{wind2} =$ "Eccentricity is inside the length of the leg"

Soil pressure

$l_{soil.wind2.4.sl} := (l_{wind2.4.sl} - e_{4.sl})(1.5) = 2.412\,m$ Length of the soil pressure zone

$A_{soil.wind2.4.sl} := 2(l_{soil.wind2.4.sl})^2 = 11.634\,m^2$ Area of the soil pressure zone

$\sigma_{soil.wind2.4.sl} := \dfrac{G_{d.4.sl} + N_d}{A_{soil.wind2.4.sl}} = 846.127 \cdot kPa$ Soil pressure

Check if the resistance of the soil is sufficient

$\dfrac{\sigma_{soil.wind2.4.sl}}{\sigma_{Rv}} = 0.846$ Utilisation

$$Check2_{wind2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.wind2.4.sl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

$Check2_{wind2} = \text{"OK! Soil resistance is sufficent"}$

Summary of the shape four-legged slim

$V_{tot.4.slim} := 4 \cdot V_{leg.4.sl} + V_{centrepiece} = 281.087 \cdot m^3$ Total volume of the concrete

$m_{tot.4.slim} := V_{tot.4.slim} \cdot \dfrac{\rho_c}{g} = 716.573 \cdot ton$ Total weight of the concrete

$m_{leg.4.slim} := V_{leg.4.sl} \cdot \dfrac{\rho_c}{g} = 147.859 \cdot ton$ Weight of one leg

$m_{centrepiece.4.slim} := V_{centrepiece} \cdot \dfrac{\rho_c}{g} = 125.138 \cdot ton$ Weight of the centerpiece

$l_{foundation.4.slim} := l_{leg.4.sl} \cdot 2 + d_{centrepiece} = 34\,m$ Total length of the foundation

4. Three-legged structure

Geometry of the foundation

$d_{leg.3} := 4m$ Width of the leg

$l_{leg.3} := 14m$ Length of the leg

$h_{leg.3} := 2m$ Height of the leg

$V_{leg.3} := d_{leg.3} \cdot l_{leg.3} \cdot h_{leg.3} = 112 \cdot m^3$ Holume of one leg

$h_{fill.3} := 0.5m$ Height of fill above the legs

$V_{fill.3} := 3 l_{leg.3} \cdot d_{leg.3} \cdot h_{fill.3} = 84 \cdot m^3$ Volume of the fill above the legs

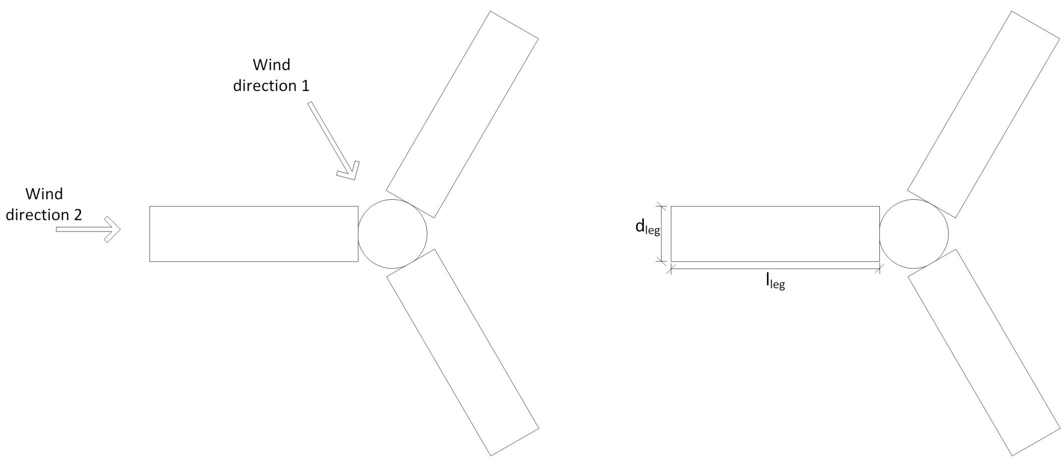


Figure 7: Three-legged structure. Calculations are performed for two different wind directions

Self-weight of the foundation and the soil

$G_{k.3} := \left(3V_{leg.3} + V_{centrepiece}\right) \cdot \rho_c + V_{fill.3} \cdot \rho_{fill} = 10.945 \cdot MN$ Characteristic load from the self-weight

$G_{d.3} := G_{k.3} \cdot \gamma_{fav} = 9.851 \cdot MN$ Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_3 := \frac{M_d + H_d \cdot h_{centrepiece}}{N_d + G_{d.3}} = 8.167\,m$$

Eccentricity for the soil pressure resultant

Wind direction 1

Check if the eccentricity fits within the length of the leg

$$l_3 := l_{leg.3} + \frac{d_{centrepiece}}{2} = 16.5\,m$$

Length of one leg defined from the center of the foundation

$$\text{Check1} := \begin{vmatrix} \text{"OK! Eccentricity is inside the length of the leg"} & \text{if } e_3 \le l_3 \\ \text{"Not OK! Eccentricity is too big"} & \text{otherwise} \end{vmatrix}$$

Check1 = "OK! Eccentricity is inside the length of the leg"

Soil pressure

$$l_{soil.3} := 2\left[\left(l_{leg.3} + \frac{d_{centrepiece}}{2}\right) - e_3\right] = 16.666\,m$$

Length of the soil pressure zone

$$A_{soil.3} := l_{soil.3} \cdot d_{leg.3} = 66.665\,m^2$$

Area of the soil pressure zone

$$\sigma_{soil.3} := \frac{N_d + G_{d.3}}{A_{soil.3}} = 188.265 \cdot kPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.3}}{\sigma_{Rv}} = 0.188$$

Utilisation

$$\text{Check2} := \begin{vmatrix} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.3}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{vmatrix}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

Check if the eccentricity fits within the length of the leg

$$l_{wind2.3} := \left(\frac{d_{centrepiece}}{2} + l_{leg.3} + \frac{d_{leg.3}}{2 \cdot \tan(30deg)}\right) \cdot \cos(60deg) = 9.982\,m$$

Length of the projection of the legs in the direction of the wind, defined from the centre of the foundation

$$\text{Check1}_2 := \begin{vmatrix} \text{"OK! Eccentricity is inside the length of the leg"} & \text{if } e_3 \le l_{wind2.3} \\ \text{"Not OK! Eccentricity is too big"} & \text{otherwise} \end{vmatrix}$$

Check1_2 = "OK! Eccentricity is inside the length of the leg"

Soil pressure

$l_{soil.wind2.3} := \left(l_{wind2.3} - e_3\right)\cdot 1.5 = 2.723\,m$

Length of the soil pressure zone

$A_{soil.wind2.3} := 2\,\dfrac{l_{soil.wind2.3}{}^2}{2\cos(30deg)\cdot\cos(60deg)} = 17.12\,m^2$

Area of the soil pressure, two legs are contributing to the soil pressure area

$\sigma_{soil.wind2.3} := \dfrac{G_{d.3} + N_d}{A_{soil.wind2.3}} = 733.086\cdot kPa$

Soil pressure

Check if the resistance of the soil is sufficient

$\dfrac{\sigma_{soil.wind2.3}}{\sigma_{Rv}} = 0.733$

Utilisation

$$Check3_2 := \begin{vmatrix} \text{"OK! Soil resistance is sufficent"} \quad \text{if } \dfrac{\sigma_{soil.wind2.3}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} \quad \text{otherwise} \end{vmatrix}$$

$Check3_2 = \text{"OK! Soil resistance is sufficent"}$

Summary of the shape three-legged

$V_{tot.3} := 3\cdot V_{leg.3} + V_{centrepiece} = 385.087\cdot m^3$

Total volume of the concrete

$m_{tot.3} := V_{tot.3}\cdot\dfrac{\rho_c}{g} = 981.7\cdot ton$

Total weight of the concrete

$m_{leg.3} := V_{leg.3}\cdot\dfrac{\rho_c}{g} = 285.521\cdot ton$

Weight of one leg

$m_{centrepiece.3} := V_{centrepiece}\cdot\dfrac{\rho_c}{g} = 125.138\cdot ton$

Weight of the centerpiece

$l_{foundation.3} := l_{leg.3} + d_{centrepiece} + l_{leg.3}\cdot\cos(60deg) = 26\,m$

Total length of the foundation

5. Square solid structure

Geometry of the foundation

$b_{sq} := 15.5m$ Width of the foundation

$h_{sq} := 2m$ Height of the foundation

$V_{sq} := \left(b_{sq}^{\ 2} - A_{centrepiece}\right) \cdot h_{sq} = 441.23 \cdot m^3$ Volume of outer part of the foundation, excluding the centrepiece

$h_{fill.sq} := 0.5m$ Height of fill above the foundation

$V_{fill.sq} := \frac{V_{sq}}{h_{sq}} \cdot h_{fill.sq} = 110.308 \cdot m^3$ Volume of the fill above the foundation

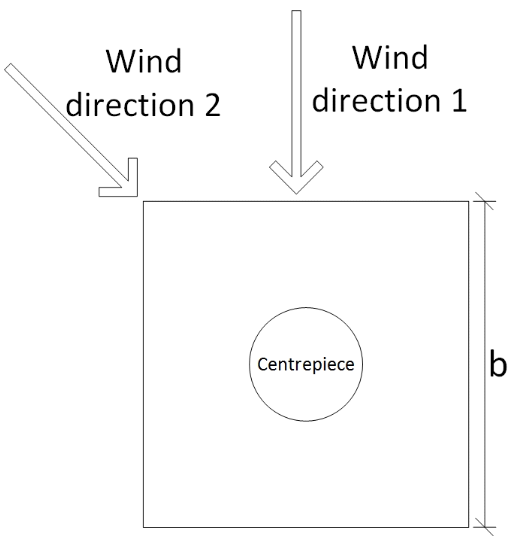


Figure 8: Square solid structure. Calculations are performed for two different wind directions

Self-weight of the foundation and the fill

$G_{k.sq} := \left[\left(V_{sq} + V_{centrepiece}\right) \cdot \rho_c + V_{fill.sq} \cdot \rho_{fill}\right] = 13.989 \cdot MN$ Characteristic load from the self-weight

$G_{d.sq} := G_{k.sq} \cdot \gamma_{fav} = 12.59 \cdot MN$ Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$e_{sq} := \frac{M_d + H_d \cdot h_{centrepiece}}{G_{d.sq} + N_d} = 6.704\,m$ Eccentricity for the soil pressure resultant

Wind direction 1

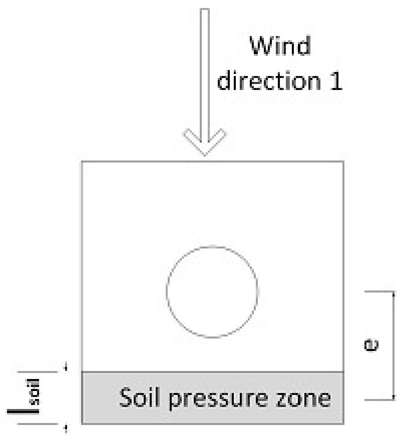


Figure 9: Wind direction 1. The grey area are the resultant area that resist the soil pressure.

Check if the eccentricity fits within the length of the foundation

$$l_{sq} := \frac{b_{sq}}{2} = 7.75\,m$$ Length of half of the foundation

$$\text{Check1} := \begin{cases} \text{"OK! Eccentricity is inside the length of the leg"} & \text{if } e_{sq} \le l_{sq} \\ \text{"Not OK! Eccentricity is too big"} & \text{otherwise} \end{cases}$$

Check1 = "OK! Eccentricity is inside the length of the leg"

Soil pressure

$$l_{soil.sq} := 2\left(l_{sq} - e_{sq}\right) = 2.092\,m$$ Length of the soil pressure zone

$$A_{soil.sq} := l_{soil.sq} \cdot b_{sq} = 32.433\,m^2$$ Area of the soil pressure zone

$$\sigma_{soil.sq} := \frac{N_d + G_{d.sq}}{A_{soil.sq}} = 471.436 \cdot kPa$$ Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.sq}}{\sigma_{Rv}} = 0.471$$ Utilisation

$$\text{Check2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.sq}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

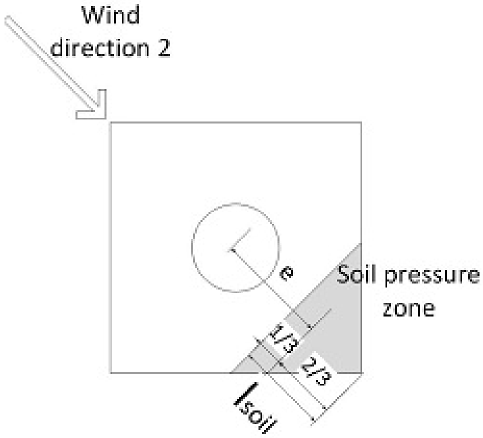


Figure 10: *Wind direction 2. The grey area are the resultant area that resist the soil pressure.*

Check if the eccentricity fits within the length of the foundation

$$l_{wind2.sq} := \sqrt{\left(\frac{b_{sq}}{2}\right)^2 2} = 10.96\,\text{m}$$

Distance from the centre of the foundation to the corner, in direction of wind 2

$$\text{Check1}_{wind2} := \begin{vmatrix} \text{"Eccentricity is inside the length of the leg"} \quad \text{if } e_{sq} \le l_{wind2.sq} \\ \text{"Eccentricity is too big"} \quad \text{otherwise} \end{vmatrix}$$

$$\text{Check1}_{wind2} = \text{"Eccentricity is inside the length of the leg"}$$

Soil pressure

$$l_{soil.wind2.sq} := \left(l_{wind2.sq} - e_{sq}\right) \cdot 1.5 = 6.385\,\text{m}$$

Length of the soil pressure zone

$$A_{soil.wind2.sq} := \frac{2l_{soil.wind2.sq}^{\;2}}{2} = 40.762\,\text{m}^2$$

Area of the soil pressure zone

$$\sigma_{soil.wind2.sq} := \frac{N_d + G_{d.sq}}{A_{soil.wind2.sq}} = 375.097 \cdot \text{kPa}$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind2.sq}}{\sigma_{Rv}} = 0.375$$

Utilisation

$$\text{Check2}_{wind2} := \begin{vmatrix} \text{"OK! Soil resistance is sufficent"} \quad \text{if } \dfrac{\sigma_{soil.wind2.sq}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} \quad \text{otherwise} \end{vmatrix}$$

$$\text{Check2}_{wind2} = \text{"OK! Soil resistance is sufficent"}$$

Summary of the shape square solid

$$V_{tot.square} := V_{sq} + V_{centrepiece} = 490.317 \cdot \text{m}^3$$

Volume of the concrete

$$m_{square} := V_{tot.square} \cdot \frac{\rho_c}{g} = 1.25 \times 10^3 \cdot \text{ton}$$

Total weight of the concrete

$$l_{foundation.square} := b_{sq} = 15.5\,\text{m}$$

Total length of the foundation

6. Circular solid structure

Geometry of the foundation

$d_{ci} := 16.5m$ Diameter of the foundation

$h_{ci} := 2m$ Height of the foundation

$V_{ci} := \left(\frac{\pi \cdot d_{ci}^{2}}{4} - A_{centrepiece}\right) \cdot h_{ci} = 388.379 \cdot m^{3}$ Volume of outer part of the foundation

$h_{fill.ci} := 0.5m$ Height of fill above the foundation

$V_{fill.ci} := \left(\frac{\pi \cdot d_{ci}^{2}}{4} - A_{centrepiece}\right) \cdot h_{fill.ci} = 97.095 \cdot m^{3}$ Volume of the fill above the foundation

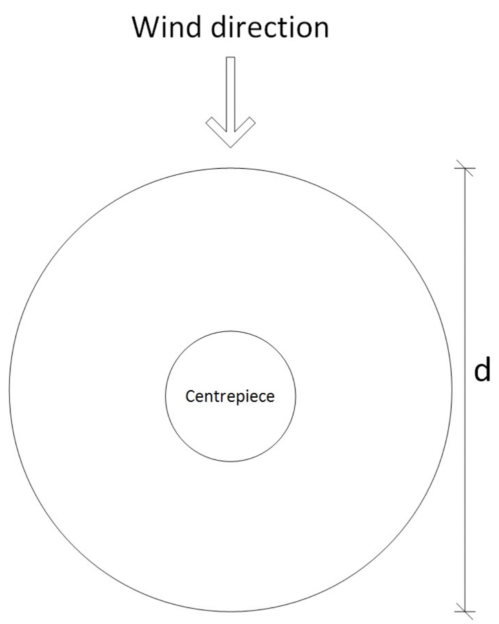


Figure 11: Circular solid structure. Calculations are performed for one wind direction, due to symmetry

Self-weight of the foundation and the fill

$G_{k.ci} := \left(V_{ci} + V_{centrepiece}\right) \cdot \rho_{c} + V_{fill.ci} \cdot \rho_{fill} = 12.46 \cdot MN$ Characteristic load from the self-weight

$G_{d.ci} := G_{k.ci} \cdot \gamma_{fav} = 11.214 \cdot MN$ Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$e_{ci} := \frac{M_{d} + H_{d} \cdot h_{centrepiece}}{G_{d.ci} + N_{d}} = 7.367\,m$ Eccentricity for the soil pressure resultant

Check that the eccentricity fits within the length of the foundation

$l_{ci} := \frac{d_{ci}}{2} = 8.25\,m$ Distance from the centre of the foundation to the edge

$$Check1 := \begin{cases} \text{"OK! Eccentricity is inside the length of the leg"} & \text{if } e_{ci} \le l_{ci} \\ \text{"Not OK! Eccentricity is too big"} & \text{otherwise} \end{cases}$$

Check1 = "OK! Eccentricity is inside the length of the leg"

Calculation of the angle of the compression zone

The soil pressure zone of the foundation should be decided. It is calculated iteratively by deciding an angle of the soil pressure zone, which gives a certein soil pressure area and a centre of gravity of the soil pressure zone.
The centre of gravity should match the eccentricity given by the equilibrium calculations above. To determine the center of gravity for the segment the areas in the figure below and respective center of gravity for each area need to be calculated.

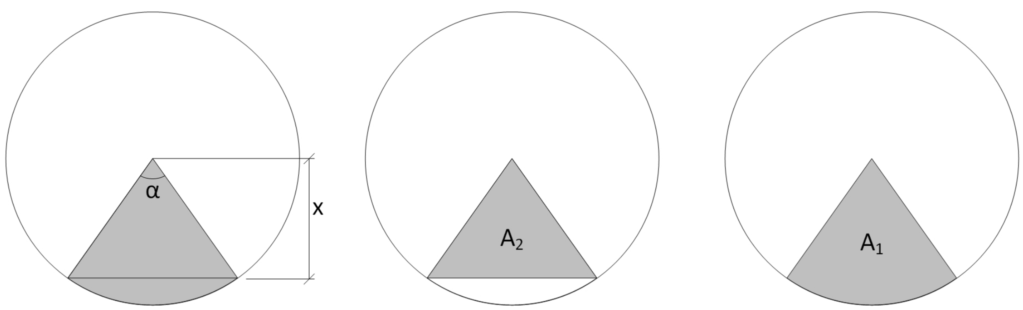


Figure 12: Illustration of the shape of the soil pressure zone for the foundation. The area of the soil pressure is calculated as $A_{sp} := A_1 - A_2$ *and thereafter the centre of gravity is calculated.*

$\alpha := 90deg$ Assumed angle, defined as shown in the figure above

$x := \cos(0.5\alpha)\cdot\frac{d_{ci}}{2} = 5.834\,m$ Distance from the center of the foundation to where the radial part of the circle sector start.

$tp_1 := \frac{2}{3}\cdot\frac{d_{ci}}{2}\cdot\frac{\sin(0.5\alpha)}{\frac{\alpha}{2}} = 4.952\,m$ Centre of gravity for the whole sector A_1

$tp_2 := \frac{2}{3}\cdot x = 3.889\,m$ Centre of gravity of the triangle A_2

$A_1 := \frac{\alpha}{2}\cdot\left(\frac{d_{ci}}{2}\right)^2 = 53.456\,m^2$ Area of the whole sector A_1

$A_2 := x\cdot x\cdot\tan(0.5\alpha) = 34.031\,m^2$ Area of the triangle A_2

$tp := \frac{A_1\cdot tp_1 - A_2\cdot tp_2}{A_1 + A_2} + x = 7.346\,m$ Centre of gravity of the segment which takes the soil pressure

Check if the assumed angle is correct. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$\text{Check2} := \begin{cases} \text{"Angle ok"} & \text{if } \left|1 - \dfrac{tp}{e_{ci}}\right| < 1\% \\ \text{"To large difference"} & \text{otherwise} \end{cases}$$

Check2 = "Angle ok"

Soil pressure

$A_{soil.ci} := A_1 - A_2 = 19.425\,m^2$ Area of the soil pressure zone

$\sigma_{soil.ci} := \dfrac{N_d + G_{d.ci}}{A_{soil.ci}} = 716.304 \cdot kPa$ Soil pressure

Check if the resistance of the soil is sufficient

$\dfrac{\sigma_{soil.ci}}{\sigma_{Rv}} = 0.716$ Utilisation

$$\text{Check3} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.ci}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check3 = "OK! Soil resistance is sufficent"

Summary of the shape circular solid

$V_{tot.circ} := V_{ci} + V_{centrepiece} = 437.467 \cdot m^3$ Volume of the concrete

$m_{circ} := V_{tot.circ} \cdot \dfrac{\rho_c}{g} = 1.115 \times 10^3 \cdot ton$ Total weight of the concrete

$m_{centrepiece.circ} := V_{centrepiece} \cdot \dfrac{\rho_c}{g} = 125.138 \cdot ton$ Weight of the centerpiece

$l_{foundation.circ} := d_{ci} = 16.5\,m$ Total length of the foundation

7. Eight legged stocky structure

Geometry of the foundation

$d_{leg.8.st} := 2m$ — Width of the leg

$l_{leg.8.st} := 9.5m$ — Length of the leg

$h_{leg.8.st} := 2m$ — Height of the leg

$V_{leg.8.st} := d_{leg.8.st} \cdot l_{leg.8.st} \cdot h_{leg.8.st} = 38 \cdot m^3$ — Volume of one leg

$h_{fill.8.st} := 0.5m$ — Height of fill above the legs

$V_{fill.8.st} := 8 l_{leg.8.st} \cdot d_{leg.8.st} \cdot h_{fill.8.st} = 76 \cdot m^3$ — Volume of the fill above the legs

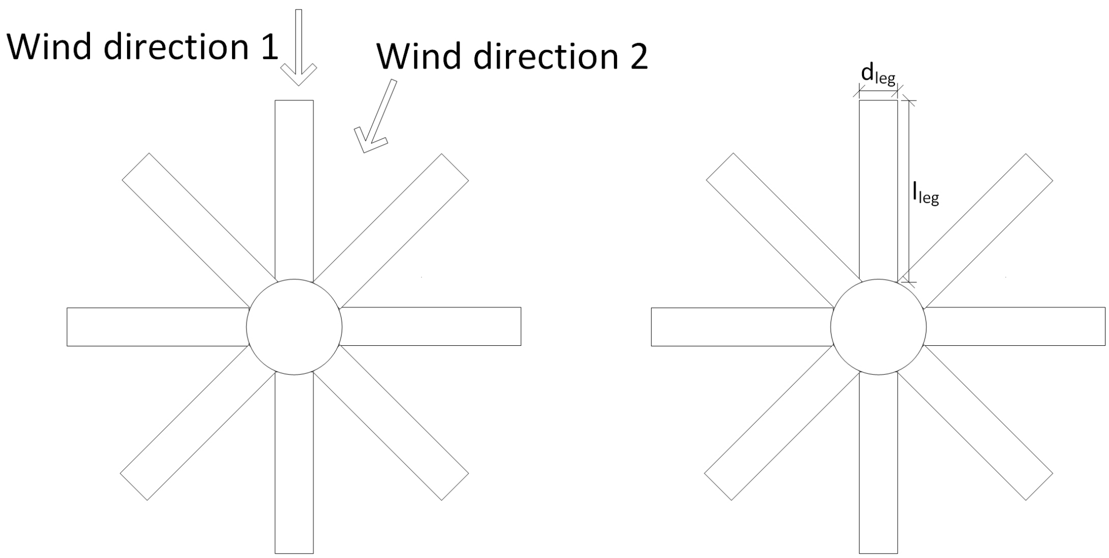


Figure 13: Eight-legged stocky structure. Calculations are performed for two different wind directions

Self-weight of the foundation and the soil

$G_{k.8.st} := \left(8V_{leg.8.st} + V_{centrepiece}\right) \cdot \rho_c + V_{fill.8.st} \cdot \rho_{fill} = 10.02 \cdot MN$ — Characteristic load from the self-weight

$G_{d.8.st} := G_{k.8.st} \cdot \gamma_{fav} = 9.018 \cdot MN$ — Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_{8.st} := \frac{M_d + H_d \cdot h_{centrepiece}}{G_{d.8.st} + N_d} = 8.747\,m$$

Eccentricity for the global stability

Wind direction 1

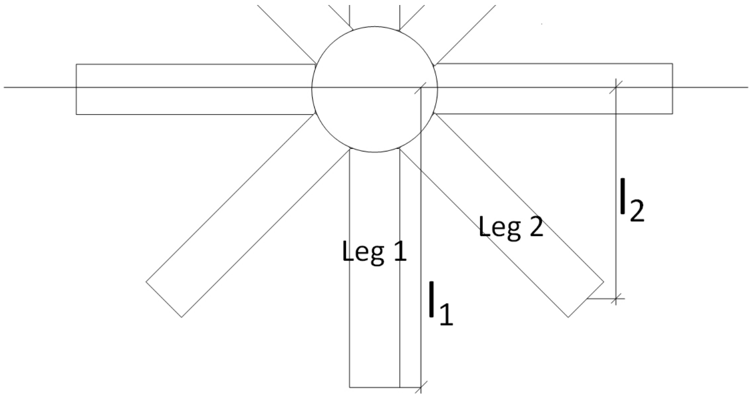


Figure 14: Definition of the lenght of the legs in the direction of the wind, defined from the centre of the foundation

$$l_{1.wind1.8.st} := l_{leg.8.st} + \frac{d_{centrepiece}}{2} = 12\,m$$

Length of leg 1, defined from the center of the foundation

$$l_{2.wind1.8.st} := \cos(45deg)\left(l_{leg.8.st} + \frac{d_{centrepiece}}{2}\right) = 8.485\,m$$

Length of projection of leg 2, defined from the center of the foundation

The eccentricty of the soil pressure should coincide with the resulant of the centre of gravity for the soil pressure area. The distance l_{sp} is iterativly changed in order to fulfill this request.

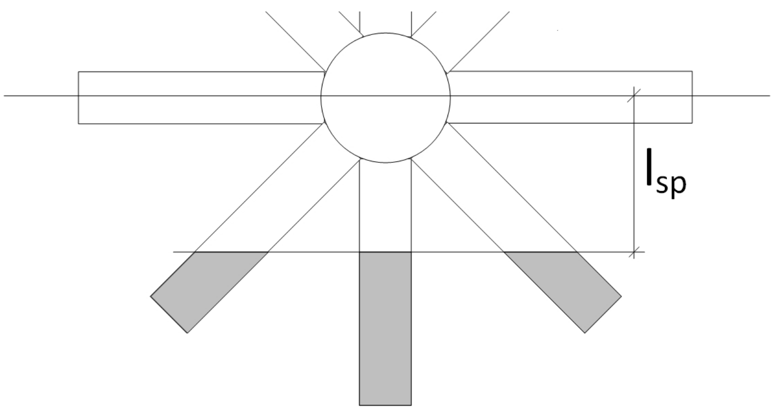


Figure 15: The definition of the lenght l.sp used to calculate the area of the soil pressure. The grey areas represent the soil pressure area.

$$l_{sp.wind1.8.st} := 6.9m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

The area is approximated for leg 2, the projected length is multiplied with the real width of the leg. This is approximate but is on safe side, since this area is slightly smaller than the real area.

$$A_{1.wind1.8st} := \left(l_{1.wind1.8.st} - l_{sp.wind1.8.st}\right)\cdot d_{leg.8.st} = 10.2\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{2.wind1.8st} := 2\cdot\left(l_{2.wind1.8.st} - l_{sp.wind1.8.st}\right)\cdot d_{leg.8.st} = 6.341\,m^2$$

Area of the soil pressure zone for legs 2

$$A_{soil.wind1.8.st} := A_{1.wind1.8st} + A_{2.wind1.8st} = 16.541\,m^2$$

Total area of soil pressure

Resultant centre of gravity of the soil pressure area for all three legs

$$tp_{wind1.8.st} := \frac{A_{1.wind1.8st} \cdot \left(l_{sp.wind1.8.st} + \frac{l_{1.wind1.8.st} - l_{sp.wind1.8.st}}{2} \right) \ldots + A_{2.wind1.8st} \cdot \left(l_{sp.wind1.8.st} + \frac{l_{2.wind1.8.st} - l_{sp.wind1.8.st}}{2} \right)}{A_{soil.wind1.8.st}} = 8.776\,\mathrm{m}$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$\text{Check} := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left| 1 - \frac{tp_{wind1.8.st}}{e_{8.st}} \right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind1.8.st} := \frac{N_d + G_{d.8.st}}{A_{soil.wind1.8.st}} = 708.398 \cdot \mathrm{kPa}$$ Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind1.8.st}}{\sigma_{Rv}} = 0.708$$ Utilisation

$$\text{Check2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.wind1.8.st}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

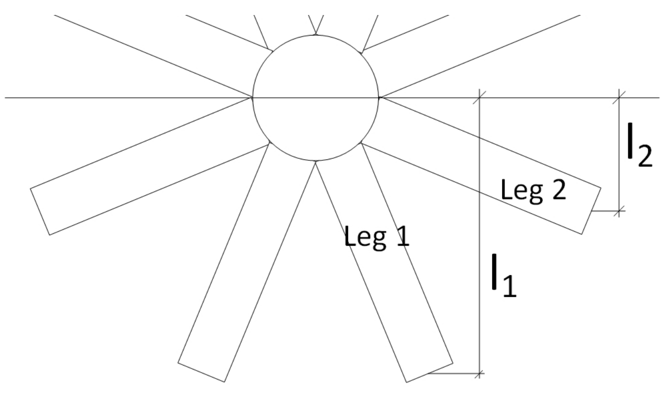


Figure 16: Definition of the lenght of the legs in the direction of the wind, defined from the centre of the foundation

$$l_{1.wind2.8.st} := \left(l_{leg.8.st} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{45}{2}deg\right) = 11.087\,m$$

Length of projection of leg 1, defined from the center of the foundation

$$l_{2.wind2.8.st} := \left(l_{leg.8.st} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{3\cdot45}{2}deg\right) = 4.592\,m$$

Length of projection of leg 2, defined from the center of the foundation

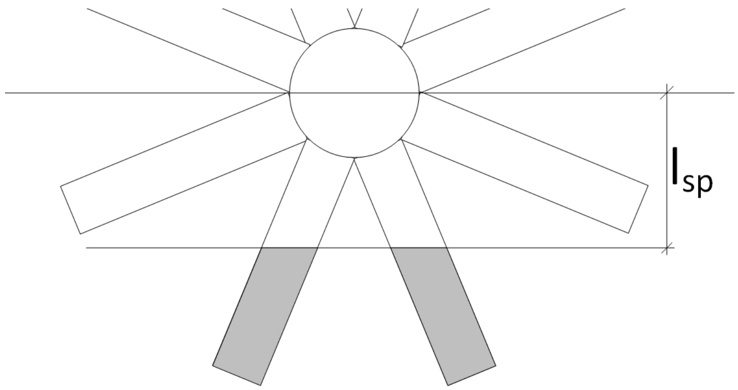


Figure 17: The definition of the lenght l.sp used to calculate the area of the soil pressure. The grey areas represent the soil pressure area.

$$l_{sp.wind2.8.st} := 6.3m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

The same method as for wind direction 1 is used for calculating the soil pressure areas

$$A_{1.wind2.8.st} := 2\left(l_{1.wind2.8.st} - l_{sp.wind2.8.st}\right)\cdot d_{leg.8.st} = 19.146\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{2.wind2.8.st} := \begin{cases} 0 & \text{if } 2\cdot\left(l_{2.wind2.8.st} - l_{sp.wind2.8.st}\right)\cdot d_{leg.8.st} < 0 \\ \left[2\cdot\left(l_{2.wind2.8.st} - l_{sp.wind2.8.st}\right)\cdot d_{leg.8.st}\right] & \text{otherwise} \end{cases} = 0\,m^2$$

Area of the soil pressure zone for legs 2

$$A_{soil.wind2.8.st} := A_{1.wind2.8.st} + A_{2.wind2.8.st} = 19.146\,m^2$$

Total area of soil pressure zone

Resultant centre of gravity of the soil pressure area for all legs

$$tp_{wind2.8.st} := \frac{A_{1.wind2.8.st}\cdot\left(l_{sp.wind2.8.st} + \frac{l_{1.wind2.8.st} - l_{sp.wind2.8.st}}{2}\right)\ ... \\ + A_{2.wind2.8.st}\cdot\left(l_{sp.wind2.8.st} + \frac{l_{2.wind2.8.st} - l_{sp.wind2.8.st}}{2}\right)}{A_{soil.wind2.8.st}} = 8.693\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp_{wind2.8.st}}{e_{8.st}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind2.8.st} := \frac{N_d + G_{d.8.st}}{A_{soil.wind2.8.st}} = 612.012\cdot kPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind2.8.st}}{\sigma_{Rv}} = 0.612$$ Utilisation

$$\text{Check2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.wind2.8.st}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

$\text{Check2} = \text{"OK! Soil resistance is sufficent"}$

Summary of the shape eight-legged stocky

$$V_{8.stocky} := 8 \cdot V_{leg.8.st} + V_{centrepiece} = 353.087 \cdot m^3$$ Total volume of the concrete

$$m_{8.stocky} := V_{8.stocky} \cdot \frac{\rho_c}{g} = 900.122 \cdot ton$$ Total weight of the concrete

$$m_{leg.8.stocky} := V_{leg.8.st} \cdot \frac{\rho_c}{g} = 96.873 \cdot ton$$ Weight of one leg

$$l_{foundation.8.stocky} := 2 \cdot l_{leg.8.st} + d_{centrepiece} = 24\,m$$ Total length of the foundation

8. Eight legged slim structure

Geometry of the foundation

$d_{leg.8.sl} := 1\text{m}$ — Width of the leg

$l_{leg.8.sl} := 13.5\text{m}$ — Length of the leg

$h_{leg.8.sl} := 2\text{m}$ — Height of the leg

$V_{leg.8.sl} := d_{leg.8.sl} \cdot l_{leg.8.sl} \cdot h_{leg.8.sl} = 27 \cdot \text{m}^3$ — Volume of one leg

$h_{fill.8.sl} := 0.5\text{m}$ — Height of fill

$V_{fill.8.sl} := 8 l_{leg.8.sl} \cdot d_{leg.8.sl} \cdot h_{fill.8.sl} = 54 \cdot \text{m}^3$ — Volume of the fill

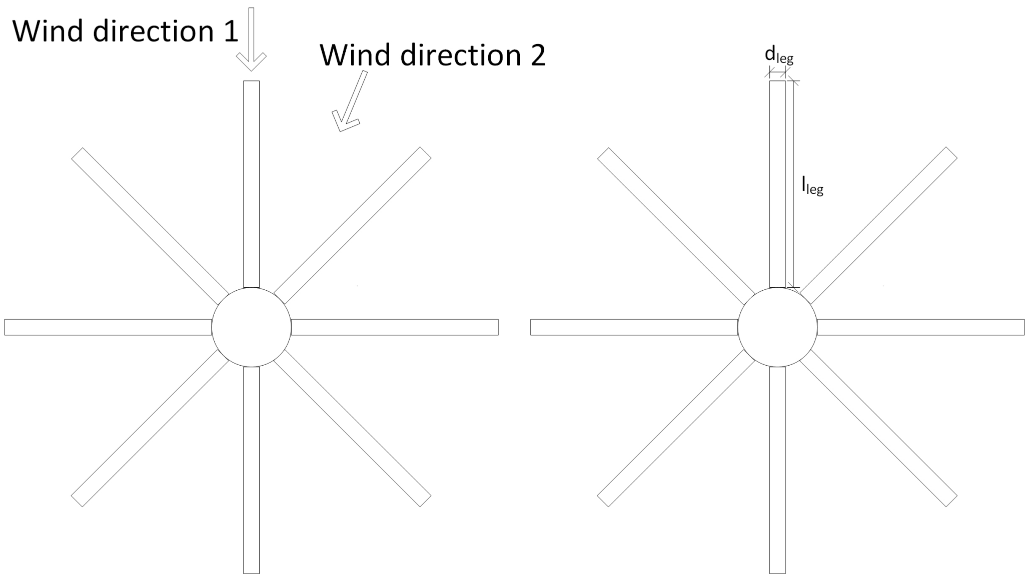


Figure 18: Eight-legged slim structure. Calculations are performed for two different wind directions

Self-weight of the foundation and the soil

$G_{k.8.sl} := \left(8V_{leg.8.sl} + V_{centrepiece}\right) \cdot \rho_c + V_{fill.8.sl} \cdot \rho_{fill} = 7.474 \cdot \text{MN}$ — Characteristic load from the self-weight

$G_{d.8.sl} := G_{k.8.sl} \cdot \gamma_{fav} = 6.727 \cdot \text{MN}$ — Design load for the self-weight, self-weight is a favourable load when calculating the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_{8.sl} := \frac{M_d + H_d \cdot h_{centrepiece}}{G_{d.8.sl} + N_d} = 10.873\,\text{m}$$

Eccentricity for the soil pressure resultant

The same method is used as for eight-legged stocky. Defintions and lengths are made in the figures in chapter above.

Wind direction 1

$$l_{1.wind1.8.sl} := l_{leg.8.sl} + \frac{d_{centrepiece}}{2} = 16\,m$$

Length of leg 1, defined from the center of the foundation

$$l_{2.wind1.8.sl} := \cos(45deg)\left(l_{leg.8.sl} + \frac{d_{centrepiece}}{2}\right) = 11.314\,m$$

Length of projection of leg 2, defined from the center of the foundation

Soil pressure

$$l_{sp.wind1.8.sl} := 7.9m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

The area is approximated for leg 2, the projected length is multiplied with the real width of the leg. This is approximate but is on safe side, since this area is slightly smaller than the real area.

$$A_{1.wind1.8sl} := \left(l_{1.wind1.8.sl} - l_{sp.wind1.8.sl}\right)\cdot d_{leg.8.sl} = 8.1\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{2.wind1.8sl} := 2\cdot\left(l_{2.wind1.8.sl} - l_{sp.wind1.8.sl}\right)\cdot d_{leg.8.sl} = 6.827\,m^2$$

Area of the soil pressure zone for legs 2

$$A_{soil.wind1.8.sl} := A_{1.wind1.8sl} + A_{2.wind1.8sl} = 14.927\,m^2$$

Total area of soil pressure

Resultant centre of gravity of the soil pressure area for all three legs

$$tp_{wind1.8.sl} := \frac{A_{1.wind1.8sl}\cdot\left(l_{sp.wind1.8.sl} + \frac{l_{1.wind1.8.sl} - l_{sp.wind1.8.sl}}{2}\right) \ldots + A_{2.wind1.8sl}\cdot\left(l_{sp.wind1.8.sl} + \frac{l_{2.wind1.8.sl} - l_{sp.wind1.8.sl}}{2}\right)}{A_{soil.wind1.8.sl}} = 10.878\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp_{wind1.8.sl}}{e_{8.sl}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind1.8.sl} := \frac{G_{d.8.sl} + N_d}{A_{soil.wind1.8.sl}} = 631.525\cdot kPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind1.8.sl}}{\sigma_{Rv}} = 0.632$$

Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.wind1.8.sl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

$$l_{1.wind2.8.sl} := \left(l_{leg.8.sl} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{45}{2}deg\right) = 14.782\,m$$

Length of projection of leg 1, defined from the center of the foundation

$$l_{2.wind2.8.sl} := \left(l_{leg.8.sl} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{3\cdot45}{2}deg\right) = 6.123\,m$$

Length of projection of leg 2, defined from the center of the foundation

Soil pressure

$$l_{sp.wind2.8.sl} := 7m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

$$A_{1.wind2.8.sl} := 2\left(l_{1.wind2.8.sl} - l_{sp.wind2.8.sl}\right)\cdot d_{leg.8.sl} = 15.564\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{2.wind2.8.sl} := \begin{cases} 0 & \text{if } 2\cdot\left(l_{2.wind2.8.sl} - l_{sp.wind2.8.sl}\right)\cdot d_{leg.8.sl} < 0 \\ 2\cdot\left(l_{2.wind2.8.sl} - l_{sp.wind2.8.sl}\right)\cdot d_{leg.8.sl} & \text{otherwise} \end{cases} = 0\,m^2$$

Area of the soil pressure zone for legs 2

$$A_{soil.wind2.8.sl} := A_{1.wind2.8.sl} + A_{2.wind2.8.sl} = 15.564\,m^2$$

Total area of soil pressure

Resultant centre of gravity of the soil pressure area for all legs

$$tp_{wind2.8.sl} := \frac{A_{1.wind2.8.sl}\cdot\left(l_{sp.wind2.8.sl} + \frac{l_{1.wind2.8.sl} - l_{sp.wind2.8.sl}}{2}\right) ... + A_{2.wind2.8.sl}\cdot\left(l_{sp.wind2.8.sl} + \frac{l_{2.wind2.8.sl} - l_{sp.wind2.8.sl}}{2}\right)}{A_{soil.wind2.8.sl}} = 10.891\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp_{wind2.8.sl}}{e_{8.sl}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind2.8.sl} := \frac{G_{d.8.sl} + N_d}{A_{soil.wind2.8.sl}} = 0.606\cdot MPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind2.8.sl}}{\sigma_{Rv}} = 0.606$$

Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.wind2.8.sl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Summary of the shape eight-legged slim

$V_{8.slim} := 8 \cdot V_{leg.8.sl} + V_{centrepiece} = 265.087 \cdot m^3$ Total volume of the concrete

$m_{8.slim} := V_{8.slim} \cdot \frac{\rho_c}{g} = 675.785 \cdot ton$ Total weight of the concrete

$m_{leg.8.slim} := V_{leg.8.sl} \cdot \frac{\rho_c}{g} = 68.831 \cdot ton$ Weight of one leg

$l_{foundation.8.slim} := 2 \cdot l_{leg.8.sl} + d_{centrepiece} = 32\,m$ Total length of the foundation

9. Eight legged structure with bottom flange

Geometry of the foundation

$d_{leg.8.fl} := 0.8m$ Width of the leg

$l_{leg.8.fl} := 10m$ Length of the leg

$h_{leg.8.fl} := 2m$ Height of the leg

$l_{foundation.8.flange} := l_{leg.8.fl} \cdot 2 + d_{centrepiece} = 25\,m$ Total lenght of the foundation

The flange is assumed to placed under the legs

$d_{flange.8} := 2m$ Width of the flange on each leg

$h_{flange.8} := 0.3m$ Height of the flange

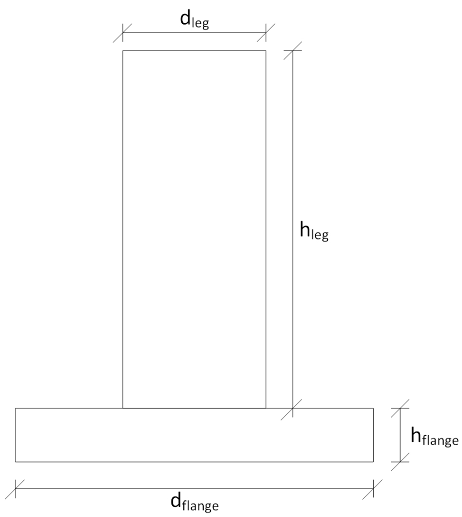


Figure 19: Cross-section of one leg with flange

$$V_{leg.8.fl} := d_{leg.8.fl} \cdot l_{leg.8.fl} \cdot h_{leg.8.fl} \cdots + d_{flange.8} \cdot h_{flange.8} \cdot l_{leg.8.fl} = 22 \cdot m^3$$ Volume of one leg

$h_{fill.leg} := 0.5m$ Height of the fill over the leg

$h_{fill.flange} := h_{leg.8.fl} + h_{fill.leg} - h_{flange.8} = 2.2\,m$ Height of the fill over the flange

$$V_{fill.8.fl} := 8 l_{leg.8.fl} \cdot d_{leg.8.fl} \cdot h_{fill.leg} \cdots + 8 l_{leg.8.fl} \cdot \left(d_{flange.8} - d_{leg.8.fl}\right) \cdot h_{fill.flange} = 243.2 \cdot m^3$$ Volume of the fill

$$\beta_8 := \frac{360deg}{8} = 45 \cdot deg$$ Angle between the legs

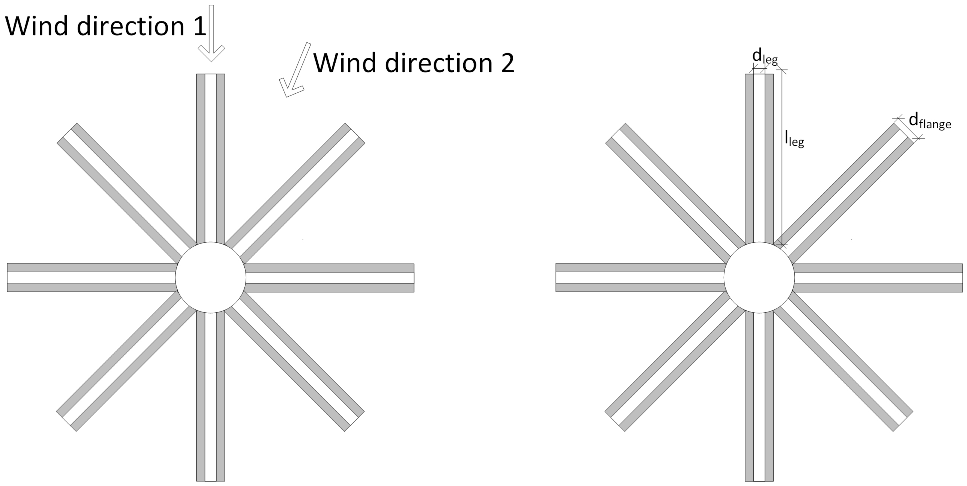


Figure 20: Eight-legged structure with bottom flange. Calculations are performed for two different wind directions

Self-weight of the foundation and the soil

$G_{k.8.fl} := \left(8V_{leg.8.fl} + V_{centrepiece}\right)\cdot\rho_c + V_{fill.8.fl}\cdot\rho_{fill} = 9.443\cdot MN$ Characteristic self-weight

$G_{d.8.fl} := G_{k.8.fl}\cdot\gamma_{fav} = 8.499\cdot MN$ Design load for the self-weight, self-weight is a favourable load when calculating the global stabilty

Verification of the choice of bottom flange instead of bottom plate

In order to determine if a whole plate or a flange is most efficient for taking the soil pressure its verified whether the effective flange is smaller or larger than the half of the span between the legs.

$a := 2\cdot\left(l_{leg.8.fl} + \frac{d_{centrepiece}}{2}\right)\cdot\sin\left(\frac{\beta_8}{2}\right) = 9.567\,m$ The maximum span between the legs, calculated with the angle β and trigonometry

$b_i := \frac{a}{2}$ Half of the largest span between the legs, at the outermost part of the foundation

$l_0 := \frac{l_{foundation.8.flange}}{2} = 12.5\,m$ The distance between the moment zero points is assumed to be half the length of the foundation

$b_{eff} := \min\left(0.2\cdot b_i + 0.1\cdot l_0, b_i\right) = 2.207\,m$ Effective width

$$Check_{eff.width} := \begin{cases} \text{"The whole plate can take the soil pressure"} & \text{if } b_{eff} = \frac{a}{2} \\ \text{"The whole plate can not transfer the load the legs"} & \text{if } b_{eff} < \frac{a}{2} \end{cases}$$

$Check_{eff.width} =$ "The whole plate can not transfer the load the legs"

Therfore a concept with 8 legs and a bottom flange is choosen instead of a concept with a whole bottom plate. The flange width is chosen according to the effective width and also adapted to the geometry of the centrepiece.

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_{8.fl} := \frac{M_d + H_d \cdot h_{centrepiece}}{G_{d.8.fl} + N_d} = 9.153\,m$$

Eccentricity for the soil pressure resultant

The same method is used as for eight-legged stocky. Defintions and lengths are made in the figures in chapter above.

