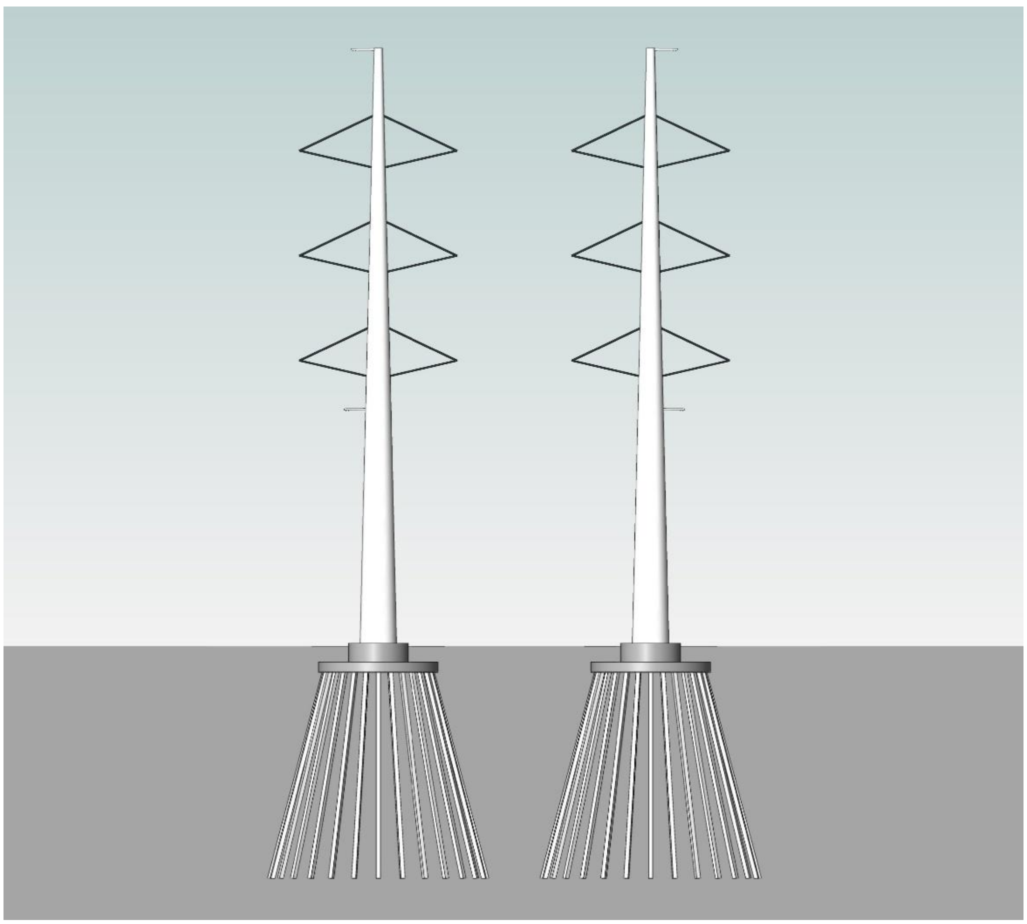
Master Thesis

Transmission Tower in (Ultra) High Strength Concrete



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Abstract

The power grid in the Netherlands will be fundamentally renovated and expanded in the coming years. Therefore new concepts of transmission towers should be considered, with an architecturally approved design not only with a long lifespan, but with the smallest visual impact on the environment, while simultaneously being execution and cost friendly, as well as environmentally sustainable.

Movares Nederland has been doing research on different alternatives to find the optimal shape of the described concept since 2005. One of the possible alternatives is to use conical tube masts of prestressed (Ultra) High Strength Concrete. The choice between the HSC and UHSC and the extent and manner of prestressing are part of the design. Aside from the costs, other important factors are the speed of construction and minimal disturbance of the environment. The challenge is the making of an intelligent design, which utilizes the material concrete in the most efficient manner, while at the same time adhering to all the design criteria.

The objective of the thesis was to determine whether (ultra) high strength concrete is a viable option for transmission towers according to the new concept.
A comparative analysis was performed with various methods of producing and execution of a transmission tower in UHSC. In order to obtain the most favorable method, the sustainability and costs of the entire project were also taken into consideration.