Wind direction 1

The length calculated below is defined similarly as for the concept; 8 legged stocky structure

$$l_{1.wind1.8.fl} := l_{leg.8.fl} + \frac{d_{centrepiece}}{2} = 12.5\,m$$

Length of leg 1, defined from the center of the foundation

$$l_{2.wind1.8.fl} := \cos(\beta_8)\left(l_{leg.8.fl} + \frac{d_{centrepiece}}{2}\right) = 8.839\,m$$

Length of projection of leg 2, defined from the center of the foundation

Soil pressure

$$l_{sp.wind1.8.fl} := 6.7m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

The area is approximated for leg 2, the projected length is multiplied with the real width of the leg. This is approximate but is on safe side, since this area is slightly smaller than the real area.

$$A_{1.wind1.8.fl} := \left(l_{1.wind1.8.fl} - l_{sp.wind1.8.fl}\right) \cdot d_{flange.8} = 11.6\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{2.wind1.8.fl} := 2 \cdot \left(l_{2.wind1.8.fl} - l_{sp.wind1.8.sl}\right) \cdot d_{flange.8} = 3.755\,m^2$$

Area of the soil pressure zone for legs 2

$$A_{soil.wind1.8.fl} := A_{1.wind1.8.fl} + A_{2.wind1.8.fl} = 15.355\,m^2$$

Total area of soil pressure

Resultant centre of gravity of the soil pressure area for all three legs

$$tp_{wind1.8.fl} := \frac{A_{1.wind1.8.fl} \cdot \left(l_{sp.wind1.8.fl} + \frac{l_{1.wind1.8.fl} - l_{sp.wind1.8.fl}}{2}\right) \ldots + A_{2.wind1.8.fl} \cdot \left(l_{sp.wind1.8.fl} + \frac{l_{2.wind1.8.fl} - l_{sp.wind1.8.fl}}{2}\right)}{A_{soil.wind1.8.fl}} = 9.152\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{array}{|l} \text{"The soil pressure zone is correct"} \quad \text{if } \left|1 - \frac{tp_{wind1.8.fl}}{e_{8.fl}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} \quad \text{otherwise} \end{array}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind1.8.fl} := \frac{G_{d.8.fl} + N_d}{A_{soil.wind1.8.fl}} = 0.729 \cdot MPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind1.8.fl}}{\sigma_{Rv}} = 0.729$$

Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.wind1.8.fl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

$$l_{1.wind2.8.fl} := \left(l_{leg.8.fl} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{\beta_8}{2}\right) = 11.548\,m$$

Length of projection of leg 1, defined from the center of the foundation

$$l_{2.wind2.8.fl} := \left(l_{leg.8.fl} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{3\cdot\beta_8}{2}\right) = 4.784\,m$$

Length of projection of leg 2, defined from the center of the foundation

Soil pressure

$$l_{sp.wind2.8.fl} := 6.8m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

$$A_{1.wind2.8.fl} := 2\left(l_{1.wind2.8.fl} - l_{sp.wind2.8.fl}\right)\cdot d_{flange.8} = 18.994\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{2.wind2.8.fl} := \begin{cases} 0 & \text{if } 2\cdot\left(l_{2.wind2.8.fl} - l_{sp.wind2.8.sl}\right)\cdot d_{flange.8} < 0 \\ 2\cdot\left(l_{2.wind2.8.fl} - l_{sp.wind2.8.sl}\right)\cdot d_{flange.8} & \text{otherwise} \end{cases} = 0\,m^2$$

Area of the soil pressure zone for leg 2

$$A_{soil.wind2.8.fl} := A_{1.wind2.8.fl} + A_{2.wind2.8.fl} = 18.994\,m^2$$

Total area of the soil pressure zone

Resultant centre of gravity of the soil pressure area for all legs

$$tp_{wind2.8.fl} := \frac{A_{1.wind2.8.fl}\cdot\left(l_{sp.wind2.8.fl} + \dfrac{l_{1.wind2.8.fl} - l_{sp.wind2.8.fl}}{2}\right) \dots + A_{2.wind2.8.fl}\cdot\left(l_{sp.wind2.8.fl} + \dfrac{l_{2.wind2.8.fl} - l_{sp.wind2.8.fl}}{2}\right)}{A_{soil.wind2.8.fl}} = 9.174\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \dfrac{tp_{wind2.8.fl}}{e_{8.fl}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind2.8.fl} := \frac{G_{d.8.fl} + N_d}{A_{soil.wind2.8.fl}} = 0.59\cdot MPa$$

Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind2.8.fl}}{\sigma_{Rv}} = 0.59$$ Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.wind2.8.fl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Summary of the shape eight-legged with flange

$$V_{8.flange} := 8 \cdot V_{leg.8.fl} + V_{centrepiece} = 225.087 \cdot m^3$$ Total volume of the concrete

$$m_{8.flange} := V_{8.flange} \cdot \frac{\rho_c}{g} = 573.813 \cdot ton$$ Total weight of the concrete

$$m_{leg.8.flange} := V_{leg.8.fl} \cdot \frac{\rho_c}{g} = 56.084 \cdot ton$$ Weight of one leg

10. Sixteen legged structure

Geometry of the foundation

$d_{leg.16} := 1m$ Width of the leg

$l_{leg.16} := 10m$ Length of the leg

$h_{leg.16} := 2m$ Height of the leg

$V_{leg.16} := d_{leg.16} \cdot l_{leg.16} \cdot h_{leg.16} = 20 \cdot m^3$ Volume of one leg

$h_{fill.16} := 0.5m$ Height of fill above the legs

$V_{fill.16} := 16 l_{leg.16} \cdot d_{leg.16} \cdot h_{fill.16} = 80 \cdot m^3$ Volume of the fill above the legs

$\beta_{16} := \frac{360deg}{16} = 22.5 \cdot deg$ Angle between the legs

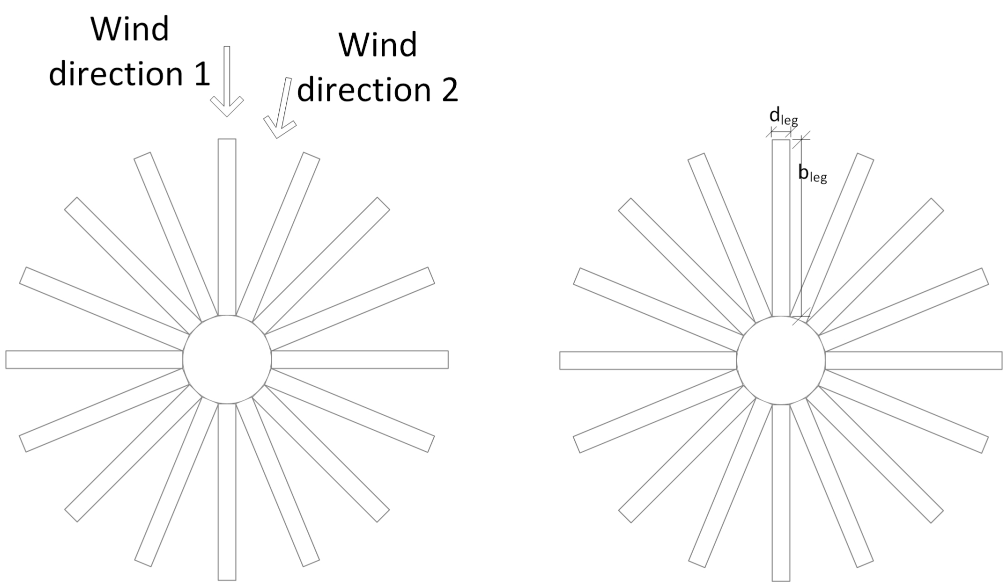


Figure 21: Sixteen-legged structure. Calculations are performed for two different wind directions

Self-weight of the foundation and the soil

$G_{k.16} := \left[\left(16V_{leg.16} + V_{centrepiece}\right) \cdot \rho_c + V_{fill.16} \cdot \rho_{fill}\right] = 10.482 \cdot MN$ Characteristic self-weight

$G_{d.16} := G_{k.16} \cdot \gamma_{fav} = 9.434 \cdot MN$ Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_{16} := \frac{M_d + H_d \cdot h_{centrepiece}}{G_{d.16} + N_d} = 8.447\,m$$

Eccentricity for the soil pressure resultant

The same method is used as for eight-legged stocky. Defintions and lengths are made in the figures in this chapter.

Wind direction 1

$$l_{1.wind1.16} := l_{leg.16} + \frac{d_{centrepiece}}{2} = 12.5\,m$$

Length of leg 1, defined from the center of the foundation

$$l_{2.wind1.16} := \cos(\beta_{16})\left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right) = 11.548\,m$$

Length of projection of leg 2, defined from the center of the foundation

$$l_{3.wind1.16} := \cos(2 \cdot \beta_{16})\left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right) = 8.839\,m$$

Length of projection of leg 3, defined from the center of the foundation

$$l_{4.wind1.16} := \cos(3 \cdot \beta_{16})\left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right) = 4.784\,m$$

Length of projection of leg 4, defined from the center of the foundation

Soil pressure

$$l_{sp.wind1.16} := 5.8m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively

The area is approximated for leg 2, the projected length is multiplied with the real width of the leg. This is approximate but is on safe side, since this area is slightly smaller than the real area.

$$A_{1.wind1.16} := (l_{1.wind1.16} - l_{sp.wind1.16}) \cdot d_{leg.16} = 6.7\,m^2$$

Area of the soil pressure zone for the different legs

$$A_{2.wind1.16} := 2(l_{2.wind1.16} - l_{sp.wind1.16}) \cdot d_{leg.16} = 11.497\,m^2$$

$$A_{3.wind1.16} := 2(l_{3.wind1.16} - l_{sp.wind1.16}) \cdot d_{leg.16} = 6.078\,m^2$$

$$A_{4.wind1.16} := \begin{cases} 0 & \text{if } 2(l_{4.wind1.16} - l_{sp.wind1.16}) \cdot d_{leg.16} < 0 \\ 2(l_{4.wind1.16} - l_{sp.wind1.16}) \cdot d_{leg.16} & \text{otherwise} \end{cases} = 0\,m^2$$

$$A_{soil.wind1.16} := A_{1.wind1.16} + A_{2.wind1.16} + A_{3.wind1.16} + A_{4.wind1.16} = 24.275\,m^2$$

Resultant centre of gravity for all the legs that contributes to the soil pressure

$$tp_{wind1.16} := \frac{\begin{aligned} & A_{1.wind1.16}\cdot\left(l_{sp.wind1.16} + \frac{l_{1.wind1.16} - l_{sp.wind1.16}}{2}\right) \ldots \\ & + A_{2.wind1.16}\cdot\left(l_{sp.wind1.16} + \frac{l_{2.wind1.16} - l_{sp.wind1.16}}{2}\right) \ldots \\ & + A_{3.wind1.16}\cdot\left(l_{sp.wind1.16} + \frac{l_{3.wind1.16} - l_{sp.wind1.16}}{2}\right) \ldots \\ & + A_{4.wind1.16}\cdot\left(l_{sp.wind1.16} + \frac{l_{4.wind1.16} - l_{sp.wind1.16}}{2}\right) \end{aligned}}{A_{soil.wind1.16}} = 8.466\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$\text{Check} := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp_{wind1.16}}{e_{16}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind1.16} := \frac{G_{d.16} + N_d}{A_{soil.wind1.16}} = 0.5\cdot MPa$$

Soil pressure

Check if the resistance of the soil is sufficient

Utilisation

$$\frac{\sigma_{soil.wind1.16}}{\sigma_{Rv}} = 0.5$$

$$\text{Check2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.wind1.16}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Wind direction 2

$$l_{1.wind2.16} := \left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{\beta_{16}}{2}\right) = 12.26\,m$$

Length of projection of leg 1, defined from the center of the foundation

$$l_{2.wind2.16} := \left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{3\cdot\beta_{16}}{2}\right) = 10.393\,m$$

Length of projection of leg 2, defined from the center of the foundation

$$l_{3.wind2.16} := \left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{5\cdot\beta_{16}}{2}\right) = 6.945\,m$$

Length of projection of leg 3, defined from the center of the foundation

$$l_{4.wind2.16} := \left(l_{leg.16} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{7\cdot\beta_{16}}{2}\right) = 2.439\,m$$

Length of projection of leg 4, defined from the center of the foundation

Soil pressure

$l_{sp.wind2.16} := 6.1m$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively

$A_{1.wind2.16} := \left(l_{1.wind2.16} - l_{sp.wind2.16}\right)\cdot d_{leg.16} = 6.16\,m^2$

Area of the soil pressure zone for the different legs

$A_{2.wind2.16} := 2\left(l_{2.wind2.16} - l_{sp.wind2.16}\right)\cdot d_{leg.16} = 8.587\,m^2$

$A_{3.wind2.16} := 2\left(l_{3.wind2.16} - l_{sp.wind2.16}\right)\cdot d_{leg.16} = 1.689\,m^2$

$$A_{4.wind2.16} := \begin{cases} 0 & \text{if } 2\left(l_{4.wind2.16} - l_{sp.wind2.16}\right)\cdot d_{leg.16} < 0 \\ 2\left(l_{4.wind2.16} - l_{sp.wind2.16}\right)\cdot d_{leg.16} & \text{otherwise} \end{cases} = 0\,m^2$$

$A_{soil.wind2.16} := A_{1.wind2.16} + A_{2.wind2.16} + A_{3.wind2.16} + A_{4.wind2.16} = 16.436\,m^2$

Resultant centre of gravity for all the legs that contributes to the soil pressure

$$tp_{wind2.16} := \frac{\begin{gathered} A_{1.wind2.16}\cdot\left(l_{sp.wind2.16} + \frac{l_{1.wind2.16} - l_{sp.wind2.16}}{2}\right) \ldots \\ + A_{2.wind2.16}\cdot\left(l_{sp.wind2.16} + \frac{l_{2.wind2.16} - l_{sp.wind2.16}}{2}\right) \ldots \\ + A_{3.wind2.16}\cdot\left(l_{sp.wind2.16} + \frac{l_{3.wind2.16} - l_{sp.wind2.16}}{2}\right) \ldots \\ + A_{4.wind2.16}\cdot\left(l_{sp.wind2.16} + \frac{l_{4.wind2.16} - l_{sp.wind2.16}}{2}\right) \end{gathered}}{A_{soil.wind2.16}} = 8.419\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp_{wind2.16}}{e_{16}}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$\sigma_{soil.wind2.16} := \frac{G_{d.16} + N_d}{A_{soil.wind2.16}} = 0.738\cdot MPa$

Soil pressure

Check if the resistance of the soil resistance is sufficient

$\frac{\sigma_{soil.wind2.16}}{\sigma_{Rv}} = 0.738$

Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.wind2.16}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Summary of shape with sixteen legs

$V_{16} := 16 \cdot V_{leg.16} + V_{centrepiece} = 369.087 \cdot m^3$ Total volume of the concrete

$m_{16} := V_{16} \cdot \frac{\rho_c}{g} = 940.911 \cdot ton$ Total weight of the concrete

$m_{leg.16} := V_{leg.16} \cdot \frac{\rho_c}{g} = 50.986 \cdot ton$ Weight of one leg

$l_{foundation.16} := 2 \cdot l_{leg.16} + d_{centrepiece} = 25\,m$ Total length of the foundation

11. Sixteen legged structure with a bottom plate

Geometry of the foundation

$d_{leg.16.pl} := 0.3m$	Width of the leg
$l_{leg.16.pl} := 6.5m$	Length of the leg
$h_{leg.16.pl} := 2m$	Height of the leg
$l_{foundation.16.plate} := 2l_{leg.16.pl} + d_{centrepiece} = 18\,m$	Total length of the foundation
$V_{leg.16.pl} := d_{leg.16.pl} \cdot l_{leg.16.pl} \cdot h_{leg.16.pl} = 3.9 \cdot m^3$	Volume of one leg
$A_{plate.16} := \left(l_{leg.16.pl} + \frac{d_{centrepiece}}{2}\right)^2 \cdot \pi \ldots + -A_{centrepiece} - 16d_{leg.16.pl} \cdot l_{leg.16.pl} = 203.634\,m^2$	Area of the bottom plate excluding the legs and the centrepiece
$h_{plate.16} := 0.3m$	Height of the bottom plate
$V_{plate.16} := h_{plate.16} \cdot A_{plate.16} = 61.09 \cdot m^3$	Volume of the bottom plate
$h_{fill.16.pl} := 0.5m$	Height of fill above the legs
$h_{fill.plate.16} := h_{leg.16.pl} + h_{fill.16.pl} - h_{plate.16} = 2.2\,m$	Height of fill above plate
$V_{fill.16.pl} := 16l_{leg.16.pl} \cdot d_{leg.16.pl} \cdot h_{fill.16.pl} \ldots + A_{plate.16} \cdot h_{fill.plate.16} = 463.595 \cdot m^3$	Total volume of the fill
$\beta_{16} := \frac{360deg}{16} = 22.5 \cdot deg$	Angle between the legs

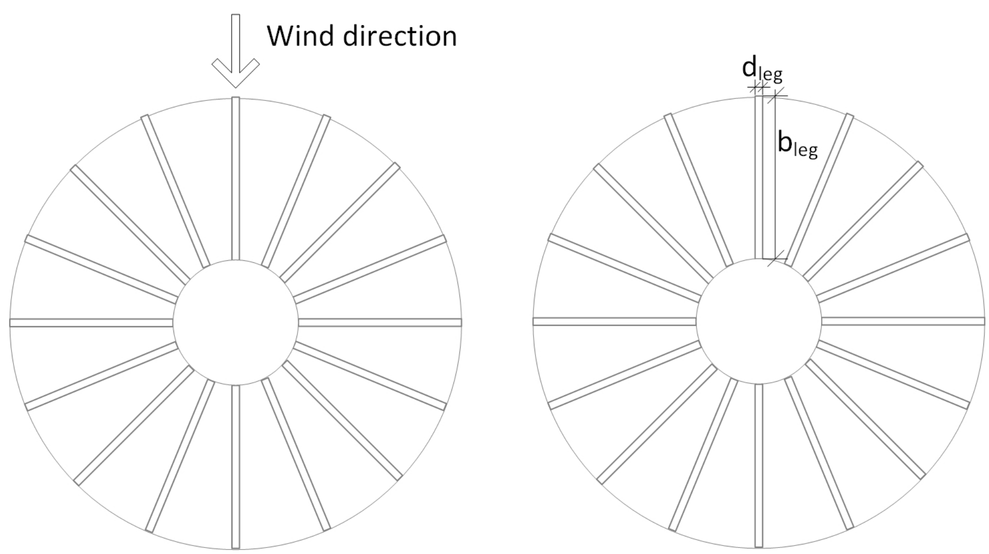


Figure 22: Sixteen legs with bottom plate. Calculations are performed for one wind direction, due to symmetry.

Self-weight of the foundation and the soil

$$G_{k.16.pl} := \left[\left(16V_{leg.16.pl} + V_{centrepiece} + V_{plate.16}\right)\cdot\rho_c + V_{fill.16.pl}\cdot\rho_{fill}\right] = 11.589\cdot MN$$ Characteristic self-weight

$$G_{d.16.pl} := G_{k.16.pl}\cdot\gamma_{fav} = 10.43\cdot MN$$ Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Effective flange width

In order to determine how much of the plate that can contribute and take the soil pressure, the effective width for the legs are calculated.

$$a := 2\cdot\left(l_{leg.16.pl} + \frac{d_{centrepiece}}{2}\right)\cdot\sin\left(\frac{\beta_{16}}{2}\right) = 3.512\,m$$ The maximum span between each leg, calculated with the angle β and trigonometry

$$b_i := \frac{a}{2}$$ Half of the largest span between the legs, at the outermost part of the foundation

$$l_0 := \frac{l_{foundation.16.plate}}{2} = 9\,m$$ The distance between the moment zero points, it is assumed to be half the length of the foundation.

$$b_{eff} := \min\left(0.2\cdot b_i + 0.1\cdot l_0, b_i\right) = 1.251\,m$$ Effective width

$$Check_{eff.width} := \begin{cases} \text{"The whole plate takes the soil pressure"} & \text{if } b_{eff} = \frac{a}{2} \\ \text{"The effective width takes the soil pressure"} & \text{if } b_{eff} < \frac{a}{2} \end{cases}$$

$$Check_{eff.width} = \text{"The effective width takes the soil pressure"}$$

Each leg with its respective effective width contribute to the area of the soil pressure.Only a small part of the plate between the legs do not contribute. It is investigated whether it is possible to disregard this and calculate the soil pressure as if it was taken by the whole plate.

$$A_{part} := \frac{\frac{a}{2} - b_{eff}}{\tan\left(\frac{\alpha}{2}\right)}\cdot\left(\frac{a}{2} - b_{eff}\right) = 0.255\,m^2$$ Area of the plate which do not contribute to the soil pressure area.

$$A_{foundation} := \frac{l_{foundation.16.plate}^2\cdot\pi}{4} = 254.469\,m^2$$ Total area of the foundation

$$A_{foundation.cont} := A_{foundation} - 16A_{part} = 250.394\,m^2$$ The total area of the plate that contributes and can take the soil pressure

$$\frac{A_{foundation.cont}}{A_{foundation}} = 0.984$$ Utilisation

The difference between the areas is very small, thus the area which does not contribute is very small. The calculation are simplified so that the whole plate is assumed to take the soil pressure.

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_{16.pl} := \frac{M_d + H_d \cdot h_{centrepiece}}{G_{d.16.pl} + N_d} = 7.807\,m$$

Eccentricity for global stability

Check that the eccentricity fits within half of the foundation:

$$Check_{eccentricity} := \begin{array}{|l} \text{"The eccentricity is ok!"} \quad \text{if } e_{16.pl} \le \dfrac{l_{foundation.16.plate}}{2} \\ \text{"Eccentricity is not ok!"} \quad \text{otherwise} \end{array}$$

$$Check_{eccentricity} = \text{"The eccentricity is ok!"}$$

Calculation of the angle of the compression zone

The soil pressure calculations follow the same rocedure as for the solid circular concept.

$$\alpha := 100deg$$

Assumed angle, defined as shown in the figure. Change iteratively

$$x := \cos(0.5\alpha)\cdot\frac{l_{foundation.16.plate}}{2} = 5.785\,m$$

Geometrical length from the center of the foundation to where the radial part of the circle sector start.

$$tp_1 := \frac{2}{3}\cdot\frac{l_{foundation.16.plate}}{2}\cdot\frac{\sin(0.5\alpha)}{\frac{\alpha}{2}} = 5.267\,m$$

Centre of gravity for the whole sector A_1

$$tp_2 := \frac{2}{3}\cdot x = 3.857\,m$$

Centre of gravity of the triangle A_2

$$A_1 := \frac{\alpha}{2}\cdot\left(\frac{l_{foundation.16.plate}}{2}\right)^2 = 70.686\,m^2$$

Area of the whole sector A_1

$$A_2 := 2\left(x \cdot x \cdot \frac{\tan(0.5\alpha)}{2}\right) = 39.885\,m^2$$ Area of the triangle A_2

$$tp := \frac{A_1 \cdot tp_1 - A_2 \cdot tp_2}{A_1 + A_2} + x = 7.761\,m$$ Centre of gravity of the segment

Check if the assumed angle is correct. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$\text{Check2} := \begin{cases} \text{"Angle ok"} & \text{if } \left|1 - \frac{tp}{e_{16.pl}}\right| < 1\% \\ \text{"Too large difference"} & \text{otherwise} \end{cases}$$

Check2 = "Angle ok"

Soil pressure

$$A_{soil.16.pl} := A_1 - A_2 = 30.801\,m^2$$ Area of the soil pressure zone

$$\sigma_{soil.16.pl} := \frac{G_{d.16.pl} + N_d}{A_{soil.16.pl}} = 426.273 \cdot kPa$$ Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.16.pl}}{\sigma_{Rv}} = 0.426$$ Utilisation

$$\text{Check3} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.16.pl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check3 = "OK! Soil resistance is sufficent"

Summary of shape sixteen-legged with bottom plate

$$V_{16.plate} := 16 \cdot V_{leg.16.pl} + V_{centrepiece} + V_{plate.16} = 172.578 \cdot m^3$$ Total volume of the concrete

$$m_{16.plate} := V_{16.plate} \cdot \frac{\rho_c}{g} = 439.95 \cdot ton$$ Total weight of the concrete

$$m_{leg.16.plate} := V_{leg.16.pl} \cdot \frac{\rho_c}{g} = 9.942 \cdot ton$$ Weight of one leg

$$m_{element.16.plate} := V_{leg.16.pl} \cdot \frac{\rho_c}{g} + \frac{V_{plate.16}}{16} \cdot \frac{\rho_c}{g} = 19.676 \cdot ton$$ Weight of one element (leg and plate)

12. Twenty legs with bottom plate

Geometry of the foundation

$d_{leg.20.pl} := 0.3m$ — Width of the leg

$l_{leg.20.pl} := 6.5m$ — Length of the leg

$h_{leg.20.pl} := 2m$ — Height of the leg

$V_{leg.20.pl} := d_{leg.20.pl} \cdot l_{leg.20.pl} \cdot h_{leg.20.pl} = 3.9 \cdot m^3$ — Volume of one leg

$$A_{plate.20} := \left(l_{leg.20.pl} + \frac{d_{centrepiece}}{2}\right)^2 \cdot \pi \ldots + -A_{centrepiece} - 20 d_{leg.20.pl} \cdot l_{leg.20.pl} = 195.834\, m^2$$

Area of the bottom plate excluding the legs and the centerpiece

$h_{plate.20} := 0.3m$ — Height of the bottom plate

$V_{plate.20} := h_{plate.20} \cdot A_{plate.20} = 58.75 \cdot m^3$ — Volume of the bottom plate

$h_{fill.20.pl} := 0.5m$ — Height of fill above the legs

$h_{fill.plate.20} := h_{leg.20.pl} + h_{fill.20.pl} - h_{plate.20} = 2.2\, m$ — Height of fill above plate

$$V_{fill.20.pl} := 20 l_{leg.20.pl} \cdot d_{leg.20.pl} \cdot h_{fill.20.pl} \ldots + A_{plate.20} \cdot h_{fill.plate.20} = 450.335 \cdot m^3$$

Total volume of the fill

$\beta_{20} := \frac{360deg}{20} = 18 \cdot deg$ — Angle between the legs

$l_{foundation.20.plate} := l_{leg.20.pl} \cdot 2 + d_{centrepiece} = 18\, m$ — Total length of the foundation

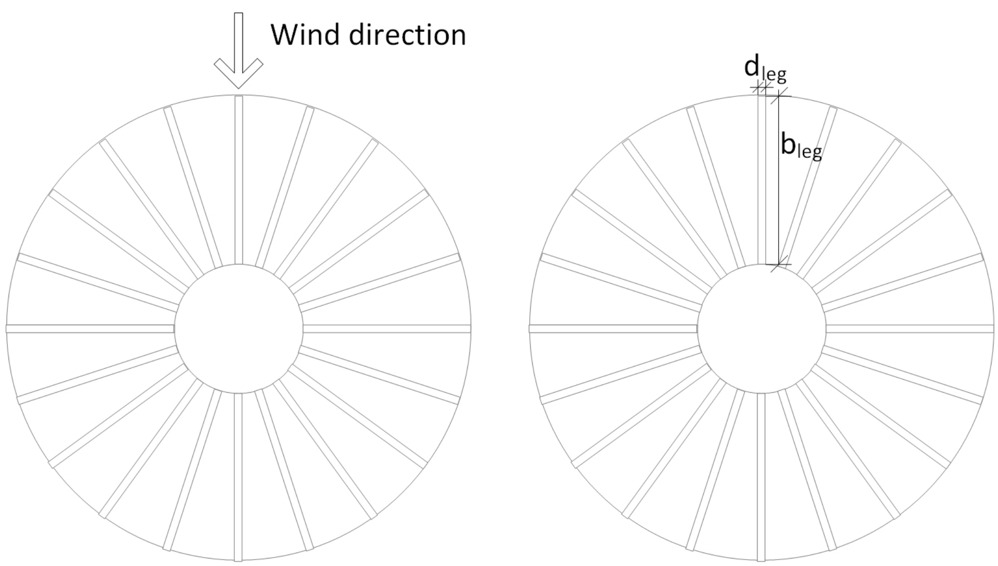


Figure 23: Twenty legs with bottom plate. Calculations are performed for one wind direction, due to symmetry.

For this concept it is only calculated for one wind direction, due to soil pressure on the bottom plate are equal for all wind directions.

Self-weight of the foundation and the soil

$$G_{k.20.pl} := \left[\left(20V_{leg.20.pl} + V_{centrepiece} + V_{plate.20}\right)\cdot\rho_c + V_{fill.20.pl}\cdot\rho_{fill}\right] = 11.712\cdot MN$$ Characteristic self-weight

$$G_{d.20.pl} := G_{k.20.pl}\cdot\gamma_{fav} = 10.541\cdot MN$$ Design load for the self-weight, self-weight is a favourable load when calculate the global stabilty

Effective flange width

In order to determine how much of the plate that can contribute and take the soil pressure, the effective width for the legs are calculated.

$$a := 2\cdot\left(l_{leg.20.pl} + \frac{d_{centrepiece}}{2}\right)\cdot\sin\left(\frac{\beta_{20}}{2}\right) = 2.816\,m$$ The maximum span between each leg, calculated with the angle β and trigonometry

$$b_i := \frac{a}{2}$$ Half of the largest span between the legs, at the outermost part of the foundation

$$l_0 := \frac{l_{foundation.20.plate}}{2} = 9\,m$$ The distance between the moment zero points, it is assumed to be half the length of the foundation.

$$b_{eff} := \min\left(0.2\cdot b_i + 0.1\cdot l_0, b_i\right) = 1.182\,m$$ Effective width

$$Check_{eff.width} := \begin{cases} \text{"The whole plate takes the soil pressure"} & \text{if } b_{eff} = \frac{a}{2} \\ \text{"The effective width takes the soil pressure"} & \text{if } b_{eff} < \frac{a}{2} \end{cases}$$

$$Check_{eff.width} = \text{"The effective width takes the soil pressure"}$$

This means that the whole plate is contributing to taking the soil pressure, therefore the soil pressure can be calculated as for a solid circular foundation.

Global equilibrium

From moment equilibrium around the resultant of the soil pressure, the eccentricity of the resultant soil pressure can be calculated. This is conceptually shown in Figure 3

$$e_{20.pl} := \frac{M_d + H_d\cdot h_{centrepiece}}{G_{d.20.pl} + N_d} = 7.741\,m$$ Eccentricity for the global stability

Check that the eccentricity fits within half of the foundation:

$$Check_{eccentricity} := \begin{cases} \text{"The eccentricity is ok!"} & \text{if } e_{20.pl} \le \frac{l_{foundation.20.plate}}{2} \\ \text{"Eccentricity is not ok!"} & \text{otherwise} \end{cases}$$

$$Check_{eccentricity} = \text{"The eccentricity is ok!"}$$

Calculation of the angle of the compression zone

The soil pressure calculations follow the same rocedure as for the solid circular concept.

$\alpha := 100\text{deg}$ — Assumed angle, defined as shown in the figure

$x := \cos(0.5\alpha)\cdot\frac{l_{foundation.20.plate}}{2} = 5.785\,\text{m}$ — Geometrical length from the center of the foundation to where the radial part of the circle sector start.

$tp_1 := \frac{2}{3}\cdot\frac{l_{foundation.20.plate}}{2}\cdot\frac{\sin(0.5\alpha)}{\frac{\alpha}{2}} = 5.267\,\text{m}$ — Centre of gravity for the whole sector A_1

$tp_2 := \frac{2}{3}\cdot x = 3.857\,\text{m}$ — Centre of gravity of the triangle A_2

$A_1 := \frac{\alpha}{2}\cdot\left(\frac{l_{foundation.20.plate}}{2}\right)^2 = 70.686\,\text{m}^2$ — Area of the whole sector A_1

$A_2 := 2\left(x\cdot x\cdot\frac{\tan(0.5\alpha)}{2}\right) = 39.885\,\text{m}^2$ — Area of the triangle A_2

$tp := \frac{A_1\cdot tp_1 - A_2\cdot tp_2}{A_1 + A_2} + x = 7.761\,\text{m}$ — Centre of gravity of the segment

Check if the assumed angle is correct. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check2 := \begin{array}{|l} \text{"Angle ok"} \quad \text{if} \left|1 - \frac{tp}{e_{20.pl}}\right| < 1\% \\ \text{"To large difference"} \quad \text{otherwise} \end{array}$$

$Check2 = \text{"Angle ok"}$

Soil pressure

$A_{soil.20.pl} := A_1 - A_2 = 30.801\,\text{m}^2$ — Area of the soil pressure zone

$\sigma_{soil.20.pl} := \frac{G_{d.20.pl} + N_d}{A_{soil.20.pl}} = 429.88\cdot\text{kPa}$ — Soil pressure

Check if the resistance of the soil is sufficient

$\frac{\sigma_{soil.20.pl}}{\sigma_{Rv}} = 0.43$ — Utilisation

$$Check3 := \begin{array}{|l} \text{"OK! Soil resistance is sufficent"} \quad \text{if } \frac{\sigma_{soil.20.pl}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} \quad \text{otherwise} \end{array}$$

$Check3 = \text{"OK! Soil resistance is sufficent"}$

Summary of the shape twenty-legged with bottom plate

$V_{20.plate} := 20 \cdot V_{leg.20.pl} + V_{centrepiece} + V_{plate.20} = 185.838 \cdot m^3$ Total volume of the concrete

$m_{20.plate} := V_{20.plate} \cdot \dfrac{\rho_c}{g} = 473.754 \cdot ton$ Total weight of the concrete

$m_{leg.20.plate} := V_{leg.20.pl} \cdot \dfrac{\rho_c}{g} = 9.942 \cdot ton$ Weight of one leg

$m_{element.20.plate} := \left(V_{leg.20.pl} + \dfrac{V_{plate.20}}{20}\right) \cdot \dfrac{\rho_c}{g} = 17.431 \cdot ton$ Weight of one element (leg and plate)

13. Summary of the shapes

	Total volume	Total weight	Weight per leg
3 legged	$V_{tot.3} = 385.087 \cdot m^3$	$m_{tot.3} = 981.7 \cdot ton$	$m_{leg.3} = 285.521 \cdot ton$
4 legged st	$V_{tot.4.stocky} = 409.087 \cdot m^3$	$m_{tot.4.stocky} = 1.043 \times 10^3 \cdot ton$	$m_{leg.4.stocky} = 229.436 \cdot ton$
4 legged sl	$V_{tot.4.slim} = 281.087 \cdot m^3$	$m_{tot.4.slim} = 716.573 \cdot ton$	$m_{leg.4.slim} = 147.859 \cdot ton$
Square	$V_{tot.square} = 490.317 \cdot m^3$	$m_{square} = 1.25 \times 10^3 \cdot ton$	
Circular	$V_{tot.circ} = 437.467 \cdot m^3$	$m_{circ} = 1.115 \times 10^3 \cdot ton$	
8 legged st	$V_{8.stocky} = 353.087 \cdot m^3$	$m_{8.stocky} = 900.122 \cdot ton$	$m_{leg.8.stocky} = 96.873 \cdot ton$
8 legged sl	$V_{8.slim} = 265.087 \cdot m^3$	$m_{8.slim} = 675.785 \cdot ton$	$m_{leg.8.slim} = 68.831 \cdot ton$
8 legged + fl	$V_{8.flange} = 225.087 \cdot m^3$	$m_{8.flange} = 573.813 \cdot ton$	$m_{leg.8.flange} = 56.084 \cdot ton$
16 legged	$V_{16} = 369.087 \cdot m^3$	$m_{16} = 940.911 \cdot ton$	$m_{leg.16} = 50.986 \cdot ton$
16 legged + pl	$V_{16.plate} = 172.578 \cdot m^3$	$m_{16.plate} = 439.95 \cdot ton$	$m_{leg.16.plate} = 9.942 \cdot ton$
20 legged + pl	$V_{20.plate} = 185.838 \cdot m^3$	$m_{20.plate} = 473.754 \cdot ton$	$m_{leg.20.plate} = 9.942 \cdot ton$

	Length of foundation	Width of leg	Height of leg	Length of leg
3 legged	$l_{foundation.3} = 26\,m$	$d_{leg.3} = 4\,m$	$h_{leg.3} = 2\,m$	$l_{leg.3} = 14\,m$
4 legged st	$l_{foundation.4.stocky} = 23\,m$	$d_{leg.4.st} = 5\,m$	$h_{leg.4.st} = 2\,m$	$l_{leg.4.st} = 9\,m$
4 legged sl	$l_{foundation.4.slim} = 34\,m$	$d_{leg.4.sl} = 2\,m$	$h_{leg.4.sl} = 2\,m$	$l_{leg.4.sl} = 14.5\,m$
Square	$l_{foundation.square} = 15.5\,m$		$h_{sq} = 2\,m$	
Circular	$l_{foundation.circ} = 16.5\,m$		$h_{ci} = 2\,m$	
8 legged st	$l_{foundation.8.stocky} = 24\,m$	$d_{leg.8.st} = 2\,m$	$h_{leg.8.st} = 2\,m$	$l_{leg.8.st} = 9.5\,m$
8 legged sl	$l_{foundation.8.slim} = 32\,m$	$d_{leg.8.sl} = 1\,m$	$h_{leg.8.sl} = 2\,m$	$l_{leg.8.sl} = 13.5\,m$
8 legged + fl	$l_{foundation.8.flange} = 25\,m$	$d_{leg.8.fl} = 0.8\,m$	$h_{leg.8.fl} = 2\,m$	$l_{leg.8.fl} = 10\,m$
16 legged	$l_{foundation.16} = 25\,m$	$d_{leg.16} = 1\,m$	$h_{leg.16} = 2\,m$	$l_{leg.16} = 10\,m$
16 legged + pl	$l_{foundation.16.plate} = 18\,m$	$d_{leg.16.pl} = 0.3\,m$	$h_{leg.16.pl} = 2\,m$	$l_{leg.16.pl} = 6.5\,m$
20 legged + pl	$l_{foundation.20.plate} = 18\,m$	$d_{leg.20.pl} = 0.3\,m$	$h_{leg.20.pl} = 2\,m$	$l_{leg.20.pl} = 6.5\,m$

13. Division into elements

In order to enable transportation of the elements, the maximum weight of each element is set 20 ton. The number of elements each leg must be divided into to fulfill this limit is calculated.

Three legs

$n_{el.3} := \mathrm{round}\left(\frac{m_{leg.3}}{20ton}\right) = 14$ Number of elements one leg must be divided into

$m_{el.3} := \frac{m_{leg.3}}{n_{el.3}} = 20.394 \cdot ton$ Weight of each element

Transversal division

$l_{el.3} := \frac{l_{leg.3}}{n_{el.3}} = 1\,m$ Length of the elements if transversally divided

$d_{leg.3} = 4\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.3} = 14\,m$ Length of the elements if transversally divided

$d_{el.3} := \frac{d_{leg.3}}{n_{el.3}} = 0.286\,m$ Width of the element if transversally divided

Four legs, stocky

$n_{el.4.stocky} := \mathrm{round}\left(\frac{m_{leg.4.stocky}}{20ton}\right) = 11$ Number of elements one leg must be divided into

$m_{el.4.stocky} := \frac{m_{leg.4.stocky}}{n_{el.4.stocky}} = 20.858 \cdot ton$ Weight of each element

Transversal division

$l_{el.4.stocky} := \frac{l_{leg.4.st}}{n_{el.4.stocky}} = 0.818\,m$ Length of the elements if transversally divided

$d_{leg.4.st} = 5\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.4.st} = 9\,m$ Length of the elements if transversally divided

$d_{el.4.st} := \frac{d_{leg.4.st}}{n_{el.4.stocky}} = 0.455\,m$ Width of the element if transversally divided

Four legs slim

$n_{el.4.slim} := \mathrm{round}\left(\frac{m_{leg.4.slim}}{20ton}\right) = 7$ Number of elements one leg must be divided into

$m_{el.4.slim} := \frac{m_{leg.4.slim}}{n_{el.4.slim}} = 21.123 \cdot ton$ Weight of each element

Transversal division

$l_{el.4.slim} := \frac{l_{leg.4.sl}}{n_{el.4.slim}} = 2.071\,m$ Length of the elements if transversally divided

$d_{leg.4.sl} = 2\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.4.sl} = 14.5\,m$ Length of the elements if transversally divided

$d_{el.4.slim} := \frac{d_{leg.4.sl}}{n_{el.4.slim}} = 0.286\,m$ Width of the element if transversally divided

Eight legs, stocky

$n_{el.8.stocky} := \mathrm{round}\left(\frac{m_{leg.8.stocky}}{20ton}\right) = 5$ Number of elements one leg must be divided into

$m_{el.8.stocky} := \frac{m_{leg.8.stocky}}{n_{el.8.stocky}} = 19.375 \cdot ton$ Weight of each element

Transversal division

$l_{el.8.stocky} := \frac{l_{leg.8.st}}{n_{el.8.stocky}} = 1.9\,m$ Length of the elements if transversally divided

$d_{leg.8.st} = 2\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.8.st} = 9.5\,m$ Length of the elements if transversally divided

$d_{el.8.stocky} := \frac{d_{leg.8.st}}{n_{el.8.stocky}} = 0.4\,m$ Width of the element if transversally divided

Eight legs, slim

$n_{el.8.slim} := \mathrm{round}\left(\frac{m_{leg.8.slim}}{20ton}\right) = 3$ Number of elements one leg must be divided into

$m_{el.8.slim} := \frac{m_{leg.8.slim}}{n_{el.8.slim}} = 22.944 \cdot ton$ Weight of each element

Transversal division

$l_{el.8.slim} := \frac{l_{leg.8.sl}}{n_{el.8.slim}} = 4.5\,m$ Length of the elements if transversally divided

$d_{leg.8.sl} = 1\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.8.sl} = 13.5\,m$ Length of the elements if transversally divided

$d_{el.8.slim} := \frac{d_{leg.8.sl}}{n_{el.8.slim}} = 0.333\,m$ Width of the element if transversally divided

Sixteen legs

$n_{el.16} := \mathrm{round}\left(\frac{m_{leg.16}}{20ton}\right) = 3$ Number of elements one leg must be divided into

$m_{el.16} := \frac{m_{leg.16}}{n_{el.16}} = 16.995 \cdot ton$ Weight of each element

Transversal division

$l_{el.16} := \frac{l_{leg.16}}{n_{el.16}} = 3.333\,m$ Length of the elements if transversally divided

$d_{leg.16} = 1\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.16} = 10\,m$ Length of the elements if transversally divided

$d_{el.16} := \frac{d_{leg.16}}{n_{el.16}} = 0.333\,m$ Width of the element if transversally divided

Eight legs, flange

$n_{el.8.flange} := \mathrm{round}\left(\frac{m_{leg.8.flange}}{20ton}\right) = 3$ Number of elements one leg must be divided into

$m_{el.8.flange} := \frac{m_{leg.8.flange}}{n_{el.8.flange}} = 18.695 \cdot ton$ Weight of each element

Transversal division

$l_{el.8.flange} := \frac{l_{leg.8.fl}}{n_{el.8.flange}} = 3.333\,m$ Length of the elements if transversally divided

$d_{leg.8.flange} := d_{leg.8.fl} = 0.8\,m$ Width of the element if transversally divided

Longitudinal division

$l_{leg.8.flange} := l_{leg.8.fl} = 10\,m$ Length of the elements if transversally divided

$d_{el.8.flange} := \frac{d_{leg.8.flange}}{n_{el.8.flange}} = 0.267\,m$ Width of the element if transversally divided

Square foundation

$n_{el.square} := \frac{m_{square}}{20ton} = 62.498$ Number of elements one leg must be divided into

$n_{el.square} := 64$ Choose 64 elements, so the elements are symmetric

$m_{el.square} := \frac{m_{square}}{n_{el.square}} = 19.531 \cdot ton$ Weight of each element

$l_{el.square} := \frac{l_{foundation.square}}{\sqrt{n_{el.square}}} = 1.938\,m$ Length of the elements

Circular foundation

$$n_{el.circ} := \mathrm{round}\left(\frac{m_{circ}}{20ton}\right) = 56$$ Number of elements one leg must be divided into

$$m_{el.circ} := \frac{m_{circ}}{n_{el.circ}} = 19.915 \cdot ton$$ Weight of each element

$$l_{el.circ} := \frac{\pi \cdot l_{foundation.circ}}{n_{el.circ}} = 0.926\,m$$ Arc length of the outer part of the element

$$l_{el.circ.cp} := \frac{\pi \cdot d_{centrepiece}}{n_{el.circ}} = 0.28\,m$$ Arc length of the inner part of the element

20 legs with bottom plate

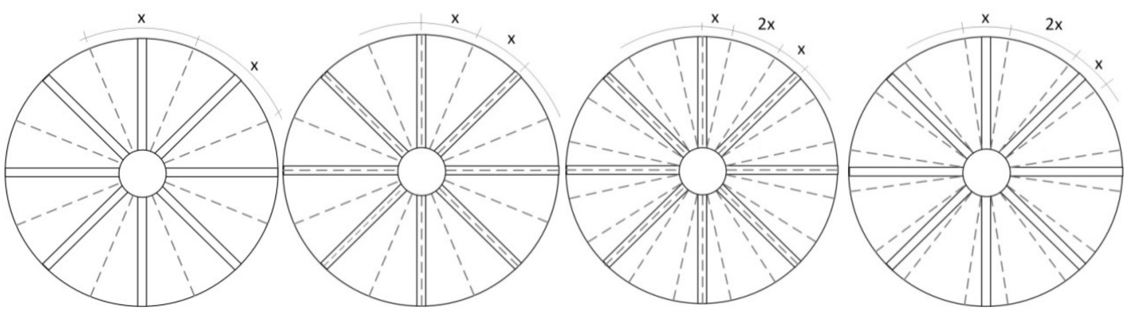


Figure 24: Division of the shapes with bottom plate. Method A, B, C and D in respective order.

Division of the elements due to the 4 different methods; A, B, C & D with and without the corresponding part of the centrepiece

Without centerpiece

$$m_{el.20.A} := \left(V_{leg.20.pl} + \frac{V_{plate.20}}{20}\right) \cdot \frac{\rho_c}{g} = 17.431 \cdot ton$$ Weight of the element with division method A

$$m_{el.20.B} := \left(\frac{V_{leg.20.pl}}{2} + \frac{V_{plate.20}}{40}\right) \cdot \frac{\rho_c}{g} = 8.715 \cdot ton$$ Weight of the element with division method B

$$m_{el.20.C.1} := \left(\frac{V_{leg.20.pl}}{2} + \frac{V_{plate.20}}{80}\right) \cdot \frac{\rho_c}{g} = 6.843 \cdot ton$$ Weigth of element C1 (with the leg) with division method C

$$m_{el.20.C.2} := \left(\frac{V_{plate.20}}{40}\right) \cdot \frac{\rho_c}{g} = 3.744 \cdot ton$$ Weight of element C2 (only plate) with division method C

$$m_{el.20.D.1} := \left(V_{leg.20.pl} + \frac{V_{plate.20}}{60}\right) \cdot \frac{\rho_c}{g} = 12.438 \cdot ton$$ Weigth of element D1 (with the leg) with division method D

$$m_{el.20.D.2} := \left(\frac{V_{plate.20}}{30}\right) \cdot \frac{\rho_c}{g} = 4.992 \cdot ton$$ Weight of element D2 (only plate) with division method D

With centerpiece

$m_{el.20.A.cp} := \left(V_{leg.20.pl} + \frac{V_{plate.20}}{20} + \frac{V_{centrepiece}}{20}\right)\cdot\frac{\rho_c}{g} = 23.688\cdot ton$ Weight of the element with division method A

$m_{el.20.B.cp} := \left(\frac{V_{leg.20.pl}}{2} + \frac{V_{plate.20}}{40} + \frac{V_{centrepiece}}{40}\right)\cdot\frac{\rho_c}{g} = 11.844\cdot ton$ Weight of the element with division method B

$m_{el.20.C1.cp} := \left(\frac{V_{leg.20.pl}}{2} + \frac{V_{plate.20}}{80} + \frac{V_{centrepiece}}{80}\right)\cdot\frac{\rho_c}{g} = 8.407\cdot ton$ Weigth of element C1 (with the leg) with division method C

$m_{el.20.C2.cp} := \left(\frac{V_{plate.20}}{40} + \frac{V_{centrepiece}}{40}\right)\cdot\frac{\rho_c}{g} = 6.873\cdot ton$ Weight of element C2 (only plate) with division method C

$m_{el.20.D.1.cp} := \left(V_{leg.20.pl} + \frac{V_{plate.20}}{60} + \frac{V_{centrepiece}}{60}\right)\cdot\frac{\rho_c}{g} = 14.524\cdot ton$ Weigth of element D1 (with the leg) with division method D

$m_{el.20.D.2.cp} := \left(\frac{V_{plate.20}}{30} + \frac{V_{centrepiece}}{30}\right)\cdot\frac{\rho_c}{g} = 9.164\cdot ton$ Weight of element D2 (only plate) with division method D

Width of the elements

$b_{el.20.A} := \frac{l_{foundation.20.plate}\cdot\pi}{20} = 2.827\,m$ Width of the element with division method A

$b_{el.20.B} := \frac{l_{foundation.20.plate}\cdot\pi}{40} = 1.414\,m$ Width of the element with division method B

$b_{el.20.C1} := \frac{l_{foundation.20.plate}\cdot\pi}{20\cdot 4} = 0.707\,m$ Width of element C1 (with the leg) with division method C

$b_{el.20.C2} := b_{el.20.C1}\cdot 2 = 1.414\,m$ Width of element C2 (only plate) with division method C

$b_{el.20.D1} := \frac{l_{foundation.20.plate}\cdot\pi}{20\cdot 3} = 0.942\,m$ Width of element D1 (with the leg) with division method D

$b_{el.20.D2} := b_{el.20.D1}\cdot 2 = 1.885\,m$ Width of element D2 (only plate) with division method D

16 legs with bottom plate

Division of the elements due to the 4 different methods; A, B, C & D.
With and without the corresponding part of the centrepiece

Without centerpiece

$m_{el.16A} := \left(V_{leg.16.pl} + \frac{V_{plate.16}}{16}\right)\cdot\frac{\rho_c}{g} = 19.676\cdot ton$ Weight of the element with division method A

$m_{el.16.B} := \left(\frac{V_{leg.16.pl}}{2} + \frac{V_{plate.16}}{32}\right)\cdot\frac{\rho_c}{g} = 9.838\cdot ton$ Weight of the element with division method B

$m_{el.16.C.1} := \left(\frac{V_{leg.16.pl}}{2} + \frac{V_{plate.16}}{64}\right)\cdot\frac{\rho_c}{g} = 7.405\cdot ton$ Weigth of element C1 (with the leg) with division method C

$$m_{el.16.C.2} := \left(\frac{V_{plate.16}}{32}\right)\cdot\frac{\rho_c}{g} = 4.867\cdot ton$$ Weight of element C2 (only plate) with division method C

$$m_{el.16D.1} := \left(V_{leg.16.pl} + \frac{V_{plate.16}}{48}\right)\cdot\frac{\rho_c}{g} = 13.187\cdot ton$$ Weigth of element D1 (with the leg) with division method D

$$m_{el.16D.2} := \left(\frac{V_{plate.16}}{24}\right)\cdot\frac{\rho_c}{g} = 6.489\cdot ton$$ Weight of element D2 (only plate) with division method D

With centerpiece

$$m_{el.16.A.cp} := \left(V_{leg.16.pl} + \frac{V_{plate.16}}{16} + \frac{V_{centrepiece}}{16}\right)\cdot\frac{\rho_c}{g} = 27.497\cdot ton$$ Weight of the element with division method A

$$m_{el.16.B.cp} := \left(\frac{V_{leg.16.pl}}{2} + \frac{V_{plate.16}}{32} + \frac{V_{centrepiece}}{32}\right)\cdot\frac{\rho_c}{g} = 13.748\cdot ton$$ Weight of the element with division method B

$$m_{el.16.C.1.cp} := \left(\frac{V_{leg.16.pl}}{2} + \frac{V_{plate.16}}{64} + \frac{V_{centrepiece}}{64}\right)\cdot\frac{\rho_c}{g} = 9.36\cdot ton$$ Weigth of element C1 (with the leg) with division method C

$$m_{el.16.C.2.cp} := \left(\frac{V_{plate.16}}{32} + \frac{V_{centrepiece}}{32}\right)\cdot\frac{\rho_c}{g} = 8.777\cdot ton$$ Weight of element C2 (only plate) with division method C

$$m_{el.16.D.1.cp} := \left(V_{leg.16.pl} + \frac{V_{plate.16}}{48} + \frac{V_{centrepiece}}{48}\right)\cdot\frac{\rho_c}{g} = 15.794\cdot ton$$ Weigth of element D1 (with the leg) with division method D

$$m_{el.16D.2.cp} := \left(\frac{V_{plate.16}}{24} + \frac{V_{centrepiece}}{24}\right)\cdot\frac{\rho_c}{g} = 11.703\cdot ton$$ Weight of element D2 (only plate) with division method D

Width of the elements

$$b_{el.16.A} := \frac{l_{foundation.16.plate}\cdot\pi}{16} = 3.534\,m$$ Width of the element with division method A

$$b_{el.16.B} := \frac{l_{foundation.16.plate}\cdot\pi}{32} = 1.767\,m$$ Width of the element with division method B

$$b_{el.16.C1} := \frac{l_{foundation.16.plate}\cdot\pi}{16\cdot 4} = 0.884\,m$$ Width of element C1 (with the leg) with division method C

$$b_{el.16.C2} := b_{el.16.C1}\cdot 2 = 1.767\,m$$ Width of element C2 (only plate) with division method C

$$b_{el.16.D1} := \frac{l_{foundation.16.plate}\cdot\pi}{16\cdot 3} = 1.178\,m$$ Width of element D1 (with the leg) with division method D

$$b_{el.16.D2} := b_{el.16.D1}\cdot 2 = 2.356\,m$$ Width of element D2 (only plate) with division method D

Preliminary Foundation Loads SWT-2.3-101

Foundation loads provided within this document cover the wind climate conditions listed under “Project information”.

Project information

Site location		Not available
Customer		Not available
Wind Turbine Type		SWT-2.3-101
Hub height		99.5 [m], **onshore**
Annual average wind speed at hub height	V_{ave}	Not available [m/s]
10 min. extreme wind speed with return period of 50 years at hub height	V_{ref}	Not available [m/s]
3 sec. gust wind speed with return period of 50 years at hub height	V_{50e}	Not available [m/s]
Average air density		Not available [kg/m^3]

Design Code information

Foundation loads given within this document are based on:

Design code		EN 61400-1:2004
IEC Class		2B
Annual average wind speed at hub height	V_{ave}	8.5 [m/s]
10 min. extreme wind speed with return period of 50 years at hub height	V_{ref}	42.5 [m/s]
3 sec. gust wind speed with return period of 50 years at hub height	V_{50e}	59.5 [m/s]
Average air density		1.225 [kg/m^3]

Foundation Design loads, design load case with highest overturning moment
The ultimate loads from the design load case with the highest overturning moment at the base of the turbine from all load cases are tabulated below.

Hub height	**H [m]**	**99.5**
Normal force	N [kN]	3,600
Shear force	Q [kN]	1,080
Overturning moment	M [kNm]	97,700
Torsion moment	T [kNm]	3,800

Table 1: Design loads, design load case with highest overturning moment (DLC6.2).
Loads are inclusive of partial safety factors according to IEC 61400-1 Ed.3.

Foundation Design loads, normal operation
The maximum load experienced at the base of the wind turbine tower during normal operation is tabulated below.

Hub height	**H [m]**	**99.5**
Normal force	N [kN]	3,600
Shear force	Q [kN]	800
Overturning moment	M [kNm]	72,500
Torsion moment	T [kNm]	7,900

Table 2: Design loads, normal operation.
Loads are inclusive of partial safety factors according to IEC 61400-1 Ed.3.

As a minimum requirement, partial factors of safety in accordance with IEC 61400-1 Ed.3 shall be applied to unfactored foundation loads given in this document.

Seismic activity is not included in foundation loads given in this document.

Foundation stiffness

A calculation of the foundation stiffness must be evaluated with possible variations depending on the soil characteristics and stiffness of foundation.

Siemens requirements to the combined stiffness of foundation and soil are:

Minimum rotational stiffness around horizontal axis	1500 [MNm/deg]
Minimum stiffness for horizontal translation	500 [MN/m]

If the foundation stiffness is below the Siemens requirements, the foundation loads are not valid for the given foundation design.

A settlement calculation of the foundation must be evaluated and at the end of the life time of the turbine a differential settlement resulting in a maximum rotation of 0.25 deg of the foundation to the horizontal plan is acceptable.

Operational loads – Fatigue load spectrum

In the following table, the fatigue load spectra calculated for 20 years operating time in accordance with IEC 61400-1 for the turbine are specified for the horizontal force, the overturning moment and the torsion. The numbers of cycles are determined according to different load ranges.
The fatigue spectrum for the shear force, the overturning moment and the torsion are given for ten mean bins. This may be used when fatigue damage in the material depends on the mean load, which is common practice for concrete and reinforcement. In case that the fatigue is independent of the mean value the load bins in the last column “All” shall be used only.
The spectrum is only valid for the specified conditions stated under “Design Code information”.

NB: In the tables comma (,) is used as thousand separator and point (.) is used as decimal separator.

Horizontal force. Peak-to-peak fatigue load bins [kN]

Number of cycles	Mean value [kN] -270 to -170	-170 to -80	-80 to 20	20 to 120	120 to 220	220 to 320	320 to 420	420 to 520	520 to 620	620 to 720	All
1.00E+09	0	0	0	0	0	0	0	0	0	0	30
5.00E+08	0	0	0	0	0	0	0	0	0	0	50
2.00E+08	0	0	0	30	30	50	0	0	0	0	70
1.00E+08	0	0	0	30	50	70	30	0	0	0	90
5.00E+07	0	0	0	50	90	90	70	0	0	0	140
2.00E+07	0	0	0	50	110	140	90	0	0	0	160
1.00E+07	0	0	0	50	160	160	110	0	0	0	200
5.00E+06	0	0	0	70	200	200	140	0	0	0	250
2.00E+06	0	0	0	70	250	250	160	0	0	0	290
1.00E+06	0	0	30	90	270	290	180	0	0	0	310
5.00E+05	0	0	30	140	310	330	200	0	0	0	350
2.00E+05	0	0	30	160	330	350	220	0	0	0	380
1.00E+05	0	0	50	160	350	380	250	30	0	0	400
5.00E+04	0	0	70	180	400	400	290	50	0	0	420
2.00E+04	0	0	90	200	420	420	310	70	0	0	460
1.00E+04	0	0	140	200	460	460	330	70	0	0	510
5.00E+03	0	0	200	220	550	490	350	70	0	0	620
2.00E+03	0	30	250	270	660	620	350	110	0	0	700
1.00E+03	110	440	600	920	1,100	1,100	940	270	160	90	1,100

Table 3: Fatigue load spectrum for horizontal force.

Overturning moment. Peak-to-peak fatigue load bins [kNm]

Number of cycles	Mean value [kNm]: -26,200 to -16,400	-16,400 to -6,500	-6,500 to 3,300	3,300 to 13,200	13,200 to 23,000	23,000 to 32,900	32,900 to 42,700	42,700 to 52,600	52,600 to 62,400	62,400 to 72,200	All
1.00E+09	0	0	0	0	0	0	0	0	0	0	10
5.00E+08	0	0	0	0	0	0	0	0	0	0	10
2.00E+08	0	0	0	10	10	10	0	0	0	0	2,000
1.00E+08	0	0	0	10	2,000	2,000	10	0	0	0	4,000
5.00E+07	0	0	10	2,000	2,000	4,000	10	0	0	0	6,000
2.00E+07	0	0	10	2,000	6,000	6,000	2,000	0	0	0	9,900
1.00E+07	0	0	10	2,000	9,900	9,900	4,000	0	0	0	11,900
5.00E+06	0	0	2,000	4,000	13,900	13,900	6,000	0	0	0	15,900
2.00E+06	0	0	2,000	4,000	15,900	17,800	8,000	0	0	0	19,800
1.00E+06	0	0	2,000	6,000	19,800	21,800	9,900	0	0	0	23,800
5.00E+05	0	0	4,000	9,900	21,800	25,800	11,900	0	0	0	27,700
2.00E+05	0	0	6,000	13,900	25,800	27,700	11,900	0	0	0	29,700
1.00E+05	0	0	8,000	15,900	27,700	29,700	13,900	0	0	0	31,700
5.00E+04	0	0	9,900	17,800	31,700	31,700	13,900	10	0	0	35,600
2.00E+04	0	0	13,900	19,800	35,600	35,600	15,900	10	0	0	39,600
1.00E+04	0	0	13,900	19,800	39,600	35,600	17,800	10	0	0	41,600
5.00E+03	0	0	19,800	21,800	47,500	39,600	25,800	10	0	0	49,500
2.00E+03	0	10	23,800	31,700	57,400	39,600	25,800	2,000	0	0	59,400
1.00E+03	2,000	41,600	59,400	87,100	89,000	98,900	79,200	8,000	2,000	2,000	98,900

Table 4: Fatigue load spectrum for overturning moment.

Torsion. Peak-to-peak fatigue load bins [kNm]

Number of cycles	Mean value [kNm]: -4,200 to -3,300	-3,300 to -2,400	-2,400 to -1,500	-1,500 to -600	-600 to 300	300 to 1,300	1,300 to 2,200	2,200 to 3,100	3,100 to 4,000	4,000 to 4,900	All
1.00E+09	0	0	0	0	0	0	0	0	0	0	0
5.00E+08	0	0	0	0	180	0	0	0	0	0	360
2.00E+08	0	0	0	0	540	0	0	0	0	0	720
1.00E+08	0	0	0	0	900	0	0	0	0	0	1,300
5.00E+07	0	0	0	540	1,500	540	0	0	0	0	1,700
2.00E+07	0	0	0	1,100	1,800	1,100	0	0	0	0	2,000
1.00E+07	0	0	0	1,500	2,200	1,500	0	0	0	0	2,400
5.00E+06	0	0	0	1,700	2,700	2,000	180	0	0	0	2,900
2.00E+06	0	0	0	2,000	3,300	2,400	900	0	0	0	3,400
1.00E+06	0	0	360	2,400	3,800	2,700	1,500	0	0	0	4,000
5.00E+05	0	0	900	2,700	4,500	3,300	1,800	0	0	0	4,500
2.00E+05	0	0	1,500	2,900	4,900	3,800	2,200	0	0	0	5,100
1.00E+05	0	0	1,700	3,300	5,200	4,700	2,600	720	0	0	5,600
5.00E+04	0	0	2,000	3,600	5,800	5,400	3,100	1,500	0	0	6,000
2.00E+04	0	0	2,600	3,800	6,000	5,800	3,300	1,800	0	0	6,300
1.00E+04	0	0	2,700	4,200	6,300	6,300	3,600	2,200	0	0	6,700
5.00E+03	0	180	2,900	5,100	6,800	6,700	4,000	2,400	180	0	7,200
2.00E+03	0	360	3,600	5,600	7,200	7,600	5,100	3,100	900	0	7,600
1.00E+03	0	2,400	4,200	7,400	9,000	8,600	7,900	4,500	3,600	1,300	9,000

Table 5: Fatigue load spectrum for torsion.

Geometry of the standard interface between tower and foundation

For details of foundation bolts, mating surface stiffness and flatness and grouting please ensure that all aspects are thoroughly discussed and agreed with Siemens prior to construction. Tolerances in relation to preparation of foundations are according to AI-WI549479.

In cases where the customer/civil contractor deliver the foundation bolts, the customer/civil contractor are also obliged to deliver interface calculations showing that the safety are acceptable both in extreme situations as well as during power production (fatigue calculations).

The tower bottom flange dimensions are in table and figure shown below.

Hub height	**H**	**99.5**
Bottom flange outer diameter	do [mm]	4200
Bottom flange inner diameter	di [mm]	3560
Bottom flange bolt circle, outer	Dbco [mm]	4060
Bottom flange bolt circle, inner	Dbci [mm]	3700
Diameter of bolt holes	dh [mm]	48
Bottom flange thickness	t [mm]	80 [+/- 2]
Number of bolts, outer bolt circle	Nb_o	100
Number of bolts, inner bolt circle	Nb_i	100
Size of bolts	-	M42 1-3/8’’ (US)
Grade	-	8.8 150 KSI ASTM A722 (US)
Bolt pretension	[kN]	400
Bolt length	[mm]	Min. 1475
Tower name	-	2.3-T99.5-05 2.3-T99.5-09

Table 6: Bottom flange geometry.

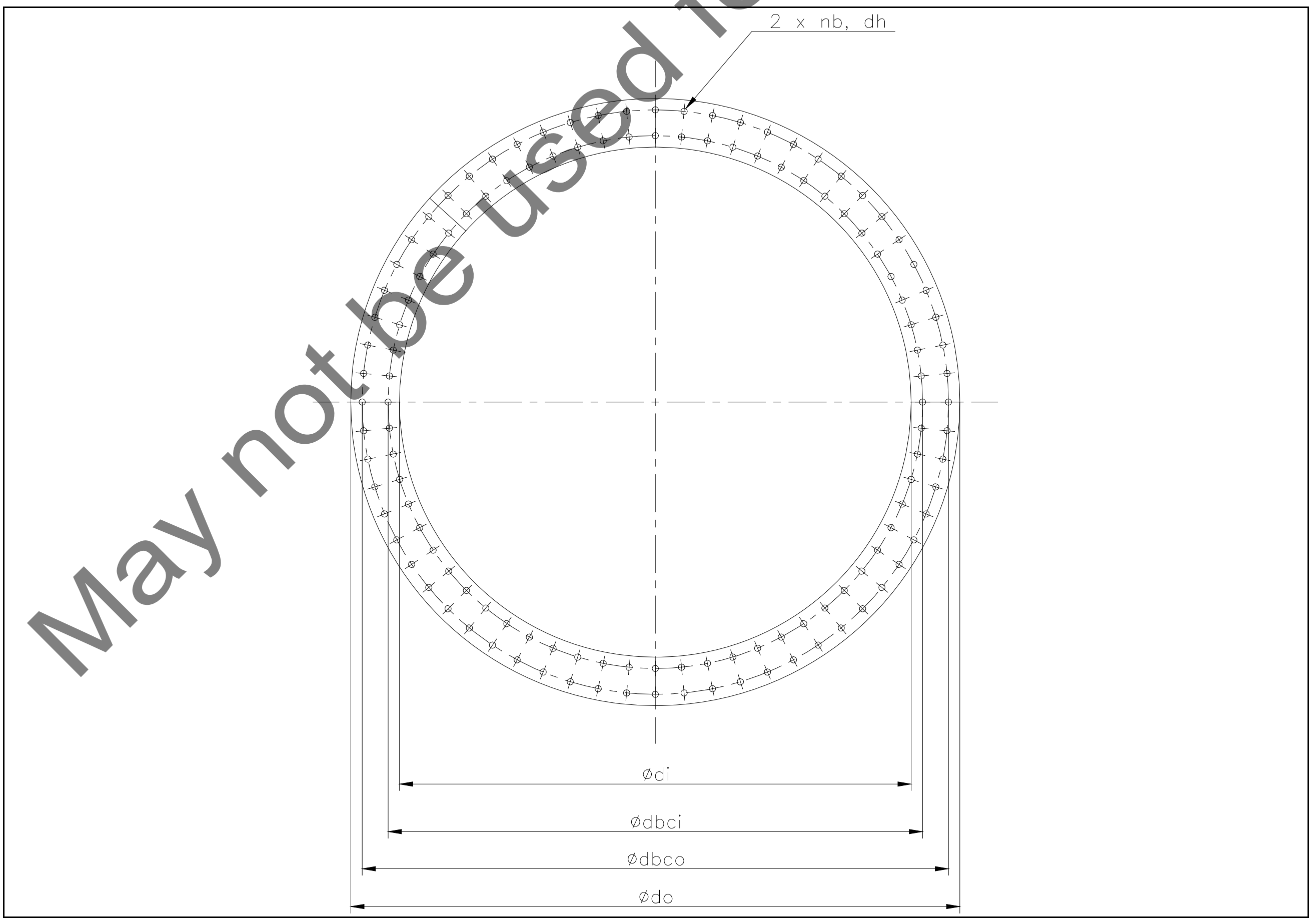


Table 7: Bottom flange

Comments to Ultimate Foundation Loads
Loads stated on page 1 are maximum design loads under normal power production and extreme conditions to be used for foundation design.

Loads in tables below are for special design analysis required by foundation designers.

The given ultimate load cases are the most onerous unfactored load cases and design load cases with safety factors applied according to IEC 61400-1 Ed.3 and Dibt, Fassung März 2004.

Most onerous ultimate loads derived from loads with a probability of exceedance of min. 10^{-2} equivalent to 1750h in 20 years (Dibt, Druckfehlerbereinigung Dezember 2006)

Design load case: 1.0		Partial safety factor	Unfactored Load	Design Load
Hub height	H [m]	N/A	99.5	99.5
Normal force	N [kN]	1.00	3,600	3,600
Shear force	Q [kN]	1.00	460	460
Overturning moment	M [kNm]	1.00	42,900	42,900
Torsion moment	T [kNm]	1.00	1,870	1,870

Table 8: Ultimate loads from DLC1.0.

Ultimate normal operation loads incl. partial safety factor 1.35 (IEC 61400-1 Ed.3)

Design load case: 1.2		Partial safety factor	Unfactored Load	Design Load
Hub height	H [m]	N/A	99.5	99.5
Normal force	N [kN]	1.00	3,600	3,600
Shear force	Q [kN]	1.35	590	800
Overturning moment	M [kNm]	1.35	53,700	72,500
Torsion moment	T [kNm]	1.35	5,900	7,900

Table 9: Ultimate loads from DLC1.2.

Ultimate operation gust loads with 1 year return period incl. partial safety factor 1.10 (IEC 61400-1 Ed.3)

Design load case: 2.3		Partial safety factor	Unfactored Load	Design Load
Hub height	H [m]	N/A	99.5	99.5
Normal force	N [kN]	1.00	3,600	3,600
Shear force	Q [kN]	1.10	810	890
Overturning moment	M [kNm]	1.10	78,800	86,600
Torsion moment	T [kNm]	1.10	3,300	3,600

Table 10: Ultimate loads from DLC2.3.

Ultimate operation gust loads with 50 years return period incl. partial safety factor 1.35 (IEC 61400-1 Ed.3)

Design load case: 4.2		Partial safety factor	Unfactored Load	Design Load
Hub height	H [m]	N/A	99.5	99.5
Normal force	N [kN]	1.00	3,600	3,600
Shear force	Q [kN]	1.35	690	940
Overturning moment	M [kNm]	1.35	66,400	89,600
Torsion moment	T [kNm]	1.35	2,500	3,300

Table 11: Ultimate loads from DLC4.2.

Ultimate parked/idling gust loads with 50 years return period incl. partial safety factor 1.35 (IEC 61400-1 Ed.3)

Design load case: 6.1		Partial safety factor	Unfactored Load	Design Load
Hub height	H [m]	N/A	99.5	99.5
Normal force	N [kN]	1.00	3,600	3,600
Shear force	Q [kN]	1.35	940	1,260
Overturning moment	M [kNm]	1.35	70,200	94,800
Torsion moment	T [kNm]	1.35	3,400	4,600

Table 12: Ultimate loads from DLC6.1.

Ultimate parked/idling gust loads with 50 years return period incl. partial safety factor 1.10 (IEC 61400-1 Ed.3)

Design load case: 6.2		Partial safety factor	Unfactored Load	Design Load
Hub height	H [m]	N/A	99.5	99.5
Normal force	N [kN]	1.00	3,600	3,600
Shear force	Q [kN]	1.10	980	1,080
Overturning moment	M [kNm]	1.10	88,800	97,700
Torsion moment	T [kNm]	1.10	3,500	3,800

Table 13: Ultimate loads from DLC6.2.

Note: As a minimum requirement, partial factors of safety in accordance with IEC 61400-1 Ed.3 shall be applied to unfactored foundation loads given in this document. Seismic activity is not included in foundation loads given in this document.

SWP Reference:

Siemens Siting report.. Not available
Siemens Extreme Load reference document .... Uls_SWT-23-101-H99.5_hcbd_IEC2B_20091012-01.xls
Siemens Fatigue Load reference document...................................... MarkovFound_hcbd_IEC2B_rev00_091015.xls
Siemens Seismic Load reference document .... N/A

Signed Electronically

Loads prepared by:	Stephan Schönrock	Date:	20091015
Loads released by:		Date:	
Site data prepared by:		Date:	
Site data approved by:		Date:	

Appendix III
Detailed calculations
Twenty webs with bottom plate

Table of contents

1. Indata

Loads from the tower

Design loads assumed for tower SWT-2.3-101 from Siemens with an height of 99.5 m high, loads are including the partial safety factors (except for the self-weight of the tower). The loads are presented in Appendix II.

$M_d := 97700\text{kN}\cdot\text{m}$ Design load on top of the foundation; overturning momen

$H_d := 1080\text{kN}$ Design load on top of the foundation; transverse load

$N_k := 3600\text{kN}$ Characteristic load on top of the foundation; dead load

Partial safety factors, wind power plants

$\gamma_{norm} := 1.35$ For normal load cases, this is included in the design loads

$\gamma_{abn} := 1.1$ For abnormal load cases

$\gamma_{dead} := 0.9$ For dead weight

$\gamma_{fat} := 1.0$ For fatigue loading

Geometry of the connection between tower and foundation

$d_{centrepiece} := 5\text{m}$ Diameter of the centrepiece

$$A_{centrepiece} := \frac{\pi \cdot d_{centrepiece}^{2}}{4} = 19.635\,\text{m}^2$$ Area of the centrepiece

$h_{centrepiece} := 2.9\text{m}$ Height of the centrepiece, defined from the top to the bottom of the foundation

$V_{centrepiece} := h_{centrepiece} \cdot A_{centrepiece} = 56.941\cdot\text{m}^3$ Volume of the centrepiece

$d_r := 4\text{m}$ Diameter of the anchor ring, see figure below

$$r_r := \frac{d_r}{2}$$ Radius of the anchor ring

$$\Delta l := \frac{d_{centrepiece} - d_r}{2} = 0.5\,\text{m}$$ Distance between the centre of the anchor ring to the outer part of the centrepiece

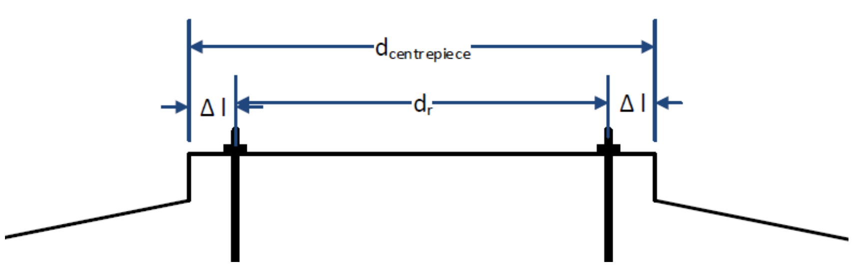


Figure 1: Definition of the dimensions of the centrepiece

The applied overturning moment can be described by a force couple F_c and F_t. The force couple acts along the bolt basket, with a stress distribution according to figure:

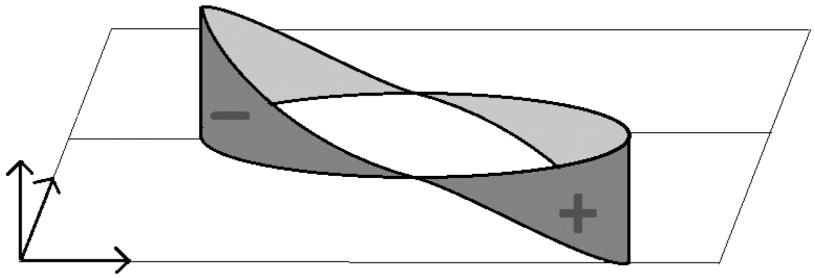


Figure 2: Stress distribution along the prestressing bolt basket

The stress distribution is simplified to be uniform along two quarters of the bolt basket, see figure, and it is assumed that the force couple can be considered to act in the centre of gravity of the arc of these two circle quarters (according to Landén & Lilljegren 2012, eq. 5.3)

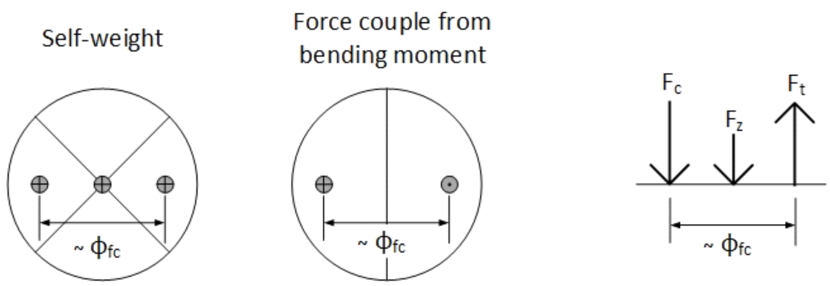


Figure 3: Illustration of how the resultants of the tensile and compressive parts of the force couple is offset towards the centre of the foundation, and not acting in the position of the anchorage ring.

This means that the attack point of the force couple will be calculated according to:

$$\phi_{fc} := 2 \cdot \left(\frac{2}{\pi \cdot r_r}\right) \cdot \int_{\frac{-\pi}{4}}^{\frac{\pi}{4}} \left(r_r\right)^2 \cdot \cos(\varphi)\, d\varphi$$

Distance between the force couple resultants F_c and F_t calculated as the centre of gravity of the arc of the two quarters.

$$r_{fc} := \text{round}\left(\frac{\phi_{fc}}{2 \cdot m}, 2\right) \cdot m = 1.8\,m$$

Distance from the center of the foundation to the force resultants F_c and $F_{t,}$ from the applied overturning moment.

Material parameters

Concrete C30/37

$$\rho_c := 25 \frac{kN}{m^3}$$

Weight of the concrete

$$\gamma_c := 1.5$$

Concrete partial safety factor

$$f_{ck} := 30MPa$$

Characteristic strength class of concrete

$$f_{cd} := \frac{f_{ck}}{\gamma_c} = 20 \cdot MPa$$

Design compressive strength of the concrete

$$f_{ctm} := 2.9MPa$$

Mean tensile strength of concrete

$$f_{ctd} := \frac{f_{ctm}}{\gamma_c} = 1.933 \cdot MPa$$

Design tensile strength of concrete

$$E_c := 33GPa$$

Elastic modulus of the concrete

Reinforcement B500B

$f_{yk} := 500MPa$ Characteristic yield strength of the steel

$\gamma_s := 1.15$ Reinforcement steel partial safety factor

$f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783 \cdot MPa$ Design yield strength of the reinforcing steel

$E_s := 200GPa$ Elastic modulus of steel

Fill

$\rho_{fill} := 1600 \frac{kg}{m^3} \cdot g = 15.691 \cdot \frac{kN}{m^3}$ Density of fill

Soil

$\sigma_{Rv} := 1000kPa$ Assumed soil resistance

Correction of units

$ton := 1000kg$

Geometry of the foundation

Plate and web

$b_{web} := 0.35m$ Width of the web

$l_{web} := 6.5m$ Length of the web

$h_{plate} := 0.4m$ Height of the bottom plate

$h_{web.0} := 1.5m$ Height of the web at the outer edge

$h_{web.r.cp} := 2m$ Height of the web next to the centrepiece

$h_{web} := \frac{h_{web.0} + h_{web.r.cp}}{2} = 1.75\,m$ Average height of the web

$h_{element.0} := h_{web.0} + h_{plate} = 1.9\,m$ Height of the element at the outer edge

$h_{element.r.cp} := h_{web.r.cp} + h_{plate} = 2.4\,m$ Height of the foundation next to the centrepiece

$h_{tot} := h_{web} + h_{plate} = 2.15\,m$ Average height of the element

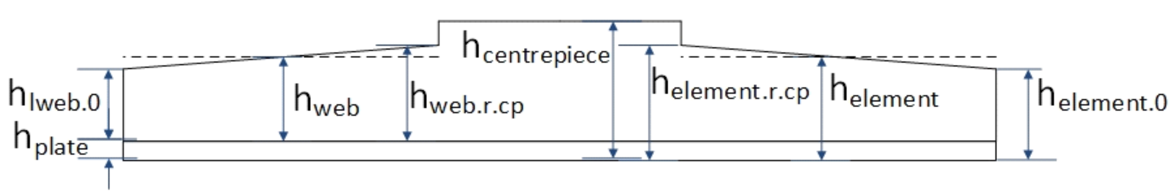


Figure 3: Heights of the foundation, along a cut through the middle of the foundation, are defined according to this figure. Element refers to the whole element including both web and plate.

$l_{tot} := l_{web} \cdot 2 + d_{centrepiece} = 18\,m$ Total length of the foundation

$V_{web} := b_{web} \cdot l_{web} \cdot h_{web} = 3.981 \cdot m^3$ Volume of one web

$\beta := \dfrac{360deg}{20} = 18 \cdot deg$ Angle between the webs

$$A_{plate} := \left(l_{web} + \frac{d_{centrepiece}}{2}\right)^2 \cdot \pi \ldots + -A_{centrepiece} - 20b_{web} \cdot l_{web} = 189.334\,m^2$$

Area of the bottom plate excluding the centrepiece and the webs

$V_{plate} := h_{plate} \cdot A_{plate} = 75.734 \cdot m^3$ Volume of the bottom plate

Fill

$h_{fill} := 0.5m$ Height of fill

$h_{fill.plate} := h_{web} + h_{fill} - h_{plate} = 1.85\,m$ Height of fill above plate

$V_{fill} := 20l_{web} \cdot b_{web} \cdot h_{fill} + A_{plate} \cdot h_{fill.plate} = 373.018 \cdot m^3$ Volume of the fill

Verification of needed thickness of bottom plate

The slab is assumed to work in one direction, therefore the calculation can be done as a beam, calculated per lenght meter.

Maximum span length of the outermost part of the piece of cake

$$l_{span} := 2 \cdot l_{web} \cdot \tan\left(\frac{\beta}{2}\right) = 2.059\,m$$

Height of the beam/slab. Assumed fixed connection, page B113 Bärande konstruktioner del 1

$$h_{plate.min} := \frac{l_{span}}{30} = 0.069\,m$$

$$Check_1 := \left| \begin{array}{ll} \text{"Sufficient height of the bottom plate"} & \text{if } h_{plate} \ge h_{plate.min} \\ \text{"Not sufficient height of the plate"} & \text{if } h_{plate} < h_{plate.min} \end{array} \right.$$

$Check_1 =$ "Sufficient height of the bottom plate"

Summary of weights and volumes

$V_{tot} := 20V_{web} + V_{centrepiece} + V_{plate} = 212.3 \cdot m^3$ Total volume of the concrete

$m_{tot} := \left(20V_{web} + V_{centrepiece} + V_{plate}\right) \cdot \dfrac{\rho_c}{g} = 541.214 \cdot ton$ Total weight of the concrete

$m_{web} := V_{web} \cdot \dfrac{\rho_c}{g} = 10.149 \cdot ton$ Weight of one web

$m_{element} := m_{web} + \dfrac{V_{plate}}{20} \cdot \dfrac{\rho_c}{g} = 19.803 \cdot ton$ Weight of one element, consisting of one web and a twentieth of the plate

Self-weight of the foundation and the soil

$$G_k := \left(20V_{web} + V_{centrepiece} + V_{plate}\right) \cdot \rho_c + V_{fill} \cdot \rho_{fill} = 11.16 \cdot MN$$

Verification of the effective flange width

In order to decide which method to use for calculation of the contribution from the bottom plate, the effective flange width for the web is calculated.
The effective width is thereafter compared to the span lenght, a, between two webs to see if the whole span accounts to take the soil pressure or if we must consider the plate as a flange.

$$a := 2 \cdot l_{web} \cdot \sin\left(\frac{\beta}{2}\right) = 2.034\,m$$

The maximum span between each web, calculated with the angle β and trigonometry

$$b_i := \frac{a}{2}$$

Half of the largest span between the webs, at the outermost part of the foundation

$$l_0 := \frac{l_{tot}}{2} = 9\,m$$

The distance between the moment zero points, it is assumed to be half the length of the foundation.

$$b_{eff} := \min\left(0.2 \cdot b_i + 0.1 \cdot l_0, b_i\right) = 1.017\,m$$

Effective width

$$Check_{eff.width} := \begin{vmatrix} \text{"The whole plate takes the soil pressure"} & \text{if } b_{eff} = \frac{a}{2} \\ \text{"The effective width takes the soil pressure"} & \text{if } b_{eff} < \frac{a}{2} \end{vmatrix}$$

$$Check_{eff.width} = \text{"The whole plate takes the soil pressure"}$$

This means that the whole plate is contributing to taking the soil pressure, therefore the soil pressure can be calculated as for a solid circular foundation.

2. Global stability of the concept

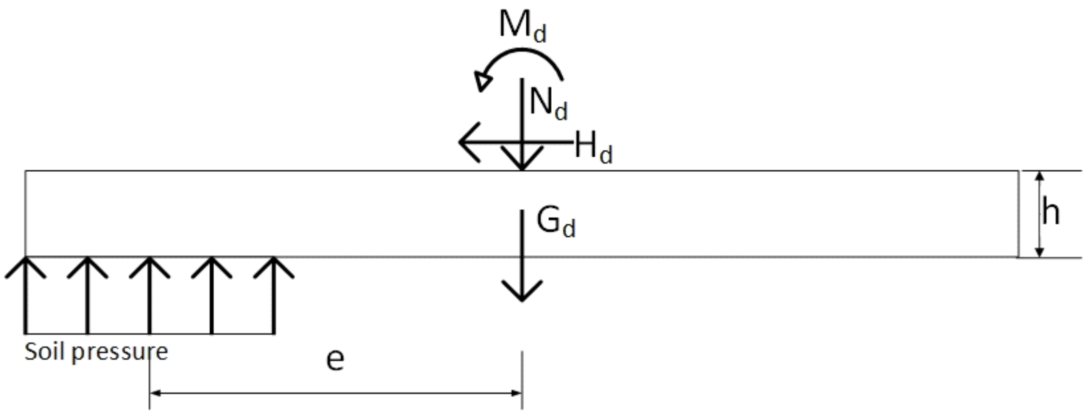


Figure 4: The loads acting on the foundation and the definition of e. The horisontal force is assumed to be resisted by the soil and is not further investigated, only the resulting moment due to its eccentricity is included. The distribution of the soil pressure is just a principal and will be changed according the geometry of the foundation.

Eccentricity

From moment equilibrium around the resultant of the soil pressure, the needed eccentricity can be calculated

$$e := \frac{M_d + H_d \cdot h_{centrepiece}}{\gamma_{dead}(G_k + N_k)} = 7.59\,m$$

Eccentricity for the soil pressure resultant

Check that the eccentricity fits within half of the foundation, otherwise it will be too large since the length of the soil pressure zone mustn't exceed the total length of the foundation in order to be in global equilibrium.

$$Check_{eccentricity} := \begin{array}{|ll} \text{"The eccentricity is ok!"} & \text{if } e \le \frac{l_{tot}}{2} \\ \text{"Eccentricity is not ok!"} & \text{otherwise} \end{array}$$

$$Check_{eccentricity} = \text{"The eccentricity is ok!"}$$

Calculation of the area of the soil pressure zone

The soil pressure zone of the foundation should be decided. It is calculated iteratively by deciding an angle of the soil pressure zone, which gives a certain soil pressure area and a centre of gravity of the soil pressure zone.
The centre of gravity should match the eccentricity given by the equilibrium calculations above. To determine the center of gravity for the segment the areas in the figure below and respective center of gravity for each area need to be calculated.

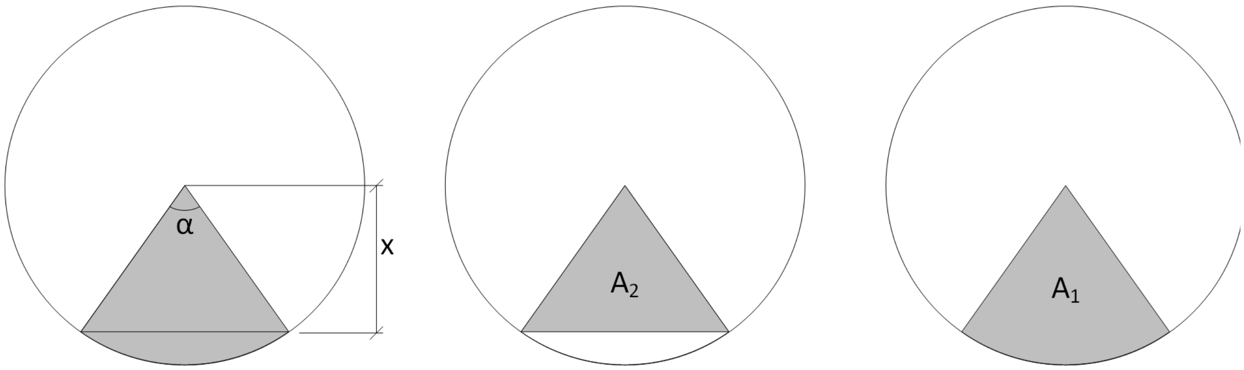


Figure 5: Illustration of the shape of the soil pressure zone for the foundation. The area of the soil pressure is calculated as $A_{sp} := A_1 - A_2$ *and thereafter the centre of gravity is calculated*

$\alpha := 105deg$ — Assumed angle of the soil pressure zone, defined as shown in the figure above

$x := \cos(0.5\alpha)\cdot\frac{l_{tot}}{2} = 5.479\,m$ — Distance from the center of the foundation to where the radial part of the circle sector start.

$A_1 := \frac{\alpha}{2}\cdot\left(\frac{l_{tot}}{2}\right)^2 = 74.22\,m^2$ — Area of the whole sector A_1

$A_2 := 2\left(x\cdot x\cdot\frac{\tan(0.5\alpha)}{2}\right) = 39.12\,m^2$ — Area of the triangle A_2

$A_{soil} := A_1 - A_2 = 35.1\,m^2$ — Area of the soil pressure

$tp_1 := \frac{2}{3}\cdot\frac{l_{tot}}{2}\cdot\frac{\sin(0.5\alpha)}{\frac{\alpha}{2}} = 5.195\,m$ — Center of gravity for the area A_1

$tp_2 := \frac{2}{3}\cdot x = 3.653\,m$ — Centre of gravity of the area A_2

$tp := \frac{A_1\cdot tp_1 - A_2\cdot tp_2}{A_1 + A_2} + x = 7.62\,m$ — Centre of gravity of the segment which takes the soil pressure

Check if the assumed angle is correct. If the difference between the eccentricity and the c.g of the soil pressure zone is smaller than 1%, we assume that we have found the correct soil pressure zone.

$$Check2 := \begin{cases} \text{"Area of the soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp}{e}\right| < 1\% \\ \text{"Too large difference"} & \text{otherwise} \end{cases}$$

Check2 = "Area of the soil pressure zone is correct"

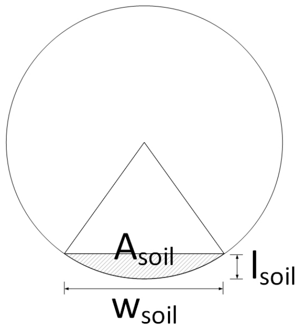


Figure 6: Illustration of the soil pressure zone and the dimensions of it

$w_{soil} := l_{tot} \cdot \sin\left(\frac{\alpha}{2}\right) = 14.28\,m$ — Maximum width of soil pressure zone

$l_{soil} := \frac{l_{tot}}{2} - \sqrt{\left(\frac{l_{tot}}{2}\right)^2 - \left(\frac{w_{soil}}{2}\right)^2} = 3.521\,m$ — Maximum height of soil pressure zone

Soil pressure acting on the bottom plate

$\sigma_{soil} := \frac{\gamma_{dead}\left(N_k + G_k\right)}{A_{soil}} = 378.47 \cdot kPa$ — Soil pressure

$\frac{\sigma_{soil}}{\sigma_{Rv}} = 0.378$ — Utilisation

Check if the given soil resistance, σ_{Rv}, is sufficient

$$\text{Check3} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil}}{\sigma_{Rv}} < 1 \\ \text{"The soil resistance is not sufficent"} & \text{otherwise} \end{cases}$$

Check3 = "OK! Soil resistance is sufficent"

Webs contribution to the global stability

Arc length per element

$\beta = 18 \cdot deg$ — Angle between each web

$l_{arc.element} := \beta \cdot \frac{l_{tot}}{2} = 2.827\,m$ — Arc length of each element

Arc length contributing to the global stability

$\alpha = 105 \cdot deg$ — Angle of the soil pressure zone

$l_{arc.soil} := \alpha \cdot \frac{l_{tot}}{2} = 16.493\,m$ — Arc length of the soil pressure zone

Number of elements contributing to the global stability

$n_{elements.soil} := \mathrm{floor}\left(\frac{l_{arc.soil}}{l_{arc.element}}\right) = 5$

So 5 elements should contribute to the global stability.

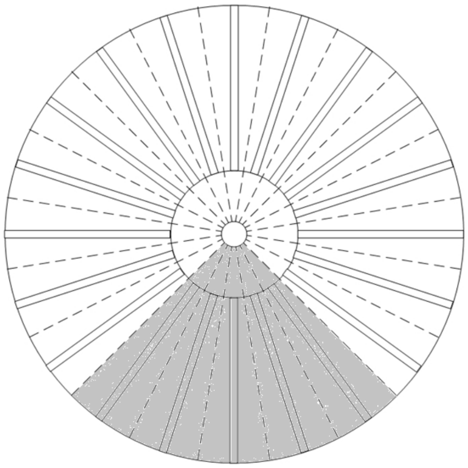


Figure 7: The elements contributing to take the soil pressure

3. Division into strips

The element is divvied into small element strips in order to easier handle the circular shape of the foundation. The length, height and width for each element are assumed to be constant over the element. Also the ground pressure are assumed to be constant over each element.

The moment is calculated around the cut, r.fc from the centre of the gravity. The moment is calculated in separate functions and diagram, one for the positive moment and one for the negative moment.

Divide the element into strips

In order to enable the calculations of the moments and shear forces, we look at only one element of the foundation, see figure below. The wind is assumed to be acting in one direction, wind direction 1, which is the worst case.

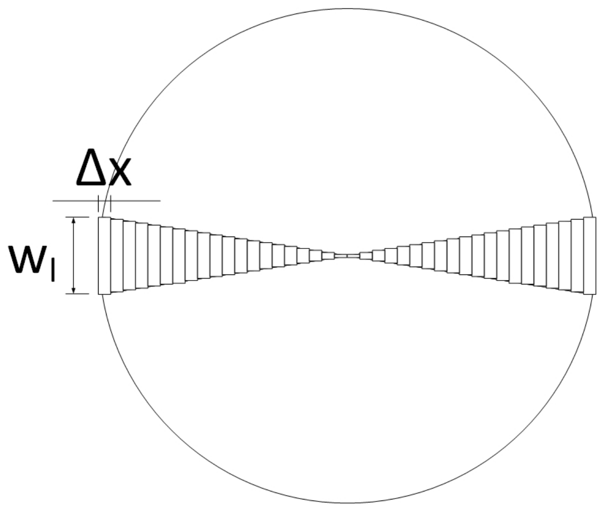


Figure 8: The foundation is divided into strips, the lenght, Δw, and width, Δx, of the strips are shown in the figure.

Definition of vectors for directions and numbering of the strips

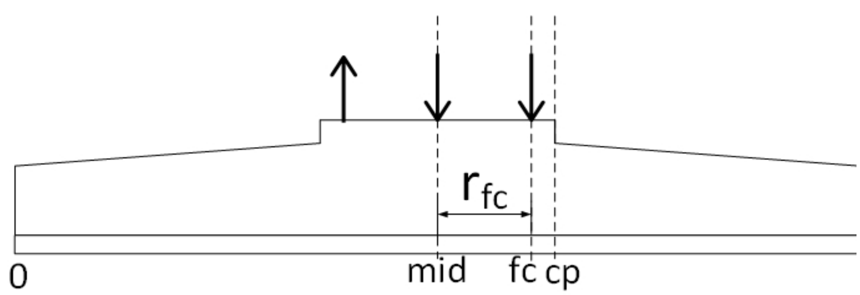


Figure 9: Definition of certain sections in the foundation that will be used in the calculations.

Vectors are defined to enable the calculations for the moment and shear force, according to figures below. Both vectors that give each strip a certain number, starting from point i or j, and vectors that give each strip a coordinate, also starting from i and j.

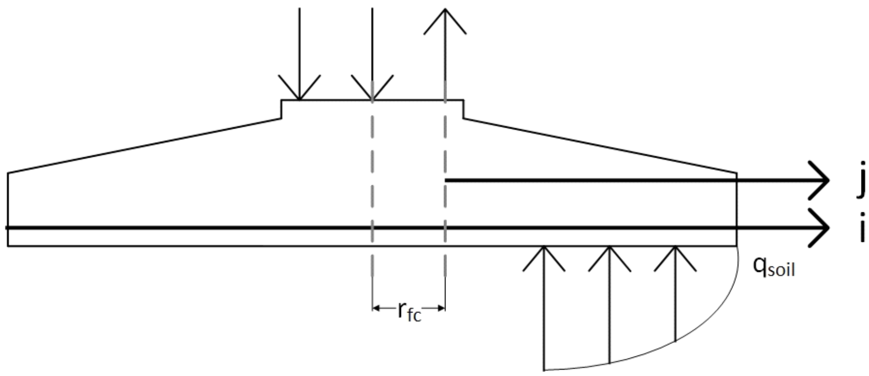


Figure 10: Vectors numbering of the strips from different locations in the foundation

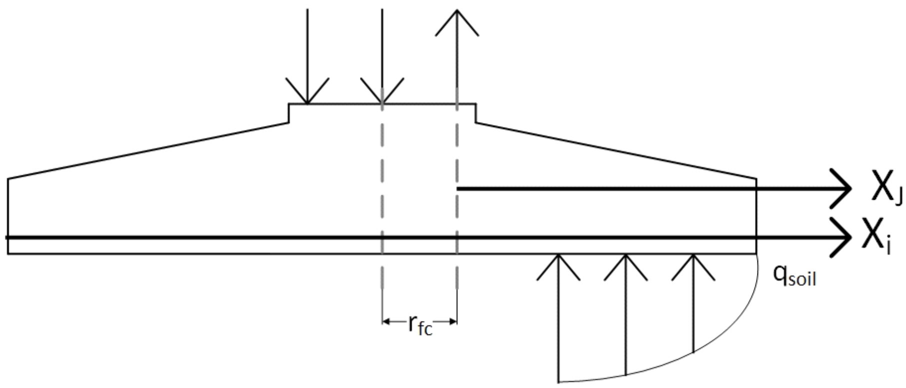


Figure 11: Vectors with coordinates of the strips, from different locations in the foundation

Vector i

$\Delta x := 10mm$ — Width of each element strip, see figure above

$i_{tot} := \frac{l_{tot}}{\Delta x} - 1 = 1.799 \times 10^3$ — Total number of element strips

$i := 1..i_{tot}$ — A vector that gives each strip a certain number, see figure above

$i_{mid} := ceil\left(\frac{i_{tot}}{2}\right) = 900$ — The number of the middle strip in vector i

$i_{r.cp} := ceil\left(\frac{\frac{l_{tot}-d_{centrepiece}}{2}}{\Delta x}\right) = 650$ — Position in vector i where the centrepiece starts

$i_{r.fc} := ceil\left(\frac{\frac{l_{tot}}{2} + r_{fc}}{\Delta x}\right) = 1.08 \times 10^3$ — Position in vector i where the force couple is acting

$i_{soil.1} := ceil\left(\frac{l_{tot} - l_{soil}}{\Delta x}\right) = 1.448 \times 10^3$ — Number of the strips in vector i where the soil pressure is no longer zero

$$X_{I_i} := i\cdot\Delta x + \frac{\Delta x}{2} - \frac{l_{tot}}{2}$$

Coordinate for each strip. X is defined from the centre of the foundation, see figure above

$$w_{tot.I_i} := 2\cdot\sqrt{\left(\frac{l_{tot}}{2} + X_{I_i}\right)\cdot\left(\frac{l_{tot}}{2} - X_{I_i}\right)}$$

The width of bottom plate along vector i, developed from the equation for the length of a chord.

Element length

The width of the element is calculated along the vector i (or X_i), w_I is defined in figure above.

$$w_{I_i} := \begin{cases} w_{tot.I_i} & \text{if } \frac{-l_{tot}}{2} \le X_{I_i} < -\cos\left(\frac{\beta}{2}\right)\cdot\frac{l_{tot}}{2} \\ 2\left(-X_I\right)_i\cdot\tan\left(\frac{\beta}{2}\right) & \text{if } \left(-\cos\left(\frac{\beta}{2}\right)\cdot\frac{l_{tot}}{2}\right) \le X_{I_i} < 0 \\ 2\,X_{I_i}\cdot\tan\left(\frac{\beta}{2}\right) & \text{if } 0 \le X_{I_i} < \cos\left(\frac{\beta}{2}\right)\cdot\frac{l_{tot}}{2} \\ w_{tot.I_i} & \text{if } \cos\left(\frac{\beta}{2}\right)\cdot\frac{l_{tot}}{2} \le X_{I_i} < \frac{l_{tot}}{2} \end{cases}$$

$$A_{I_i} := \Delta x\cdot w_{I_i} = ...$$

A vector defining the area of each strip in vector i

Vector j

$$j_{tot} := \frac{\frac{l_{tot}}{2} - r_{fc}}{\Delta x} - 1 = 719$$

Number of elements in vector j

$$j := 0..j_{tot}$$

A vector that gives each strip a certain number. Defined from the section fc to the end of the foundation, see figure above

$$X_{J_j} := X_{I_{j+i_{r.fc}}} = ...$$

A vector that gives the coordinates for the elements in vector j

$$w_{tot_j} := w_{tot.I_{j+i_{r.fc}}} = ...$$

The width of bottom plate along vector j

$$w_{J_j} := w_{I_{j+i_{r.fc}}} = ...$$

A vector that gives the length of each strip in vector j

$$A_{J_j} := w_{J_j}\cdot\Delta x = ...$$

A vector that gives the area of each strip in vector j

$$s_{J_j} := X_{J_j} - X_{J_0} + \frac{\Delta x}{2} = ...$$

A vector that gives the lever arm for each strip in vector j, The lever arm reaches from g.c. of the strip to the section fc

The height of the foundation

Height of element with adjusted height of the centrepiece

The approximated variation of the height of the element, assuming that the centrepiece has the same height as the element at r.cp

$$\Delta h_{I_i} := \begin{cases} h_{element.0} + \dfrac{h_{element.r.cp} - h_{element.0}}{\left(\dfrac{l_{tot} - d_{centrepiece}}{2}\right)} \cdot \Delta x \cdot i & \text{if } \Delta x \cdot i \le \dfrac{l_{tot}}{2} - \dfrac{d_{centrepiece}}{2} \\ h_{element.r.cp} & \text{if } \dfrac{l_{tot}}{2} - \dfrac{d_{centrepiece}}{2} < \Delta x \cdot i \le \dfrac{l_{tot}}{2} + \dfrac{d_{centrepiece}}{2} \\ h_{element.r.cp} \cdots \\ \quad + -\left[\dfrac{h_{element.r.cp} - h_{element.0}}{\left(\dfrac{l_{tot} - d_{centrepiece}}{2}\right)}\right] \cdot \Delta x \cdot \left(i - \operatorname{ceil}\left(\dfrac{\dfrac{l_{tot}+d_{centrepiece}}{2}}{\Delta x}\right)\right) & \text{if } \Delta x \cdot i > \dfrac{l_{tot}}{2} + \dfrac{d_{centrepiece}}{2} \end{cases}$$

Height of the web, with adjusted height of the centrepiece

The variation of the height of the legs, excluding the height of the plate. The height at the centrepiece is treated in the same way as for Δh_i

$$\Delta h_{web.I_i} := \Delta h_{I_i} - h_{plate} = \ldots$$

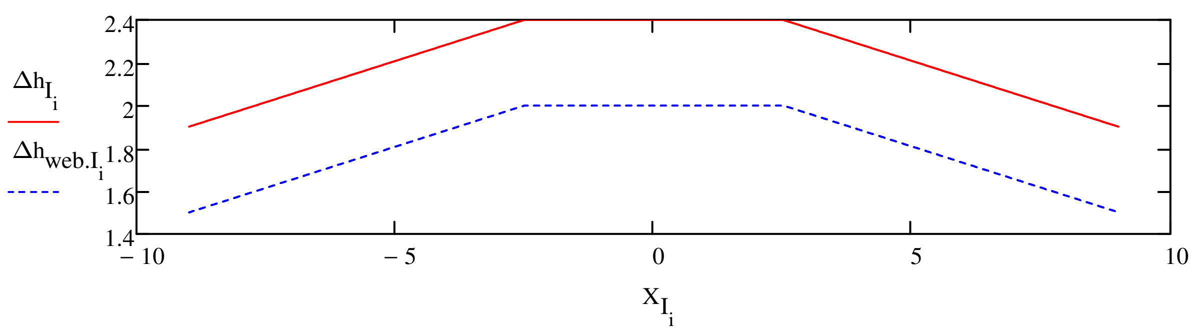


Diagram: Variation of the height along vector i. Δh_{I2_i} *showing the height disconsidering the centrepiece and* $\Delta h_{web.I_i}$ *considering only the height of the web (without the bottom plate)*

The variation of the height of the foundation along vector i

Defined along vector j, from f.c to the edge of the foundation

$$\Delta h_{J_j} := \Delta h_{I_{j+i_{r.fc}}} = \ldots$$

The variation of the height of the foundation, disregarding the extra height over the centrepiece, which will not be accessible for the reinforcement.

$$\Delta h_{web.J_j} := \Delta h_{web.I_{j+i_{r.fc}}} = \ldots$$

The variation of the height of the web along vector j

Variation of the height of the fill

Due to the variation of the height of the leg, also the height of the fill will vary along the element.

$$h_{fill.leg.I_i} := h_{centrepiece} - h_{plate} - \Delta h_{web.I_i} = \ldots$$

Variation of the height of the fill along vector i

$$h_{fill.leg.J_j} := h_{centrepiece} - h_{plate} - \Delta h_{web.J_j} = \ldots$$

Variation of the height of the fill along vector j

4. Loads acting on the foundation, locally

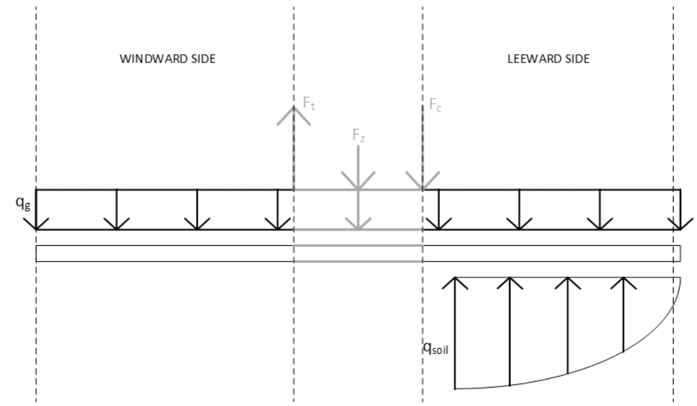


Figure 12: Loads acting on the foundation, in the local analysis only the part of the foundation outside the force couple resultant is considered.