The design process of the transmission was divided in several steps. First to attain an objective result a number of variants were introduced. These variants were based on three different concrete strength classes, namely Ordinary Strength Concrete (OSC), High Strength Concrete (HSC) and Ultra High Strength Concrete. For each of the concrete strengths both the reinforced variant, as well as the prestressed variant were examined. Secondly calculations were performed for all these variants. These results were verified with a mathematical program, resulting in a preliminary design.

After the preliminary design was established, the production and execution process could be studied. To ensure a smooth execution process at the building site later on, an efficient production method had to be devised. Various production methods and materials were studied, each with their own pros and cons. The use of steel moulds for each separate segment proved to be a reasonable choice.

The execution of transmission towers in concrete, is a vital aspect of the entire process. While examining this process, factors that influence the initial design should be scrutinized. One such an important aspect is the prestressing. Because the transmission towers will be designed as conical tube masts with a small wall thickness, it was decided that internal prestressing would be too difficult to apply. Thus external prestressing was chosen to be applied on the inside of the masts. The anchorage of the prestressing can then be achieved through blind anchors. Furthermore it was found that the amount of prestressing can be reduced at increasing heights. After a segment is placed, the prestressing can then be anchored at the top of this segment.

Another important aspect of the execution process are the various connections. The transmission tower has three main connections that must be worked out in detail, namely the foundation-column connection, the segment-segment connection and the column-isolator connection. After considering a wide variety of options, suggestions were made as to which connections would be the most effective for the tube masts.
The parameters established for the production and execution process, yielded the final design. In this final design both the mast dimensions and the prestressing configuration were optimized. It was found that for a tube mast with a fixed foot diameter of 2200 mm, a top diameter of 500 mm and a height of 57 m, the necessary wall thickness for a tube mast made in OSC is 420 mm, in HSC is 145 mm and in UHSC is 75 mm.

Subsequently, the sustainability and costs of the masts were looked at in detail. The sustainability of the transmission tower depends mainly on the amount of emitted and stored energy (i.e. insulating properties), throughout the entire life of the structure. For both the sustainability and the cost considerations, two extra variants were introduced, namely a traditional steel truss mast and steel conical tube masts. Comparison of these two steel variants with the three concrete variants in terms of sustainability, revealed that the steel truss variant was the most sustainable alternative followed by the UHSC and the HSC alternative.

For the cost consideration the steel truss mast once again proved superior with a total cost of € 421.000. Following closely was the steel tube mast with € 534.000, the UHSC variant with € 566.000 and the HSC variant with € 597.000.

Finally, all acquired results were considered in a comparative analysis of the variants. Both a Multi-Criteria Decision Analysis (MCDA) and a Value Matrix(VM) were used to determine a ranking for the variants, based on:
Lowest total costs (1st steel truss, 2nd steel tube, 3rd UHSC);
Best total performance (1st UHSC, 2nd HSC, 3rd steel tube);
Best performance/costs ratio (1st steel truss, 2nd UHSC tube, 3rd steel tube).

The outcome of this thesis reveals that UHSC, with the best total performance, is a viable option for transmission towers.
In addition, this comparative analysis also provides an easy tool for variant selection, since the choice of the best variant is governed by the specific requirement of the client.

Keywords: High Strength Concrete (HSC), Ultra High Strength Concrete (UHSC), Transmission tower, Tubular columns, Prefab segments, Reinforced concrete, External prestressing, Sustainability, MCDA, Value Engineering.

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Preface

This thesis is the capstone of my master Structural Engineering, Concrete Structures at TU Delft and this research was conducted for the company Movares Nederland.

The main objective of the thesis was to determine the feasibility of construction of transmission towers in Ultra High Strength Concrete.
This report features an overview of the design steps, the various production and execution methods are discussed and conclusions and recommendations are offered.

Acknowledgments
Sincere thanks go out to my thesis supervisor René Braam from the TU Delft, for his helpful insights, suggestions, comments and reviewing of the thesis.
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The enthusiasm and support of Dick Hordijk despite his busy schedule was inspiring.

Furthermore, I am indebted to Movares for providing me with the opportunity to study this fascinating subject. I am grateful to Jan van Wolfswinkel, who was assigned as my daily supervisor and guided me every step of the way, with insightful comments and constant counsel. The pleasant and stimulating working environment created by all my colleagues at Movares was highly appreciated.