In the local calculations the moment distribution is calculated both for the windward and the leeward side in separate calculations. The assumed resulting loading is shown in figure below.

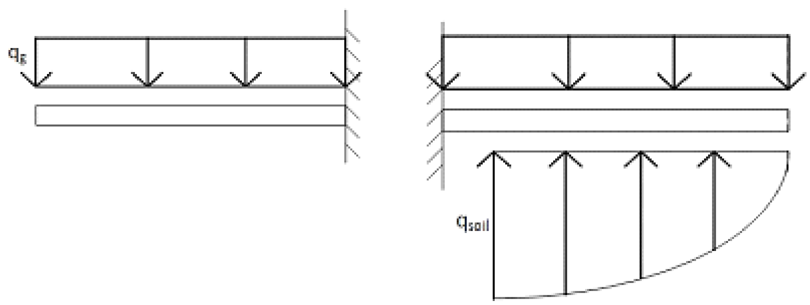


Figure 13: Assumed resulting loading a) for the windward side b) for the leeward side

Self-weight

The total self-weight of the foundation including the fill above the foundation, is assumed to be uniformly distributed over the total length of the foundation. In reality this distribution is not uniform, but depends on the shape of the foundation.

$$q_g := \frac{\gamma_{dead} G_k}{l_{tot}} = 558.02 \cdot \frac{kN}{m}$$

Distributed self-weight over the length of the foundation

Soil pressure

Element 1

$$l_1 := \frac{l_{tot}}{2} = 9\,m$$

Length of second elements, defined from the centre of the foundation

$$l_{soil.1} := l_{soil} = 3.521\,m$$

Length of the soil pressure zone in element 1

$$l_{sp.1} := l_1 - l_{soil.1} = 5.479\,m$$

Distance from the centre of the foundation to the section s.p.

$$\sigma_{g.soil.1_j} := \begin{vmatrix} 0 \ \text{ if } \ X_{J_j} < \dfrac{l_{tot}}{2} - l_{soil.1} \\ \sigma_{soil} \ \text{ otherwise} \end{vmatrix}$$

Vector with the soil pressure for each strip in element 1. The soil pressure is zero outside the soil pressure area.

Element 2

$$l_2 := \frac{l_{tot}}{2} = 9\,\text{m}$$

Length of second elements, defined from the centre of the foundation

$$l_{2.comp} := \frac{l_{tot}}{2}\cdot\cos(\beta) = 8.56\,\text{m}$$

Composant of the length, in l_1-direction

$$l_{soil.2.comp} := l_{soil.1} - \left(l_1 - l_{2.comp}\right) = 3.081\,\text{m}$$

Composant of the length of the soil pressure zone in element 2 in l_1-direction.

$$l_{soil.2} := \frac{l_{soil.2.comp}}{\cos(\beta)} = 3.239\,\text{m}$$

Length of the soil pressure zone in element 2

$$\sigma_{g.soil.2_j} := \begin{vmatrix} 0 \ \text{ if } \ X_{J_j} < \dfrac{l_{tot}}{2} - l_{soil.2} \\ \sigma_{soil} \ \text{ otherwise} \end{vmatrix}$$

Vector with the soil pressure for each strip in element 2 The soil pressure is zero outside the soil pressure area.

Element 3

$$l_3 := \frac{l_{tot}}{2} = 9\,\text{m}$$

Length of third elements, defined from the centre of the foundation

$$l_{3.comp} := \frac{l_{tot}}{2}\cdot\cos(2\beta) = 7.281\,\text{m}$$

Composant of the length, in l_1-direction

$$l_{soil.3.comp} := l_{soil.1} - \left(l_1 - l_{3.comp}\right) = 1.802\,\text{m}$$

Composant of the length of the soil pressure zone in element 3 in l_1-direction.

$$l_{soil.3} := \frac{l_{soil.3.comp}}{\cos(2\beta)} = 2.228\,\text{m}$$

Length of the soil pressure zone in element 3

$$\sigma_{g.soil.3_j} := \begin{vmatrix} 0 \ \text{ if } \ X_{J_j} < \dfrac{l_{tot}}{2} - l_{soil.3} \\ \sigma_{soil} \ \text{ otherwise} \end{vmatrix}$$

Vector with the soil pressure for each strip in element 3. The soil pressure is zero outside the soil pressure area.

Resulting loads for each element

For the positive moment the self-weight is favorable and the partial factor 0.9 is used on the loads.
For the negative moment the foundation is assumed to be "hanging" from the centrepiece, and therefore the self-weight is unfavorable and the partial factor 1.1 is used.
The moment is calculated in the cut fc at a distance r_{fc} from the centre of the foundation. Therefore the loads below are calculated for elements in vector j.

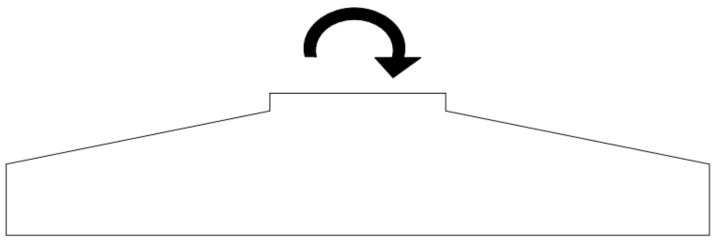


Figure 14: The moment on the windward side is denoted the negative moment and the moment on the leeward side is denoted the positive moment.

Dead load, leeward side

Vector with dead load for each element, used for calculate the positive moment

$$N_{dead.p_j} := \gamma_{dead}\left(\Delta x \cdot \Delta h_{web.J_j} \cdot b_{web} + \Delta x \cdot h_{plate} \cdot w_{J_j}\right) \cdot \rho_c \cdots + \left[\Delta x \cdot h_{fill.leg.J_j} \cdot b_{web} + \Delta x \cdot h_{fill.plate} \cdot \left(w_{J_j} - b_{web}\right)\right] \cdot \rho_{fill} = \ldots$$

Dead load, windward side

Vector with dead load for each element, used for calculate the negative moment

$$N_{dead.n_j} := \gamma_{abn}\left(\Delta x \cdot \Delta h_{web.J_j} \cdot b_{web} + \Delta x \cdot h_{plate} \cdot w_{J_j}\right) \cdot \rho_c \cdots + \left[\Delta x \cdot h_{fill.leg.J_j} \cdot b_{web} + \Delta x \cdot h_{fill.plate} \cdot \left(w_{J_j} - b_{web}\right)\right] \cdot \rho_{fill} = \ldots$$

5. Moment distribution, locally

Moment distribution - element 1

$N_{soil.1_j} := \sigma_{g.soil.1_j} \cdot w_{J_j} \cdot \Delta x = ...$

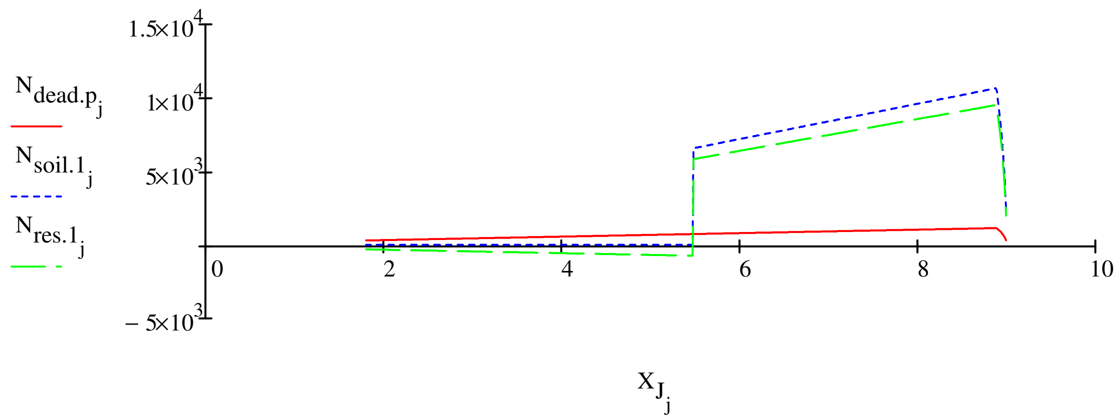
Vector with ground pressure as a total force acting on each element. The ground pressure has only a value in the soil pressure zone, otherwise it is zero.

It is only the leeward side that it is of interest to calculate the resultant. The reason is that the soil pressure is not acting on the leeward side, so there the resultant is equal to the dead-load.

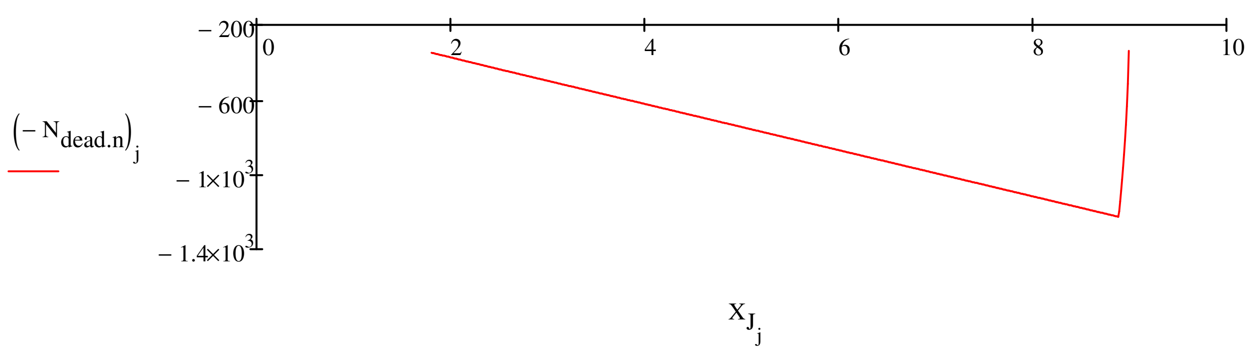
$N_{res.1_j} := N_{soil.1_j} - N_{dead.p_j} = ...$

Resulting force vector, when summing up the dead load and the soil pressure together with their directions.

Positive side



Negative side



Positive moment distribution, on the leeward side

The moment distribution, M_p is calculated from the cut fc to the end of the foundation. The positive moment is defined on the leeward side
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side, The resulting force, N_{res}, for each element is multiplied with its lever arm, s_J, for its respective element and then summed up for the all the elements into the positive moment M_p .

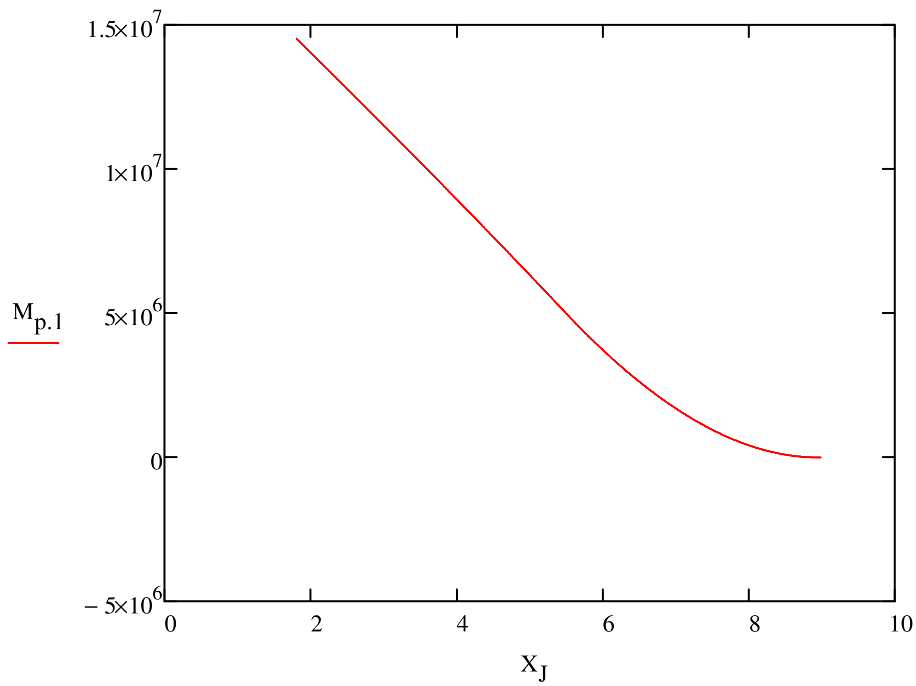
Moment distribution from r_{fc} to the edge of the foundation

$$M_{p.1} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.1_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$M_{p.1.max} := M_{p.1_0} = 1.453 \times 10^4 \cdot kN \cdot m$ Moment in critical cut, r_{fc}, gives the maximum moment on the positive side

Moment diagram of the positive moment



Negative distribution, on the windward side

The moment distribution, M_n is calculated from the cut fc to the edge of the foundation. The negative moment is defined on the windward side.
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side, The resulting force, N_{res}, for each element is multiplied with the lever arm, s_J ,for its respective element and then summed up for the all the elements into the negative moment M_n .

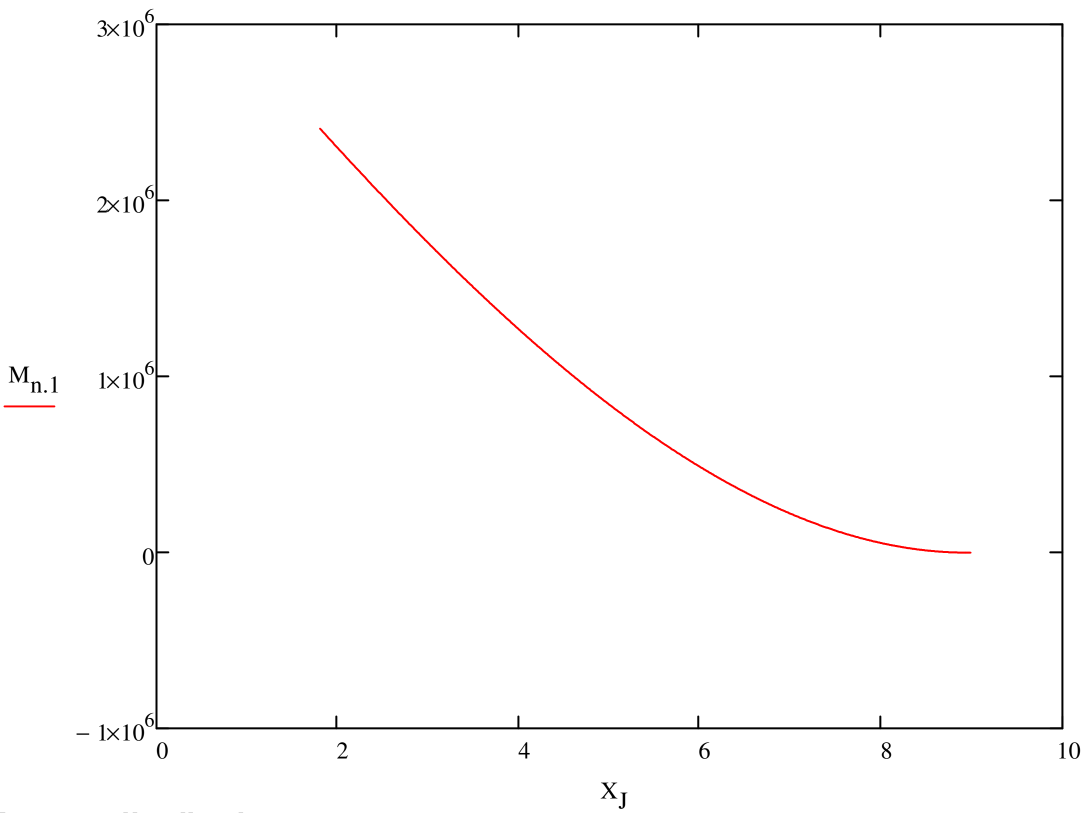
Moment distribution from r_{fc} to the edge of the foundation

$$M_{n.1} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{dead.n_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$M_{n.1.max} := M_{n.1_0} = 2.41 \times 10^3 \cdot kN \cdot m$ Moment in critical cut, r_{fc}, gives the maximum moment on the negative side

Moment diagram of the negative moment



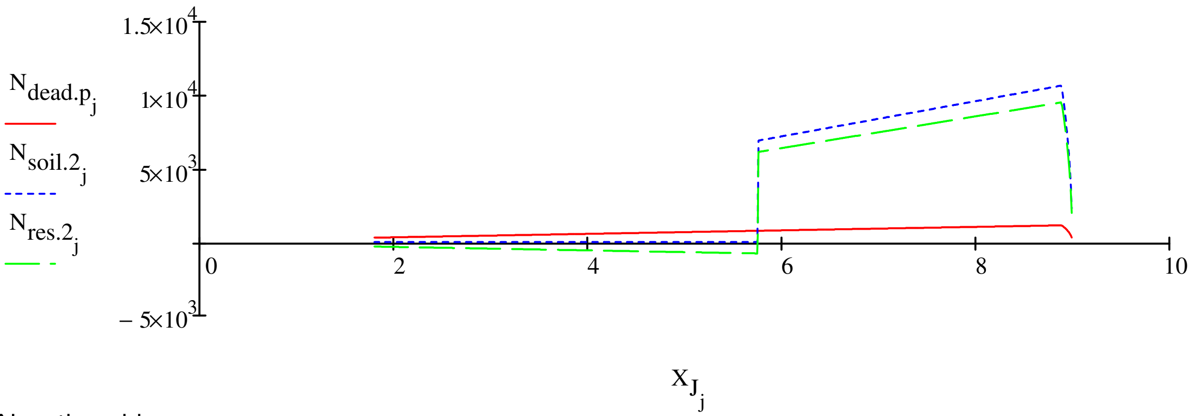
Moment distribution - element 2

$N_{soil.2_j} := \sigma_{g.soil.2_j} \cdot w_{J_j} \cdot \Delta x = ...$ Vector with ground pressure as a total force acting on each element. The ground pressure has only a value in the soil pressure zone, otherwise it is zero.

It is only the leeward side that it is of interest to calculate the resultant. The reason is that the soil pressure is not acting on the leeward side, so there the resultant is equal to the dead-load.

$N_{res.2_j} := N_{soil.2_j} - N_{dead.p_j} = ...$ Resulting force vector, when summing up the dead load and the soil pressure together with their directions.

Positive side



Negative side

The negative moment distribution is equal to the one in element 1, since only the self-weight is acting and it is not effected by the change in soil pressure.

Positive moment distribution, on the leeward side

The moment distribution in element 2, $M_{p.2}$, is calculated in the same manner as for element 1, $M_{p.1}$

Moment distribution from r_{fc} to the edge of the foundation

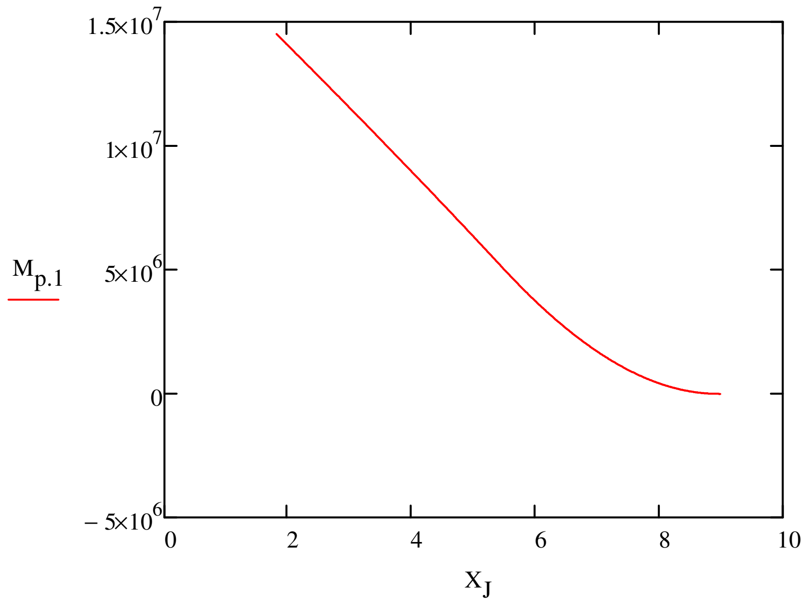
$$M_{p.2} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.2_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$$M_{p.2.max} := M_{p.2_0} = 1.381 \times 10^4 \cdot kN \cdot m$$

Moment in critical cut, r_{fc}, gives the maximum moment on the positive side

Moment diagram of the positive moment



Negative distribution, on the windward side

The negative moment distribution in element 2 is equal to the negative moment distribution in element 1, since only the self-weight is acting on the negative side and it is not effected by the change in soil pressure.

$M_{n.2} := M_{n.1}$

$M_{n.2.max} := M_{n.1.max} = 2.41 \times 10^{3} \cdot kN \cdot m$

Moment distribution - element 3

$N_{soil.3_j} := \sigma_{g.soil.3_j} \cdot w_{J_j} \cdot \Delta x = ...$

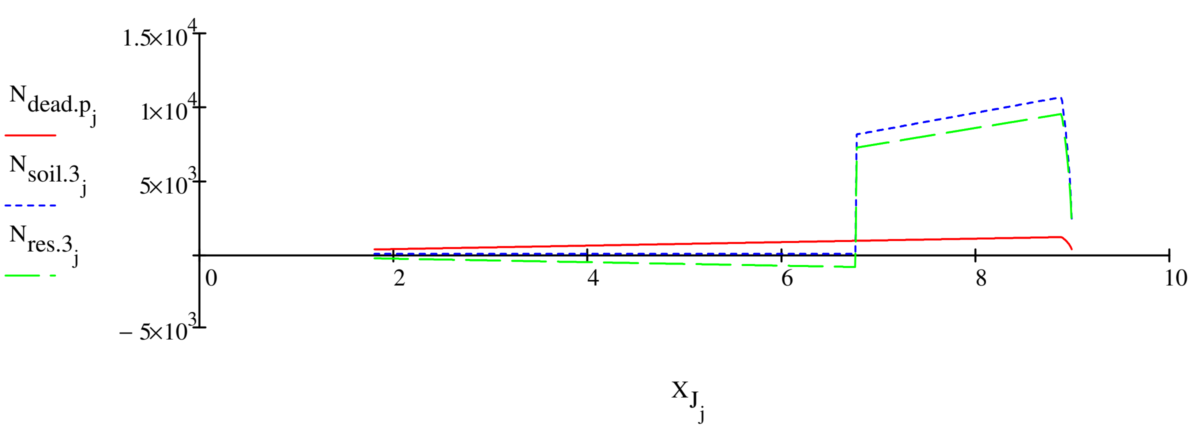
Vector with ground pressure as a total force acting on each element. The ground pressure has only a value in the soil pressure zone, otherwise it is zero.

It is only the leeward side that it is of interest to calculate the resultant. The reason is that the soil pressure is not acting on the leeward side, so there the resultant is equal to the dead.

$N_{res.3_j} := N_{soil.3_j} - N_{dead.p_j} = ...$

Resulting force vector, when summing up the dead load and the soil pressure together with their directions.

Positive side



Negative side

The negative moment distribution is equal to the one in element 1, since only the self-weight is acting and it is not effected by the change in soil pressure.

Positive moment distribution, on the leeward side

The moment distribution in element 3, $M_{p.3}$, is calculated in the same manner as for element 1, $M_{p.1}$

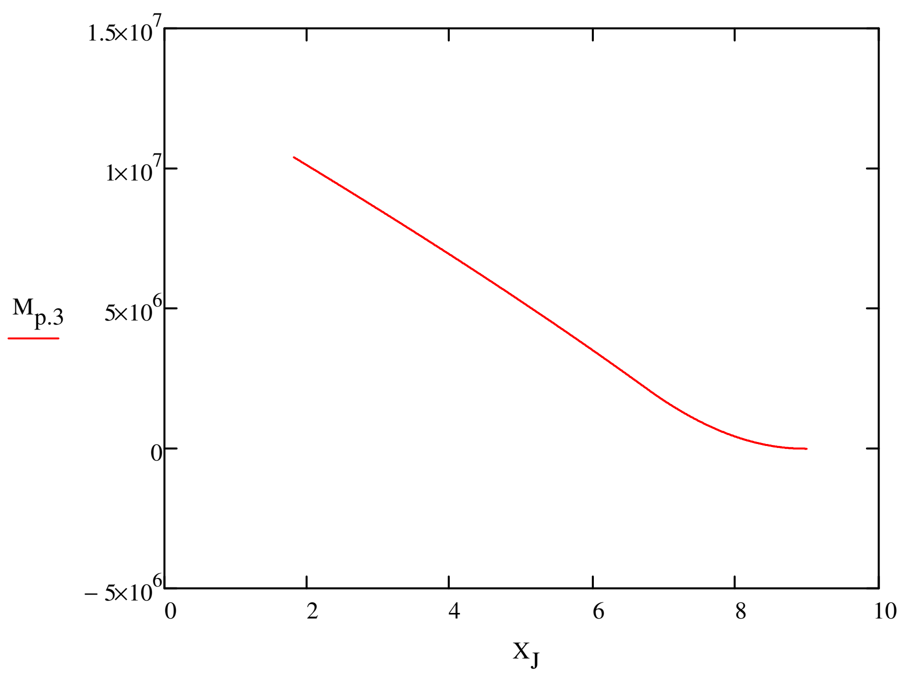
Moment distribution from r_{fc} to the edge of the foundation

$$M_{p.3} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.3_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$M_{p.3.max} := M_{p.3_0} = 1.041 \times 10^4 \cdot kN \cdot m$ Moment in critical cut, r_{fc}, gives the maximum moment on the positive side

Moment diagram of the positive moment



Negative distribution, on the windward side

The negative moment distribution in element 3 is equal to the negative moment distribution in element 1, since only the self-weight is acting on the negative side and it is not effected by the change in soil pressure.

$M_{n.3} := M_{n.1}$

$M_{n.3.max} := M_{n.1.max} = 2.41 \times 10^3 \cdot kN \cdot m$

Summary of the moment calculations

$M_{p.1.max} = 1.453 \times 10^4 \cdot kN \cdot m$ Maximum moment on the positive side, in element 1

$M_{p.2.max} = 1.381 \times 10^4 \cdot kN \cdot m$ Maximum moment on the positive side, in element 2

$M_{p.3.max} = 1.041 \times 10^4 \cdot kN \cdot m$ Maximum moment on the positive side, in element 3

$M_{n.1.max} = 2.41 \times 10^3 \cdot kN \cdot m$ Maximum moment on the negative side, in all three elements

Due to the fact that the wind can blow from any side, all sides of the foundation must be dimensioned for the maximum moment both the positive and negative moment.
The largest positive moment is dimensioning for the bottom reinforcement and the largest negative moment for the top reinforcement.

$M_{p.max} := \max(M_{p.1.max}, M_{p.2.max}, M_{p.3.max}) = 1.453 \times 10^4 \cdot kN \cdot m$ Maximum positive moment

$M_{n.max} := M_{n.1.max} = 2.41 \times 10^3 \cdot kN \cdot m$ Maximum negative moment

Composant's of the moments

Due to the angle between the legs, the composants of each leg is calculated. The composants of the moments from the different legs are calculated, the main direction (x) is in the same direction as the moment in element 1, while the secondary moment (y) is perpendicular to element 1.

$M_{p.1.x} := M_{p.1.max} \cdot \cos(0) = 1.453 \times 10^4 \cdot kN \cdot m$ $M_{n.1.x} := M_{n.1.max} \cdot \cos(0) = 2.41 \times 10^3 \cdot kN \cdot m$

$M_{p.1.y} := M_{p.1.max} \sin(0) = 0 \cdot kN \cdot m$ $M_{n.1.y} := M_{n.1.max} \sin(0) = 0 \cdot kN \cdot m$

$M_{p.2.x} := M_{p.2.max} \cdot \cos(\beta) = 1.313 \times 10^4 \cdot kN \cdot m$ $M_{n.2.x} := M_{n.2.max} \cdot \cos(\beta) = 2.292 \times 10^3 \cdot kN \cdot m$

$M_{p.2.y} := M_{p.2.max} \cdot \sin(\beta) = 4.267 \times 10^3 \cdot kN \cdot m$ $M_{n.2.y} := M_{n.2.max} \sin(\beta) = 0.745 \cdot 10^3 kN \cdot m$

$M_{p.3.x} := M_{p.3.max} \cdot \cos(2\beta) = 8.422 \times 10^3 \cdot kN \cdot m$ $M_{n.3.x} := M_{n.3.max} \cdot \cos(2\beta) = 1.949 \times 10^3 \cdot kN \cdot m$

$M_{p.3.y} := M_{p.3.max} \sin(2\beta) = 6.119 \times 10^3 \cdot kN \cdot m$ $M_{n.3.y} := M_{n.3.max} \cdot \sin(2\beta) = 1.416 \times 10^3 \cdot kN \cdot m$

Bending reinforcement in the bottom

The maximum moment is acting in section fc, inside the centrepiece. This maximum moment should be used to design the needed reinforcement amount. Due to the change of geometry between the cross-section of the centrepiece and the outer parts, the placement of reinforcement is based on the cross-section in section cp.

Chosen dimension of the bars

$\phi_b := 25mm$ — Diameter of the bars for the bottom reinforcement

$A_{si.b} := \dfrac{\phi_b^{\,2} \cdot \pi}{4} = 4.909 \times 10^{-4}\,m^2$ — Area of each reinforcement bar

Indata from Eurocode 2 SS-EN 1992-1-1:2005 8.2 (2)

$k_1 := 1 \qquad k_2 := 5mm$ — Factors from EC2

$d_g := 32mm$ — Assumed dimension of the aggregates

$c_{bar.b} := \max\left(d_g + k_2, 20mm, k_1 \cdot \phi_b\right) = 0.037\,m$ — Distance between bars and layers

$\Delta c_{dev} := 10mm$ — Factors from EC2

$c_{min.b} := \phi_b + \Delta c_{dev} = 0.035\,m$ — Minimum concrete cover (eq 4.1)

$c_{cover.b} := c_{min.b}$ — Chosen concrete cover

Maximum number of bars that fits in the element

The reinforcement is placed in the section cp.

$w_{r.cp} := w_{I_{i_{r.cp}}} = 0.79\,m$ — Width of the element at section cp

$n_{max.bars.plate.b} := \mathrm{floor}\left(\dfrac{w_{r.cp} - 2 \cdot c_{cover.b} - 2 \cdot \phi_b}{\phi_b + c_{bar.b}}\right) = 10$ — Number of bars that fits into one layer in the bottom plate

$n_{max.layer.plate.b} := \mathrm{floor}\left(\dfrac{h_{plate} - 2 \cdot c_{cover.b} - 2 \cdot \phi_b}{\phi_b + c_{bar.b}}\right) = 4$ — Number of layers that fits into the bottom plate in the height direction.

$n_{max.bars.leg.b} := \mathrm{floor}\left(\dfrac{b_{web} - 2 \cdot c_{cover.b} - 2 \cdot \phi_b}{\phi_b + c_{bar.b}}\right) = 3$ — Number of bars that fits into one layer in the leg

Number of bars needed

$n_{bars.b} := 34$ — Number of bars needed in the section, iteratively changed according to calculations below. In order to achieve the correct centre of gravity for the bars

$n_{bars.plate.b} := \min\left(n_{bars.b}, n_{max.layer.plate.b} \cdot n_{max.bars.plate.b}\right) = 34$ — Number of bars in the bottom plate

$n_{bars.leg.b} := \max\left(n_{bars.b} - n_{bars.plate.b}, 0\right) = 0$ — Number of bars in the leg of the element

$$n_{layer.b} := \begin{cases} \text{ceil}\left(\dfrac{n_{bars.b}}{n_{max.bars.plate.b}}\right) & \text{if } n_{bars.b} \le n_{max.layer.plate.b} \cdot n_{max.bars.plate.b} \\ \text{ceil}\left(n_{max.layer.plate.b} + \dfrac{n_{bars.b} - n_{max.layer.plate.b} \cdot n_{max.bars.plate.b}}{n_{max.bars.leg.b}}\right) & \text{otherwise} \end{cases}$$

$n_{layer.b} = 4$ Total number of layers

Centre of gravity of the bars

Centre of gravity of the different layers in the bottom flange, different equations are valid for different types of layers. All equations are shown below, and in the and a if-loop will get the correct value for the actual number of bars and layers. me of the equations are never used, depending on number of bars and layers.

$$tp_{layer.1} := \frac{n_{bars.b} \cdot \left(c_{cover.b} + \dfrac{\phi_b}{2}\right)}{n_{bars.b}} = 0.048\,m$$

$$tp_{layer.2} := \frac{n_{max.bars.plate.b} \cdot \left(c_{cover.b} + \dfrac{\phi_b}{2}\right) \dots + \left(n_{bars.b} - n_{max.bars.plate.b}\right) \cdot \left(c_{cover.b} + \dfrac{3}{2} \cdot \phi_b + c_{bar.b}\right)}{n_{bars.b}} = 0.091\,m$$

$$tp_{layer.3} := \frac{n_{max.bars.plate.b} \cdot \left(2 \cdot c_{cover.b} + \dfrac{4 \cdot \phi_b}{2} + c_{bar.b}\right) \dots + \left(n_{bars.b} - 2 \cdot n_{max.bars.plate.b}\right) \cdot \left(c_{cover.b} + \dfrac{5}{2} \cdot \phi_b + 2 \cdot c_{bar.b}\right)}{n_{bars.b}} = 0.117\,m$$

$$tp_{layer.4} := \frac{n_{max.bars.plate.b} \cdot \left(3 \cdot c_{cover.b} + \dfrac{9 \cdot \phi_b}{2} + 3 \cdot c_{bar.b}\right) \dots + \left(n_{bars.b} - 3 \cdot n_{max.bars.plate.b}\right) \cdot \left(c_{cover.b} + \dfrac{7}{2} \cdot \phi_b + 3 \cdot c_{bar.b}\right)}{n_{bars.b}} = 0.124\,m$$

$$tp_{layer.5} := \frac{n_{max.bars.plate.b} \cdot \left(4 \cdot c_{cover.b} + \dfrac{16 \cdot \phi_b}{2} + 6 \cdot c_{bar.b}\right) \dots + \left(n_{bars.b} - 4 \cdot n_{max.bars.plate.b}\right) \cdot \left(c_{cover.b} + \dfrac{9}{2} \cdot \phi_b + 4 \cdot c_{bar.b}\right)}{n_{bars.b}} = 0.113\,m$$

Centre of gravity of all the bars, defined from the bottom of the foundation:

$$y_{tp.b} := \begin{vmatrix} tp_{layer.1} & \text{if } n_{layer.b} = 1 \\ tp_{layer.2} & \text{if } n_{layer.b} = 2 \\ tp_{layer.3} & \text{if } n_{layer.b} = 3 \\ tp_{layer.4} & \text{if } n_{layer.b} = 4 \\ tp_{layer.5} & \text{if } n_{layer.b} = 5 \end{vmatrix}$$

$y_{tp.b} = 0.124\,m$

$d_{b_i} := \Delta h_{I_i} - y_{tp.b} = ...$ Effective depth, from top of the section to the tensile resultant. Along vector i.

$d_{b.J_j} := d_{b_{j+i_{r.fc}}} = ...$ Effective depth, from top of the section to the tensile resultant. Along vector j.

$d_{b.r.cp} := d_{b_{i_{r.cp}}} = 2.276\,m$ Effective depth, from top of the section to the tensile resultant, at section cp.

The need for bottom bending reinforcement

$A_{s.b} := \dfrac{M_{p.1.max}}{0.9 \cdot d_{b.r.cp} \cdot f_{yd}} = 0.016\,m^2$ Needed amount of reinforcement in the bottom in section cp, calculated for the maximum moment.

$n_{bars.b} := \dfrac{A_{s.b}}{A_{si.b}} = 33.234$ Number of bars needed in section cp, based on the maximum moment in fc.

$n_{bars.b} := \mathrm{ceil}(\max(n_{bars.b})) = 34$ Number of bars needed, rounded value

$A_{Rd.b} := A_{si.b} \cdot n_{bars.b} = 1.669 \times 10^4 \cdot mm^2$ Actual amount of reinforcement with the dimension of the bars given as indata above.

$M_{p.Rd} := 0.9 \cdot d_{b.J_0} \cdot f_{yd} \cdot A_{Rd.b} = 1.486 \times 10^4 \cdot kN \cdot m$ Bending moment capacity for the calculated number of bars

$\dfrac{M_{p.1.max}}{M_{p.Rd}} = 0.977$ Ok! Utilisation

Needed anchorage length for the bottom reinforcement

The calculation of needed anchorage lenght is done according to SS-EN 1992-1-1:2005 section 8.4

$\eta_1 := 1.0$ Coefficient related to the quality of the bond condition. =1.0 for "good" conditions, =0.7 for all other conditions. 1.0 is chosen due to the criteria d in EC

$$\eta_2 := \begin{vmatrix} 1.0 & \text{if } \phi_b \le 32mm \\ \dfrac{\left(132 - \dfrac{\phi_b}{mm}\right)}{100} & \text{otherwise} \end{vmatrix} = 1$$

Coefficient related to the bar diameter

$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 4.35 \cdot MPa$ Design value of the ultimate bond stress

The calculation of the required anchorage length is base on EC 8.4.3 (2), using the yield stress of the steel bars, σ_{sd} is set to the yield stress f_{yd}. This is an assumption on the safe side, since the bars are not fully utilised in all sections of the element.

$$l_{b.rqd.b} := \left(\frac{\phi_b}{4}\right)\cdot\left(\frac{f_{yd}}{f_{bd}}\right) = 0.625\,m$$ Basic required anchorage length.

Coefficient α_1 - α_5 is given in table 8.2

$\alpha_1 := 1.0$ Effect of the form of the bars, assuming adequate cover

$$a := \frac{w_{r.cp} - 2\cdot c_{cover.b}}{n_{max.bars.plate.b} - 1} = 0.08\,m$$ Distance between bars in the critical section

$$c_d := \min\left(\frac{a}{2}, c_{cover.b}\right) = 0.035\,m$$

Effect of concrete minimum cover

$$\alpha_2 := \begin{cases} 0.7 & \text{if } 1 - 0.15\cdot\frac{(c_{cover.b} - \phi_b)}{\phi_b} \le 0.7 \\ 1 - 0.15\cdot\frac{(c_d - \phi_b)}{\phi_b} & \text{if } 0.7 < 1 - 0.15\cdot\frac{(c_{cover.b} - \phi_b)}{\phi_b} < 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.94$$

α_3 and α_5 are assumed to be 1.0, which is on the safe sic

$\alpha_3 := 1.0$ Effect of confinement by transverse reinforcement, set to 1.0 (safe side)

$\alpha_5 := 1.0$ Effect of pressure transverse to the plane of splitting along the design anchorage length, set to1.0 (safe side).

$\alpha_4 := 0.7$ Influence of welded transverse bars, set to 0.7 according to EC

$\text{Check} := \alpha_2\cdot\alpha_3\cdot\alpha_5 > 0 = 1$ Check ok

$$l_{b.min.b} := \max(0.3\cdot l_{b.rqd.b}, 10\cdot\phi_b, 100mm) = 0.25\,m$$ Minimum anchorage length for anchorage in tension.

$$l_{bd.bottom} := \max(\alpha_1\cdot\alpha_2\cdot\alpha_3\cdot\alpha_4\cdot\alpha_5\cdot l_{b.rqd.b}, l_{b.min.b}) = 0.411\,m$$ Design anchorage length

Bending reinforcement in the top

The cross-section is turned upside down in order to calculate the reinforcement amount
Chosen dimension of the bars

$\phi_t := 25mm$ Diameter of the bars for the bottom reinforcement

$$A_{si.t} := \frac{\phi_t^{\,2}\cdot\pi}{4} = 4.909\times 10^{-4}\,m^2$$ Area of each reinforcement bar

Indata from Eurocode 2 SS-EN 1992-1-1:2005 8.2 (2)

$k_1 := 1 \qquad k_2 := 5mm$ — Factors from EC2

$d_g := 32mm$ — Assumed size of aggregates

$c_{bar.t} := \max(d_g + k_2, 20mm, k_1 \cdot \phi_t) = 0.037\,m$ — Distance between bars and layers

$\Delta c_{dev} := 10mm$ — Factor from EC2

$c_{min.t} := \phi_t + \Delta c_{dev} = 0.035\,m$ — Minimum concrete cover

$c_{cover.t} := c_{min.t}$ — Chosen concrete cover

Maximum number of bars that fits in the element

The top reinforcement is only placed in the leg

$$n_{max.bars.leg.t} := \mathrm{floor}\left(\frac{b_{web} - 2 \cdot c_{cover.t} + c_{bar.t}}{\phi_t + c_{bar.t}}\right) = 5$$

Number of bars that fits into one layer in the leg

Number of bars needed

$n_{bars.t} := 6$

Number of bars needed in the section, iteratively changed according to calculations below. In order to achieve the correct centre of gravity for the bars

$$n_{layer.t} := \mathrm{ceil}\left(\frac{n_{bars.t}}{n_{max.bars.leg.t}}\right) = 2$$

Total number of layers needed

Centre of gravity for the bars

Centre of gravity of all the bars, defined from the bottom of the foundation

$$y_{tp.t} := \begin{cases} \left(c_{cover.t} + \dfrac{\phi_t}{2}\right) & \text{if } n_{layer.t} \le 1 \\[2ex] \dfrac{n_{max.bars.leg.t} \cdot \left(c_{cover.t} + \dfrac{\phi_t}{2}\right) + (n_{bars.t} - n_{max.bars.leg.t}) \cdot \left(c_{cover.t} + \dfrac{3}{2} \cdot \phi_t + c_{bar.t}\right)}{n_{bars.t}} & \text{if } 1 < n_{layer.t} \\[2ex] \dfrac{n_{max.bars.leg.t} \cdot \left(c_{cover.t} + \dfrac{\phi_t}{2}\right) + (n_{max.bars.leg.t}) \cdot \left(c_{cover.t} + \dfrac{3}{2} \cdot \phi_t + c_{bar.t}\right) \cdots + \left[(n_{bars.t} - 2 \cdot n_{max.bars.leg.t}) \cdot \left(c_{cover.t} + \dfrac{5}{2} \cdot \phi_t + 2c_{bar.t}\right)\right]}{n_{bars.t}} & \text{if } 2 < n_{layer.t} \\[2ex] \dfrac{n_{max.bars.leg.t} \cdot \left(c_{cover.t} + \dfrac{\phi_t}{2}\right) + (n_{max.bars.leg.t}) \cdot \left(2c_{cover.t} + \dfrac{8}{2} \cdot \phi_t + 3c_{bar.t}\right) \cdots + \left[(n_{bars.t} - 3 \cdot n_{max.bars.leg.t}) \cdot \left(c_{cover.t} + \dfrac{7}{2} \cdot \phi_t + 3c_{bar.t}\right)\right]}{n_{bars.t}} & \text{if } 3 < n_{layer.t} \le 4 \end{cases}$$

$y_{tp.t} = 0.058\,m$ — Centre of gravity for the bars

$d_{t_i} := \Delta h_{I_i} - y_{tp.t}$ — Effective depth, from top of the section to the tensile resultant. Along vector i.

$d_{t.J_j} := d_{t_{j+i_{r.fc}}}$ — Effective depth, from top of the section to the tensile resultant. Along vector j.

$d_{t.r.cp} := d_{t_{i_{r.cp}}} = 2.342\,m$ — Effective depth, from top of the section to the tensile resultant, at section cp.

The need for top bending reinforcement

$A_{s.t} := \dfrac{M_{n.1.max}}{0.9 \cdot d_{t.r.cp} \cdot f_{yd}}$ — Needed amount of reinforcement in the bottom in section cp, calculated for the maximum moment.

$n_{bars.t} := \dfrac{A_{s.t}}{A_{si.t}} = 5.356$ — Number of bars needed in section cp, based on the maximum moment in fc.

$n_{bars.t} := \mathrm{ceil}\left(\max\left(n_{bars.t}\right)\right) = 6$ — Number of bars needed, rounded value

$A_{Rd.t} := A_{si.t} \cdot n_{bars.t} = 2.945 \times 10^{3} \cdot mm^{2}$ — Actual amount of reinforcement with the dimension of the bars given as indata above.

$M_{p.Rd} := 0.9 \cdot d_{t.r.cp} \cdot f_{yd} \cdot A_{Rd.b} = 1.53 \times 10^{4} \cdot kN \cdot m$ — Bending moment capacity for the calculated number of bars

$\dfrac{M_{p.1.max}}{M_{p.Rd}} = 0.95$ Ok! — Utilisation

Needed anchorage length for the top reinforcement

The calculation of needed anchorage lenght is done according to SS-EN 1992-1-1:2005 section 8.4

$\eta_1 := 1.0$ — Coefficient related to the quality of the bond condition. =1.0 for "good" conditions, =0.7 for all other conditions. 1.0 is chosen due to the criteria d in EC

$$\eta_2 := \begin{cases} 1.0 & \text{if } \phi_t \le 32mm \\ \dfrac{\left(132 - \dfrac{\phi_t}{mm}\right)}{100} & \text{otherwise} \end{cases} = 1$$

Coefficient related to the bar diameter

$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 4.35 \cdot MPa$ — Design value of the ultimate bond stress

The calculation of the required anchorage length is base on EC 8.4.3 (2), using the yield stress of the steel bars, σ_{sd} is set to the yield stress f_{yd}. This is an assumption on the safe side, since the bars are not fully utilised in all sections of the element.

$l_{b.rqd} := \left(\dfrac{\phi_t}{4}\right) \cdot \left(\dfrac{f_{yd}}{f_{bd}}\right) = 0.625\,m$ — Basic required anchorage length.

Coefficient α_1 - α_5 is given in table 8.2

$\alpha_1 := 1.0$ — Effect of the form of the bars, assuming adequate cover, according to EC

$a := \dfrac{b_{web} - 2 \cdot c_{cover.t}}{n_{max.bars.leg.t} - 1} = 0.07\,m$ — Distance between bars in the critical section

$c_d := \min\left(\dfrac{a}{2}, c_{cover.t}\right) = 0.035\,m$

$$\alpha_2 := \begin{vmatrix} 0.7 \ \text{ if } \ 1 - 0.15 \cdot \dfrac{(c_{cover.t} - \phi_t)}{\phi_t} \le 0.7 \\ 1 - 0.15 \cdot \dfrac{(c_d - \phi_t)}{\phi_t} \ \text{ if } \ 0.7 < 1 - 0.15 \cdot \dfrac{(c_{cover.t} - \phi_t)}{\phi_t} < 1.0 \\ 1.0 \ \text{ otherwise} \end{vmatrix} = 0.94$$

Effect of concrete minimum cover, according to EC

α_3 and α_5 are assumed to be 1.0, which is on the safe side

$\alpha_3 := 1.0$ — Effect of confinement by transverse reinforcement, set to 1.0 (safe side)

$\alpha_5 := 1.0$ — Effect of pressure transverse to the plane of splitting along the design anchorage length, set to1.0 (safe side).

$\alpha_4 := 0.7$ — Influence of welded transverse bars, set to 0.7 according to EC

$\text{Check} := \alpha_2 \cdot \alpha_3 \cdot \alpha_5 > 0 = 1$ — Check is ok

$l_{b.min} := \max\left(0.3 \cdot l_{b.rqd}, 10 \cdot \phi_t, 100mm\right) = 0.25\,m$ — Minimum anchorage length for anchorage in tension.

$l_{bd.top} := \max\left(\alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd}, l_{b.min}\right) = 0.411\,m$ — Design anchorage length

Transversal bending reinforcement in the flanges

Loads and moments in the bottom flange - only the self-weight of the plate and fill is considered

Load acting on the flange, calculated per meter at the worst place (outermost part).
Loads are: weight of the fill, self-weight of the slab

$q_{d.plate} := \left(h_{plate} \cdot \rho_c + h_{fill.plate} \cdot \rho_{fill}\right) 1m = 39.028 \cdot \dfrac{kN}{m}$

$M_{support} := q_{d.plate} \cdot \dfrac{l_{span}^2}{12} = 13.788 \cdot kN \cdot m$ — Support moment

$M_{field} := -q_{d.plate} \cdot \dfrac{l_{span}^2}{24} = -6.894 \cdot kN \cdot m$ — Field moment

$M_{Ed} := \max\left(\left|M_{support}\right|, \left|M_{field}\right|\right) = 13.788 \cdot kN \cdot m$ — Maximum moment

Reinforcement amount in the bottom flange

$\phi_{bar.plate} := 16mm$ Assumed bar diameter

Assume one layer of reinforcement

$d_{plate} := h_{plate} - c_{cover.b} - \frac{\phi_{bar.plate}}{2} = 0.357\,m$ Distance from top of the flange to the reinforcement

$A_{s.plate} := \frac{M_{Ed}}{f_{yd} \cdot 0.9 \cdot d_{plate}} = 9.87 \times 10^{-5}\,m^2$ Needed amount of reinforcement

$A_{si.plate} := \phi_{bar.plate}^{2} \cdot \frac{\pi}{4} = 2.011 \times 10^{-4}\,m^2$ Area of one bar

$n_{bar.plate} := \frac{A_{s.plate}}{A_{si.plate}} = 0.491$ Number of bars needed per meter

Loads and moments in the bottom flange - the self-weight of fill and plate and the soil pressure acting on the plate is considered

Load acting on the flange, calculated per meter at the worst place (outermost part). Loads are: weight of the fill, self-weight of the slab and the soil pressure action on the plate

$$q_{d.plate.soil} := -\left(h_{plate} \cdot \rho_c + h_{fill.plate} \cdot \rho_{fill}\right) 1\text{m} + \sigma_{soil} \cdot \frac{l_{span}}{2} = 350.607 \cdot \frac{\text{kN}}{\text{m}}$$

$$M_{support} := q_{d.plate.soil} \cdot \frac{l_{span}^{\ 2}}{12} = 123.866 \cdot \text{kN} \cdot \text{m}$$ Support moment

$$M_{field} := -q_{d.plate.soil} \cdot \frac{l_{span}^{\ 2}}{24} = -61.933 \cdot \text{kN} \cdot \text{m}$$ Field moment

$$M_{Ed} := \max\left(\left|M_{support}\right|, \left|M_{field}\right|\right) = 123.866 \cdot \text{kN} \cdot \text{m}$$ Maximum moment

Reinforcement amount in the bottom flange

$$\phi_{bar.plate} := 16\text{mm}$$ Assumed bar diameter

Assume one layer of reinforcement

$$d_{plate} := h_{plate} - c_{cover.b} - \frac{\phi_{bar.plate}}{2} = 0.357\,\text{m}$$ Distance from top of the flange to the reinforcement

$$A_{s.plate} := \frac{M_{Ed}}{f_{yd} \cdot 0.9 \cdot d_{plate}} = 8.867 \times 10^{-4}\,\text{m}^2$$ Needed amount of reinforcement

$$\phi_{bar.plate} := 16\text{mm}$$ Assumed bar diameter

$$A_{si.plate} := \phi_{bar.plate}^{\ 2} \cdot \frac{\pi}{4} = 2.011 \times 10^{-4}\,\text{m}^2$$ Area of one bar

$$n_{bar.plate} := \frac{A_{s.plate}}{A_{si.plate}} = 4.41$$ Number of bars needed per meter

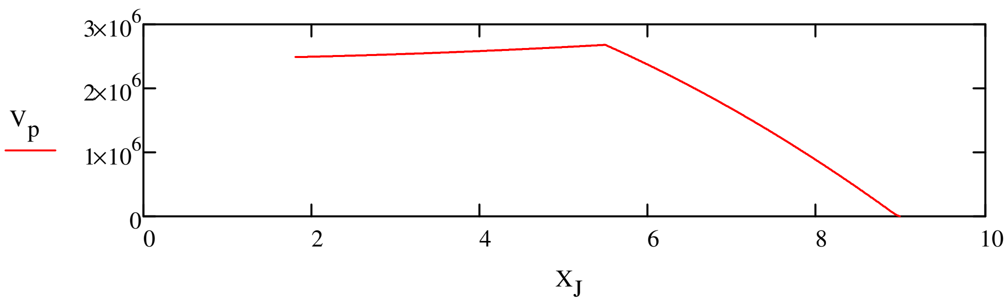
6. Shear force distribution in the legs

The shear force distribution is calculated in the same manner as for the moment distribution. Summing up the shear force acting on each strip, into a shear force distribution vector V_p. Only the shear force distribution in element 1 is calculated since this value is used for dimensioning the reinforcement, since it has the largest magnitude. The shear force distribution is calculated for both the leeward and the windward side.

Shear force for the leeward side

Shear force distribution from the section fc to the edge of the foundation

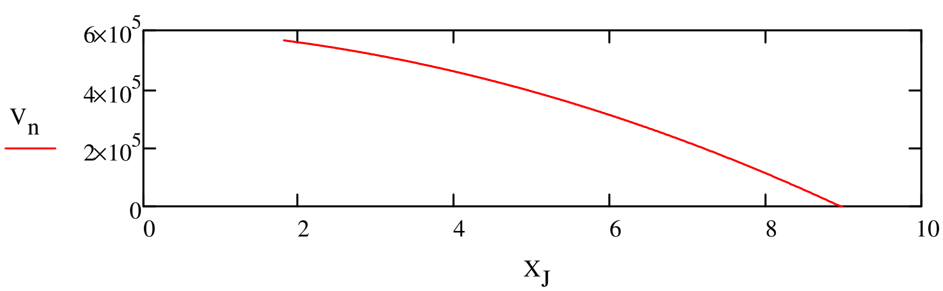
$$V_p := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad V_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left(N_{res.1_i}\right) \\ V \end{array} \right.$$



Shear force for the windward side

Shear force distribution from the section fc to the edge of the foundation

$$V_n := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad V_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left(N_{dead.n_i}\right) \\ V \end{array} \right.$$



Dimensioning shear force in the critical sections

Dimensioning shear force at the section r_{fc} , where the force couple is acting

$$V_{Ed.r.fc} := \max\left(V_{p_0}, V_{n_0}\right) = 2.493\times 10^3\cdot kN$$

Dimensioning shear force at l_{sp} , where the soil pressure zone starts

$$V_{Ed.l.sp} := V_{p\left(i_{soil.1}-i_{r.fc}\right)} = 2.684\times 10^3\cdot kN$$

Shear force reinforcement, in cut fc

$$V_{Ed.r.fc} = 2.493\times 10^6 N$$

Required capacity of the shear force in section r_{fc}

Check if the shear capacity is sufficient without shear reinforcement

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.2
Calculations are done in order to investigate if shear reinforcement is needed in section r_{fc}

$$d_{r.fc} := d_{b_{i_{r.fc}}} = 2.276 m$$

Distance from the top to the c.g of the bottom bending reinforcement

$$z_{r.fc} := 0.9\cdot d_{r.fc} = 2.048 m$$

Internal lever arm

$$C_{Rd.c} := \frac{0.18}{\gamma_c} = 0.12$$

National parameter, recommended value

$$k := \min\left(1 + \sqrt{\frac{200}{\frac{d_{r.fc}}{mm}}}, 2\right) = 1.296$$

$$A_{sl} := A_{Rd.b} = 0.017 m^2$$

Cross-sectional area of fully anchored main reinforcement in tensile zone

$$b_w := b_{web}$$

Least cross-sectional width in side tensile part of cross-section.

$$\rho_l := \min\left(\frac{A_{sl}}{b_w\cdot d_{r.fc}}, 0.02\right) = 0.02$$

$$\nu_{min} := 0.035k^{\frac{3}{2}}\cdot\sqrt{\frac{f_{ck}}{MPa}}\cdot MPa = 2.83\times 10^5 Pa$$

National parameter, recommended expression

Shear capacity without shear reinforcement

$$V_{Rd.r.fc} := \max\left[C_{Rd.c}\cdot k\cdot\left(100\rho_l\cdot\frac{f_{ck}}{MPa}\right)^{\frac{1}{3}}\cdot b_w\cdot d_{r.fc}\cdot MPa, \nu_{min}\cdot b_w\cdot d_{r.fc}\right] = 485.148\cdot kN$$

$$\frac{V_{Ed.r.fc}}{V_{Rd.r.fc}} = 5.139$$

Utilisation

Shear reinforcement is needed! Therefore further calculations must be performed in order to calculate the required reinforcement area.

Shear force capacity

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.3
The required shear force capacity is used to calculated the required reinforcement area.

$\theta := 45deg$ — Choice of shear angle. The angle must be 21.8 deg < θ < 45 deg.

$s_{r.fc} := 1m$ — Distance between stirrups in the direction of the webs

$$A_{sw.r.fc} := \frac{V_{Ed.r.fc} \cdot s_{r.fc}}{z_{r.fc} \cdot f_{yd} \cdot \cot(\theta)} = 2.799 \times 10^{3} \cdot mm^{2}$$

Required shear reinforcement area

For members with vertical shear reinforcement, the maximum shear resistance is calculated

$\alpha_{cw} := 1.0$ — National parameter, no prestressing

$b_w := b_{web}$ — Least cross-sectional width between compressive and tensile part of cross-section.

$$\nu_1 := 0.6 \cdot \left(1 - \frac{f_{ck}}{250MPa}\right) = 0.528$$

Reduction factor for the compressive strength of the concrete, national parameter

Shear capacity

$$V_{Rd.r.fc} := \alpha_{cw} \cdot b_w \cdot z_{r.fc} \cdot \nu_1 \cdot f_{cd} \cdot \frac{1}{\tan(\theta) + \cot(\theta)} = 3.785 \times 10^{3} \cdot kN$$

$$\frac{V_{Ed.r.fc}}{V_{Rd.r.fc}} = 0.659$$

Utilisation

The utilisation of the shear force capacity ok, this means that the calculated shear reinforcement is sufficient in ULS in section r_{fc}

Shear force reinforcement, in cut l_{sp}

Shear force reinforcement is also designed for the section where the soil pressure zone starts, section l_{sp} , see figure below for the definition of the section.

$V_{Ed.l.sp} = 2.684 \times 10^{6} N$ — Required capacity of the shear force in section sp

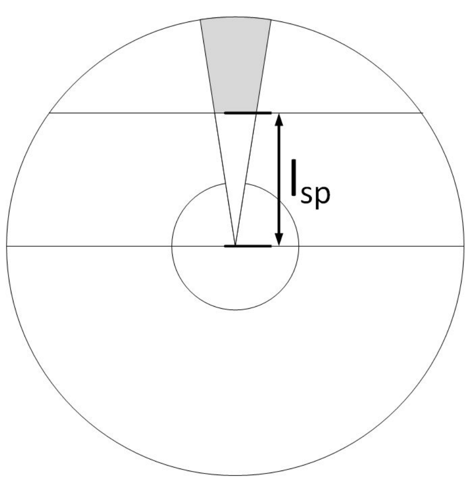


Figure 15: Definition of the section l_{sp}

Check if the shear capacity is sufficient without shear reinforcement

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.2
Calculations are done in order to investigate if shear reinforcement is needed in section l_{sp}

$d_{l.sp} := d_b(i_{soil.1}) = 2.047\,m$ — Distance from the top to the centre of gravity of the bottom reinforcement

$z_{l.sp} := 0.9 \cdot d_{l.sp} = 1.842\,m$ — Internal lever arm

$C_{Rd.c} := \frac{0.18}{\gamma_c} = 0.12$ — National parameter, recommended value

$$k := \min\left(1 + \sqrt{\frac{200}{\frac{d_{l.sp}}{mm}}}, 2\right) = 1.313$$

$A_{sl} := A_{Rd.b}$ — Cross-sectional area of fully anchored main reinforcement in tensile zone

$b_w := b_{web}$ — Least cross-sectional width in side tensile part of cross-section.

$$\rho_l := \min\left(\frac{A_{Rd.b}}{b_w \cdot d_{l.sp}}, 0.02\right) = 0.02$$

$$\nu_{min} := 0.035k^{\frac{3}{2}} \cdot \sqrt{\frac{f_{ck}}{MPa}} \cdot MPa = 2.883 \times 10^5\,Pa$$

National parameter, recommended expression

Shear capacity without shear reinforcement

$$V_{Rd.l.sp} := \max\left[C_{Rd.c} \cdot k \cdot \left(100\rho_l \cdot \frac{f_{ck}}{MPa}\right)^{\frac{1}{3}} \cdot b_w \cdot d_{l.sp} \cdot MPa, \nu_{min} \cdot b_w \cdot d_{l.sp}\right] = 441.722 \cdot kN$$

$\frac{V_{Ed.l.sp}}{V_{Rd.l.sp}} = 6.077$ — Utilisation

Shear reinforcement is needed! Therefore further calculations must be performed in order to decide the reinforcement area

Shear force capacity

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.3
The required shear force capacity is used to calculated the required reinforcement area.

$\theta := 45deg$ — Choice of shear angle. The angle must be 21.8 deg < θ < 45 deg.

$s_{l.sp} := 1m$ — Distance between stirrups in the direction of the webs

$A_{sw.l.sp} := \frac{V_{Ed.l.sp} \cdot s_{l.sp}}{z_{l.sp} \cdot f_{yd} \cdot \cot(\theta)} = 3.352 \times 10^3 \cdot mm^2$ — Required shear reinforcement area

For members with vertical shear reinforcement, the maximum shear resistance is calculated

$\alpha_{cw} := 1.0$ National parameter, no prestressing

$b_w := b_{web}$ Least cross-sectional width between compressive and tensile part of cross-section.

$\nu_1 := 0.6 \cdot \left(1 - \frac{f_{ck}}{250MPa}\right) = 0.528$ Reduction factor for the compressive strength of the concrete, national parameter

Shear capacity

$$V_{Rd.l.sp} := \alpha_{cw} \cdot b_w \cdot z_{l.sp} \cdot \nu_1 \cdot f_{cd} \cdot \frac{1}{\cot(\theta) + \tan(\theta)} = 3.404 \times 10^3 \cdot kN$$

$\frac{V_{Ed.l.sp}}{V_{Rd.l.sp}} = 0.789$ Utilisation

The utilisation of the shear force capacity is ok. This means that the calculated shear reinforcement is sufficient in ULS

7. Global analysis

Loads acting on the foundation, for the global analysis

In the global analysis of the foundation the whole foundation and all the loads acting on the foundation is considered, see Figure below

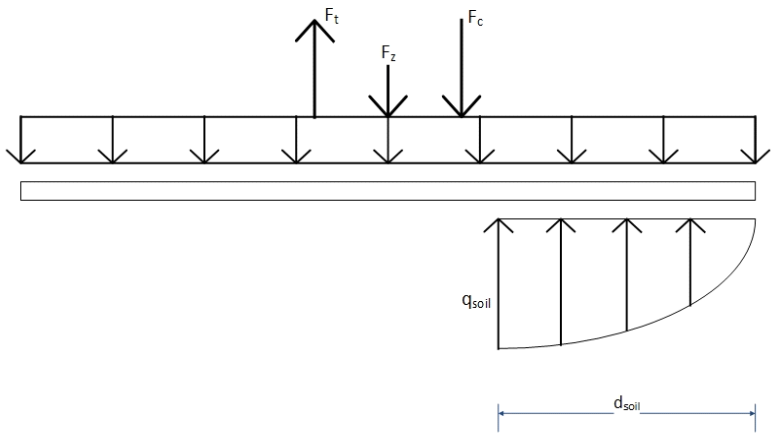


Figure 16: Loads acting on the foundation

Loads from the tower

Transformation of the moment into a force couple; The bending moment is transformed into one compressive force and one tensile force. The attack point is calculated to be acting in the c.g of the arc of the tower.

The normal force from the tower is divided in four parts, one part is applied at each component of the force couple. Two parts of if, ie half of the normal force, is applied in the middle of the beam.

$$F_c := \frac{M_d + H_d \cdot h_{centrepiece}}{\phi_{fc}} + \frac{\gamma_{dead} N_k}{4} = 28.809 \cdot MN$$

Compressive component of the force couple and a quarter of the normal force

$$F_t := \frac{M_d + H_d \cdot h_{centrepiece}}{\phi_{fc}} - \frac{\gamma_{dead} N_k}{4} = 27.189 \cdot MN$$

Tensile component of the force couple and a quarter of the normal force

$$F_z := \frac{\gamma_{dead} N_k}{2} = 1.62 \cdot MN$$

Half of the normal force

The bending moment in the foundation is now calculated for the whole foundation, globally, in order to find the global moment distribution. For the global moment distribution the whole foundation is included in the analysis.

Self-weight

$$N_{dead.tot.p_j} := \frac{\gamma_{dead} G_k}{\frac{l_{tot}}{\Delta x}} = ...$$

A vector with the dead load acting on each strip, when calculating the positive moment. The dead load is uniformly spread over the lenght of the foundation.

$$N_{dead.tot.n_j} := \frac{\gamma_{abn} G_k}{\frac{l_{tot}}{\Delta x}}$$

A vector with the dead load acting on each strip, when calculate the negative moment. The dead load is uniformly spread over the lenght of the foundation.

Soil pressure

Vectors with the total soil pressure for each strip. The soil pressure is zero outside the soil pressure area.

$$\sigma_{soil_j} := \begin{vmatrix} 0 \ \text{ if } \ X_{J_j} < \dfrac{l_{tot}}{2} - l_{soil.1} \\ \sigma_{soil} \ \text{ otherwise} \end{vmatrix}$$

Soil pressure contribution from the secondary legs.

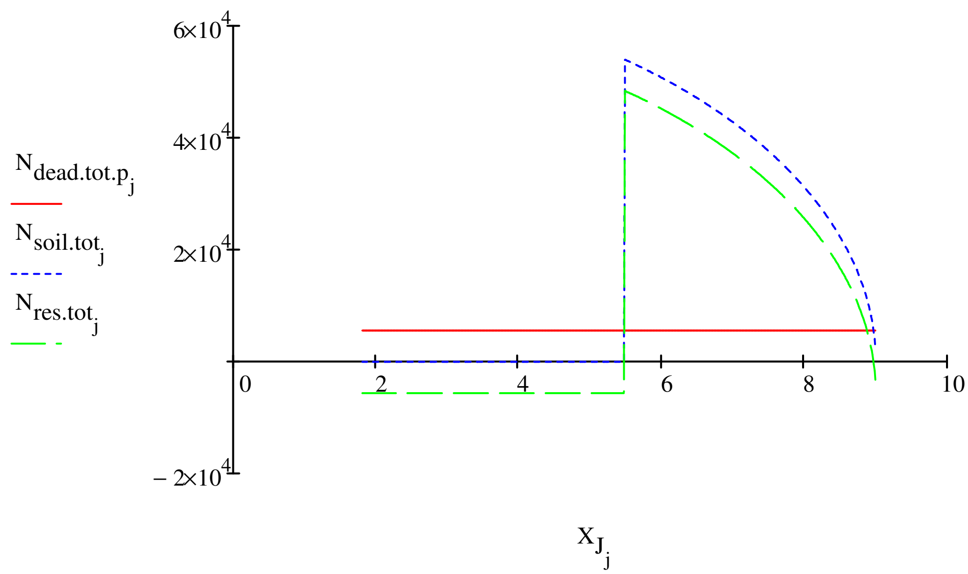
Summary of the loads acting on the foundation, globally

$$N_{soil.tot_j} := \sigma_{soil_j} \cdot w_{tot_j} \cdot \Delta x = ...$$

Vector with ground pressure as a total force acting on each strip along vector j

$$N_{res.tot_j} := N_{soil.tot_j} - N_{dead.tot.p_j} = ...$$

Resulting force vector for each strip along vector j



Moment distribution - globally, from cut fc to the edge

Positive moment distribution, on the leeward side

The moment distribution, M_p is calculated from the cut fc to the end of the foundation. The positive moment is defined on the leeward side
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side, The resulting force, N_{res}, for each strip is multiplied with its lever arm, s_J, for its respective element and then summed up for the all the elements into the positive moment M_p .

Moment distribution from r_{fc} to the edge of the foundation

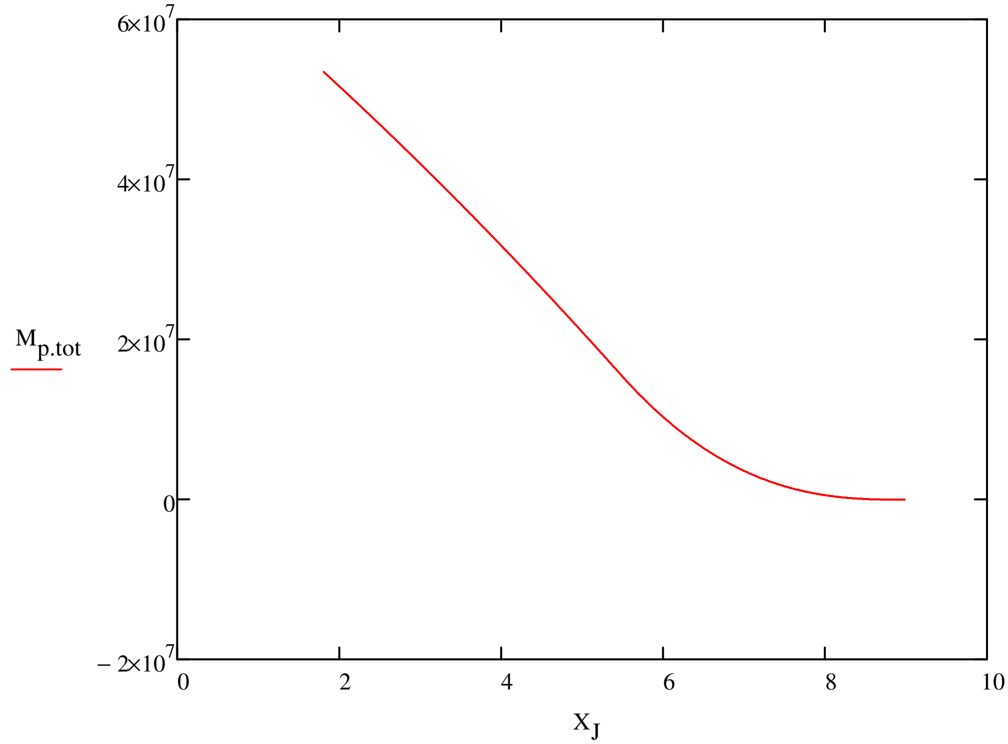
$$M_{p.tot} := \begin{vmatrix} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } \ k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.tot_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{vmatrix}$$

Maximum moment

$M_{p.tot.max} := M_{p.tot_0} = 5.347 \times 10^4 \cdot kN \cdot m$

Moment in critical cut, r_{fc}, gives the maximum moment on the positive side

Moment diagram of the positive moment



Negative distribution, on the windward side

The moment distribution, M_n is calculated from the cut fc to the edge of the foundation. The negative moment is defined on the windward side.
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side,
The resulting force, $N_{dead.tot.n}$, for each element is multiplied with the lever arm, s_J ,for its respective element and then summed up for the all the elements into the negative moment M_n .

Moment distribution from r_{fc} to the edge of the foundation

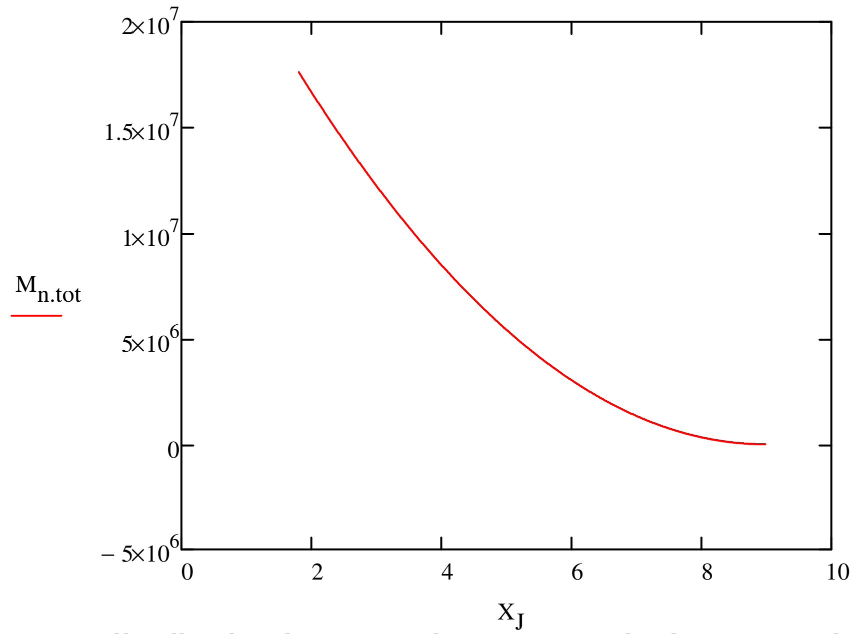
$$M_{n.tot} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{dead.tot.n_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$M_{n.tot.max} := M_{n.tot_0} = 1.763 \times 10^4 \cdot kN \cdot m$

Moment in critical cut, r_{fc}, gives the maximum moment on the negative side

Moment diagram of the negative moment



Moment distribution between the elements in the centrepiece

In an approximated manner it is assumed that the moment distribution is linear between the maximum positive moment and the maximum negative moment. The negative minimum moment and positive maximum moment, are both acting at the distance r.fc from the centre of the foundation. The approximated moment distribution in the centrepiece is plotted.

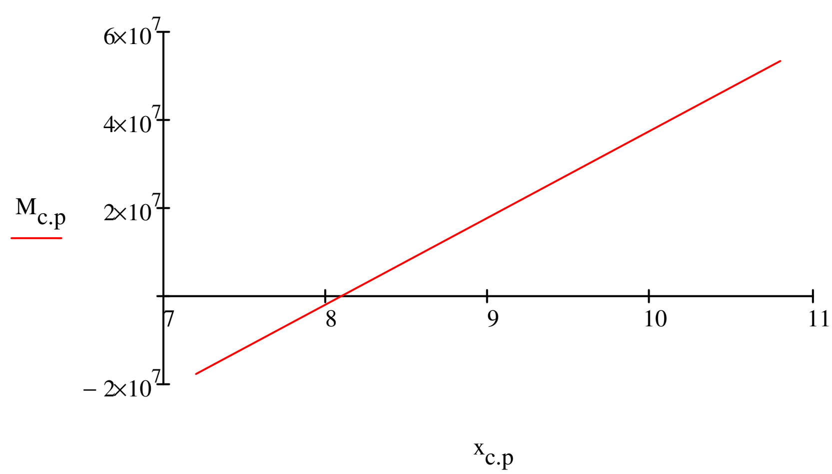
$$M_{c.p} := \begin{pmatrix} -M_{n.tot.max} \\ M_{p.tot.max} \end{pmatrix}$$

The minimum negative moment and the maximum positive moment

$$x_{c.p} := \begin{pmatrix} \dfrac{l_{tot}}{2} - r_{fc} \\ \dfrac{l_{tot}}{2} + r_{fc} \end{pmatrix} = \begin{pmatrix} 7.2 \\ 10.8 \end{pmatrix} m$$

The location of the maximum moment and the negative moment along the foundation

Moment distribution between F_c and F_t , in the centrepiece



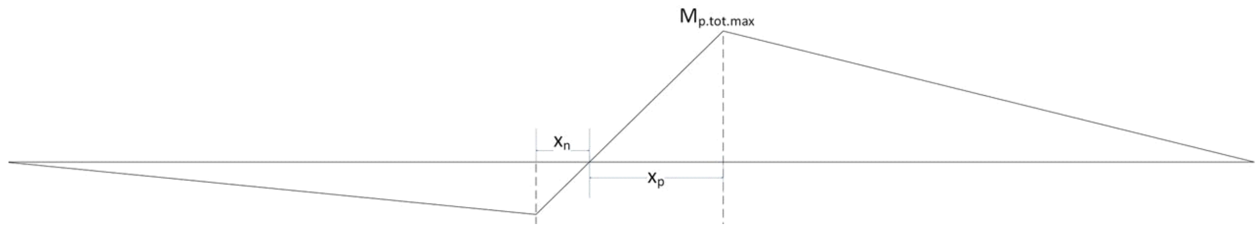


Figure 17: Principle distribution of the moment, with approximate distribution in the centrepiece, between the force resultant.

Distance from the maximum moment and the minimum moment to the zero moment

$$x_p := \frac{M_{p.tot.max}}{M_{p.tot.max} + M_{n.tot.max}} \cdot \phi_{fc} = 2.708\,m$$

Distance from maximum negative moment to zero moment section

$$x_n := \frac{M_{n.tot.max}}{M_{p.tot.max} + M_{n.tot.max}} \cdot \phi_{fc} = 0.893\,m$$

Distance from maximum positive moment to zero moment section

Shear force distribution in the centrepiece

The shear force distribution, conceptually in a wind power plant foundation

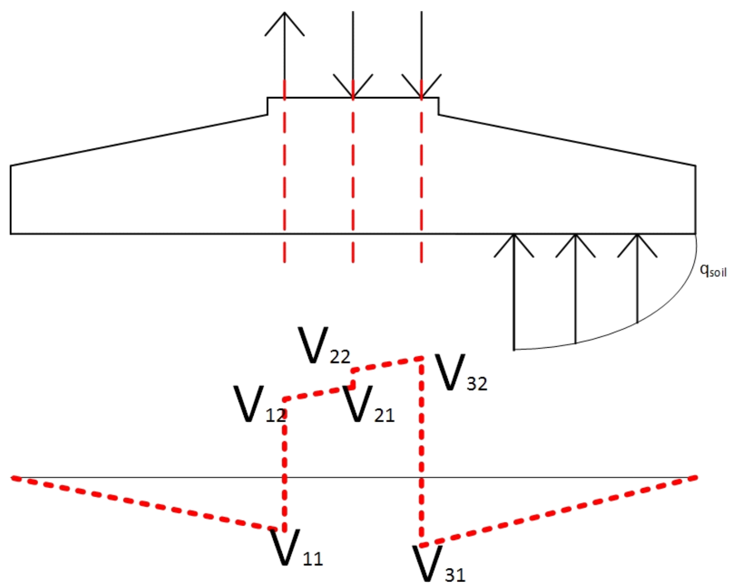


Figure 18: The shear force is critical between the force couple, and therefore this is calculated. It is characterized by the critical points $V_{11}, V_{12}, V_{21}, V_{22}, V_{31}, V_{32}$ *as illustrated.*

Shear forces in the critical points

Calculated for the points defined in the figure above

$$V_{11} := -\sum N_{dead.tot.n} = -4.911 \times 10^{3} \cdot kN$$

V_{11} is calculated by summing the dead load

$$V_{12} := F_t - V_{11} = 3.21 \times 10^{4} \cdot kN$$

$V_{31} := -\sum N_{res.tot} = -9.261\times 10^{3}\cdot kN$

V_{31} is calculated by summing the resultant loads

$V_{32} := F_c - V_{31} = 3.807\times 10^{4}\cdot kN$

$V_{22} := V_{32} + \dfrac{-V_{32} + V_{12} + F_z}{\phi_{fc}}\cdot\dfrac{\phi_{fc}}{2} = 3.589\times 10^{4}\cdot kN$

$V_{21} := V_{22} - F_z = 3.427\times 10^{4}\cdot kN$

For plotting of the shear force

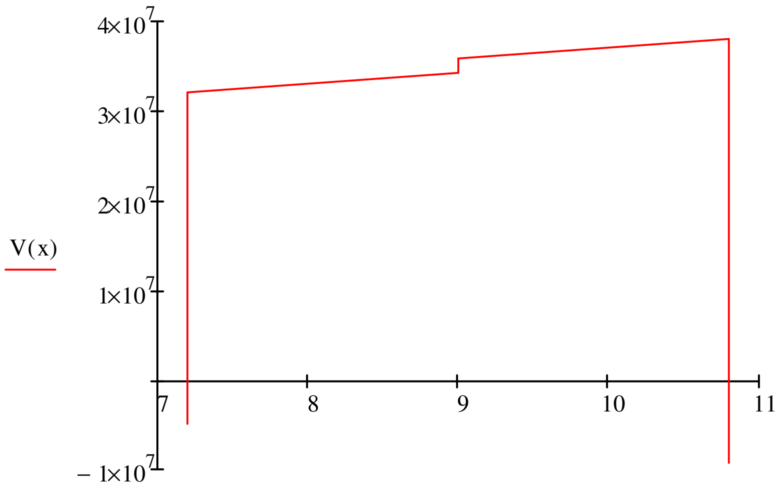
$V(x) := \left(V_{11}\ \ V_{12}\ \ V_{21}\ \ V_{22}\ \ V_{32}\ \ V_{31}\right)^{T}$

Vector with the critical points

$x := \left(\dfrac{l_{tot}-\phi_{fc}}{2}\ \ \dfrac{l_{tot}-\phi_{fc}}{2}\ \ \dfrac{l_{tot}}{2}\ \ \dfrac{l_{tot}}{2}\ \ \dfrac{l_{tot}+\phi_{fc}}{2}\ \ \dfrac{l_{tot}+\phi_{fc}}{2}\right)^{T}$

Vector with the position of the critical points

Shear force diagram in the tower between the force couple



Dimensioning shear force in the critical sections

Dimensioning shear force at the middle of the tower

$V_{Ed.mid} := \max\left(\left|V_{11}\right|, \left|V_{12}\right|, \left|V_{21}\right|, \left|V_{22}\right|, \left|V_{31}\right|, \left|V_{32}\right|\right) = 3.807\times 10^{4}\cdot kN$

Shear force reinforcement, in the middle of the tower

Shear force capacity

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.3.
The required shear force capacity is used to calculated the required reinforcement area.

$V_{Ed.mid} = 3.807\times 10^{7}\,N$

Maximum shear force inside the tower

$v_{Ed.mid} := \dfrac{V_{Ed.mid}}{d_{centrepiece}} = 7.614\times 10^{3}\cdot\dfrac{kN}{m}$

Shear force spread over the width of the centrepiece

$\theta := 45deg$

Choice of shear angle

$d_{mid} := d_{b_{i_{mid}}} = 2.276\,m$

Height of the foundation inside the tower excluding the reinforcement. Not including the extra height of the centrepiece.

$s_{w.mid} := 1$

Distance between stirrups

$A_{sw.mid} := \dfrac{v_{Ed.mid} \cdot s_{w.mid}}{0.9 \cdot d_{mid} \cdot f_{yd} \cdot \cot(\theta)} = 8.549 \times 10^{3} \cdot \dfrac{mm^2}{m^2}$

Needed reinforcement amount per square meter

$\phi_{c.p} := 16mm$

Assumed diameter of shear reinforcement

$A_{si.c.p} := \dfrac{\phi_{c.p}^{\;2} \cdot \pi}{4} = 201.062 \cdot mm^2$

Area of the shear reinforcement

$n_{shear.mid} := \dfrac{A_{sw.mid}}{A_{si.c.p}} = 42.522 \dfrac{1}{m^2}$

Number of shear reinforcement bars per square meter

For members with vertical shear reinforcement, the maximum shear resistance is calculated

$\alpha_{cw.mid} := 1.0$

National parameter, no prestressing

$\nu_{1.mid} := 0.6 \cdot \left(1 - \dfrac{f_{ck}}{250MPa}\right) = 0.528$

Reduction factor for the compressive strength of the concrete, national parameter

Shear force capacity

$V_{Rd.mid} := \alpha_{cw.mid} \cdot 0.9 \cdot d_{mid} \nu_{1.mid} \cdot f_{cd} \cdot \dfrac{1}{\cot(\theta) + \tan(\theta)} = 1.082 \times 10^{4} \cdot \dfrac{kN}{m}$

$\dfrac{v_{Ed.mid}}{V_{Rd.mid}} = 0.704$ Utilisation

The utilisation of the shear force capacity ok, this means that the calculated shear reinforcement is sufficient in ULS in the middle of the section.

Appendix IV
Detailed calculations
Eight legs with bottom flange

Table of contents

1. Indata

Loads from the tower

Design loads assumed for tower SWT-2.3-101 from Siemens with an height of 99.5 m high, loads are including the partial safety factors (except for the self-weight of the tower). The loads are presented in Appendix II.

$M_d := 97700 kN \cdot m$	Design load on top of the foundation; overturning moment
$H_d := 1080 kN$	Design load on top of the foundation; transverse load
$N_k := 3600 kN$	Characteristic load on top of the foundation; dead load

Partial safety factors, wind power plants

$\gamma_{norm} := 1.35$	For normal load cases, this is included in the design loads
$\gamma_{abn} := 1.1$	For abnormal load cases
$\gamma_{dead} := 0.9$	For dead weight
$\gamma_{fat} := 1.0$	For fatigue loading

Geometry of the connection between tower and foundation

$d_{centrepiece} := 5m$	Diameter of the centrepiece
$A_{centrepiece} := \frac{\pi \cdot d_{centrepiece}^2}{4} = 19.635\,m^2$	Area of the centrepiece
$h_{centrepiece} := 2.8m$	Height of the centrepiece, defined from the top to the bottom of the foundation (incl flange)
$V_{centrepiece} := h_{centrepiece} \cdot A_{centrepiece} = 54.978 \cdot m^3$	Volume of the centrepiece
$d_r := 4m$	Diameter of the anchor ring, see figure below
$r_r := \frac{d_r}{2}$	Radius of the anchor ring
$\Delta l := \frac{d_{centrepiece} - d_r}{2} = 0.5\,m$	Distance between the centre of gravity of anchor ring and the outer part of the centrepiece

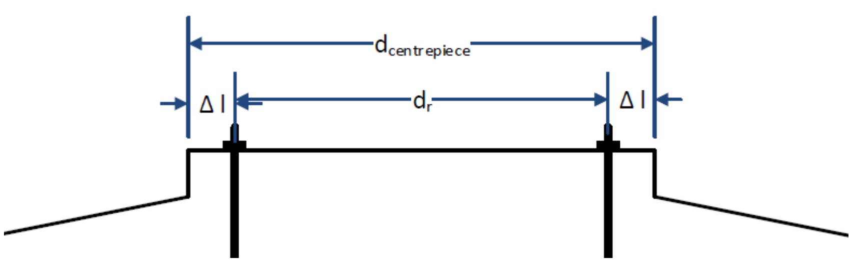


Figure 1: Definition of the dimensions of the centrepiece.

The applied overturning moment can be described by a force couple F_c and F_t. The force couple acts along the bolt basket, with a stress distribution according to figure:

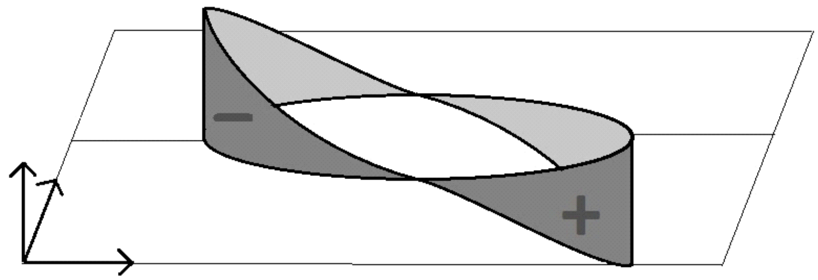


Figure 2: Stress distribution along the prestressing bolt basket

The stress distribution is simplified to be uniform along two quarters of the bolt basket, see figure, and it is assumed that the force couple can be considered to act in the centre of gravity of the arc of these two circle quarters (according to Landén & Lilljegren 2012, eq. 5.3)

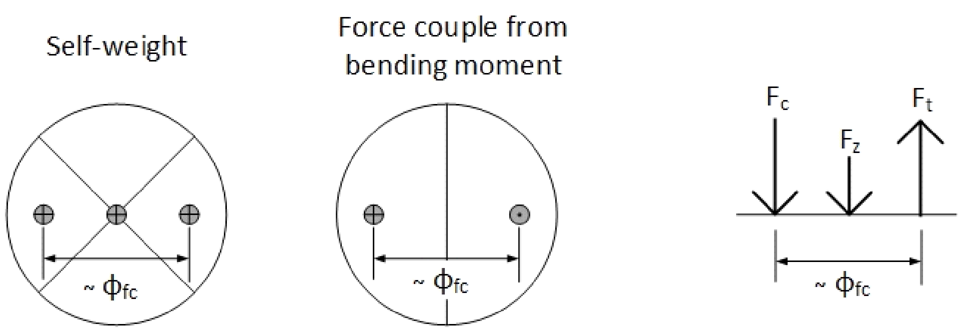


Figure 3: Illustration of how the resultants of the tensile and compressive parts of the force couple is offseted towards the centre of the foundation, and not acting in the position of the anchorage ring

The means that the attack point of the force couple will be calculated according to:

$$\phi_{fc} := 2\cdot\left(\frac{2}{\pi\cdot r_r}\right)\cdot\int_{\frac{-\pi}{4}}^{\frac{\pi}{4}} \left(r_r\right)^2\cdot\cos(\varphi)\,d\varphi$$

Distance between the force couple resultants F_c and F_t calculated as the centre of gravity of the arc of the two quarters.

$$r_{fc} := \text{round}\left(\frac{\phi_{fc}}{2\cdot m}, 2\right)\cdot m = 1.8\,m$$

Distance from the center of the foundation to the force resultants F_c and $F_{t,}$ from the applied overturning moment.

Material parameters

Concrete C30/37

$\rho_c := 25\frac{kN}{m^3}$ Weight of the concrete

$\gamma_c := 1.5$ Concrete safety factor

$f_{ck} := 30MPa$ Characteristic strength of concrete

$f_{cd} := \frac{f_{ck}}{\gamma_c} = 20\cdot MPa$ Design compressive strength of the concrete

$f_{ctm} := 2.9MPa$ Mean tensile strength of concrete

$f_{ctd} := \frac{f_{ctm}}{\gamma_c} = 1.933 \cdot MPa$ Design tensile strength of concrete

$E_c := 33GPa$ Elastic modulus of the concrete

Reinforcement B500B

$f_{yk} := 500MPa$ Characteristic yield strength of the reinforcement

$\gamma_s := 1.15$ Reinforcement steel partial safety factor

$f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783 \cdot MPa$ Dimensioning yield strength of the reinforcement

$E_s := 200GPa$ Elastic modulus of steel

Fill

$\rho_{fill} := 1600 \frac{kg}{m^3} \cdot g = 15.691 \cdot \frac{kN}{m^3}$ Density of fill

Soil

$\sigma_{Rv} := 1000kPa$ Assumed soil resistance

Correction of units

$ton := 1000kg$

Geometry of the foundation

$b_{leg} := 0.75m$ Width of the leg

$l_{leg} := 9.0m$ Length of the leg

$b_{flange} := 2m$ Width of the flange on each leg

$h_{flange} := 0.6m$ Height of the flange

$h_{leg.0} := 1.3m$ Height of the leg at the outer edge

$h_{leg.r.cp} := 1.8m$ Height of the leg next to the centrepiece

$h_{leg} := \frac{h_{leg.0} + h_{leg.r.cp}}{2} = 1.55\,m$ Average height of the leg

$h_{element.0} := h_{leg.0} + h_{flange} = 1.9\,m$ Total height of the element at the outer edge

$h_{element.r.cp} := h_{leg.r.cp} + h_{flange} = 2.4\,m$ Height of the foundation next to the centrepiece

$h_{tot} := h_{leg} + h_{flange} = 2.15\,m$ Average height of the element

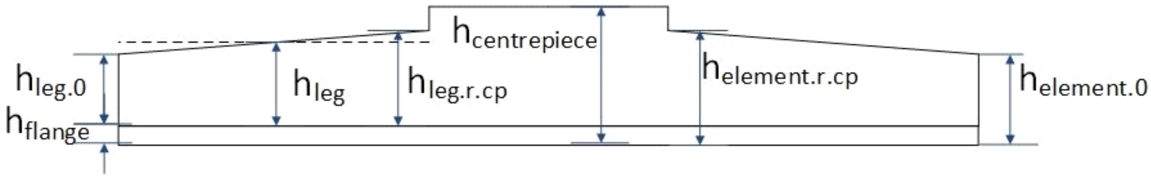


Figure 3: Heights of the foundation, along a cut through the middle of the foundation, are defined according to this figure. Element refers to the whole element including both leg and flange.

$l_{tot} := 2l_{leg} + d_{centrepiece} = 23\,m$ Total length of the foundation

$V_{leg} := b_{leg} \cdot l_{leg} \cdot h_{leg} + b_{flange} \cdot h_{flange} \cdot l_{leg} = 21.262 \cdot m^3$ Volume of one leg including flange

$\alpha := \frac{360deg}{8} = 45 \cdot deg$ Angle between the legs

$h_{fill.leg} := h_{centrepiece} - h_{flange} - h_{leg} = 0.65\,m$ Average height of the fill over the leg

$h_{fill.flange} := h_{centrepiece} - h_{flange} = 2.2\,m$ Height of the fill over the flange

$V_{fill} := 8l_{leg} \cdot b_{leg} \cdot h_{fill.leg} + 8l_{leg} \cdot (b_{flange} - b_{leg}) \cdot h_{fill.flange} = 233.1 \cdot m^3$ Volume of the fill

Verification of plate thickness

The flanges are assumed to work in one direction, therefore the calculation can be done as cantilever, calculated per length meter.
Minimum height of the flange is calculated, assumed cantilevering, according to page B113 Bärande konstruktioner del 1

$$h_{flange.min} := \frac{\frac{b_{flange}}{2}}{8} = 0.125\,m$$

$$Check_1 := \begin{cases} \text{"Sufficient height of the bottom flange"} & \text{if } h_{flange} \geq h_{flange.min} \\ \text{"Not sufficient height of the flange"} & \text{if } h_{flange} < h_{flange.min} \end{cases}$$

$Check_1 = \text{"Sufficient height of the bottom flange"}$

Summary of weights and volumes

$V_{tot} := 8 \cdot V_{leg} + V_{centrepiece} = 225.078 \cdot m^3$ Total volume of the concrete

$m_{tot} := V_{tot} \cdot \frac{\rho_c}{g} = 573.789 \cdot ton$ Total weight of the concrete

$m_{leg} := V_{leg} \cdot \frac{\rho_c}{g} = 54.204 \cdot ton$ Weight of one leg

Due to the weight of one leg, it needs to be divided into smaller elements within the limitations of transportation. The legs are divided into three elements, assuming that 1/8 of the centrepiece is attached to each leg.

$$m_{element} := \frac{V_{leg} \cdot \frac{\rho_c}{g} + \frac{V_{centrepiece} \cdot \frac{\rho_c}{g}}{8}}{3} = 23.908 \cdot ton$$ Weight of one element

$$l_{element.1} := \frac{m_{element} - \frac{V_{centrepiece} \cdot \frac{\rho_c}{g}}{8}}{\left(b_{leg} \cdot h_{leg} + b_{flange} \cdot h_{flange}\right) \frac{\rho_c}{g}} = 1.061\,m$$

Length of element 1 closest to the centrepiece Defined from section cp

$$l_{element.2} := \frac{l_{leg} - l_{element.1}}{2} = 3.97\,m$$

Length of element 2. Element 3 has equal length as element 2

Self-weight of the foundation and the fill

$$G_k := \left(8V_{leg} + V_{centrepiece}\right) \cdot \rho_c + V_{fill} \cdot \rho_{fill} = 9.284 \cdot MN$$

Total self-weight of the foundation and the fill

2. Global stability of the concept

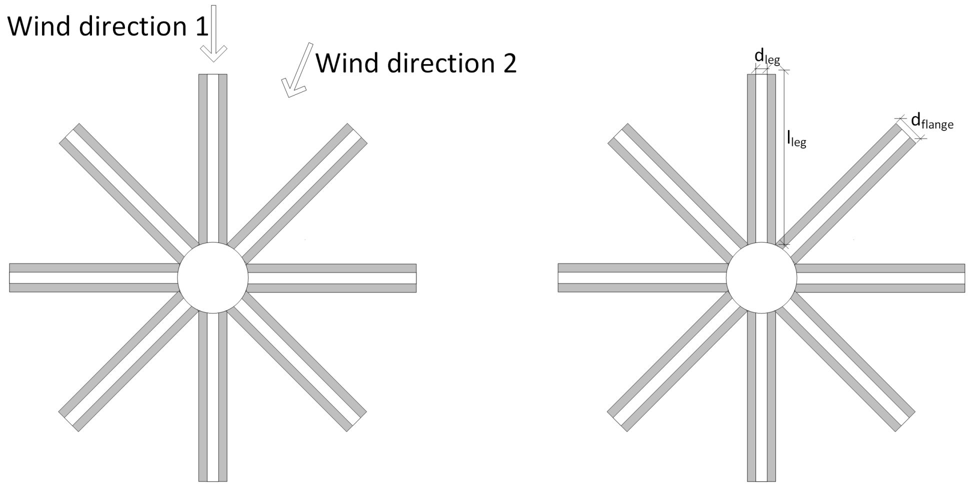


Figure 4: The foundation seen from above. The calculations are performed for two different wind directions, see the left figure. The right figure shows the definition of the dimensions of the legs.

Eccentricity

From moment equilibrium around the resultant of the soil pressure, the eccentricity can be calculated

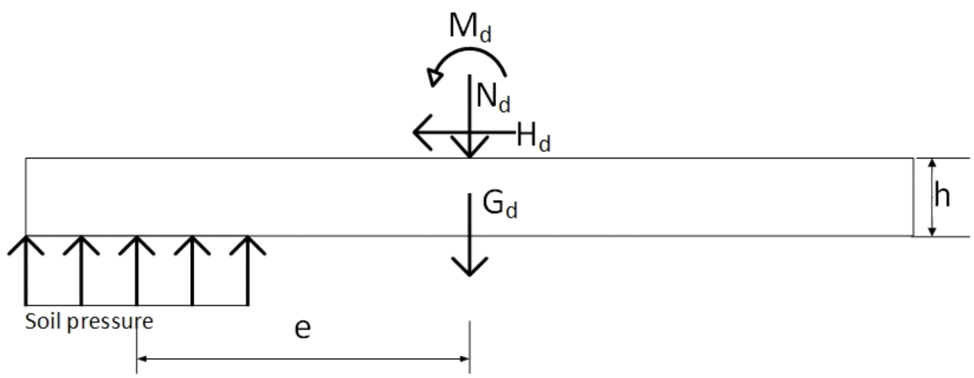


Figure 5: The loads acting on the foundation and the definition of e. The horisontal force is assumed to be resisted by the soil and is not further investigated, only the resulting moment due to its eccentricity is included. The distribution of the soil pressure is just a principal and will be changed according the geometry of the foundation.

$$e := \frac{M_d + H_d \cdot h_{centrepiece}}{\gamma_{dead}\left(G_k + N_k\right)} = 8.686\,m$$

Eccentricity for the soil pressure resultant

Global stability for wind direction 1

The length calculated below is defined similarly as for the concept; 8 legged stocky structure

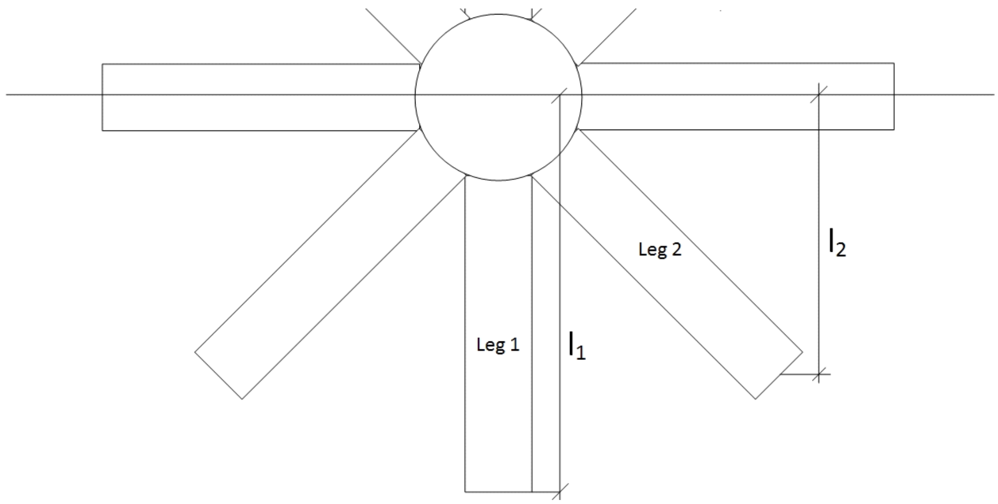


Figure 6: Definition of the length of the legs in the direction of the wind, defined from the centre of the foundation

$$l_1 := l_{leg} + \frac{d_{centrepiece}}{2} = 11.5\,m$$

Length of leg 1, defined from the centre of the foundation

$$l_2 := \cos(\alpha)\left(l_{leg} + \frac{d_{centrepiece}}{2}\right) = 8.132\,m$$

Length of legs 2, defined from the centre of the foundation

Soil pressure

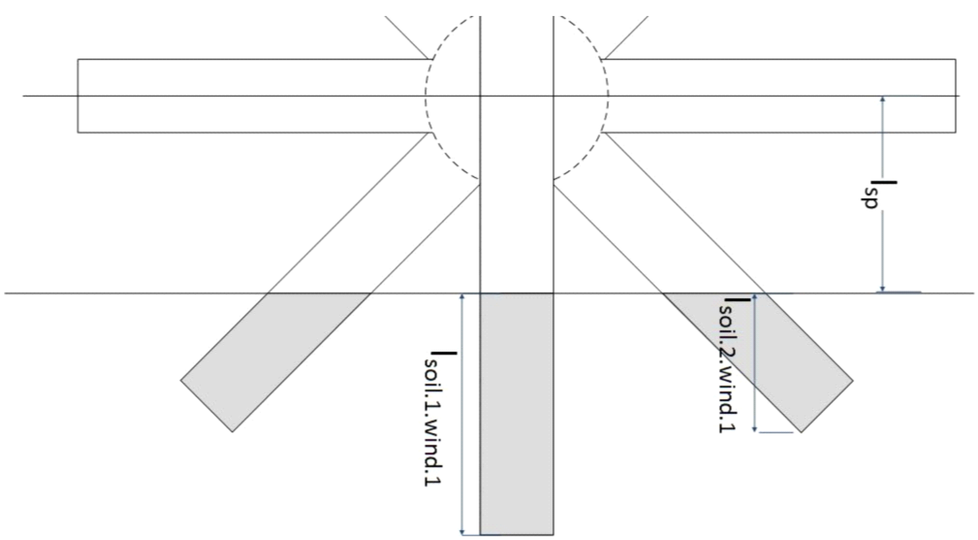


Figure 7: The definition of the length l.sp used to calculate the area of the soil pressure. The gray areas represent the soil pressure area.

$$l_{sp.1} := 7.1m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

The area is approximated for leg 2, the projected length is multiplied with the real width of the leg. This is approximate but is on safe side, since this area is slightly smaller than the real area.