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1. Introduction

The power grid in the Netherlands will be largely renovated and expanded in the coming years. Therefore a new concept of transmission towers is considered, with an architecturally approved design that yields the smallest visual pollution for the environment, while simultaneously being execution and cost friendly, as well as environmentally sustainable.
Movares Nederland has been doing research on different alternatives to find the optimal shape of the described concept since 2005. One of the possible alternatives is to use conical tube masts of prestressed (Ultra) High Strength Concrete. The choice between the HSC and UHSC and the extent and manner of prestressing are part of the design. Aside from the costs, other important factors are the speed of construction and the limitation of disturbance of the environment. The challenge is the making of an intelligent design, which makes use of the material concrete in the most efficient manner, while at the same time satisfying all the design criteria.

At TU Delft the Master program for Civil Engineering (CIE) is concluded with a thesis worth 40 ECTS. The thesis is expected to last 8-9 months and will bring the graduate's Structural Design program to a close. The thesis is concluded with a presentation by the graduate, where all commission members and interested parties will be present. During the thesis, the graduate from the Delft University of Technology is under the guidance of René Braam and Sander Pasterkamp. Furthermore, Dick Hordijk is involved in the project as the leading professor from TU Delft. At Movares the graduate is under the daily supervision of Jan van Wolfswinkel.

1.1 Problem statement and research questions

"Is the use of (ultra) high strength concrete viable for transmission towers?"

The study consists of various sections based on the various aspects of the construction process. First, the pros and cons of using UHSC are analyzed. Secondly, the current systems are also examined. The main question of the research can then be divided into a number of sub questions:

1. What are the characteristics of UHSC?
2. How is UHSC covered in the European Standards (EC)?
3. How is the current technology of transmission towers (steel and concrete)?
4. Are conical tube masts the best option for a mast in UHSC?
5. What is the optimal way of producing and executing a mast in UHSC?
6. What are the costs of the whole project?
7. How sustainable is a mast of UHSC (LCC)?
8. How much is the visual impact?
9. How does the new alternative relate to the current systems, if all aspects are taken into account?

1.2 Thesis objectives

UHSC is a relatively new type of material. Therefore the industry is (understandably) hesitant to make much use of it at the moment. At the moment there aren't that many applications of UHSC in the Netherlands or in the world, partially because the existing codes for designing with UHSC are very limited. The challenge of this thesis is to research if transmission towers constructed with UHSC are a viable option, compared to the standard steel transmission towers.

1.3 Scope of the thesis

To define the scope of the thesis, some limitations are applied. These restrictions may originate from the environment, the company, the graduate himself, etc.

Boundary conditions

- The transmission tower must have a lifespan of at least 50 years;
- The transmission tower may not be a danger to public safety;
- The mast should fit in the surrounding area.

Requirements

- The design should be able to be implemented;
- The costs of the design should be acceptable;
- The design should aim to provide the least amount of visual impact;
- Inconvenience to the surrounding area during construction should be minimal;
- The end product must be sustainable, durable and have low maintenance.

Assumptions

- The design is based on the information on UHSC present in the Eurocodes and French Recommendations;
- UHSC is defined as concrete with a compressive strength greater than 150 N/mm^2;
- The magnetic fields will only be examined to some extent;
- The type of grounding will not be studied in detail;
- The type of conductors used will be decided by the client;
- Only the decisive load cases will be considered.

Restrictions

- The total height of the transmission tower is fixed, as well as the top and bottom diameter;
- Only one type of mast will be considered namely the support mast.

1.4 Method of research

Phase 1: Introduction and start of the project

- Problem analysis
- Objective
- Requirements
- Method of research
- Literature study
- Sub-questions
- Hypotheses

Phase 2: Research and analysis

- Variants (MCAs)
- Preliminary design
- Preliminary models

Phase 3: Execution and cost assessment
- Production technique
- Cost

Phase 4: *Influence on environment*
- Sustainability
- Visual impact

Phase 5: *Comparative analysis*
- Comparison design (final design)
- Comparison models (final model)

Phase 6: Discussion
- Results
- Conclusions
- Recommendations

1.5 Outline of the thesis

The main body of the thesis consists of 11 chapters which can be divided in 5 parts. In the first part the subject matter is introduced and the literature study is presented. In the second part a theoretical approach to the problem is given which culminates in a preliminary design. The third part covers the practical aspects of the problem, namely the production and execution. The fourth part is comprised of the comparison of the alternatives on various factors, which eventually leads to a conclusion in the last part.