$A_{soil.1.wind.1} := (l_1 - l_{sp.1}) \cdot b_{flange} = 8.8\,m^2$ Area of the soil pressure zone for leg 1

$A_{soil.2.wind.1} := 2 \cdot (l_2 - l_{sp.1}) \cdot b_{flange} = 4.127\,m^2$ Area of the soil pressure zone for legs 2

$A_{soil.tot.wind.1} := A_{soil.1.wind.1} + A_{soil.2.wind.1} = 12.927\,m^2$ Total area of soil pressure

$l_{soil.1.wind.1} := l_1 - l_{sp.1} = 4.4\,m$ Length of the soil pressure zone for leg 1

$l_{soil.2.wind.1} := l_2 - l_{sp.1} = 1.032\,m$ Length of the soil pressure zone for leg 2

Resultant centre of gravity of the soil pressure area for all three legs

$$tp_{wind1} := \frac{A_{soil.1.wind.1} \cdot \left(l_{sp.1} + \frac{l_1 - l_{sp.1}}{2}\right) \ldots + A_{soil.2.wind.1} \cdot \left(l_{sp.1} + \frac{l_2 - l_{sp.1}}{2}\right)}{A_{soil.tot.wind.1}} = 8.762\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$Check := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \frac{tp_{wind1}}{e}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind.1} := \frac{\gamma_{dead}(G_k + N_k)}{A_{soil.tot.wind.1}} = 897.043 \cdot kPa$$ Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind.1}}{\sigma_{Rv}} = 0.897$$ Utilisation

$$Check2 := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \frac{\sigma_{soil.wind.1}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Global stability for wind direction 2

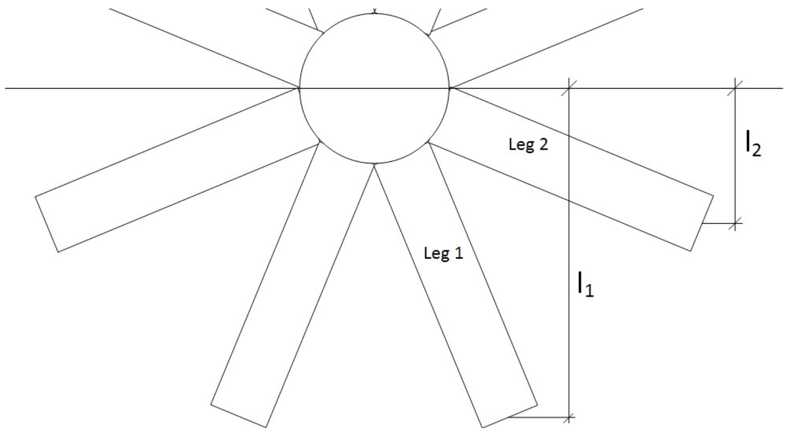


Figure 8: Definition of the length of the legs in the direction of the wind, defined from the centre of the foundation

$$l_1 := \left(l_{leg} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{45}{2}deg\right) = 10.625\,m$$

Length of leg 1, defined from the centre of the foundation

$$l_2 := \left(l_{leg} + \frac{d_{centrepiece}}{2}\right)\cdot\cos\left(\frac{3\cdot45}{2}deg\right) = 4.401\,m$$

Length of legs 2, defined from the centre of the foundation

Soil pressure

$$l_{sp.2} := 5.1m$$

Assumed distance from the centre of the foundation to the section where the soil pressure starts. Change iteratively.

The area is approximated for leg 1 and 2, the projected length is multiplied with the real width of the leg. This is approximate but is on safe side, since this area is slightly smaller than the real area.

$$A_{soil.1.wind.2} := 2\left(l_1 - l_{sp.2}\right)\cdot b_{flange} = 22.098\,m^2$$

Area of the soil pressure zone for leg 1

$$A_{soil.2.wind.2} := \begin{cases} 0 & \text{if } 2\cdot\left(l_2 - l_{sp.2}\right)\cdot b_{flange} < 0 \\ \left[2\cdot\left(l_2 - l_{sp.2}\right)\cdot b_{flange}\right] & \text{otherwise} \end{cases} = 0\,m^2$$

Area of the soil pressure zone for legs 2

$$A_{soil.tot.wind.2} := A_{soil.1.wind.2} + A_{soil.2.wind.2} = 22.098\,m^2$$

Total area of soil pressure

$$l_{soil.1.wind.2} := l_1 - l_{sp.2} = 5.525\,m$$

Length of the soil pressure zone for leg 1

$$l_{soil.2.wind.2} := l_2 - l_{sp.2} = -0.699\,m$$

Length of the soil pressure zone for leg 2

Resultant centre of gravity of the soil pressure area for all contributing legs

$$tp_{wind2} := \frac{A_{soil.1.wind.2}\cdot\left(l_{sp.2} + \frac{l_1 - l_{sp.2}}{2}\right) \ldots + A_{soil.2.wind.1}\cdot\left(l_{sp.2} + \frac{l_2 - l_{sp.2}}{2}\right)}{A_{soil.tot.wind.2}} = 8.749\,m$$

Check if the assumed length of the soil pressure is ok. If the difference between the eccentricity and the centre of gravity of the soil pressure zone is smaller than 1%, it is assumed that correct soil pressure zone is found.

$$\text{Check} := \begin{cases} \text{"The soil pressure zone is correct"} & \text{if } \left|1 - \dfrac{tp_{wind2}}{e}\right| < 1\% \\ \text{"Too large difference between the eccentricity and the soil pressure zone"} & \text{otherwise} \end{cases}$$

Check = "The soil pressure zone is correct"

$$\sigma_{soil.wind.2} := \frac{\gamma_{dead}(G_k + N_k)}{A_{soil.tot.wind.2}} = 524.742 \cdot kPa$$ Soil pressure

Check if the resistance of the soil is sufficient

$$\frac{\sigma_{soil.wind.2}}{\sigma_{Rv}} = 0.525$$ Utilisation

$$\text{Check2} := \begin{cases} \text{"OK! Soil resistance is sufficent"} & \text{if } \dfrac{\sigma_{soil.wind.2}}{\sigma_{Rv}} < 1 \\ \text{"Not sufficent resistance"} & \text{otherwise} \end{cases}$$

Check2 = "OK! Soil resistance is sufficent"

Indata from the global equilibrium calculations to the further calculations

Worst case - wind direction

In order to decide which wind direction to use in the further calculations, the soil pressure resultant Q.soil for one leg is calculated for both wind directions. The highest Q.soil will give the worst case when looking at only one element.

$$Q_{soil.wind1} := \sigma_{soil.wind.1} \cdot A_{soil.1.wind.1} = 7.894 \times 10^6 N$$ Soil pressure resultant for wind direction 1

$$Q_{soil.wind2} := A_{soil.1.wind.2} \cdot \frac{\sigma_{soil.wind.2}}{2} = 5.798 \times 10^6 N$$ Soil pressure resultant for wind direction 2

$$Q_{soil} := \max(Q_{soil.wind1}, Q_{soil.wind2}) = 7.894 \times 10^6 N$$ Maximum soil pressure resultant

It is therefore chosen to calculate for wind direction 1 in the detailed dimensioning calculations. In the further calculations it will be necessary to also verify wind direction 2 and all other wind directions that might cause any critical stresses in any part.

$$l_{soil.1} := l_{soil.1.wind.1} = 4.4 m$$ Length of soil pressure zone, for leg 1 in wind direction 1

$$l_{soil.2} := l_{soil.2.wind.1} = 1.032 m$$ Length of soil pressure zone, for leg 2 in wind direction 1

Summary of the concept - weight and volume of elements

$$V_{8.flange} := 8 \cdot V_{leg} + V_{centrepiece} = 225.078 \cdot m^3$$ Total volume of the concrete

$$m_{8.flange} := V_{8.flange} \cdot \frac{\rho_c}{g} = 573.789 \cdot ton$$ Total weight of the concrete

$$m_{leg.8.flange} := V_{leg} \cdot \frac{\rho_c}{g} = 54.204 \cdot ton$$

Weight of one leg

Division of the foundation to element within the limitation of transportation

In order to be able to transport the elements, the maximum weight of each element is set 20 ton. The number of elements each leg must be divided into to fulfill this limit is calculated.

$$n_{element.8.flange} := \frac{m_{leg.8.flange}}{20ton} = 2.71$$

Number of elements that one leg must be divided into

$$m_{element.8.flange} := \frac{m_{leg.8.flange}}{ceil\left(n_{element.8.flange}\right)} = 18.068 \cdot ton$$

Weight of each element

3. Division into strips

In order to easier calculate the moment and shear force distribution of the foundation, we chose to only look at one leg, defined from the section r.fc. The element is then divided in small strips. The length and width for each strip are assumed to be constant over the strip. Also the ground pressure are assumed to be constant over each strip

The moment can then be calculated around the section, r.fc from the centre of the foundation and the moment distribution can be plot for the element. The moment is calculated in two separate functions and diagrams, one for the positive moment and one for the negative moment. The same thing is then done for the shear force distribution.

Divide the element into strips

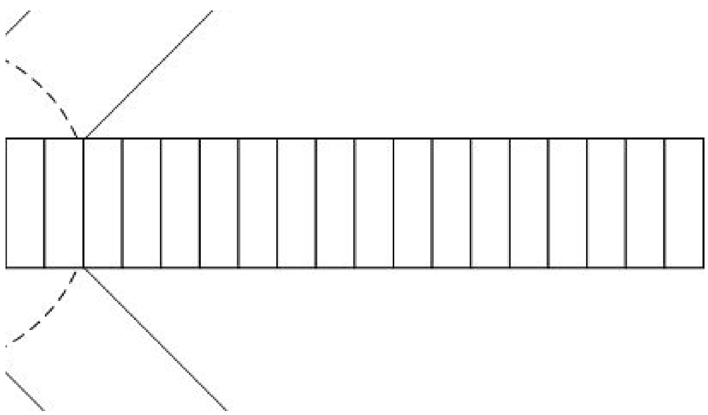


Figure 9: The element is divided into strips, as shown in the figure.

Definition of vectors for directions and numbering of the strips

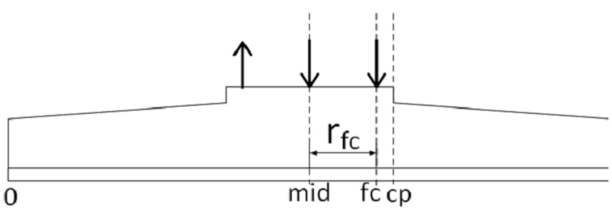


Figure 10: Definition of certain sections in the foundation that will be used in the calculations.

Vectors are defined to enable the calculations for the moment and shear force, according to figures below. Both vectors that give each strip a certain number, starting from point i or j, and vectors that give each strip a coordinate, also starting from i or j.

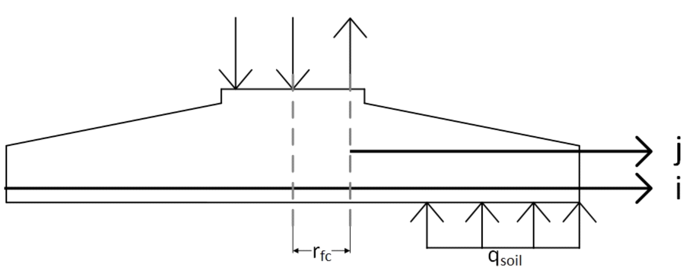


Figure 11: Vectors with numbering of the strips

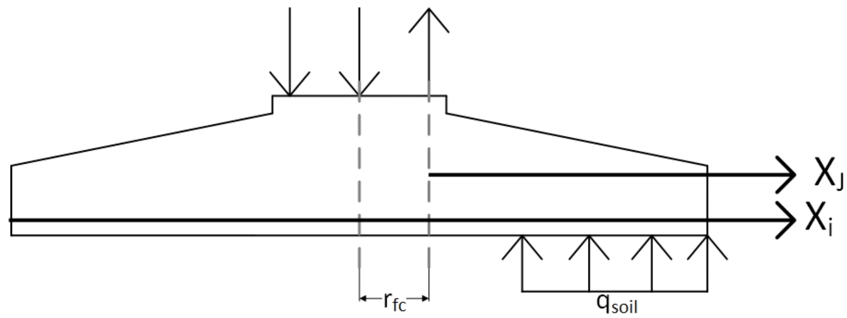


Figure 12: Vectors with the coordinates of the strip

Vector i

$\Delta x := 10mm$ — Width of each strip, see figure above

$i_{tot} := \frac{l_{tot}}{\Delta x} - 1 = 2.299 \times 10^3$ — Total number of strips in vector i

$i := 0 .. i_{tot}$ — Numbering of each strip in vector i

$i_{mid} := ceil\left(\frac{i_{tot}}{2}\right) = 1.15 \times 10^3$ — Number of the middle strip in vector i

$i_{r.cp} := ceil\left(\frac{\frac{l_{tot}-d_{centrepiece}}{2}}{\Delta x}\right) = 900$ — Number of the strip in vector i which is next to the centerpiece

$i_{r.fc} := ceil\left(\frac{\frac{l_{tot}}{2} + r_{fc}}{\Delta x}\right) = 1.33 \times 10^3$ — Number of the strip in vector i which is in the cut fc

$i_{soil.1} := ceil\left(\frac{l_{tot} - l_{soil.1}}{\Delta x}\right) - 1 = 1.859 \times 10^3$ — Number of the strips in vector i where the soil pressure is n longer zero

$i_{soil.2} := ceil\left(\frac{l_{tot} - l_{soil.2}}{\Delta x}\right) - 1 = 2.196 \times 10^3$ — Number of the strips in vector i where the soil pressure is n longer zero

$X_{I_i} := i \cdot \Delta x + \frac{\Delta x}{2} - \frac{l_{tot}}{2}$ — Coordinate for each strip, X_I is defined as zero in the centre of the foundation. The coordinates on the left side of the foundation are defined as negative and on the right side they are defined as positive.

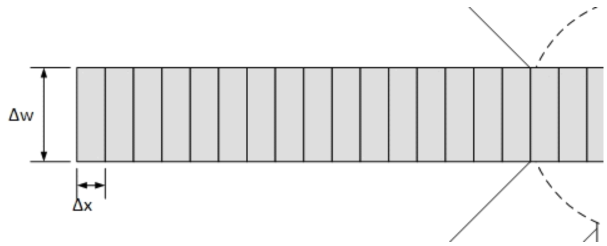


Figure 13: Definition of the lengths and widths of the strips along the vectors

The width of the strips is only valid in the sections outside the centrepiece. This is ok since we will not calculate the moment inside the centrepiece.

$b_{I_i} := b_{flange} = ...$ — The width of the strip vary along the vector i, valid outside the centrepiece only.

$A_{I_i} := \Delta x \cdot b_{I_i} = ...$ — Area of each strip in vector i

Vector j

$$j_{tot} := \frac{\frac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 = 968$$

Number of elements in vector j

$j := 0..j_{tot}$ — A vector that gives each strip a certain number, see figure above. Defined from the section fc to the end of the foundation, see figure above

$X_{J_j} := X_{I_{j+i_{r.fc}}} = ...$ — Coordinates for the elements in vector j, see figure above

$b_{J_j} := b_{I_{j+i_{r.fc}}} = ...$ — Width of each strip in vector j

$A_{J_j} := b_{J_j} \cdot \Delta x = ...$ — Area of each strip in vector j

$s_{J_j} := X_{J_j} - X_{J_0} + \frac{\Delta x}{2} = ...$ — Lever arm for each strip along vector j

$$j_{splice1} := \mathrm{ceil}\left[\frac{\left(\frac{d_{centrepiece}}{2} - r_{fc}\right) + l_{element.1}}{\Delta x}\right] = 177$$

Point in vector j where Splice 1 (between element 1 and 2) is located

$$j_{splice2} := \mathrm{ceil}\left[\frac{\left(\frac{d_{centrepiece}}{2} - r_{fc}\right) + l_{element.1} + l_{element.2}}{\Delta x}\right] = 574$$

Point in vector j where Splice 2 (between element 2 and 3) is located

The height of the foundation

Height of the element with adjusted height of the centrepiece

Approximated variation of the height of the element, assuming that the centrepiece has the same height as the strip at the cut r.cp. This will not effect the calculations since the moment distribution is only calculated outside the centrepiece.

$$\Delta h_i := \begin{cases} h_{element.0} + \dfrac{h_{element.r.cp} - h_{element.0}}{\left(\dfrac{l_{tot} - d_{centrepiece}}{2}\right)} \cdot \Delta x \cdot i & \text{if } \Delta x \cdot i \le \dfrac{l_{tot}}{2} - \dfrac{d_{centrepiece}}{2} \\ h_{element.r.cp} & \text{if } \dfrac{l_{tot}}{2} - \dfrac{d_{centrepiece}}{2} < \Delta x \cdot i \le \dfrac{l_{tot}}{2} + \dfrac{d_{centrepiece}}{2} \\ h_{element.r.cp} - \dfrac{h_{element.r.cp} - h_{element.0}}{\left(\dfrac{l_{tot} - d_{centrepiece}}{2}\right)} \cdot \Delta x \cdot \left(i - \text{ceil}\left(\dfrac{\frac{l_{tot}+d_{centrepiece}}{2}}{\Delta x}\right)\right) & \text{if } \Delta x \cdot i > \dfrac{l_{tot}}{2} + \dfrac{d_{centrepiece}}{2} \end{cases}$$

Height of the web, with adjusted height of the centrepiece

The variation of the height of the legs, excluding the height of the bottom flange. The height at the centrepiece is treated in the same way as for Δh_i

$$\Delta h_{leg.I_i} := \Delta h_i - h_{flange} = ...$$

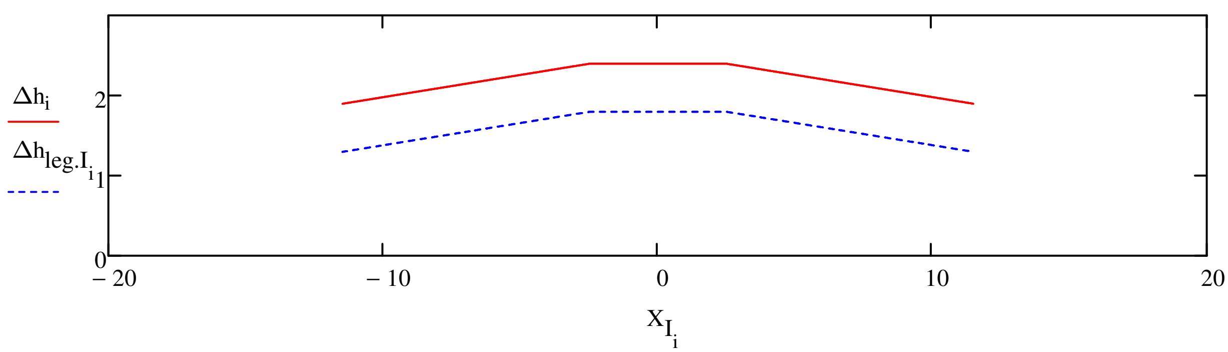


Diagram: Variation of the height along vector i, from f.c to the edge of the foundation.

Variation of the height of the element along vector j

Defined along vector j, from the section r.fc to the edge of the foundation

$$\Delta h_{leg.J_j} := \Delta h_{leg.I_{j+i_{r.fc}}} = ...$$ The variation of the height of the legs along vector j

Variation of the height of the fill

Due to the variation of the height of the leg, also the height of the fill will vary along the element.

$$h_{fill.leg.I_i} := h_{centrepiece} - h_{flange} - \Delta h_{leg.I_i} = ...$$ Variation of the height of the fill along vector i

$$h_{fill.leg.J_j} := h_{centrepiece} - h_{flange} - \Delta h_{leg.J_j} = ...$$ Variation of the height of the fill along vector j

4. Loads acting on the foundation, locally

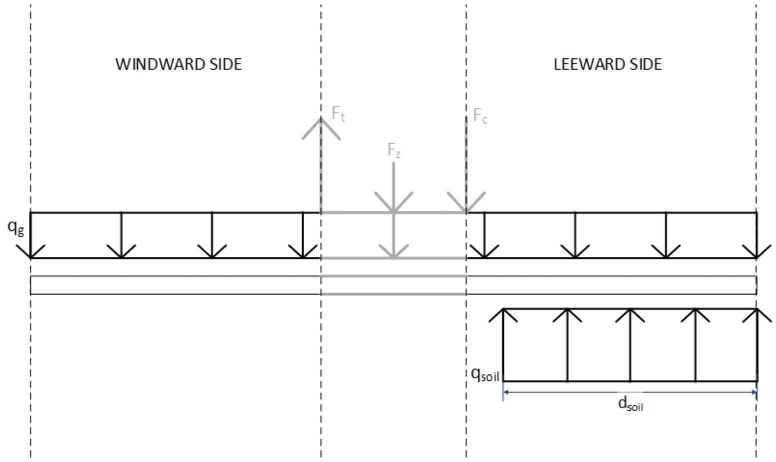


Figure 14: Loads acting on the foundation, in the local analysis only the part of the foundation outside the force couple resultant is considered.

In the local calculations the moment distribution is calculated both for the windward and the leeward side in separate calculations. The assumed resulting loading is shown in figure below.

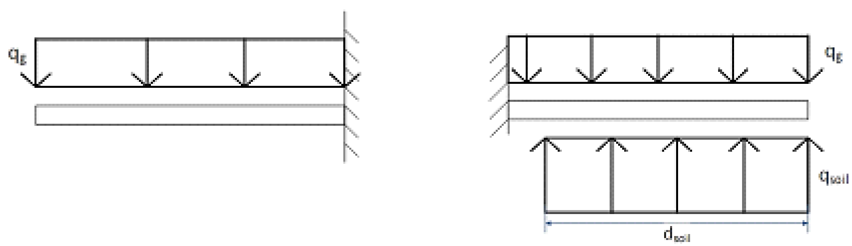


Figure 15: Assumed resulting loading a) for the windward side b) for the leeward side

Self-weight

The total self-weight of the foundation including the fill above the foundation, is assumed to be uniformly distributed over the total length of the foundation. In reality this distribution is not uniform, but depends on the shape of the foundation.

$$g_d := \frac{\gamma_{dead} \cdot G_k}{l_{tot}} = 363.304 \cdot \frac{kN}{m}$$

Distributed self-weight over the length of the foundation

Soil pressure

The total reaction force of the soil is divided with the length of the soil pressure zone, in order to have the reaction force per unit length. This is done for the two different legs.

$$q_{soil.1} := \frac{Q_{soil}}{l_{soil.1}} = 1.794 \times 10^3 \cdot \frac{kN}{m}$$

Soil reaction force per unit length in leg 1

$$q_{soil.2} := \frac{Q_{soil}}{l_{soil.2}} = 7.651 \times 10^3 \cdot \frac{kN}{m}$$

Soil reaction force per unit length in leg 2, this is larger than the reaction force in leg 1 due to the shorter length of the soil pressure zone.

Resulting loads for each element

For the positive moment the self-weight is favorable and the partial factor 0.9 is used on the loads.
For the negative moment the foundation is assumed to be "hanging" from the centrepiece, and therefore the self-weight is unfavorable and the partial factor 1.1 is used.
The moment is calculated in the cut fc at a distance r_{fc} from the centre of the foundation. Therefore the loads below are calculated for elements in vector j.

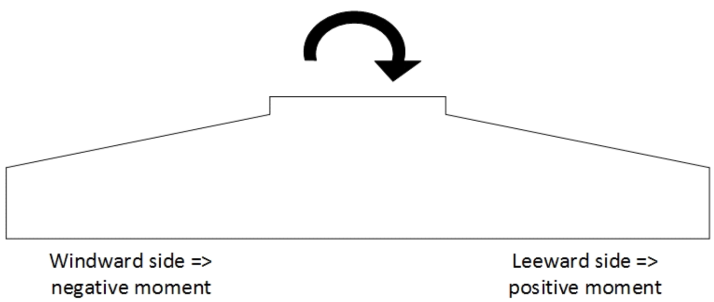


Figure 16: The moment on the windward side is denoted the negative moment and the moment on the leeward side is denoted the positive moment.

Dead load, leeward side

Vector with dead load for each element, used for calculate the positive moment

$$N_{dead.p_j} := \gamma_{dead}\left[\left(\Delta x \cdot \Delta h_{leg.J_j} \cdot b_{leg} + \Delta x \cdot h_{flange} \cdot b_{J_j}\right) \cdot \rho_c \cdots \atop + \left[\Delta x \cdot h_{fill.leg.J_j} \cdot b_{leg} + \Delta x \cdot h_{fill.flange} \cdot \left(b_{J_j} - b_{leg}\right)\right] \cdot \rho_{fill}\right] = \ldots$$

Vector with dead load for each element, used for calculate the negative moment

$$N_{dead.n_j} := \gamma_{abn}\left[\left(\Delta x \cdot \Delta h_{leg.J_j} \cdot b_{leg} + \Delta x \cdot h_{flange} \cdot b_{J_j}\right) \cdot \rho_c \cdots \atop + \left[\Delta x \cdot h_{fill.leg.J_j} \cdot b_{leg} + \Delta x \cdot h_{fill.flange} \cdot \left(b_{J_j} - b_{leg}\right)\right] \cdot \rho_{fill}\right] = \ldots$$

5. Moment distribution, locally

Moment distribution - leg 1

$$\sigma_{soil.1_j} := \begin{vmatrix} 0 \;\text{ if }\; X_{J_j} < \frac{l_{tot}}{2} - l_{soil.1} \\ \sigma_{soil.wind.1} \;\;\text{otherwise} \end{vmatrix}$$

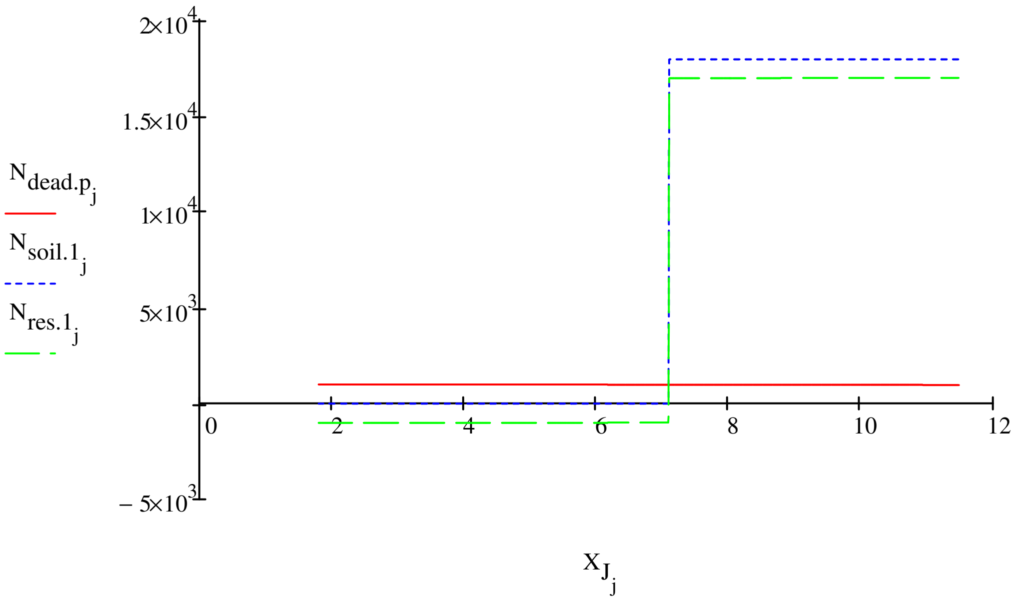
Vector with the soil pressure for each strip. The soil pressure is zero outside the soil pressure area. The ground pressure has only a value in the soil pressure zone, otherwise it is zero.

$$N_{soil.1_j} := \sigma_{soil.1_j} \cdot b_{J_j} \cdot \Delta x = ...$$

Vector with ground pressure as a total force acting on each strip.

$$N_{res.1_j} := N_{soil.1_j} - N_{dead.p_j} = ...$$

Resulting force vector



Positive moment distribution, on the leeward side

The moment distribution, M_p is calculated from the cutfc to the end of the foundation. It is defined on the leeward side
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side,
The resulting force, N_{res}, for each element is multiplied with its lever arm, s_J, for its respective element and then summed up for the all the elements into the positive moment M_p .

Moment distribution from r_{fc} to the edge of the foundation

$$M_{p.1} := \begin{vmatrix} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } \; k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.1_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{vmatrix}$$

Maximum moment

$M_{p.1.max} := M_{p.1_0} = 5.441 \times 10^4 \cdot kN \cdot m$

Moment in critical cut, r_{fc}, gives the maximum moment on the positive side

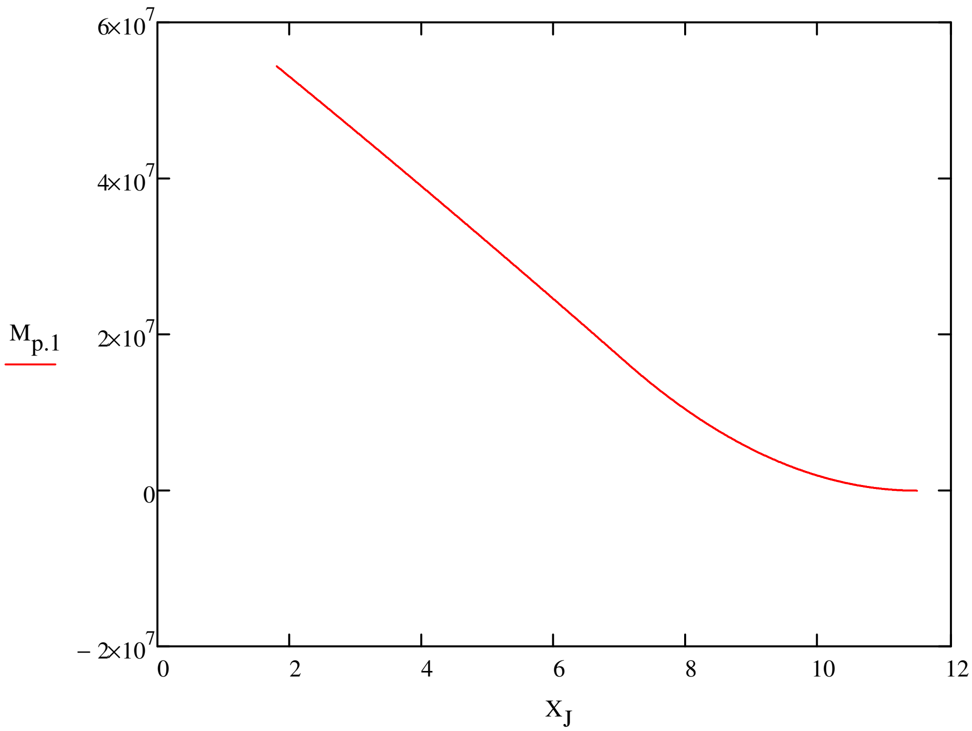
$M_{p.1.splice1} := M_{p.1_{j_{splice1}}} = 4.2 \times 10^4 \cdot kN \cdot m$

Moment in splice 1

$M_{p.1.splice.2} := M_{p.1_{j_{splice2}}} = 1.321 \times 10^4 \cdot kN \cdot m$

Moment in splice 2

Moment diagram of the positive moment



Negative distribution, on the windward side

The moment distribution, M_n is calculated from the cut fc to the edge of the foundation. It is defined on the windward side.
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side,
The resulting force, N_{res}, for each element is multiplied with the lever arm, s_J ,for its respective element and then summed up for the all the elements into the negative moment M_n .

Moment distribution from r_{fc} to the edge of the foundation

$$M_{n.1} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{dead.n_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$M_{n.1.max} := M_{n.1_0} = 5.648 \times 10^3 \cdot kN \cdot m$

Moment in critical cut, fc, gives the maximum moment on the negative side

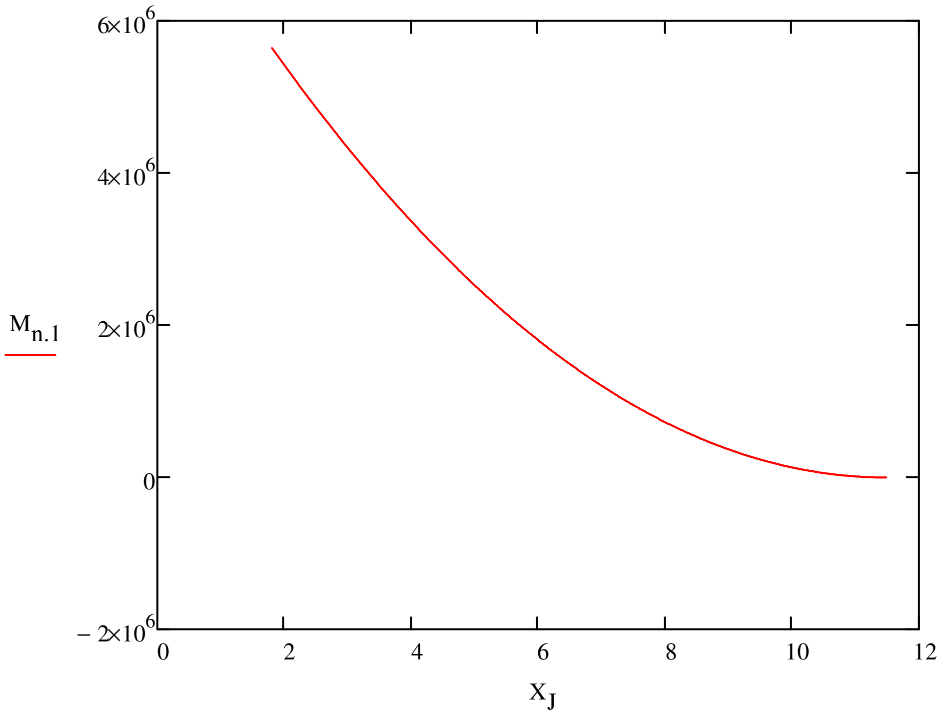
$M_{n.1.splice.1} := M_{n.1_{j_{splice1}}} = 3.765 \times 10^3 \cdot kN \cdot m$

Negative moment in splice 1

$M_{n.1.splice.2} := M_{n.1_{j_{splice2}}} = 930.809 \cdot kN \cdot m$

Negative moment in splice 2

Moment diagram of the negative moment



Moment distribution - leg 2

$$\sigma_{soil.2_j} := \begin{cases} 0 & \text{if } X_{J_j} < \frac{l_{tot}}{2} - l_{soil.2} \\ \sigma_{soil.wind.1} & \text{otherwise} \end{cases}$$

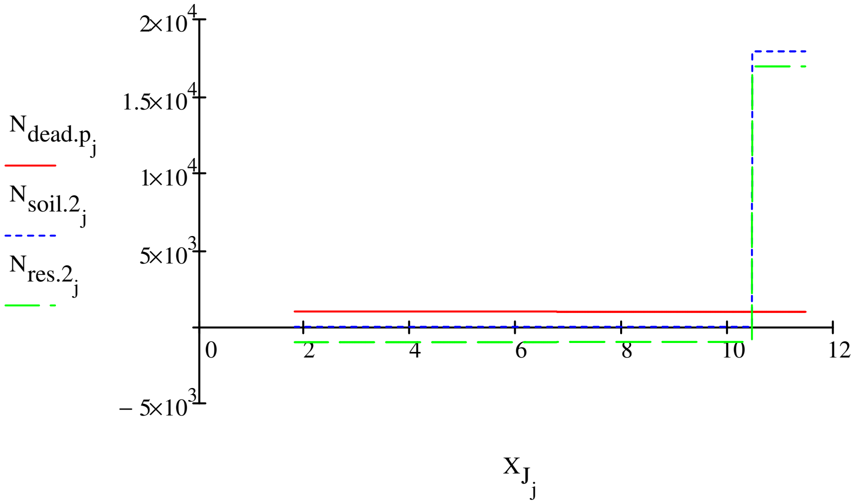
Vector with the soil pressure for each strip. The soil pressure is zero outside the soil pressure area.

$N_{soil.2_j} := \sigma_{soil.2_j} \cdot b_{J_j} \cdot \Delta x = ...$

Vector with ground pressure as a total force acting on each strip

$N_{res.2_j} := N_{soil.2_j} - N_{dead.p_j} = ...$

Resulting force vector



Positive moment distribution, on the leeward side

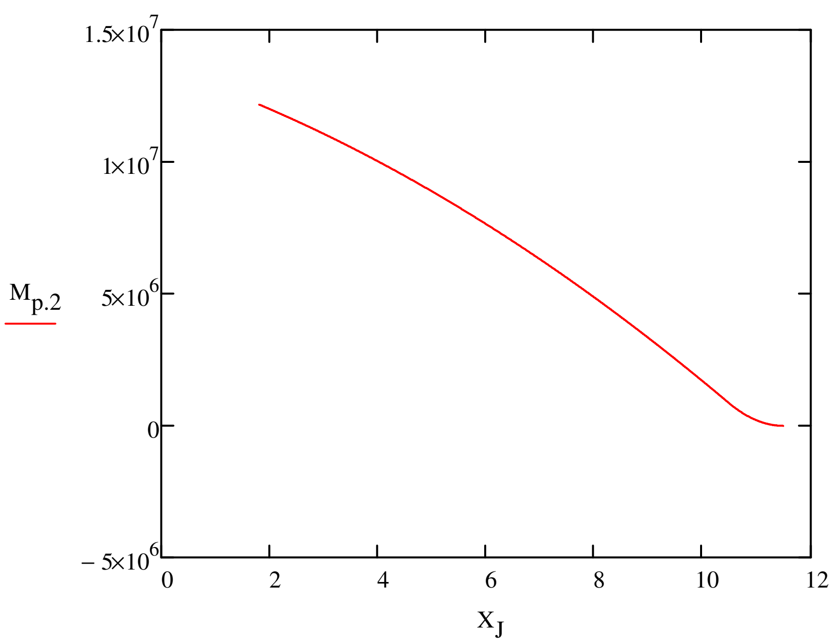
$$M_{p.2} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.2_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$$M_{p.2.max} := M_{p.2_0} = 1.218 \times 10^4 \cdot kN \cdot m$$

Moment in critical cut, fc, gives the maximum moment on the positive side

Moment diagram of the positive moment



Negative distribution, on the windward side

The negative moment distribution in leg 2 is equal to the negative moment distribution in leg 1, since only the self-weight is acting on the negative side and it is not effected by the change in soil pressure.

$M_{n.2} := M_{n.1}$

$M_{n.2.max} := M_{n.1.max} = 5.648 \times 10^{3} \cdot kN \cdot m$

Summary of the moment calculations

Maximum moment

Due to the fact that the wind can blow from any side any time, all the parts of the foundation must be dimensioned for the maximum moment both the positive and negative moment.

$M_{n.1.max} = 5.648 \times 10^{3} \cdot kN \cdot m$ Maximum moment on the negative side in leg 1

$M_{p.1.max} = 5.441 \times 10^{4} \cdot kN \cdot m$ Maximum moment on the positive side in leg 1

$M_{n.2.max} = 5.648 \times 10^{3} \cdot kN \cdot m$ Maximum moment on the negative side in leg 2

$M_{p.2.max} = 1.218 \times 10^{4} \cdot kN \cdot m$ Maximum moment on the positive side in leg 2

The largest positive moment is dimensioning for the bottom reinforcement and the largest negative moment for the top reinforcement. Leg 1 is used for dimensioning the reinforcement, since the local moment in leg 1 is higher.

Composants of the moments in x and y-direction

The composants of the moments from the different legs are calculated, the main direction (x) is in the same direction as the moment in leg 1, while the secondary direction of the bending moment (y) is perpendicular to the moment in leg 1.
Due to the angle between the legs, the composants of each leg is calculated

$M_{p.1.x} := M_{p.1.max} \cdot \cos(0) = 5.441 \times 10^{4} \cdot kN \cdot m$ $M_{n.1.x} := M_{n.1.max} \cdot \cos(0) = 5.648 \times 10^{3} \cdot kN \cdot m$

$M_{p.1.y} := M_{p.1.max} \sin(0) = 0 \cdot kN \cdot m$ $M_{n.1.y} := M_{n.1.max} \sin(0) = 0 \cdot kN \cdot m$

$M_{p.2.x} := M_{p.2.max} \cdot \cos(\alpha) = 8.611 \times 10^{3} \cdot kN \cdot m$ $M_{n.2.x} := M_{n.2.max} \cdot \cos(\alpha) = 3.994 \times 10^{3} \cdot kN \cdot m$

$M_{p.2.y} := M_{p.2.max} \cdot \sin(\alpha) = 8.611 \times 10^{3} \cdot kN \cdot m$ $M_{n.2.y} := M_{n.2.max} \cdot \sin(\alpha) = 3.994 \times 10^{3} \cdot kN \cdot m$

Bending reinforcement in the bottom

Chosen dimension of the bars

$\phi_b := 25mm$ Diameter of the bars for the bottom reinforcement

$A_{si.b} := \frac{\phi_b^{2} \cdot \pi}{4} = 4.909 \times 10^{-4} m^{2}$ Area of each reinforcement bar

Indata from Eurocode 2 SS-EN 1992-1-1:2005 8.2 (2)

$k_1 := 1$ $k_2 := 5mm$ Factors from EC2

$d_g := 32mm$ Assumed dimension of the aggregates

$c_{bar.b} := \max\left(d_g + k_2, 20mm, k_1 \cdot \phi_b\right) = 0.037\,m$ — Distance between bars and layers

$\Delta c_{dev} := 10mm$ — Factor from EC2

$c_{min.b} := \phi_b + \Delta c_{dev} = 0.035\,m$ — Minimum concrete cover (eq 4.1)

$c_{cover.b} := c_{min.b}$ — Chosen concrete cover

Maximum number of bars that fits in the element

The reinforcement is placed in the section cp

$$n_{max.bars.flange.b} := \mathrm{floor}\left(\frac{b_{flange} - 2 \cdot c_{cover.b} - 2 \cdot \phi_b}{\phi_b + c_{bar.b}}\right) = 30$$

Number of bars that fits into one layer in the bottom flange

$$n_{max.layer.flange.b} := \mathrm{floor}\left(\frac{h_{flange} - 2 \cdot c_{cover.b} - 2 \cdot \phi_b}{\phi_b + c_{bar.b}}\right) = 7$$

Number of layers that fits into the bottom flange in the height direction.

$$n_{max.bars.leg.b} := \mathrm{floor}\left(\frac{b_{leg} - 2 \cdot c_{cover.b} - 2 \cdot \phi_b}{\phi_b + c_{bar.b}}\right) = 10$$

Number of bars that fits into one layer in the leg

Number of bars needed

$n_{bars.b} := 126$

Number of bars needed in the section, iteratively changed according to calculations below. In order to achieve the correct centre of gravity for the bars

$n_{bars.flange.b} := \min\left(n_{bars.b}, n_{max.layer.flange.b} \cdot n_{max.bars.flange.b}\right) = 126$ — Number of bars in the bottom flange

$n_{bars.leg.b} := \max\left(n_{bars.b} - n_{bars.flange.b}, 0\right) = 0$ — Number of bars in the leg of the element

$$n_{layer.b} := \begin{cases} \mathrm{ceil}\left(\dfrac{n_{bars.b}}{n_{max.bars.flange.b}}\right) & \text{if } n_{bars.b} \le n_{max.layer.flange.b} \cdot n_{max.bars.flange.b} \\ \mathrm{ceil}\left(n_{max.layer.flange.b} + \dfrac{n_{bars.b} - n_{max.layer.flange.b} \cdot n_{max.bars.flange.b}}{n_{max.bars.leg.b}}\right) & \text{otherwise} \end{cases}$$

$n_{layer.b} = 5$ — Total number of layers

Centre of gravity of the bars

Centre of gravity of the different layers in the bottom flange, different equations are valid for different types of layers. All equations are shown below, and in the and a if-loop will get the correct value for the actual number of bars and layers.

$$tp_{layer.1} := \frac{n_{bars.b} \cdot \left(c_{cover.b} + \dfrac{\phi_b}{2}\right)}{n_{bars.b}} = 0.048\,m$$

$$tp_{layer.2} := \frac{n_{max.bars.flange.b} \cdot \left(c_{cover.b} + \frac{\phi_b}{2}\right) \ldots + \left(n_{bars.b} - n_{max.bars.flange.b}\right) \cdot \left(c_{cover.b} + \frac{3}{2} \cdot \phi_b + c_{bar.b}\right)}{n_{bars.b}} = 0.095\,m$$

$$tp_{layer.3} := \frac{n_{max.bars.flange.b} \cdot \left(2 \cdot c_{cover.b} + \frac{4 \cdot \phi_b}{2} + c_{bar.b}\right) \ldots + \left(n_{bars.b} - 2 \cdot n_{max.bars.flange.b}\right) \cdot \left(c_{cover.b} + \frac{5}{2} \cdot \phi_b + 2 \cdot c_{bar.b}\right)}{n_{bars.b}} = 0.127\,m$$

$$tp_{layer.4} := \frac{n_{max.bars.flange.b} \cdot \left(3 \cdot c_{cover.b} + \frac{9 \cdot \phi_b}{2} + 3 \cdot c_{bar.b}\right) \ldots + \left(n_{bars.b} - 3 \cdot n_{max.bars.flange.b}\right) \cdot \left(c_{cover.b} + \frac{7}{2} \cdot \phi_b + 3 \cdot c_{bar.b}\right)}{n_{bars.b}} = 0.145\,m$$

$$tp_{layer.5} := \frac{n_{max.bars.flange.b} \cdot \left(4 \cdot c_{cover.b} + \frac{16 \cdot \phi_b}{2} + 6 \cdot c_{bar.b}\right) \ldots + \left(n_{bars.b} - 4 \cdot n_{max.bars.flange.b}\right) \cdot \left(c_{cover.b} + \frac{9}{2} \cdot \phi_b + 4 \cdot c_{bar.b}\right)}{n_{bars.b}} = 0.148\,m$$

Centre of gravity of all the bars for the correct numbers of bars and layers

$$y_{tp.b} := \left| \begin{array}{l} tp_{layer.1} \text{ if } n_{layer.b} = 1 \\ tp_{layer.2} \text{ if } n_{layer.b} = 2 \\ tp_{layer.3} \text{ if } n_{layer.b} = 3 \\ tp_{layer.4} \text{ if } n_{layer.b} = 4 \\ tp_{layer.5} \text{ if } n_{layer.b} = 5 \end{array} \right.$$

$$y_{tp.b} = 0.148\,m$$

$$d_{b_i} := \Delta h_i - y_{tp.b} = \ldots$$

Effective depth, from top of the section to the tensile resultant. Along vector i.

$$d_{b.J_j} := d_{b_{j+i_{r.fc}}}$$

Effective depth, from top of the section to the tensile resultant. Along vector j.

The need for bottom bending reinforcement

The required moment is taken in the maximum moment section, fc. The reinforcement amount is then calculated for the section cp where the leg and flange starts. The reinforcement is placed according to the cross-section in section cp. Since the reinforcement continues into the centrepiece, it will therefore have sufficient capacity also in the maximum section fc.

$$A_{s.b} := \frac{M_{p.1.max}}{0.9 \cdot d_{b.J_0} \cdot f_{yd}} = 0.062\,m^2$$

Needed amount of reinforcement in section cp, calculated for the maximum moment.

$n_{bars.b} := \frac{A_{s.b}}{A_{si.b}} = 125.776$ Number of bars needed in section cp, based on the maximum moment in fc.

$n_{bars.b} := \mathrm{ceil}(n_{bars.b}) = 126$ Number of bars needed, rounded value.

$A_{Rd.b} := A_{si.b} \cdot n_{bars.b} = 6.185 \times 10^4 \cdot mm^2$ Actual amount of reinforcement with the dimension of the bars given as indata above.

$M_{p.Rd} := 0.9 \cdot d_{b.J_0} \cdot f_{yd} \cdot A_{Rd.b} = 5.451 \times 10^4 \cdot kN \cdot m$ Bending moment capacity for the calculated number of bars

$\frac{M_{p.1.max}}{M_{p.Rd}} = 0.998$ Utilisation

Needed anchorage length

The calculation of needed anchorage length is done according to SS-EN 1992-1-1:2005 section 8.4

$\eta_1 := 1.0$ Coefficient related to the quality of the bond condition. =1.0 for "good" conditions, =0.7 for all other conditions. 1.0 is chosen due to the criteria d in EC

$$\eta_2 := \begin{cases} 1.0 & \text{if } \phi_b \le 32mm \\ \frac{\left(132 - \frac{\phi_b}{mm}\right)}{100} & \text{otherwise} \end{cases} = 1$$

Coefficient related to the bar diameter

$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 4.35 \cdot MPa$ Design value of the ultimate bond stress

The calculation of the required anchorage length is base on EC 8.4.3 (2), using the yield stress of the steel bars, σ_{sd} is set to the yield stress f_{yd} . This is an assumption on the safe side, since the bars are not fully utilised in all sections of the element.