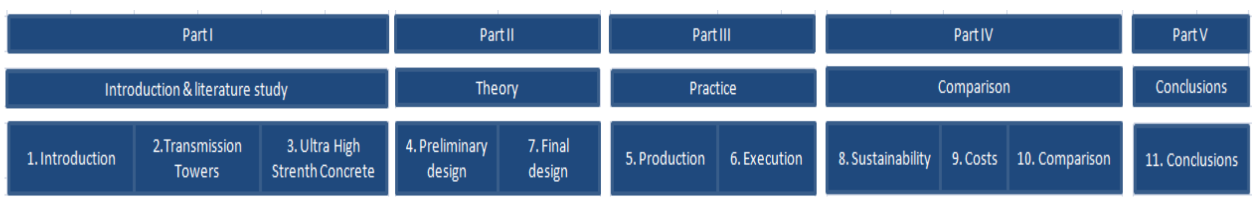


Figure 1: Thesis outline

2. Transmission towers

In this chapter the basic principle of a transmission tower is discussed. First the need for transmission towers is substantiated, after which the formation of the tower is looked at. Subsequently the various type of masts are discussed and finally the electric system and magnetism of the mast, as well as the effect on the environment is addressed.

2.1 Electricity

Electricity is the set of physical phenomena associated with the presence and flow of electric charge [I]. In the latter half of the 20th century, electricity started more and more to become part of everyday life. Nowadays electricity is a familiar concept, used for nearly everything from lighting and cooking to powering tram lines and trains.
Electricity is mainly utilized as a way to transfer energy and as such has an enormous amount of possible applications, usually benefitting the community.

The process of delivering electricity from the place of generation to the end users can be divided in four parts:

- Generation
- Transmission
- Distribution
- Marketing

Generation of electricity is usually done at power plants or electrical substations. This electricity is then *transmitted* via transmission- or power lines. The electricity is then *distributed* to the consumers after being made available through *marketing.*

The power plants are usually based in areas with few people and wide open spaces. The generated electricity must then travel long distances to the urban areas through *high voltage transmission lines*. These lines are extremely efficient for transmitting the electricity, but the high voltage electricity on these lines is not yet useable by consumers. For that reason substations, otherwise known as local power stations, are built near urban areas to receive and lower the voltage from the high voltage transmission lines. This is done by so called 'step-down' transformers. The reduced voltage is then transmitted via *subtransmission lines* in the urban area. However the reduced voltage is still too high for commercial use. Therefore another transformer is used, usually per neighborhood or district, to further reduce the voltage. This transformer than transmits the electricity via *local distribution lines* to common households, offices etc. These local distribution lines can be either above or underground, depending on the location.

2.2 Power lines

Electricity is usually transmitted at very high voltages (> 100 kV), to decrease the amount of energy lost during transmission. At the power plants though, the power is generated at a fairly low voltage (between about 2.3 kV and 30 kV), depending on the size of the generator. The voltage is then increased by a power transformer to a higher voltage (115 kV to 765 kV AC, depending on the transmission device) for transmission over long distances [II].

Because of these dangerous high voltages, electricity has to be transmitted away from civilians. For safety reasons it is thus transmitted either high above ground or underground.

Overhead power lines are a familiar sight, especially in rural areas. These power lines are used for transmitting and distributing electricity over large distances. Usually there are one to four conductors suspended by towers or poles. Because the power lines are suspended in the air, they can make use of the natural isolation of the air, making them more cost effective. Underground power lines have less of an impact on the environment, but have higher costs and are difficult to maintain. Another advantage of the suspension of the overhead power lines is that a sufficient clearance is kept between the power lines and the ground area, preventing fatal contact by wandering civilians or animals.

Overhead power lines can be classified based on the wide scope of existing voltages [II]:

- Low voltage – less than 1000 Volts, used for connection between a residential or small commercial customer and the utility.
- Medium Voltage (distribution) – between 1000 Volts (1 kV) and to about 33 kV, used for distribution in urban and rural areas.
- High Voltage (sub transmission less than 100 kV; sub transmission or transmission at voltage such as 115 kV and 138 kV), used for sub-transmission and transmission of bulk quantities of electric power and connection to very large consumers.
- Extra High Voltage (transmission) – over 230 kV, up to about 800 kV, used for long distance, very high power transmission.
- Ultra High Voltage – higher than 800 kV.

At present day transmission-level voltages are typically considered to be at least 110 kV. Voltages below this level are generally considered as subtransmission voltage (33 kV & 66 kV), but are sometimes utilized for long lines with small loads. Voltages lower than 33 kV are generally only used for distribution. Extra high voltages (> 230 kV) demand different design requirements compared to lower voltages [II].

The overhead conductors are generally bare and can be made of a variety of materials e.g. aluminum, composites, copper etc, with aluminum being the most common. Copper is a great conductor of electricity and can easily be welded together. Both its advantage and disadvantage is the high strength, making it resistant to high stress and at the same time very difficult to work with. Steel conductors are only one tenth as effective as copper and oxidize easily [1], which is why they are usually used along with other materials. However they have a higher strength than copper conductors and can be galvanized to reduce rusting as well. Aluminum is less conductive than copper, has lower strength, a higher coefficient of thermal expansion and can oxidize very fast. The reason why it is preferred over copper is that it is lighter and more cost effective. For high voltage power lines usually a combination of steel and aluminum is used called Aluminum Conductor Steel-Reinforced (ACSR) cables (see Figure 2).

Figure 2: Cable sample: mid 7 conductors steel & the bulk being aluminum (ACSR) [III]

These ACSR cables have high strength, high conductivity and are light as well. They are composed of a central core of steel (reinforcement) embedded in aluminum strands. The main purpose of the aluminum is the conductivity (see skin effect), while the steel is used to support the conductor.

As the bare power lines use the air as insulation, minimum clearance between the power lines and the ground is necessary for safety reasons. Unfavorable weather (strong winds and freezing temperatures) can result in power outages. Critical wind speeds can cause power lines to enter the space reserved for clearance, sometimes resulting in flashover and/or loss of supply and, in general, unsafe situations.

Nowadays overhead power lines are a common product, supplied by various companies around the world. The conductor material and shapes can be improved to obtain higher capacity and modernize transmission circuits. Conductor sizes range from 12 mm^2 to 750 mm^2, with varying resistance and current-carrying capacity [II]. The reason why wires are not spontaneously increased in thickness is because of a phenomenon called the skin effect.

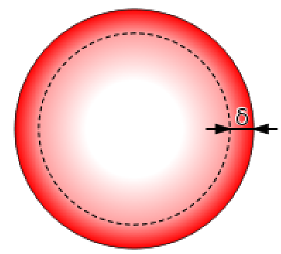


Figure 3: Distribution of current flow in a cylindrical conductor, shown in cross section

"Skin effect is the tendency of an alternating electric current (AC) to become distributed within a conductor such that the current density is largest near the surface of the conductor, and decreases with greater depths in the conductor"[IV]. This means that the electricity flows mostly at the 'skin' of the power lines, between the most outer fiber and a level defined as the skin depth (see Figure 3). At higher frequencies the skin depth of the conductor becomes smaller than at lower frequencies. So when the skin effect causes the effective resistance of the conductor to increase, the effective cross section of the conductor is reduced. The skin effect is caused by Foucault currents, induced by the shifting magnetic field which in turn is caused by the alternating current.

So while larger power lines may have less energy loss due to the skin effect, they are more expensive than smaller power lines. An optimization rule for deciding the size of the conductors exists known as Kelvin's Law which states: *"the optimum size of conductor for a line is found when the cost of the energy wasted in the conductor is equal to the annual interest paid on that portion of the line construction cost due to the size of the conductors"* [III]. Of course there are various other underlying factors which complicate the choice. Since using thicker wires for conductors would result in a rather small increase in capacity due to the skin effect, multiple parallel cables, also known as bundle conductors (see Figure 2), are used when a high capacity is needed (usually at very high voltages). These multiple parallel cables have the added advantage of reducing energy losses caused by corona discharge as well as increasing the amount of current that can be carried in a line. A disadvantage of the bundle conductors is the higher sensitivity to wind load.

2.3 Underground cables

Opposed to the overhead power lines option we have the underground cables. By placing the conductors underground, radiation of the electrical field beyond the conductors is greatly reduced along with the magnetic field. The visual impact is gone as well, along with the danger presented by the overhead power lines. The reason why underground cables are not applied most of the time is because the cost of placing these cables underground and maintaining them is several times higher than for overhead power lines. Another disadvantage according to most (TU Delft) is that at the moment it is not possible to apply a reliable 380 kV underground cable longer than 20 km.