$l_{b.rqd.b} := \left(\frac{\phi_b}{4}\right) \cdot \left(\frac{f_{yd}}{f_{bd}}\right) = 0.625\,m$ Basic required anchorage length.

Coefficient α_1 - α_5 is given in table 8.2

$\alpha_1 := 1.0$ Effect of the form of the bars, assuming adequate cover

$a := \frac{b_{flange} - 2 \cdot c_{cover.b}}{n_{max.bars.flange.b} - 1} = 0.067\,m$ Distance between bars in the critical section

$c_d := \min\left(\frac{a}{2}, c_{cover.b}\right) = 0.033\,m$

Effect of concrete minimum cover

$$\alpha_2 := \begin{cases} 0.7 \ \text{if} \ 1 - 0.15 \cdot \dfrac{(c_{cover.b} - \phi_b)}{\phi_b} \le 0.7 \\ 1 - 0.15 \cdot \dfrac{(c_d - \phi_b)}{\phi_b} \ \text{if} \ 0.7 < 1 - 0.15 \cdot \dfrac{(c_{cover.b} - \phi_b)}{\phi_b} < 1.0 \\ 1.0 \ \text{otherwise} \end{cases} = 0.95$$

α_3 and α_5 are assumed to be 1.0, which is on the safe side

$\alpha_3 := 1.0$ — Effect of confinement by transverse reinforcement, set to 1.0 (safe side)

$\alpha_5 := 1.0$ — Effect of pressure transverse to the plane of splitting along the design anchorage length, set to1.0 (safe side).

$\alpha_4 := 0.7$ — Influence of welded transverse bars, set to 0.7 according to EC

$\text{Check} := \alpha_2 \cdot \alpha_3 \cdot \alpha_5 > 0 = 1$ — Check ok

$l_{b.min.b} := \max(0.3 \cdot l_{b.rqd.b}, 10 \cdot \phi_b, 100\text{mm}) = 0.25\,\text{m}$ — Minimum anchorage length for anchorage in tension.

$l_{bd.b} := \max(\alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd.b}, l_{b.min.b}) = 0.416\,\text{m}$ — Design anchorage length

Bending reinforcement in the top

The cross-section is turned upside down in order to calculate the reinforcement amount

Chosen dimension of the bars

$\phi_t := 25\text{mm}$ — Diameter of the bars for the bottom reinforcement

$A_{si.t} := \dfrac{\phi_t^{\,2} \cdot \pi}{4} = 4.909 \times 10^{-4}\,\text{m}^2$ — Area of each reinforcement bar

Indata from Eurocode 2 SS-EN 1992-1-1:2005 8.2 (2)

$k_1 := 1 \qquad k_2 := 5\text{mm}$ — Factors from EC2

$d_g := 32\text{mm}$ — Assumed size of the aggregates

$c_{bar.t} := \max(d_g + k_2, 20\text{mm}, k_1 \cdot \phi_t) = 0.037\,\text{m}$ — Distance between bars and layers

$\Delta c_{dev} := 10\text{mm}$ — Factor from EC2

$c_{min.t} := \phi_t + \Delta c_{dev} = 0.035\,\text{m}$ — Minimum concrete cover

$c_{cover.t} := c_{min.t}$ — Chosen concrete cover

Maximum number of bars that fits in the element

The top reinforcement is only placed in the leg

$$n_{max.bars.leg.t} := \mathrm{floor}\left(\frac{b_{leg} - 2\cdot c_{cover.t} + c_{bar.t}}{\phi_t + c_{bar.t}}\right) = 11$$

Number of bars that fits into one layer in the leg

Number of bars needed

$$n_{bars.t} := 13$$

Number of bars needed in the section, iteratively changed according to calculations below. In order to achieve the correct centre of gravity for the bars

$$n_{layer.t} := \mathrm{ceil}\left(\frac{n_{bars.t}}{n_{max.bars.leg.t}}\right) = 2$$

Total number of layers needed

Centre of gravity for the bars

$$y_{tp.t} := \begin{cases} \left(c_{cover.t} + \dfrac{\phi_t}{2}\right) & \text{if } n_{layer.t} \le 1 \\[2ex] \dfrac{n_{max.bars.leg.t}\cdot\left(c_{cover.t} + \dfrac{\phi_t}{2}\right) + \left(n_{bars.t} - n_{max.bars.leg.t}\right)\cdot\left(c_{cover.t} + \dfrac{3}{2}\cdot\phi_t + c_{bar.t}\right)}{n_{bars.t}} & \text{if } 1 < n_{layer.t} \\[2ex] \dfrac{n_{max.bars.leg.t}\cdot\left(c_{cover.t} + \dfrac{\phi_t}{2}\right) + \left(n_{max.bars.leg.t}\right)\cdot\left(c_{cover.t} + \dfrac{3}{2}\cdot\phi_t + c_{bar.t}\right) \ldots + \left[\left(n_{bars.t} - 2\cdot n_{max.bars.leg.t}\right)\cdot\left(c_{cover.t} + \dfrac{5}{2}\cdot\phi_t + 2c_{bar.t}\right)\right]}{n_{bars.t}} & \text{if } 2 < n_{layer.t} \\[2ex] \dfrac{n_{max.bars.leg.t}\cdot\left(c_{cover.t} + \dfrac{\phi_t}{2}\right) + \left(n_{max.bars.leg.t}\right)\cdot\left(2c_{cover.t} + \dfrac{8}{2}\cdot\phi_t + 3c_{bar.t}\right) \ldots + \left[\left(n_{bars.t} - 3\cdot n_{max.bars.leg.t}\right)\cdot\left(c_{cover.t} + \dfrac{7}{2}\cdot\phi_t + 3c_{bar.t}\right)\right]}{n_{bars.t}} & \text{if } 3 < n_{layer.t} \le 4 \end{cases}$$

$$y_{tp.t} = 0.057\,\mathrm{m}$$

$$d_{t_i} := \Delta h_i - y_{tp.t}$$

Effective depth, from top of the section to the tensile resultant. Along vector i.

$$d_{t.J_j} := d_{t_{j+i_{r.fc}}}$$

Effective depth, from top of the section to the tensile resultant. Along vector j.

The need for top bending reinforcement

The required moment is taken in the maximum moment section, fc. The reinforcement amount is then calculated for the section cp where the leg and flange starts. Sections fc and cp has the same available height for placement of reinforcement. The reinforcement is placed according to the cross-section in section cp. Since the reinforcement continues into the centrepiece, it will therefore have sufficient capacity also in the maximum section fc.

$A_{s.t} := \dfrac{M_{n.1.max}}{0.9 \cdot d_{t.J_0} \cdot f_{yd}} = 6.161 \times 10^{-3}\, m^2$

Needed amount of reinforcement in section cp, calculated for the maximum moment in

$n_{bars.t} := \dfrac{A_{s.t}}{A_{si.t}} = 12.551$

Number of bars needed in section cp, based on the maximum moment in fc.

$n_{bars.t} := \mathrm{ceil}(n_{bars.t}) = 13$

Number of bars needed.

$A_{Rd.t} := A_{si.t} \cdot n_{bars.t} = 6.381 \times 10^3 \cdot mm^2$

Actual amount of reinforcement with the dimension of the bars given as indata above.

$M_{n.Rd} := 0.9 \cdot d_{b.J_0} \cdot f_{yd} \cdot A_{Rd.t} = 5.624 \times 10^3 \cdot kN \cdot m$

Bending moment capacity for the calculated number of bars

$\dfrac{M_{n.1.max}}{M_{n.Rd}} = 1.004$

Utilisation

Needed anchorage length

The calculation of needed anchorage length is done according to SS-EN 1992-1-1:2005 section 8.4

$\eta_1 := 1.0$

Coefficient related to the quality of the bond condition. =1.0 for "good" conditions, =0.7 for all other conditions. 1.0 is chosen due to the criteria d in EC

$$\eta_2 := \begin{cases} 1.0 & \text{if } \phi_t \le 32mm \\ \dfrac{\left(132 - \dfrac{\phi_t}{mm}\right)}{100} & \text{otherwise} \end{cases} = 1$$

Coefficient related to the bar diameter

$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 4.35 \cdot MPa$

Design value of the ultimate bond stress

The calculation of the required anchorage length is base on EC 8.4.3 (2), using the yield stress of the steel bars, σ_{sd} is set to the yield stress f_{yd}. This is an assumption on the safe side, since the bars are not fully utilised in all sections of the element.

$l_{b.rqd.t} := \left(\dfrac{\phi_t}{4}\right) \cdot \left(\dfrac{f_{yd}}{f_{bd}}\right) = 0.625\, m$

Basic required anchorage length.

Coefficient α_1 - α_5 is given in table 8.2

$\alpha_1 := 1.0$

Effect of the form of the bars, assuming adequate cover, according to EC

$a := \dfrac{b_{leg} - 2 \cdot c_{cover.b}}{n_{max.bars.leg.t} - 1} = 0.068\, m$

Distance between bars in the critical section

$c_d := \min\left(\dfrac{a}{2}, c_{cover.t}\right) = 0.034\, m$

Effect of concrete minimum cover, according to EC

$$\alpha_2 := \begin{cases} 0.7 & \text{if } 1 - 0.15\cdot\dfrac{(c_{cover.t} - \phi_t)}{\phi_b} \le 0.7 \\ 1 - 0.15\cdot\dfrac{(c_d - \phi_t)}{\phi_t} & \text{if } 0.7 < 1 - 0.15\cdot\dfrac{(c_{cover.t} - \phi_t)}{\phi_t} < 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.946$$

α_3 and α_5 are assumed to be 1.0, which is on the safe side

$\alpha_3 := 1.0$ — Effect of confinement by transverse reinforcement, set to 1.0 (safe side)

$\alpha_5 := 1.0$ — Effect of pressure transverse to the plane of splitting along the design anchorage length, set to1.0 (safe side).

$\alpha_4 := 0.7$ — Influence of welded transverse bars, set to 0.7 according to EC

$\text{Check} := \alpha_2\cdot\alpha_3\cdot\alpha_5 > 0 = 1$ — Check ok

$l_{b.min.t} := \max(0.3\cdot l_{b.rqd.t}, 10\cdot\phi_b, 100mm) = 0.25\,m$ — Minimum anchorage length for anchorage in tension.

$l_{bd.t} := \max(\alpha_1\cdot\alpha_2\cdot\alpha_3\cdot\alpha_4\cdot\alpha_5\cdot l_{b.rqd.t}, l_{b.min.t}) = 0.414\,m$ — Design anchorage length

Transversal bending reinforcement in the flanges

Loads and moments in the bottom flange - only the self-weight of the plate and fill is considered

Load acting on the flange, calculated per meter at the worst place (outermost part). Loads are: weight of the soil, self-weight of the slab

$$q_{d.flange} := (h_{flange}\cdot\rho_c + h_{fill.flange}\cdot\rho_{fill})1m = 49.519\cdot\frac{kN}{m}$$

$$M_{Ed} := q_{d.flange}\cdot\frac{\left(\dfrac{b_{flange}}{2}\right)^2}{2} = 24.76\cdot kN\cdot m$$ Design moment

Reinforcement amount in the bottom flange

$$A_s := \frac{M_{Ed}}{f_{yd}\cdot 0.9\cdot h_{flange}} = 1.055\times 10^{-4}\,m^2$$ Needed amount of reinforcement

$\phi_{bar.flange} := 16mm$ — Assumed bar diameter

$$A_{si} := \phi_{bar.flange}^{\,2}\cdot\frac{\pi}{4} = 2.011\times 10^{-4}\,m^2$$ Area of one bar

$$n_{bar} := \frac{A_s}{A_{si}} = 0.525$$ Number of bars needed per meter

Loads and moments in the bottom flange - the self-weight of fill and plate and the soil pressure acting on the plate is considered

Load acting on the flange, calculated per meter at the worst place (outermost part). Loads are: weight of the fill, self-weight of the slab and the soil pressure action on the flange

$$q_{d.flange.soil} := -\left(h_{flange}\cdot\rho_c + h_{fill.flange}\cdot\rho_{fill}\right)1m + \sigma_{soil.wind.1}\cdot\frac{b_{flange}}{2} = 847.523\cdot\frac{kN}{m}$$

$$M_{Ed} := q_{d.flange.soil}\cdot\frac{\left(\frac{b_{flange}}{2}\right)^2}{2} = 423.762\cdot kN\cdot m$$ Design moment

Reinforcement amount in the bottom flange

$$A_s := \frac{M_{Ed}}{f_{yd}\cdot 0.9\cdot h_{flange}} = 1.805\times 10^{-3}\,m^2$$ Needed amount of reinforcement

$$\phi_{bar.flange} := 16mm$$ Assumed bar diameter

$$A_{si} := \phi_{bar.flange}^{\;2}\cdot\frac{\pi}{4} = 2.011\times 10^{-4}\,m^2$$ Area of one bar

$$n_{bar} := \frac{A_s}{A_{si}} = 8.977$$ Number of bars needed per meter

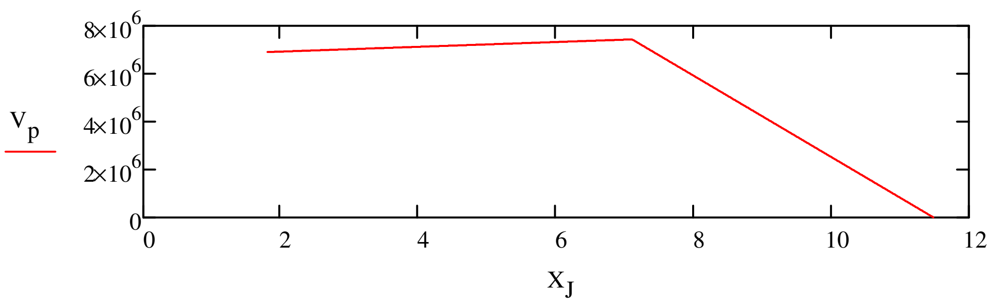
6. Shear force distribution in the legs

The shear force distribution is calculated in the same manner as for the moment distribution. Summing up the shear force acting on each strip, into a shear force distribution vector V_p. Only the shear force distribution in leg 1 is calculated since this value is used for dimensioning the reinforcement, since it has the largest magnitude. The shear force distribution is calculated for both the leeward and the windward side.

Shear force for the leeward side

Shear force distribution from the section fc to the edge of the foundation

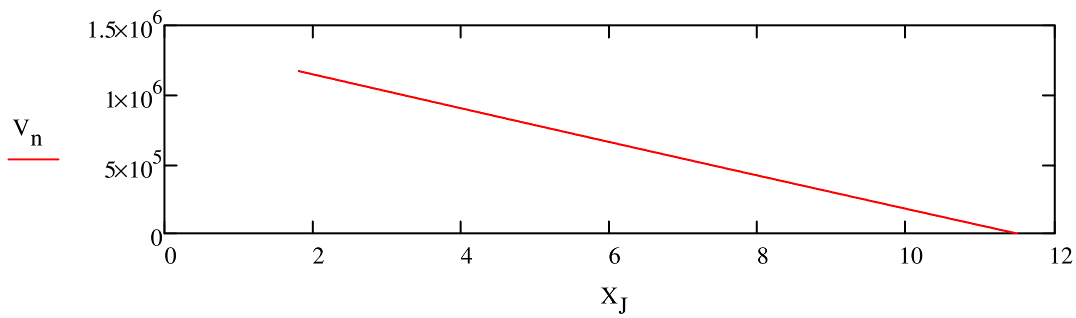
$$V_p := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad V_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left(N_{res.1_i}\right) \\ V \end{array} \right.$$



Shear force for the windward side

Shear force distribution from the section fc to the edge of the foundation

$$V_n := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad V_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left(N_{dead.n_i}\right) \\ V \end{array} \right.$$



Dimensioning shear force in the critical sections

Dimensioning shear force at the section r_{fc} , where the force couple is acting

$$V_{Ed.r.fc} := \max\left(V_{p_0}, V_{n_0}\right) = 6.917 \times 10^3 \cdot kN$$

Dimensioning shear force at l_{sp} , where the soil pressure zone starts

$$V_{Ed.l.sp} := V_{p\left(i_{soil.1}-i_{r.fc}\right)} = 7.444 \times 10^3 \cdot kN$$

Shear force reinforcement, in cut fc

$V_{Ed.r.fc} = 6.917 \times 10^6 N$ — Required shear force capacity in section fc

Check if the shear capacity is sufficient without shear reinforcement

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.2

Calculations are done in order to investigate if shear reinforcement is needed in section r_{fc}

$d_{r.fc} := d_{b_{i_{r.fc}}} = 2.252 m$ — Distance from the top to the centre of gravity of the bottom bending reinforcement

$z_{r.fc} := 0.9 \cdot d_{r.fc} = 2.027 m$ — Internal lever arm

$C_{Rd.c} := \dfrac{0.18}{\gamma_c} = 0.12$ — National parameter, recommended value

$$k := \min\left(1 + \sqrt{\frac{200}{\frac{d_{r.fc}}{mm}}}, 2\right) = 1.298$$

$A_{sl} := A_{Rd.b} = 6.185 \times 10^4 \cdot mm^2$ — Cross-sectional area of fully anchored main reinforcement in tensile zone

$b_w := b_{leg} = 0.75 m$ — Least cross-sectional width in side tensile part of cross-section.

$$\rho_l := \min\left(\frac{A_{Rd.b}}{b_{leg} \cdot d_{r.fc}}, 0.02\right) = 0.02$$

$\nu_{min} := 0.035 k^{\frac{3}{2}} \cdot \sqrt{\dfrac{f_{ck}}{MPa}} \cdot MPa = 2.835 \times 10^5 Pa$ — National parameter, recommended expression

Shear capacity without shear reinforcement

$$V_{Rd.r.fc} := \max\left[C_{Rd.c} \cdot k \cdot \left(100 \rho_l \cdot \frac{f_{ck}}{MPa}\right)^{\frac{1}{3}} \cdot b_w \cdot d_{r.fc} \cdot MPa, \nu_{min} \cdot b_w \cdot d_{r.fc}\right] = 1.03 \times 10^3 \cdot kN$$

$\dfrac{V_{Ed.r.fc}}{V_{Rd.r.fc}} = 6.716$ — Utilisation

Shear reinforcement is needed! Therefore further calculations must be performed in order to calculate the required reinforcement area.

Shear force capacity

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.3
The required shear force capacity is used to calculated the required reinforcement area.

$\theta := 45\text{deg}$ — Choice of shear angle. The angle must be 21.8 deg < θ < 45 deg.

$s_{r.fc} := 1\text{m}$ — Distance between stirrups in the direction of the legs

$$A_{sw.r.fc} := \frac{V_{Ed.r.fc} \cdot s_{r.fc}}{z_{r.fc} \cdot f_{yd} \cdot \cot(\theta)} = 7.849 \times 10^{3} \cdot \text{mm}^{2}$$

Required shear reinforcement area

$$\phi_{r.fc} := \text{ceil}\left(\sqrt{\frac{2A_{sw.r.fc}}{\pi} \cdot \frac{1}{\text{mm}^{2}}}\right) \cdot \text{mm} = 71 \cdot \text{mm}$$

For members with vertical shear reinforcement, the maximum shear resistance is calculated

$\alpha_{cw} := 1.0$ — National parameter, no prestressing

$b_{w} := b_{leg}$ — Least cross-sectional width between compressive and tensile part of cross-section.

$$\nu_{1} := 0.6 \cdot \left(1 - \frac{f_{ck}}{250\text{MPa}}\right) = 0.528$$

Reduction factor for the compressive strength of the concrete, national parameter

Shear capacity

$$V_{Rd.r.fc} := \alpha_{cw} \cdot b_{w} \cdot z_{r.fc} \cdot \nu_{1} \cdot f_{cd} \cdot \frac{1}{\tan(\theta) + \cot(\theta)} = 8.027 \times 10^{3} \cdot \text{kN}$$

$$\frac{V_{Ed.r.fc}}{V_{Rd.r.fc}} = 0.862$$

Utilisation

The utilisation of the shear force capacity ok, this means that the calculated shear reinforcement is sufficient in ULS in section r_{fc}

Shear force reinforcement, in cut l_{sp}

Shear force reinforcement is also designed for the section where the soil pressure zone starts, section l_{sp} , see figure below for the definition of the section.

$V_{Ed.l.sp} = 7.444 \times 10^{6}\,\text{N}$ — Required capacity of the shear force in section sp

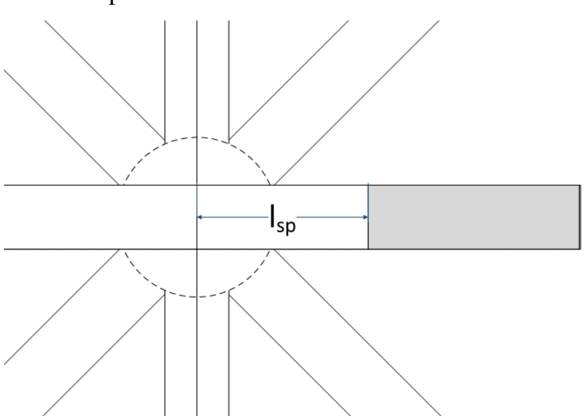


Figure 17: Definition of the section l_{sp}

Check if the shear capacity is sufficient without shear reinforcement

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.2
Calculations are done in order to investigate if shear reinforcement is needed in section l_{sp}

$d_{l.sp} := d_b\left(i_{soil.1}\right) = 1.997\,m$ — Distance from the top to the centre of gravity of the bottom reinforcement

$z_{l.sp} := 0.9 \cdot d_{l.sp} = 1.797\,m$ — Internal lever arm

$C_{Rd.c} := \frac{0.18}{\gamma_c} = 0.12$ — National parameter, recommended value

$$k := \min\left(1 + \sqrt{\frac{200}{\frac{d_{l.sp}}{mm}}}, 2\right) = 1.316$$

$A_{sl} := A_{Rd.b}$ — Cross-sectional area of fully anchored main reinforcement in tensile zone

$b_w := b_{leg}$ — Least cross-sectional width in side tensile part of cross-section.

$$\rho_l := \min\left(\frac{A_{Rd.b}}{b_{leg} \cdot d_{l.sp}}, 0.02\right) = 0.02$$

$$\nu_{min} := 0.035k^{\frac{3}{2}} \cdot \sqrt{\frac{f_{ck}}{MPa}} \cdot MPa = 2.896 \times 10^5\,Pa$$ National parameter, recommended expression

Shear capacity without shear reinforcement

$$V_{Rd.l.sp} := \max\left[C_{Rd.c} \cdot k \cdot \left(100\rho_l \cdot \frac{f_{ck}}{MPa}\right)^{\frac{1}{3}} \cdot b_w \cdot d_{l.sp} \cdot MPa, \nu_{min} \cdot b_w \cdot d_{l.sp}\right] = 926.339 \cdot kN$$

$\frac{V_{Ed.l.sp}}{V_{Rd.l.sp}} = 8.036$ — Utilisation

Shear reinforcement is needed! Therefore further calculations must be performed in order to decide the reinforcement area

Shear force capacity

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.3. The required shear force capacity is used to calculated the required reinforcement area.

$\theta := 45deg$ — Choice of shear angle. The angle must be 21.8 deg < θ < 45 deg.

$s_{l.sp} := 1m$ — Distance between stirrups in the direction of the legs

$$A_{sw.l.sp} := \frac{V_{Ed.l.sp} \cdot s_{l.sp}}{z_{l.sp} \cdot f_{yd} \cdot \cot(\theta)} = 9.526 \times 10^3 \cdot mm^2$$ Required shear reinforcement area

$$\phi_{l.sp} := \text{ceil}\left(\sqrt{\frac{2A_{sw.l.sp}}{\pi} \cdot \frac{1}{mm^2}}\right) \cdot mm = 78 \cdot mm$$

For members with vertical shear reinforcement, the maximum shear resistance is calculated

$\alpha_{cw} := 1.0$ National parameter, no prestressing

$b_w := b_{leg}$ Least cross-sectional width between compressive and tensile part of cross-section.

$\nu_1 := 0.6 \cdot \left(1 - \frac{f_{ck}}{250MPa}\right) = 0.528$ Reduction factor for the compressive strength of the concrete, national parameter

Shear capacity

$$V_{Rd.l.sp} := \alpha_{cw} \cdot b_w \cdot z_{l.sp} \cdot \nu_1 \cdot f_{cd} \cdot \frac{1}{\cot(\theta) + \tan(\theta)} = 7.118 \times 10^3 \cdot kN$$

$\frac{V_{Ed.l.sp}}{V_{Rd.l.sp}} = 1.046$ Utilisation

This is close enough, perhaps it is necessary in a more detail dimensioning to add compressive reinforcement.

7. Global analysis

Loads acting on the foundation, for the global analysis

In the global analysis of the foundation the whole foundation and all the loads acting on the foundation is considered, see Figure below.

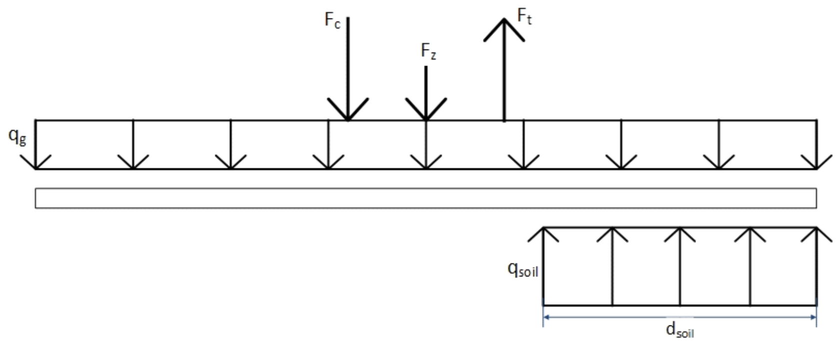


Figure 18: Loads acting on the foundation

Loads from the tower

Transformation of the moment into a force couple; The bending moment is transformed into one compressive force and one tensile force. The force couple resultants are assumed to be acting in the centre of gravity of the arc of the tower, as described above.
The normal force from the tower is divided in four parts, one part is applied at each component of the force couple. Two parts of if, ie half of the normal force, is applied in the centre of the beam.

$$F_c := \frac{M_d + H_d \cdot h_{centrepiece}}{\phi_{fc}} + \frac{\gamma_{dead} N_k}{4} = 28.779 \cdot MN$$

Compressive component of the force couple

$$F_t := \frac{M_d + H_d \cdot h_{centrepiece}}{\phi_{fc}} - \frac{\gamma_{dead} N_k}{4} = 27.159 \cdot MN$$

Tensile component of the force couple

$$F_z := \frac{\gamma_{dead} N_k}{2} = 1.62 \cdot MN$$

Half of the normal force acting in the centre of the element

The bending moment in the foundation is now calculated for the whole foundation, globally, in order to find the global moment distribution. For the global moment distribution the whole foundation is included in the analysis.

Self-weight

$$N_{dead.tot.p_j} := \frac{0.9G_k}{\frac{l_{tot}}{\Delta x}} = ...$$

A vector with the dead load acting on each strip, when calculating the positive moment. The dead load is uniformly spread over the length of the foundation.

$$N_{dead.tot.n_j} := \frac{1.1G_k}{\frac{l_{tot}}{\Delta x}}$$

A vector with the dead load acting on each strip, when calculating the negative moment. The dead load is uniformly spread over the length of the foundation.

Vector with the soil pressure for each strip. The soil pressure is zero outside the soil pressure area. The soil pressure from the two secondary legs are acting on a shorter length than leg 1.

$$\sigma_{soil.1_j} := \begin{cases} 0 & \text{if } X_{J_j} < \dfrac{l_{tot}}{2} - l_{soil.1} \\ \sigma_{soil.wind.1} & \text{otherwise} \end{cases}$$

Soil pressure contribution from leg 1.

$$\sigma_{soil.2_j} := \begin{cases} \sigma_{soil.wind.1} & \text{if } \dfrac{l_{tot}}{2} - l_{soil.1} < X_{J_j} < \dfrac{l_{tot}}{2} - l_{soil.1} + l_{soil.2} \\ 0 & \text{otherwise} \end{cases}$$

Soil pressure contribution from the secondary legs.

Soil pressure

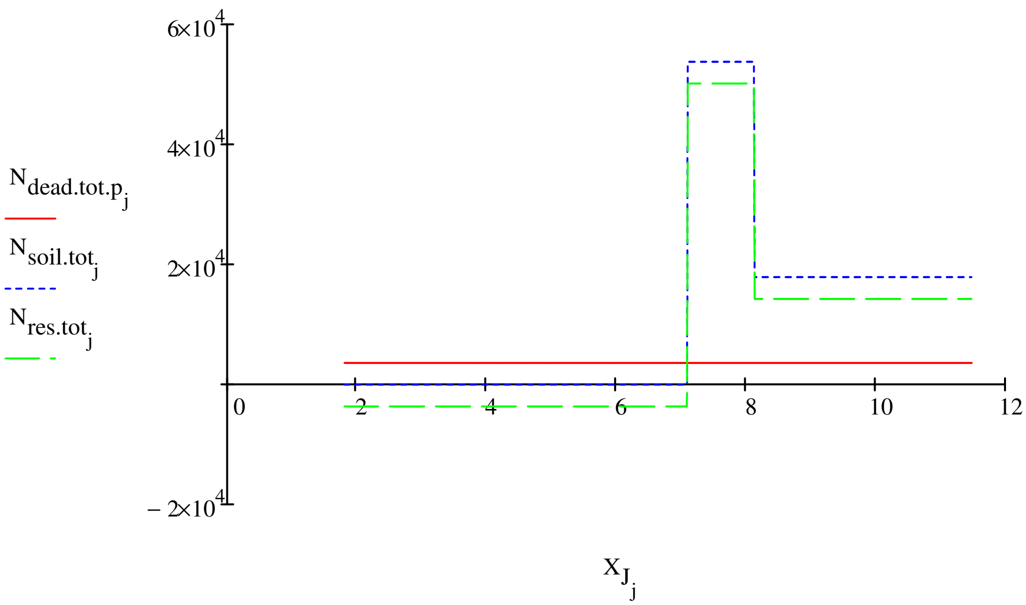
$$N_{soil.tot_j} := \sigma_{soil.1_j} \cdot b_{flange} \cdot \Delta x + 2\,\sigma_{soil.2_j} \cdot b_{flange} \cdot \Delta x = ...$$

Vector with the resultant soil pressure as a total force acting on each strip

$$N_{res.tot_j} := N_{soil.tot_j} - N_{dead.tot.p_j} = ...$$

Resulting force vector, when dead weight and soil pressure is considered

Summary of the loads acting on the foundation, globally



Moment distribution in the foundation - globally, from the cut fc to the edge

Positive moment distribution, on the leeward side

The moment distribution, M_p is calculated from the cut fc to the end of the foundation. The positive moment is defined on the leeward side
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side, The resulting force, N_{res}, for each strip is multiplied with its lever arm, s_J, for its respective element and then summed up for the all the elements into the positive moment M_p .

Moment distribution from r_{fc} to the edge of the foundation

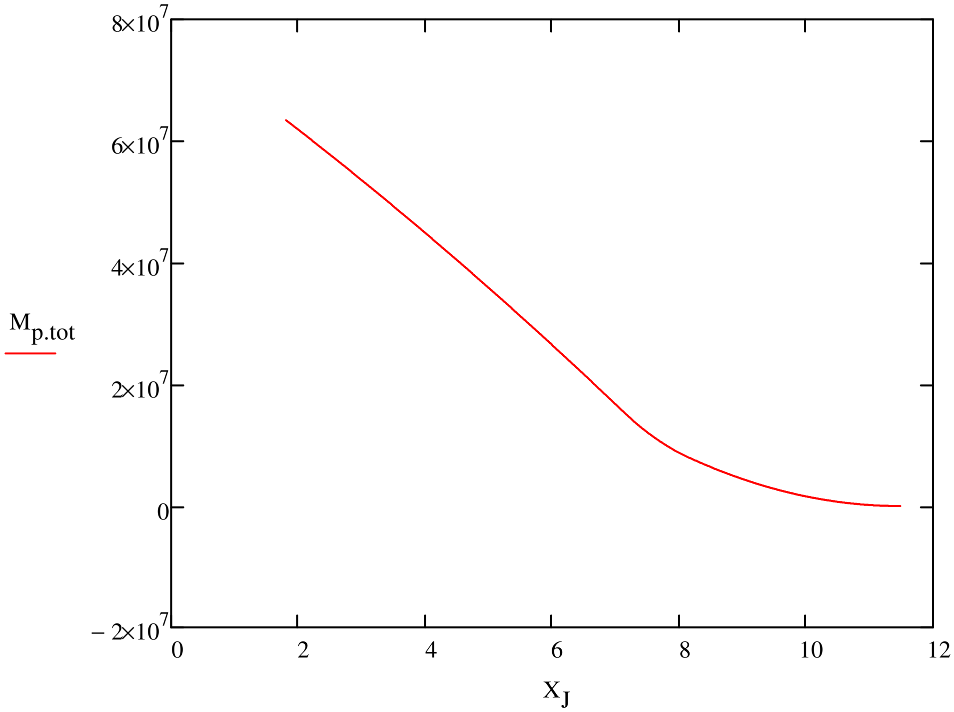
$$M_{p.tot} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{res.tot_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$$M_{p.tot.max} := M_{p.tot_0} = 6.347 \times 10^4 \cdot kN \cdot m$$

Moment in critical cut, r_{fc}, gives the maximum moment on the positive side

Moment diagram of the positive moment



Negative distribution, on the windward side

The moment distribution, M_n is calculated from the cut, r_{fc} to the edge of the foundation. It is defined on the windward side.
It is calculated in steps, with the length Δx, from fc to the edge of the foundation on the compressed side,
The resulting force, $N_{dead.tot.n}$, for each element is multiplied with the lever arm, s_J, for its respective element and then summed up for the all the elements into the negative moment M_n .

Moment distribution from r_{fc} to the edge of the foundation

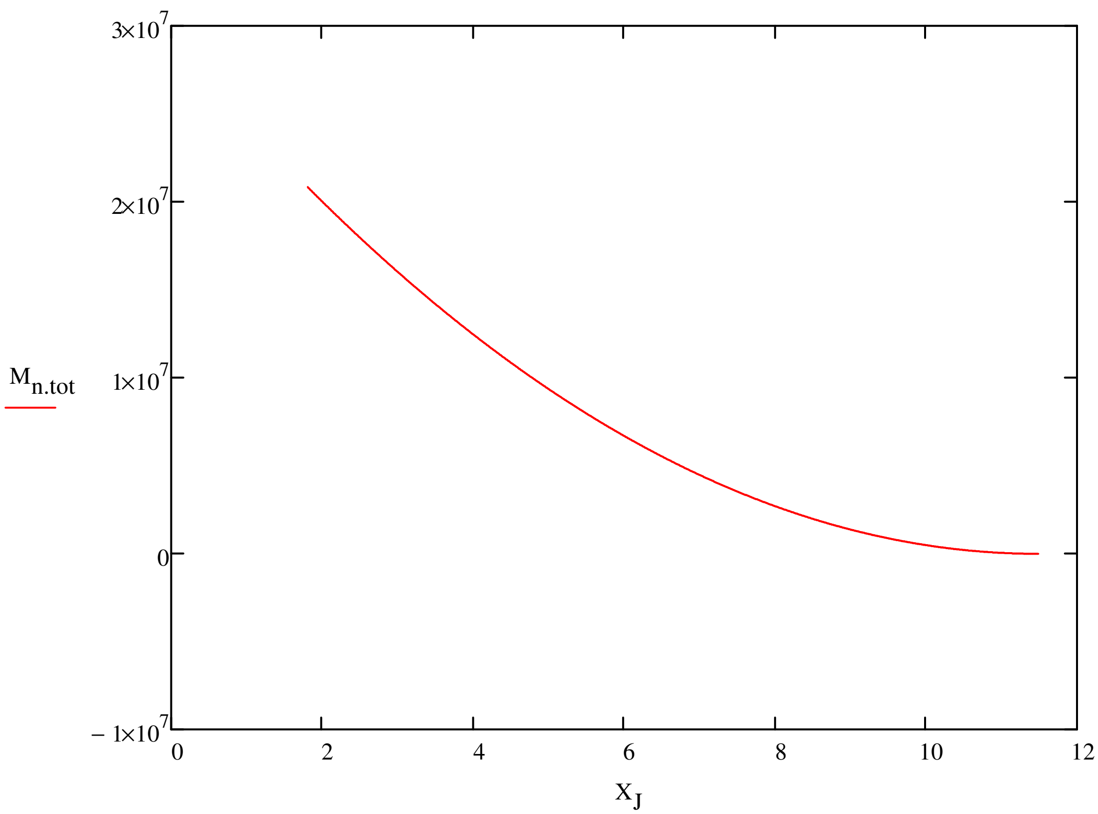
$$M_{n.tot} := \left| \begin{array}{l} \Delta \leftarrow \dfrac{\dfrac{l_{tot}}{2} - r_{fc}}{\Delta x} - 2 \\ \text{for } k \in 0..\Delta \\ \quad M_k \leftarrow \displaystyle\sum_{i=k}^{\Delta} \left[N_{dead.tot.n_i} \cdot \left(s_{J_i} - \frac{k}{\Delta} \cdot \max(s_J) \right) \right] \\ M \end{array} \right.$$

Maximum moment

$$M_{n.tot.max} := M_{n.tot_0} = 2.085 \times 10^4 \cdot kN \cdot m$$

Moment in critical cut, r_{fc}, gives the maximum moment on the negative side

Moment diagram of the negative moment



Moment distribution between the elements in the centrepiece

In an approximated manner it is assumed that the moment distribution is linear between the maximum positive moment and the maximum negative moment. The negative minimum moment and positive maximum moment, are both acting at the distance r.fc from the centre of the foundation. The approximated moment distribution in the centrepiece is plotted.

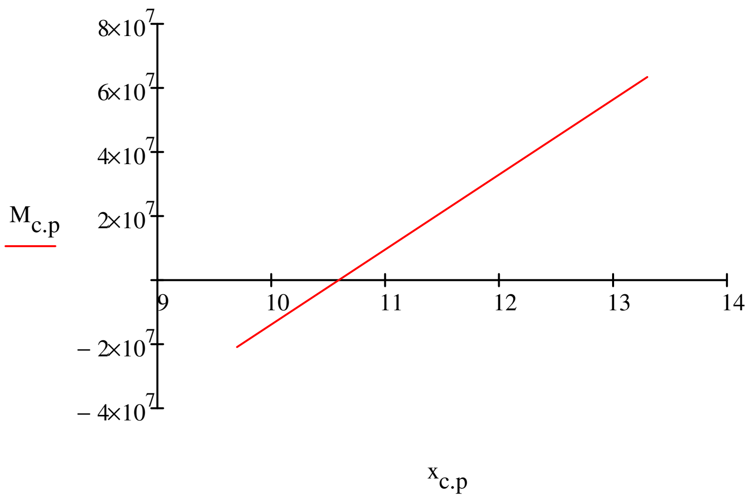
$$M_{c.p} := \begin{pmatrix} -M_{n.tot.max} \\ M_{p.tot.max} \end{pmatrix}$$

The minimum negative moment and the maximum positive moment

$$x_{c.p} := \begin{pmatrix} \frac{l_{tot}}{2} - r_{fc} \\ \frac{l_{tot}}{2} + r_{fc} \end{pmatrix} = \begin{pmatrix} 9.7 \\ 13.3 \end{pmatrix} m$$

The location of the maximum moment and the negative moment along the foundation

Moment distribution between F_c and F_t in the centrepiece



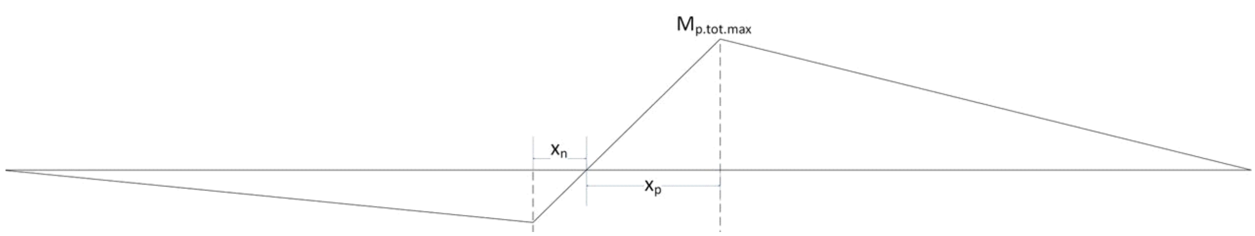


Figure 19: Principle distribution of the moment, with approximate distribution in the centrepiece, between the force resultant.

Distance from the maximum moment and the minimum moment to the zero moment

$$x_n := \frac{M_{n.tot.max}}{M_{p.tot.max} + M_{n.tot.max}} \cdot \phi_{fc} = 0.89\,m$$

Distance from maximum positive moment to zero moment section

$$x_p := \frac{M_{p.tot.max}}{M_{p.tot.max} + M_{n.tot.max}} \cdot \phi_{fc} = 2.711\,m$$

Distance from maximum negative moment to zero moment section

Shear force distribution in the centrepiece

The shear force distribution, conceptually in a wind power plant foundation

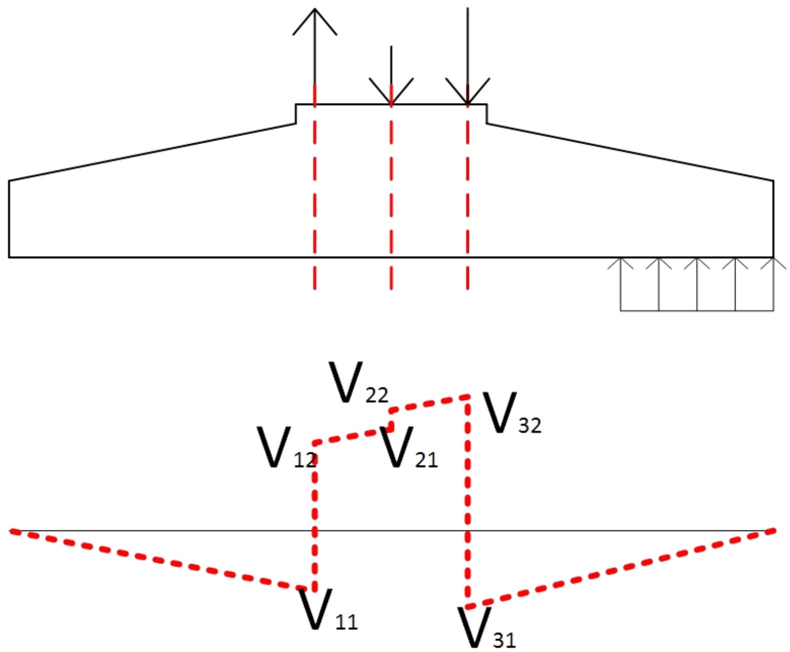


Figure 20: A conceptual sketch over the shear force distribution. The shear force is critical between the force couple, and it is characterized by the critical points V_{11}, V_{12}, V_{21}, V_{22}, V_{31}, V_{32} *as illustrated.*

Shear forces in the critical points

Calculated for the points defined in the figure above

$$V_{11} := -\sum N_{dead.n} = -1.172 \times 10^{3} \cdot kN$$

V_{11} is calculated by summing the dead load

$$V_{12} := F_t - V_{11} = 2.833 \times 10^{4} \cdot kN$$

$$V_{31} := -\sum N_{res.1} = -6.917 \times 10^{3} \cdot kN$$

V_{31} is calculated by summing the resultant loads

$$V_{32} := F_c - V_{31} = 3.57 \times 10^{4} \cdot kN$$

$$V_{22} := V_{32} + \frac{-V_{32} + V_{12} + F_z}{\phi_{fc}} \cdot \frac{\phi_{fc}}{2} = 3.282 \times 10^{4} \cdot kN$$

$$V_{21} := V_{22} - F_z = 3.12 \times 10^{4} \cdot kN$$

For plotting of the shear force

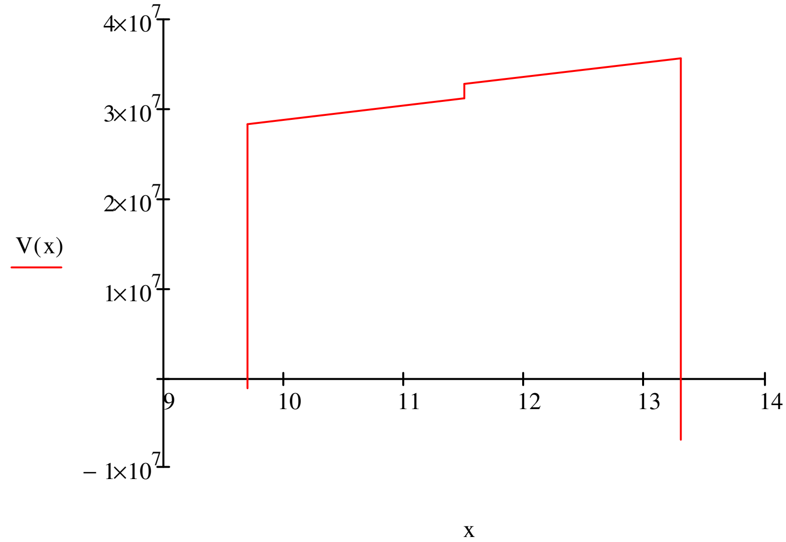
$$V(x) := \begin{pmatrix} V_{11} & V_{12} & V_{21} & V_{22} & V_{32} & V_{31} \end{pmatrix}^T$$

Vector with the critical points

$$x := \begin{pmatrix} \frac{l_{tot} - \phi_{fc}}{2} & \frac{l_{tot} - \phi_{fc}}{2} & \frac{l_{tot}}{2} & \frac{l_{tot}}{2} & \frac{l_{tot} + \phi_{fc}}{2} & \frac{l_{tot} + \phi_{fc}}{2} \end{pmatrix}^T$$

Vector with the position of the critical points

Shear force diagram in the tower between the force couple



Dimensioning shear force in the critical sections

Dimensioning shear force at the middle of the tower

$$V_{Ed.mid} := \max\left(\left|V_{11}\right|, \left|V_{12}\right|, \left|V_{21}\right|, \left|V_{22}\right|, \left|V_{31}\right|, \left|V_{32}\right|\right) = 3.57 \times 10^{4} \cdot kN$$

Shear force reinforcement, in the middle of the tower

Shear force capacity

The calculations are done according to SS-EN 1992-1-1:2005 section 6.2.3
The required shear force capacity is used to calculated the required reinforcement area.

$V_{Ed.mid} = 3.57 \times 10^{7}\,N$ Maximum shear force inside the tower

$v_{Ed.mid} := \dfrac{V_{Ed.mid}}{d_{centrepiece}} = 7.139 \times 10^{3} \cdot \dfrac{kN}{m}$ Shear force spread over the width of the centrepiece

$\theta := 45deg$ Choice of shear angle

$d_{mid} := d_{bi_{mid}} = 2.252\,m$ Height of the foundation inside the tower excluding the reinforcement. Not including the extra height of the centerpiece.

$s_{w.mid} := 1$ Distance between stirrups

$A_{sw.mid} := \dfrac{v_{Ed.mid} \cdot s_{w.mid}}{0.9 \cdot d_{mid} \cdot f_{yd} \cdot \cot(\theta)} = 8.101 \times 10^{3} \cdot \dfrac{mm^2}{m^2}$ Needed reinforcement amount per square meter

$\phi_{c.p} := 16mm$ Assumed diameter of shear reinforcement

$A_{si.c.p} := \dfrac{\phi_{c.p}^{2} \cdot \pi}{4} = 201.062 \cdot mm^{2}$ Area of the shear reinforcement

$n_{shear.mid} := \dfrac{A_{sw.mid}}{A_{si.c.p}} = 40.291 \dfrac{1}{m^2}$ Number of shear reinforcement bars per square meter

For members with vertical shear reinforcement, the maximum shear resistance is calculated

$\alpha_{cw.mid} := 1.0$ National parameter, no prestressing

$\nu_{1.mid} := 0.6 \cdot \left(1 - \frac{f_{ck}}{250MPa}\right) = 0.528$ Reduction factor for the compressive strength of the concrete, national parameter

Shear force capacity

$$V_{Rd.mid} := \alpha_{cw.mid} \cdot 0.9 \cdot d_{mid} \nu_{1.mid} \cdot f_{cd} \cdot \frac{1}{\cot(\theta) + \tan(\theta)} = 1.07 \times 10^{4} \cdot \frac{kN}{m}$$

$\frac{v_{Ed.mid}}{V_{Rd.mid}} = 0.667$ Utilisation

The utilisation of the shear force capacity ok, this means that the calculated shear reinforcement is sufficient in ULS in the middle of the section.

Appendix V

Investigation of connections

Table of contents

1. Indata from Appendix III and IV

$\alpha := 45\mathrm{deg}$	Angle between legs
$d_{centrepiece} := 4.83\mathrm{m}$	Diameter of the centrepiece
$b_{flange} := 2\mathrm{m}$	Width of the bottom flange
$l_{leg.8} := 9.5\mathrm{m}$	Length of the legs, 8 legged concept
$l_{leg.20} := 6.5\mathrm{m}$	Length of the legs, 20 legged concept
$h_{plate} := 0.4\mathrm{m}$	Height of the plate
$h_{centrepiece} := 2.8\mathrm{m}$	Height of the centrepiece
$h_{leg} := 1.55\mathrm{m}$	Height of the leg
$b_{leg} := 0.75\mathrm{m}$	Width of the leg
$r_{fc} := 1.8\mathrm{m}$	Distance between force couple and the centre of the foundation

Concrete

$\rho_c := 25\dfrac{\mathrm{kN}}{\mathrm{m}^3}$	Concrete density

Reinforcement

$f_{yd} := 435\mathrm{MPa}$	Steel yield strength
$\phi_b := 25\mathrm{mm}$	Diameter of the bars
$n_{layer.b} := 3$	Number of layers
$c_{cover.b} := 35\mathrm{mm}$	Concrete cover
$c_{layer.b} := 37\mathrm{mm}$	Distance between layers
$n_{max.bars.plate.b} := 31$	Number of bars in a layer in the bottom plate
$l_{lap} := 0.42\mathrm{m}$	Overlapping length

Bending moment distribution

$M_{p.tot.max} := 5.500\cdot 10^4\cdot\mathrm{kN}\cdot\mathrm{m}$	Maximum positive bending moment
$x_p := 2.623\mathrm{m}$	Distance from centre to zero moment, positive side
$x_n := 0.978\mathrm{m}$	Distance from centre to zero moment, negative side
$M_{p.1.x} := 4.099\cdot 10^4\mathrm{kN}\cdot\mathrm{m}$	Composant of moment in x and y-direction, from appendix IV
$M_{p.2.x} := 1.186\times 10^4\ \mathrm{kN}\cdot\mathrm{m}$	Composant of moment in x and y-direction, from appendix IV

2. Wet connection with protruding reinforcement

Needed amount of fresh reinforcement

Investigation of the needed amount of fresh concrete in order to achieve sufficient overlapping length in the connections.

Eight legs, wet connections

$$V_{c.lap.8} := 8 \cdot 2 \cdot h_{leg} \cdot b_{leg} \cdot l_{lap} + d_{centrepiece} \cdot 4 \cdot h_{centrepiece} \cdot l_{lap} = 30.532 \cdot m^3$$

Volume of conrete in concept with eight legs

$$m_{c.lap.8} := V_{c.lap.8} \cdot \frac{\rho_c}{g} = 85.799 \cdot ton$$

Weight of conrete in concept with eight legs

Twenty legs, wet connections

$$V_{c.lap.20} := d_{centrepiece} \cdot 10 \cdot h_{centrepiece} \cdot l_{lap} \cdots + \left(l_{leg.20} - d_{centrepiece}\right) \cdot 10 \cdot l_{lap} \cdot h_{plate} = 59.606 \cdot m^3$$

Volume of conrete in concept with twenty legs

$$m_{c.lap.20} := V_{c.lap.20} \cdot \frac{\rho_c}{g} = 167.501 \cdot ton$$

Weight of conrete in concept with twenty legs

Eight legs, centrepiece onsite cast

$$V_{centrepiece} := \pi \cdot \frac{d_{centrepiece}^{\;2}}{4} \cdot h_{centrepiece} = 51.303 \cdot m^3$$

Volume of the centrepiece

$$V_{c.lap.8.cp} := V_{centrepiece} + \left(l_{leg.20} - d_{centrepiece}\right) \cdot 10 \cdot l_{lap} \cdot h_{plate} = 54.109 \cdot m^3$$

Volume of conrete in concept with eight legs, onsite centrepiece

$$m_{c.lap.8.cp} := V_{c.lap.8.cp} \cdot \frac{\rho_c}{g} = 152.051 \cdot ton$$

Weight of conrete in concept with eight legs, onsite centrepiece

Twenty legs, centrepiece onsite cast

$$V_{c.lap.20.cp} := V_{centrepiece} + \left(l_{leg.8} - d_{centrepiece}\right) \cdot 10 \cdot l_{lap} \cdot h_{plate} = 59.149 \cdot m^3$$

Volume of conrete in concept with twenty legs

$$m_{c.lap.20.cp} := V_{c.lap.20.cp} \cdot \frac{\rho_c}{g} = 166.214 \cdot ton$$

Weight of conrete in concept with twenty legs

From these calculations it is possible to see that the concept with wet connections is only possible for the concept with eight legs. For the concept with twenty legs, the width of the element is thinner than the needed overlapping length, therefore the needed amount of fresh concrete is larger for the wet connections in the centrepiece than for having the whole centrepiece cast onsite. These conclusions are verified according to:

Verification if it is possible to have wet connection in the centrepiece

$$w_{element.20} := \frac{\pi \cdot d_{centrepiece}}{20} = 0.759\,m$$

Width of the element at the intersection between centrepiece and outer part of the foundation.

$$w_{element.mean.20} := \frac{w_{element.20} + 0m}{2} = 0.379\,m$$

Mean width of the element inside the centrepiece

$l_{lap} < w_{element.20} = 1$

Verification if the lep length or mean width of element is largest

$$\frac{l_{lap}}{w_{element.20}} = 0.554$$

Quota between the lap length and the mean width of the element

Therefore, the connections between the elements are only investigated for the concept with eight legs. The concept with twenty legs is only further investigated as a concept with onsite cast centrepiece.

Investigation of connections and its reinforcement, concept with eight legs

If the elements are joined together with with onsite castingaccording to the caclulations above.
The elements are placed with a distance $l_{lap} = 0.42\,\mathrm{m}$ whihc gives sufficient space for the prodtruding reinforcement to overlap, the gap is then filled with fresh concrete in order to get full interaction between the elements.
In order to calculate the bending moment that must be transferred across the edge of the elements, a calculation model is assumed according to the calculations below.

Connection 1

The centrepiece is assumed to be divided into 8 elements, like pieces of a cake, each centrepiece element attached to its corresponding leg. Element 1 is the element opposite the wind, it is the element that is exposed to the largest part of the bending moment.
Connection 1 is the connection between element 1 and element 2, whilst connection 2 is between element 2 and element 3, see figure.

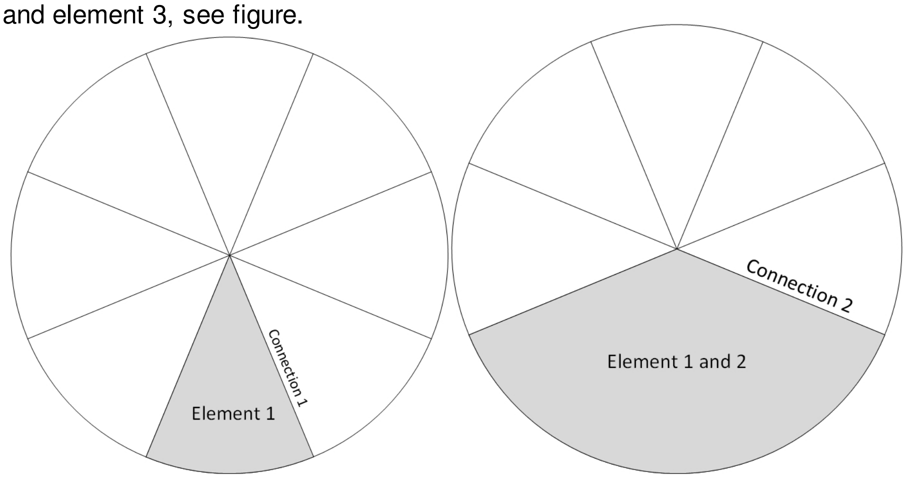


Figure 1: Connection 1 is between element 1 and 2, connection 2 is between element 2 and 3.

Division into segments

We choose to look at the centrepiece in 5 sections, at section fc and then 1/4 at a time inwards towards the centre.

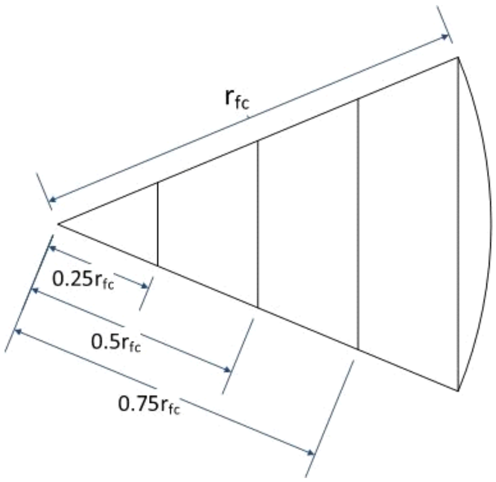


Figure 2: Definition of the sections, where the moment is calculated

In each section, the width will vary, decreasing towards the centre of the foundation where the width is zero.

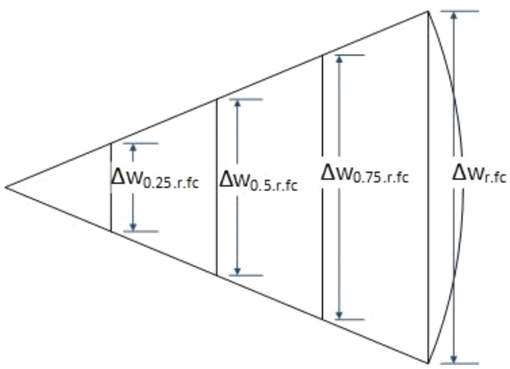


Figure 3: The width will vary for the different secions

Width in the different sections according to Figure above

$$\Delta w_{r.fc} := 2 \cdot \sin\left(\frac{\alpha}{2}\right) \cdot r_{fc} - l_{lap} = 0.958\,\text{m}$$

$$\Delta w_{0.75.r.fc} := 2 \cdot \sin\left(\frac{\alpha}{2}\right) \cdot \frac{3r_{fc}}{4} - l_{lap} = 0.613\,\text{m}$$

$$\Delta w_{0.5.r.fc} := 2 \cdot \sin\left(\frac{\alpha}{2}\right) \cdot \frac{r_{fc}}{2} - l_{lap} = 0.269\,\text{m}$$

$$\Delta w_{0.25.r.fc} := \left| \begin{array}{l} 2\sin\left(\frac{\alpha}{2}\right) \cdot \frac{r_{fc}}{4} - l_{lap} \quad \text{if } 2\sin\left(\frac{\alpha}{2}\right) \cdot \frac{r_{fc}}{4} - l_{lap} > 0 \\ 0 \quad \text{otherwise} \end{array} \right. = 0\,\text{m}$$

$$\Delta w_{0.r.fc} := 0\text{m}$$

Global bending moment in the centrepiece

The global moment will also vary in the five different sections, it will reach zero beyond the centre of the centrepiece so all the sections will have a positive moment. Here is a principle sketch of how the global

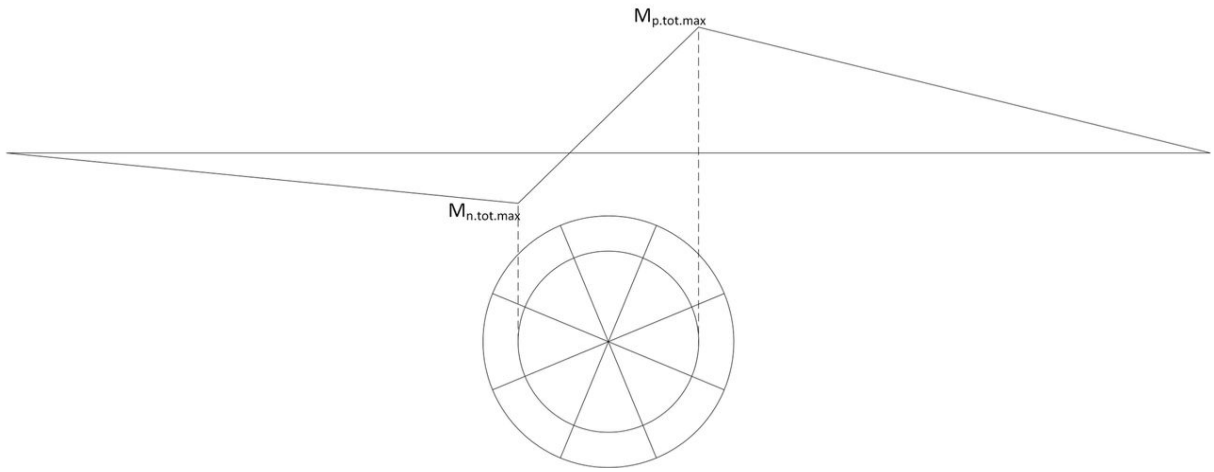


Figure 4: Principle sketch over the variation of the global moment along the foundation.

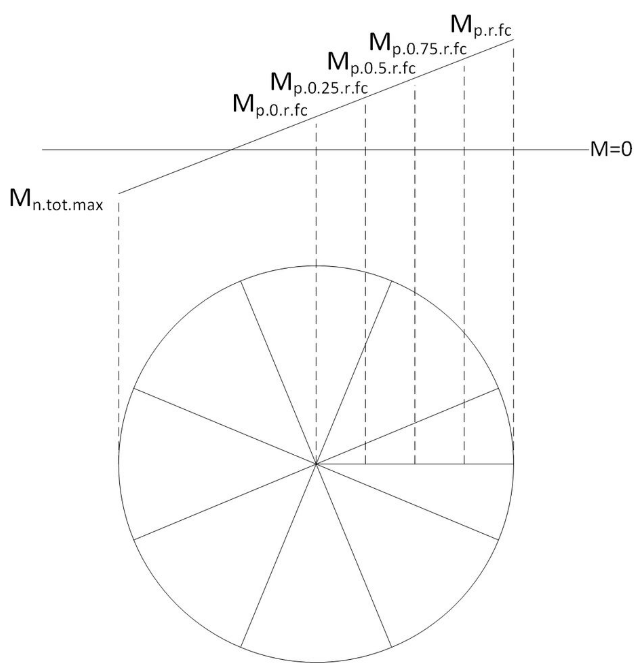


Figure 5: Variation of the moment over the centrepiece, marked for the different sections.

Variation of the bending moment in the different sections

$M_{p.r.fc} := M_{p.tot.max} = 55 \cdot MN \cdot m$ Section r.fc

$M_{p.0.75.r.fc} := \frac{M_{p.tot.max}}{x_p} \cdot \left(x_p - \frac{r_{fc}}{4} \right) = 45.564 \cdot MN \cdot m$ Section 0.75*r.fc

$M_{p.0.5.r.fc} := \frac{M_{p.tot.max}}{x_p} \cdot \left(x_p - \frac{r_{fc}}{2} \right) = 36.128 \cdot MN \cdot m$ Section 0.5.*r.fc

$M_{p.0.25.r.fc} := \frac{M_{p.tot.max}}{x_p} \cdot \left(x_p - 0.75 r_{fc} \right) = 26.693 \cdot MN \cdot m$ Section 0.25*r.fc

$M_{p.0.r.fc} := \frac{M_{p.tot.max}}{x_p} \cdot \left(x_p - r_{fc} \right) = 17.257 \cdot MN \cdot m$ Section 0 (equal to the mid of the centrepiece)

Quota between the moments in the legs

Based on the calculations of the local moment distribution in the legs, a quota between the local moment in leg 1 and the x-composant of the local moment in leg 2 is achieved. This quota is assumed to be valid also for the global moment distribution, and is used for determine the part of the global moment that is taken by element 1, and how much is taken by element 2.

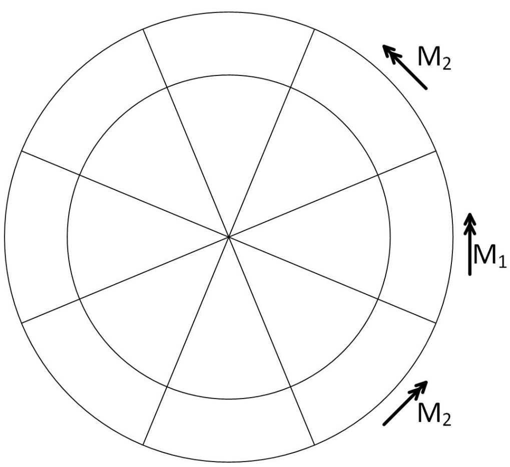


Figure 6: Local moment for leg 1 and leg 2.

$\chi := \frac{M_{p.1.x}}{M_{p.1.x} + 2M_{p.2.x}} = 0.633$

Quota between the moment for leg one and legs
2. Caclulated in the local part of this document.

Part of the global moment that should is taken by element 1

Each element is assumed to take a part of the global moment equal to the quota of the global moment.

$M_{p.r.fc.\chi} := \chi \cdot M_{p.r.fc} = 34.839 \cdot MN \cdot m$

$M_{p.0.75.r.fc.\chi} := \chi \cdot M_{p.0.75.r.fc} = 28.862 \cdot MN \cdot m$

$M_{p.0.5.r.fc.\chi} := \chi \cdot M_{p.0.5.r.fc} = 22.885 \cdot MN \cdot m$

$M_{p.0.25.r.fc.\chi} := \chi \cdot M_{p.0.25.r.fc} = 16.908 \cdot MN \cdot m$

$M_{p.0.r.fc.\chi} := \chi \cdot M_{p.0.r.fc} = 10.931 \cdot MN \cdot m$

The width will decrease to zero over a shorter length than the moment will decrease to zero, when moving from section fc towards the centre of the centrepiece. The ratio of decrease therefore will be higher for the width. This means that the cross section of the element will be thinner, and the residual moment must be transferred over the connection to the neighbouring element.

Moment that must pass over connection 1

The difference between "what is possible to pass through the cross-section of the sections due to the area decrease" and " the part of the global moment that should be taken by each section (that wants to pass through the section)".
This is calculated in the different sections and is the moment that passes over the connection, calculated for each segment.

$$M_{p.r.fc.to.0.75} := M_{p.0.75.r.fc.\chi} - \frac{\Delta w_{0.75.r.fc}}{\Delta w_{r.fc}} \cdot M_{p.r.fc.\chi} = 6.553 \cdot MN \cdot m$$

What must go to the nearby the element, from section fc to 0.75fc.

$$M_{p.0.75.to.0.5} := M_{p.0.5.r.fc.\chi} - \frac{\Delta w_{0.5.r.fc}}{\Delta w_{0.75.r.fc}} \cdot M_{p.0.75.r.fc.\chi} = 10.233 \cdot MN \cdot m$$

What must go to the nearby the element, from section 0.75fc to 0.5fc.

$$M_{p.0.5.to.0.25} := M_{p.0.25.r.fc.\chi} - \frac{\Delta w_{0.25.r.fc}}{\Delta w_{0.5.r.fc}} \cdot M_{p.0.5.r.fc.\chi} = 16.908 \cdot MN \cdot m$$

What must go to the nearby the element, from section 0.5fc to 0.25fc.

$$M_{p.0.25.to.0} := M_{p.0.r.fc.\chi} - \Delta w_{0.r.fc} \cdot M_{p.0.25.r.fc.\chi} = 10.931 \cdot MN \cdot m$$

What must pass through the mid point, but cant and instead must go around.

The mean value between these sections

$$M_{p.mean} := \frac{M_{p.r.fc.to.0.75} + M_{p.0.75.to.0.5} + M_{p.0.5.to.0.25} + M_{p.0.25.to.0}}{4} = 11.156 \cdot MN \cdot m$$

What must pass over the edge as a distributed moment

$$m_{p.mean} := \frac{M_{p.mean}}{r_{fc}} = 6.198 \cdot \frac{MN \cdot m}{m}$$

Calculations of needed reinforcement due to the moment that must pass over the connections

Assume the same indata for reinforcement calculations as for the bottom bending reinforcement in the legs.

Chosen dimension of the bars

$$\phi_b := 25mm$$

Diameter of the bars for the bottom reinforcement

$$A_{si.connection.1} := \frac{\phi_b^{\,2} \cdot \pi}{4} = 4.909 \times 10^{-4} \, m^2$$

Area of each reinforcement bar

In a simplified manner, assume one layer of reinforcement

$$tp := c_{cover.b} + \frac{\phi_b}{2} = 0.048 \, m$$

Centre of gravity for one layer of reinforcement

$$d_{b.connection.1} := h_{centrepiece} - tp = 2.752 \, m$$

The need for bottom bending reinforcement, from r_{fc} to the edge of the foundation

$$A_{s.connection.1} := \frac{m_{p.mean}}{0.9 \cdot d_{b.connection.1} \cdot f_{yd}} = 5.752 \times 10^{-3}\,m$$

Needed amount of reinforcement along connection 1

$$n_{bars.connection.1} := \frac{A_{s.connection.1}}{A_{si.connection.1}} = 11.717\,\frac{1}{m}$$

Number of bars needed

$$n_{bars.connection.1} := \mathrm{ceil}\left(n_{bars.connection.1} \cdot m\right) \cdot \frac{1}{m} = 12\,\frac{1}{m}$$

Rounded number of bars needed

$$s := \frac{1}{n_{bars.connection.1}} = 0.083\,m$$

Distance between the bars in connection 1

Connection 2

When looking at connection 2, elemens 1 and 2 are contributing in transferring the global moment. The element is divided into segments in the same manner as in connection 1.

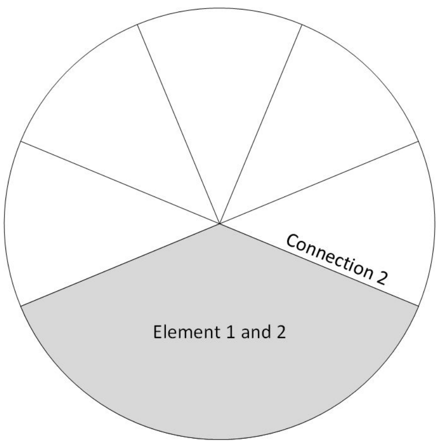


Figure 7: Connection 2

In order to calculate the moment that must pass through connection two, the angle β is used, which is the angle between the centre of element 1 and the edge of element 2 (connection 2).

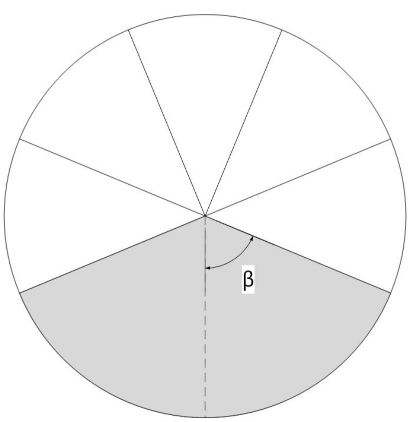


Figure 8: Angle of the elements 1 and 2

$$\beta := \alpha \cdot \frac{3}{2} = 67.5 \cdot deg$$

Width of sections of the centrepiece

In each section, the width will vary, decreasing towards the centre of the foundation where the width is zero.

$$\Delta w_{2.r.fc} := 2\cdot\sin(\beta)\cdot r_{fc} = 3.326\,m$$ Section r.fc

$$\Delta w_{2.0.75.r.fc} := 2\cdot\sin(\beta)\cdot\frac{3r_{fc}}{4} = 2.494\,m$$ Section 0.75*r.fc

$$\Delta w_{2.0.5.r.fc} := 2\cdot\sin(\beta)\cdot\frac{r_{fc}}{2} = 1.663\,m$$ Section 0.5.*r.fc

$$\Delta w_{2.0.25.r.fc} := 2\cdot\sin(\beta)\cdot\frac{r_{fc}}{4} = 0.831\,m$$ Section 0.25*r.fc

$$\Delta w_{2.0.r.fc} := 0m$$ Section 0 (equal to the mid of the centrepiece)

Variation of the global moment along connection 2

$$M_{p.2.r.fc} := \frac{M_{p.tot.max}}{x_p}\cdot\left[\left(x_p - r_{fc}\right) + r_{fc}\cdot\cos(\beta)\right] = 31.701\cdot MN\cdot m$$ Section r.fc

$$M_{p.2.0.75.r.fc} := \frac{M_{p.tot.max}}{x_p}\cdot\left[\left(x_p - r_{fc}\right) + 0.75r_{fc}\cdot\cos(\beta)\right] = 28.09\cdot MN\cdot m$$ Section 0.75*r.fc

$$M_{p.2.0.5.r.fc} := \frac{M_{p.tot.max}}{x_p}\cdot\left[\left(x_p - r_{fc}\right) + 0.5r_{fc}\cdot\cos(\beta)\right] = 24.479\cdot MN\cdot m$$ Section 0.5.*r.fc

$$M_{p.2.0.25.r.fc} := \frac{M_{p.tot.max}}{x_p}\cdot\left[\left(x_p - r_{fc}\right) + 0.25r_{fc}\cdot\cos(\beta)\right] = 20.868\cdot MN\cdot m$$ Section 0.25*r.fc

$$M_{p.2.0.r.fc} := \frac{M_{p.tot.max}}{x_p}\cdot\left(x_p - r_{fc}\right) = 17.257\cdot MN\cdot m$$ Section 0 (equal to the mid of the centrepiece)

Moment that must pass over connection 2

The difference between "what is possible to pass through the cross-section of the sections due to the area decrease" and " the part of the global moment that should be taken by each section (that wants to pass through the section"). This is calculated in the different sections and is the moment that passes over the connection, calculated for each segment.

$$M_{p.2.r.fc.to.0.75} := M_{p.2.0.75.r.fc} - \frac{\Delta w_{2.0.75.r.fc}}{\Delta w_{2.r.fc}}\cdot M_{p.2.r.fc} = 4.314\cdot MN\cdot m$$ What must go to the nearby the element, from section fc to 0.75fc.

$$M_{p.2.0.75.to.0.5} := M_{p.2.0.5.r.fc} - \frac{\Delta w_{2.0.5.r.fc}}{\Delta w_{2.0.75.r.fc}}\cdot M_{p.2.0.75.r.fc} = 5.752\cdot MN\cdot m$$ What must go to the nearby the element, from section 0.75fc to 0.5fc.

$$M_{p.2.0.5.to.0.25} := M_{p.2.0.25.r.fc} - \frac{\Delta w_{2.0.25.r.fc}}{\Delta w_{2.0.5.r.fc}}\cdot M_{p.2.0.5.r.fc} = 8.628\cdot MN\cdot m$$ What must go to the nearby the element, from section 0.5fc to 0.25fc.

$$M_{p.2.0.25.to.0} := M_{p.2.0.r.fc} - \frac{\Delta w_{2.0.r.fc}}{\Delta w_{2.0.25.r.fc}}\cdot M_{p.2.0.25.r.fc} = 17.257\cdot MN\cdot m$$ The value that must pass through the mid point, but cant and instead must go around.

The mean value between these sections:

$$M_{p.mean.connection.2} := \frac{M_{p.2.r.fc.to.0.75} + M_{p.2.0.75.to.0.5} + M_{p.2.0.5.to.0.25} + M_{p.2.0.25.to.0}}{4} = 8.988 \times 10^{3} \cdot kN \cdot r$$

What must pass over the edge, calculated as a distributed moment

$$m_{p.mean.connection.2} := \frac{M_{p.mean.connection.2}}{r_{fc}} = 4.993 \cdot \frac{MN \cdot m}{m}$$

Aproximate calculations of needed reinforcement

Assume the same indata for reinforcement calculations as for the bottom bending reinforcement in the legs.

Chosen dimension of the bars

$$\phi_{connection.2} := 25mm$$

Diameter of the bars for the bottom reinforcement

$$A_{si.connection.2} := \frac{\phi_{connection.2}^{2} \cdot \pi}{4} = 4.909 \times 10^{-4} m^{2}$$

Area of each reinforcement bar

In a simplified manner, assume one layer of reinforcement

$$tp_{connection.2} := c_{cover.b} + \phi_{connection.2} = 0.06\, m$$

Centre of gravity for one layer of reinforcement

$$d_{b.connection.2} := h_{centrepiece} - tp_{connection.2} = 2.74\, m$$

The need for bottom bending reinforcement in connection 2

$$A_{s.connection.2} := \frac{m_{p.mean.connection.2}}{0.9 \cdot d_{b.connection.2} \cdot f_{yd}} = 4.655 \times 10^{3} \cdot \frac{mm^{2}}{m}$$

Needed amount of reinforcement along connection 2

$$n_{bars.connection.2} := \frac{A_{s.connection.2}}{A_{si.connection.2}} = 9.483 \frac{1}{m}$$

Number of bars needed

$$n_{bars.connection.2} := \text{ceil}\left(n_{bars.connection.2} \cdot m\right) \cdot \frac{1}{m} = 10 \frac{1}{m}$$

Rounded number of bars needed

$$s_{connection.2} := 1 \frac{1}{n_{bars.connection.2}} = 0.1\, m$$

Distance between the bars in connection 2

The number of bars needed to transfer the bending moment between each segment is therefore decided. This is the reinforcement that should be placed as overlapping reinforcement in the gaps between the elements. The same verifications should be done for the negative bending momen, however these calculations are performed to verify that the reinforcemet will fit into the section. Since the largest moment is the positive moment, this will be dimensioning.

Interaction between the elements

If the elements had not been joined by wet connections, it would have been important to verify how they effect each other. Like hollow core slabs interact when a point load is applied on one element, the loaded element will deflect, and the adjacent elements wants to deflect as well since they are attached to each other, which will lead to twisting of the element.
In this case, this wont be a problem, due to the full interaction between the elements due to the wet connection.

3. Longitudinal prestressing

Indata

Division into sections

The calculations are performed in five different sections along the legs.
Section 0 in the edge section. Section 1 in splice 1. Section 2 in splice 2. Section 3 in the section beteween legs and centrepiece. Section 4 in the section where the bending moment is largest.

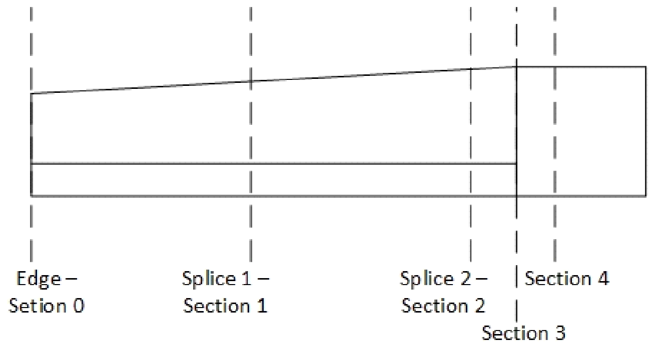


Figure 9: The five different sections that are investigated in the prestressing calculations.

For the first four sections, section 0-3, the cross-sections are T-sections. In the centrepiece, section 4, the cross-section area is rectangular.

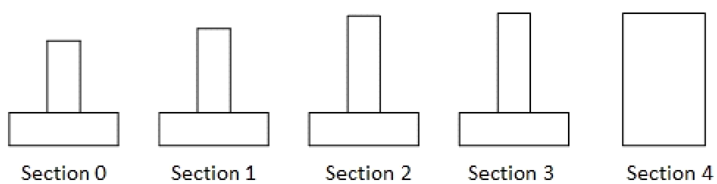


Figure 10: The cross-sections in the different sections

Sign convention

- The edge where the main load for each load case is applied is the top
- The centre of gravity is defined from the compressed side
- Compresive stresses are negative
- Normal force positive in tension
- Bending moment is postive where the down side is in tension
- The coordinate z and the eccentricity is defined as positive downwards, from the centre of gravity

Bending moments in the sections

From the calculations in the evaluation phase, the bending moments in the critical sections are found. The moment distribution calculations are performed with partial safety factor 1.0 on all the loads.

For load case 1, performed in Appendix III for the design load case with highest overturning moment to find the characteristic bending moment and the design loads for normal operation to find the quasi-permanent bending moment. The bending moment for the characteristic and the quasi-permanent bending moment are defined negative due to the sign convention in Naviers forumal.
For load case 2, performed in Appndix III for the selfweight only. This bending moment has a positive sign due to the sign convention.

The calculations on the prestressing is done in SLS

$$M_{char} := -\begin{pmatrix} 0\text{kN}\cdot\text{m} \\ 8.838\cdot 10^{3}\text{kN}\cdot\text{m} \\ 3.247\cdot 10^{4}\text{kN}\cdot\text{m} \\ 3.905\cdot 10^{4}\text{kN}\cdot\text{m} \\ 4.329\cdot 10^{4}\text{kN}\cdot\text{m} \end{pmatrix}$$

Bending moment, characteristic

$$M_g := \begin{pmatrix} 0\text{kN}\cdot\text{m} \\ 846.19\text{kN}\cdot\text{m} \\ 3.422\cdot 10^{3}\text{kN}\cdot\text{m} \\ 4.416\cdot 10^{3}\text{kN}\cdot\text{m} \\ 5.135\cdot 10^{3}\text{kN}\cdot\text{m} \end{pmatrix}$$

Bending moment, dead weight only

Stress limitations

$f_{ck} := 30\text{MPa}$ — Compressive strength of comcrete

$f_{c.tk.0.05} := 2\text{MPa}$ — Tensile strength of concrete

$\sigma_{cc.inf.max} := 0.45\cdot f_{ck} = 13.5\cdot\text{MPa}$ — Maximum compressive stress, long term

$\sigma_{ct.inf.max} := 0$ — Maximum tensile stress, long term

$\sigma_{cci.max} := 0.6\cdot f_{ck} = 1.8\times 10^{7}\,\text{Pa}$ — Maximum compressive stress, at release

$\sigma_{cti.max} := f_{c.tk.0.05} = 2\times 10^{6}\,\text{Pa}$ — Maximum tensile stress, at release

Geometry of the concept

$l_{leg} := l_{leg.8}$ Length of the legs

$l_{element.1} := 4.075m$ Length of element 1 and 2

$l_{element.2} := 0.849m$ Lenght of element 3

$h_{flange} := 0.6m$ Height of the flange

$h_{leg.min} := 1.3m$ Minimum height of the legs

$h_{leg.max} := 1.8m$ Maximum height of the legs

$$h_{leg} := \begin{bmatrix} h_{leg.min} \\ \dfrac{\left(h_{leg.max} - h_{leg.min}\right)\cdot l_{element.1}}{l_{leg}} + h_{leg.min} \\ \dfrac{\left(h_{leg.max} - h_{leg.min}\right)\cdot 2l_{element.1}}{l_{leg}} + h_{leg.min} \\ h_{leg.max} \end{bmatrix}$$

Height of the legs in the critical sections 0-3

$h_{tot} := h_{flange} + h_{leg}$ Total height of the flange and legs, section 0-3

$h_{tot_4} := h_{tot_3} = 2.4\,m$ Total height of the flange and legs, section 4

$$h_{tot} = \begin{pmatrix} 1.9 \\ 2.114 \\ 2.329 \\ 2.4 \\ 2.4 \end{pmatrix} m$$

$$b_4 := \frac{b_{flange}\cdot r_{fc}}{\dfrac{d_{centrepiece}}{2}} = 1.491\,m$$

Widht of the cross-section in section 4.

$$A_{leg} := h_{leg}\cdot b_{leg} = \begin{pmatrix} 0.975 \\ 1.136 \\ 1.297 \\ 1.35 \end{pmatrix} m^2$$

Cross-section area of the legs in the critical sections 0-3

$A_{flange} := h_{flange} \cdot b_{flange} = 1.2\,m^2$ — Cross-section area of the flanges

$A_c := A_{leg} + A_{flange}$ — Gross cross-sectional area, section 0-3

$A_{c_4} := b_4 \cdot h_{tot_4} = 3.578\,m^2$ — Gross cross-sectional area, section 4

$$A_c = \begin{pmatrix} 2.175 \\ 2.336 \\ 2.497 \\ 2.55 \\ 3.578 \end{pmatrix} m^2$$

Gross cross-sectional area in all critical sections

Centre of gravity of the cross-section

When the soil pressure is decisive

The cross-section is turned upside down, so the load is applied from its top.
Centre of gravity is defined from the tensioned side, the flange side

$$tp_{flange} := \frac{h_{flange}}{2} = 0.3\,m$$

Centre of gravity of the flange in section 0-3

$$tp_{leg} := h_{flange} + \frac{h_{leg}}{2} = \begin{pmatrix} 1.25 \\ 1.357 \\ 1.464 \\ 1.5 \end{pmatrix} m$$

Centre of gravity of the leg in section 0-3

$$tp := \left| \begin{array}{l} \Delta \leftarrow 3 \\ \text{for } k \in 0..\Delta \\ \quad tp_k \leftarrow \dfrac{A_{leg_k} \cdot tp_{leg_k} + A_{flange} \cdot tp_{flange}}{A_{c_k}} \\ tp \end{array} \right.$$

Centre of gravity of the cross-section in section 0-3

$$tp_4 := \frac{h_{tot_4}}{2}$$

Centre of gravity of the cross-section in section 4

$$tp = \begin{pmatrix} 0.726 \\ 0.814 \\ 0.905 \\ 0.935 \\ 1.2 \end{pmatrix} m$$

When the dead-weight is decisive

The cross-setion is not rotated, so the load (the self-weight) is applied from above.
The centre of gravity is defined from the tensioned side, the web

$$tp_{flange.g} := h_{leg} + \frac{h_{flange}}{2} = \begin{pmatrix} 1.6 \\ 1.814 \\ 2.029 \\ 2.1 \end{pmatrix} m$$

Centre of gravity of the flange in section 0-3

$$tp_{leg.g} := \frac{h_{leg}}{2} = \begin{pmatrix} 0.65 \\ 0.757 \\ 0.864 \\ 0.9 \end{pmatrix} m$$

Centre of gravity of the leg in section 0-3

$$tp_g := \left| \begin{array}{l} \Delta \leftarrow 3 \\ \text{for } k \in 0..\Delta \\ \quad tp_k \leftarrow \dfrac{A_{leg_k} \cdot tp_{leg.g_k} + A_{flange} \cdot tp_{flange.g_k}}{A_{c_k}} \\ tp \end{array} \right.$$

Centre of gravity of the cross-section in section 0-3

$$tp_{g_4} := \frac{h_{tot_4}}{2}$$

Centre of gravity of the cross-section in section 4

$$tp_g = \begin{pmatrix} 1.174 \\ 1.3 \\ 1.424 \\ 1.465 \\ 1.2 \end{pmatrix} m$$

Second moment of inertia of the cross-section

$$I_c := \left| \begin{array}{l} \Delta \leftarrow 3 \\ \text{for } k \in 0..\Delta \\ \quad I_k \leftarrow \dfrac{b_{flange} \cdot h_{flange}^{\ 3}}{12} + A_{flange} \cdot \left(tp_{flange} - tp_k\right)^2 \ldots \\ \qquad + \dfrac{b_{leg} \cdot \left(h_{leg_k}\right)^3}{12} + A_{leg_k} \cdot \left(tp_{leg_k} - tp_k\right)^2 \\ I \end{array} \right.$$

Second moment of inertia for the section 0-3

$$I_{c_4} := \frac{b_4 \cdot \left(h_{tot_4}\right)^3}{12}$$

Second moment of inertia for section 4

$$I_c = \begin{pmatrix} 0.659 \\ 0.905 \\ 1.204 \\ 1.315 \\ 1.717 \end{pmatrix} m^4$$

Maximum eccentricity of the tendon cable

$d_g := 32mm$ — Assumed aggregate size

$\phi_{duct} := 60mm$ — Assumed diameter of the duct

$c_{duct} := \max(d_g + 5mm, \phi_{duct}, 50mm) = 0.06\,m$ — Mimimum distance between the layers of prestressing reinforcement

$e_{max} := h_{tot_3} - tp_3 - c_{duct} = 1.405\,m$ — Maximum eccentricity, defined in section 3

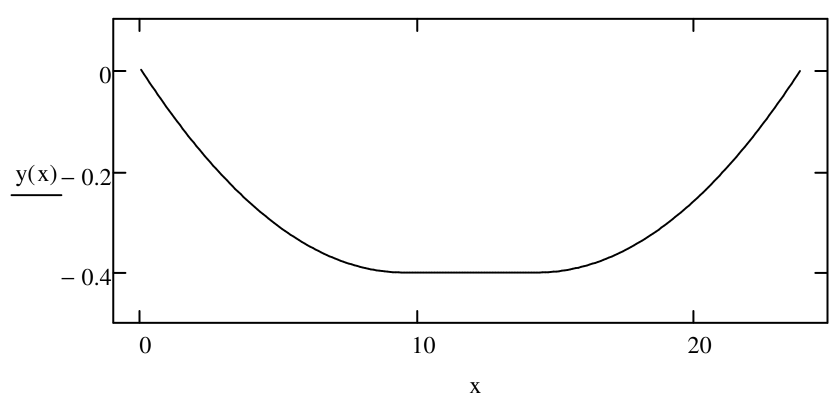
$e_{max} := 0.4m$

$e_{min} := 0m$ — Mimunmum eccentricity, defined in edge section

$x := 0m, 0.1m .. 2l_{leg} + d_{centrepiece}$

$$y(x) := \begin{vmatrix} (e_{max} - e_{min}) \cdot \left(\dfrac{l_{leg} - x}{l_{leg}}\right)^2 - e_{max} \ \text{ if } x \le l_{leg} \\ -e_{max} \ \text{ if } l_{leg} < x < d_{centrepiece} + l_{leg} \\ \left[(e_{max} - e_{min}) \cdot \left[\dfrac{x - (l_{leg} + d_{centrepiece})}{l_{leg}}\right]^2 - e_{max}\right] \ \text{ otherwise} \end{vmatrix}$$

Placement of the tendon cable



$$e_{max.1} := \begin{pmatrix} y(0) \\ y(l_{element.1}) \\ y(2l_{element.1}) \\ y(2l_{element.1} + l_{element.2}) \\ y(2l_{element.1} + l_{element.2}) \end{pmatrix} = \begin{pmatrix} 0 \\ -0.27 \\ -0.392 \\ -0.399 \\ -0.399 \end{pmatrix} m$$

Design eccentricity, when soil pressure is decisive

$$e_{max.2} := -e_{max.1}$$

Design eccentricity, when self-weight is decisive

$$\eta := 0.85$$

Acccording to Design and analysis of prestressed concrete structures: $\eta := 0.75 - 0.85$

Minimum eccentricity of the tendon cable

The cable is placed in the centre of gravity

$$e_{max} := 0m$$

Maximum eccentricity, defined in the mid section

$$e_{min} := 0m$$

Mimunmum eccentricity, defined in edge section

$$x := 0m, 0.1m..\, 2l_{leg} + d_{centrepiece}$$

$$y(x) := \begin{cases} (e_{max} - e_{min}) \cdot \left(\dfrac{l_{leg} - x}{l_{leg}}\right)^2 - e_{max} & \text{if } x \le l_{leg} \\ -e_{max} & \text{if } l_{leg} < x < d_{centrepiece} + l_{leg} \\ \left[(e_{max} - e_{min}) \cdot \left[\dfrac{x - (l_{leg} + d_{centrepiece})}{l_{leg}}\right]^2 - e_{max}\right] & \text{otherwise} \end{cases}$$

Placement of tendon

$$e_0 := \begin{pmatrix} y(0) \\ y(l_{element.1}) \\ y(2l_{element.1}) \\ y(2l_{element.1} + l_{element.2}) \\ y(2l_{element.1} + l_{element.2}) \end{pmatrix} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

Minimum eccentricity

Distance from the centre of gravity to the edges

Load case 1, soil pressure is determining

$$z_{flange.1} := -tp$$

Distance to flange edge, soil pressure decisive

$$z_{web.1} := h_{tot} - tp$$

Distance to web edge, soil pressure decisive

Load case 2, self-weight is determining

$$z_{web.2} := -tp_g$$

Distance to web edge, dead-weight decisive

$$z_{flange.2} := h_{tot} - tp_g$$

Distance to flange edge, dead-weight decisive

Dimensioning the prestressing force for Load case 1, maximum eccentricity

Load case 1

For load case 1, the critical edge due to loading is the flange edge. The soil pressure gives tension while the prestressing force give compression.

In order to prevent tensile stresses in the web edge due to the loading in load case 1, the needed prestressing force is:

Dimensioning for no tensile stresses in the flange

$$P_{i.char.max.e} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad P_{i.char.1.t_k} \leftarrow \dfrac{M_{char_k} + \dfrac{I_{c_k}}{z_{flange.1_k}} \cdot \sigma_{ct.inf.max}}{e_{max.1_k} + \dfrac{I_{c_k}}{z_{flange.1_k} \cdot A_{c_k}}} \\ P_{i.char.1.t} \end{array} \right. \qquad P_{i.char.max.e} = \begin{pmatrix} 0 \\ 11.853 \\ 35.104 \\ 41.089 \\ 54.188 \end{pmatrix} \cdot MN$$

Minimum required prestressing force to keep the flange edge in tension:

$$P_i := \max\left(P_{i.char.max.e}\right) = 54.188 \cdot MN$$

Resulting stresses in the web

$$\sigma_{web.1} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad \sigma_{web.1_k} \leftarrow \dfrac{-P_i}{A_{c_k}} + \dfrac{-P_i \cdot e_{max.1_k} + M_{char_k}}{I_{c_k}} \cdot z_{web.1_k} \\ \sigma_{web.1} \end{array} \right. \qquad \sigma_{web.1} = \begin{pmatrix} -24.914 \\ -14.912 \\ -34.989 \\ -40.665 \\ -30.293 \end{pmatrix} \cdot MPa$$

$$\sigma_{web.1} := \min\left(\sigma_{web.1}\right) = -40.665 \cdot MPa$$

$$\text{Check} := \left| \begin{array}{ll} \text{"Compression in the web"} & \text{if } \sigma_{web.1} < 0 \\ \text{"Tension in the web"} & \text{otherwise} \end{array} \right.$$

Check = "Compression in the web"

$$\text{Check}_c := \left| \begin{array}{ll} \text{"Compressive stresses are NOT ok!"} & \text{if } \left|\sigma_{web.1}\right| > \sigma_{cc.inf.max} \\ \text{"Compressive stresses are OK"} & \text{otherwise} \end{array} \right.$$

Check_c = "Compressive stresses are NOT ok!"

Load case 2

For load case 2, the critical edge due to loading is the web edge. There will never be compression on the web edge since both the self-weight and the prestressing force give tensile stresses.

The stresses are given by Naviers formula:

Resulting stresses in the web

$$\sigma_{web.2} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad \sigma_{web.2_k} \leftarrow \dfrac{-P_i}{A_{c_k}} + \dfrac{-P_i \cdot e_{max.2_k} + M_{g_k}}{I_{c_k}} \cdot z_{web.2_k} \\ \sigma_{web.2} \end{array} \right. \qquad \sigma_{web.2} = \begin{pmatrix} -24.914 \\ -3.433 \\ -0.633 \\ -2.098 \\ -3.63 \end{pmatrix} \cdot \text{MPa}$$

$$\sigma_{web.2} := \max(\sigma_{web.2}) = -0.633 \cdot \text{MPa}$$

$$\text{Check} := \left| \begin{array}{ll} \text{"Compression in the web"} & \text{if } \sigma_{web.2} < 0 \\ \text{"Tension in the web"} & \text{otherwise} \end{array} \right.$$

Check = "Compression in the web"

$$\text{Check}_c := \left| \begin{array}{ll} \text{"Compresive stresses are NOT ok!"} & \text{if } \left|\sigma_{web.2}\right| > \sigma_{cc.inf.max} \\ \text{"Compressive stresses are OK"} & \text{otherwise} \end{array} \right.$$

Check_c = "Compressive stresses are OK"

Resulting stresses in the flange

$$\sigma_{flange.2} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad \sigma_{flange.2_k} \leftarrow \dfrac{-P_i}{A_{c_k}} + \dfrac{-P_i \cdot e_{max.2_k} + M_{g_k}}{I_{c_k}} \cdot z_{flange.2_k} \\ \sigma_{flange.2} \end{array} \right. \qquad \sigma_{flange.2} = \begin{pmatrix} -24.914 \\ -35.572 \\ -35.09 \\ -33.48 \\ -26.662 \end{pmatrix} \cdot \text{MPa}$$

$$\sigma_{flange.2} := \min(\sigma_{flange.2}) = -35.572 \cdot \text{MPa}$$

$$\text{Check} := \left| \begin{array}{ll} \text{"Compression in the flange"} & \text{if } \sigma_{flange.2} < 0 \\ \text{"Tension in the flange"} & \text{otherwise} \end{array} \right.$$

Check = "Compression in the flange"

$$\text{Check}_c := \left| \begin{array}{ll} \text{"Compressive stresses are NOT ok!"} & \text{if } \left|\sigma_{flange.2}\right| > \sigma_{cc.inf.max} \\ \text{"Compressive stresses are OK"} & \text{otherwise} \end{array} \right.$$

Check_c = "Compressive stresses are NOT ok!"

Dimensioning the prestressing force for Load case 1, minimum eccentricity

Load case 1

For load case 1, the critical edge due to loading is the flange edge. The soil pressure gives tension while the prestressing force give compression.

In order to prevent tensile stresses in the web edge due to the loading in load case 1, the needed prestressing force is:

$$P_{i.char.min.e} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad P_{i.char.1.t_k} \leftarrow \dfrac{M_{char_k} + \dfrac{I_{c_k}}{z_{flange.1_k}} \cdot \sigma_{ct.inf.max}}{e_{0_k} + \dfrac{I_{c_k}}{z_{flange.1_k} \cdot A_{c_k}}} \\ P_{i.char.1.t} \end{array} \right. \qquad P_{i.char.min.e} = \begin{pmatrix} 0 \\ 18.564 \\ 60.915 \\ 70.807 \\ 108.225 \end{pmatrix} \cdot MN$$

Minimum required prestressing force to keep the flange edge in tension:

$$P_i := \max\left(P_{i.char.min.e}\right) = 108.225 \cdot MN$$

Resulting stresses in the web

$$\sigma_{web.1} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad \sigma_{web.1_k} \leftarrow \dfrac{-P_i}{A_{c_k}} + \dfrac{-P_i \cdot e_{0_k} + M_{char_k}}{I_{c_k}} \cdot z_{web.1_k} \\ \sigma_{web.1} \end{array} \right. \qquad \sigma_{web.1} = \begin{pmatrix} -49.759 \\ -59.026 \\ -81.75 \\ -85.926 \\ -60.501 \end{pmatrix} \cdot MPa$$

$$\sigma_{web.1} := \min\left(\sigma_{web.1}\right) = -85.926 \cdot MPa$$

$$Check := \left| \begin{array}{ll} \text{"Compression in the web"} & \text{if } \sigma_{flange.2} < 0 \\ \text{"Tension in the web"} & \text{otherwise} \end{array} \right.$$

Check = "Compression in the web"

$$Check_c := \left| \begin{array}{ll} \text{"Compressive stresses are NOT ok "} & \text{if } \left|\sigma_{web.1}\right| > \sigma_{cc.inf.max} \\ \text{"Compressive stresses are OK"} & \text{otherwise} \end{array} \right.$$

$Check_c$ = "Compressive stresses are NOT ok "

Load case 2

For load case 2, the critical edge due to loading is the web edge. There will never be compression on the web edge since both the self-weight and the prestressing force give tensile stresses.

The stresses are given by Naviers formula:

Resulting stresses in the web

$$\sigma_{web.2} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad \sigma_{web.2_k} \leftarrow \dfrac{-P_i}{A_{c_k}} + \dfrac{-P_i \cdot e_{0_k} + M_{g_k}}{I_{c_k}} \cdot z_{web.2_k} \\ \sigma_{web.2} \end{array} \right. \qquad \sigma_{web.2} = \begin{pmatrix} -49.759 \\ -47.547 \\ -47.394 \\ -47.359 \\ -33.839 \end{pmatrix} \cdot \text{MPa}$$

$$\sigma_{web.2} := \min(\sigma_{web.2}) = -49.759 \cdot \text{MPa}$$

$$\text{Check} := \left| \begin{array}{l} \text{"Compression in the web"} \quad \text{if } \sigma_{web.2} < 0 \\ \text{"Tension in the web"} \quad \text{otherwise} \end{array} \right.$$

Check = "Compression in the web"

$$\text{Check}_c := \left| \begin{array}{l} \text{"Compressive stresses are NOT ok!"} \quad \text{if } \left|\sigma_{web.2}\right| > \sigma_{cc.inf.max} \\ \text{"Compressive stresses are OK"} \quad \text{otherwise} \end{array} \right.$$

Check_c = "Compressive stresses are NOT ok!"

Resulting stresses in the flange

$$\sigma_{flange.2} := \left| \begin{array}{l} \Delta \leftarrow 4 \\ \text{for } k \in 0..\Delta \\ \quad \sigma_{flange.2_k} \leftarrow \dfrac{-P_i}{A_{c_k}} + \dfrac{-P_i \cdot e_{0_k} + M_{g_k}}{I_{c_k}} \cdot z_{flange.2_k} \\ \sigma_{flange.2} \end{array} \right. \qquad \sigma_{flange.2} = \begin{pmatrix} -49.759 \\ -45.571 \\ -40.776 \\ -39.301 \\ -26.662 \end{pmatrix} \cdot \text{MPa}$$

$$\sigma_{flange.2} := \min(\sigma_{flange.2}) = -49.759 \cdot \text{MPa}$$

$$\text{Check} := \left| \begin{array}{l} \text{"Compression in the flange"} \quad \text{if } \sigma_{flange.2} < 0 \\ \text{"Tension in the flange"} \quad \text{otherwise} \end{array} \right.$$

Check = "Compression in the flange"

$$\text{Check}_c := \left| \begin{array}{l} \text{"Compressive stresses are NOT ok!"} \quad \text{if } \left|\sigma_{flange.2}\right| > \sigma_{cc.inf.max} \\ \text{"Compressive stresses are OK"} \quad \text{otherwise} \end{array} \right.$$

Check_c = "Compressive stresses are NOT ok!"