The build of a 110 kV underground cable is almost identical to an upscaled version of the common electricity wires in households. It consists of a conductive core, a synthetic cover and sealing. The cable must be isolated against moisture and current leakage. With regards to the safety as well as the heat dissipation, the cable must be placed deep in the ground (at least 1 meter).

Placing cables underground is much more difficult than one would imagine. It demands expensive and special made cables. Any decent high voltage cable has difficulties dispensing heat underground. To allow this heat to dissipate without problems, in some cases soil improvement is needed. Soil improvement is especially of interest when heavy cables are used. Aside from the high costs, the execution aspects can be very inconvenient as well, in particular for farmers. These farmers have to be compensated since the farming can be stalled for as long as a whole season. After the placing of the cables, there is a (reasonable) demand from the farmers to neatly fill the excavated trench and restore the ground (including the original top humus cover) [V].

An underground cable needs less maintenance than an overhead power line. But when maintenance is actually needed, it is nearly impossible to reach the underground cable. So an underground cable must be very robust and if possible maintenance-free for forty years or more. Unlike overhead power lines, underground cables are not affected by the weather or changing temperatures, but instead by soil moisture, tree roots and the acidic and chemical properties of the soil itself. A big disadvantage of underground cables is that they cannot be upgraded, while overhead power lines have no such problem. Therefore placing cables underground must be done by taking future developments into consideration [III].

2.4 Transmission towers

Transmission towers (also called pylons or masts) are tall structures utilized to support overhead power lines.
These towers can be made of wood (as-grown or laminated), steel (either lattice structures or tubular poles), concrete, aluminum, and sometimes even reinforced plastics. Transmission towers come in a wide variety of shapes and sizes. Their form can be as simple as poles set directly in the ground, usually carrying one or two cross-arm beams to support the power lines, or as complicated as steel lattice towers with the overhead lines supported on insulators connected to the side of the tower. In urban areas we usually see steel poles (occasionally even wooden), while in rural areas steel lattice towers are more often applied. These steel lattice towers consist of hundreds of steel slats that are bolted to each other. Welding is seldom utilized. Concrete poles tend to be used with less frequency. Towers made with reinforced plastics are also an option, but their high cost usually excludes their application.

Transmission towers must be designed to resist the loads imposed on it by the power lines. Aside from the weight of the conductors, the dynamic loads must be taken into consideration as well, mainly wind load, frost accumulation and vibrations.
Conductors are usually flexible and supported at their ends. Their form approximates a catenary and much of the design and execution is based on this principle. For normal (supporting) masts, with conductors travelling in straight lines, the towers only need to resist the weight of the conductors and the wind loads. Since the horizontal tension in the conductors on either side of the mast approximately balance each other, no resultant horizontal (longitudinal) forces on the structure are caused by the conductors. The only horizontal loads that have to be taken into account are the wind load and the horizontal forces caused by the wind load on the conductors (orthogonal to the trace).

An important issue for transmission towers is the foundation. Because the ground conditions in the Netherlands are generally poor (wetlands), foundations may be large and expensive. Based on the ground conditions and the type of mast, a variety of foundation methods can be applied.

A large complaint about transmission towers is that they are a form of visual pollution. However there are also some admirers, though they are a definite minority.

2.5 Mast parts

For most type of towers, the formation tends to be very similar. To explain the function of various tower parts, a lattice steel tower will be examined.

At the bottom the *legs* of the tower can be identified. These can either be four legs in pyramid formation or one single leg. The legs are usually connected with the ground through foundation piles. The bottom of the mast is extremely important since it is the part that clamps the whole tower and thus decides how well the tower will fare against wind. For small masts usually a singular leg is used to minimize the occupied space. On the other hand for large masts four legs are usually used with large enough spacing in between, for potential cattle or agricultural machines to pass under.

On top of the legs is the *tower*, which is the determining part for maximum height of the mast and the amount of crossbars possible. Generally in the Netherlands the form of the tower is conical, though simple straight towers are used as well. Conical forms are used more often, because they use less material, receive less wind load and are more pleasant to look at.

Near the top of the tower *crossbars* are attached. On these crossbars one or two insulator strings are attached which support the conductors. These insulator strings can be made of porcelain, glass or composite polymer materials. The length of the strings varies according to the applied voltage, with the length increasing proportional to the voltage increase.

The *conductors* are the reason why high voltage masts exist. The masts usually support three or multiples of three (6 or 9 and seldom 12) conductors, because the power plants utilize a three-phase electrical power system. This three-phase system with three conductors is called a circuit. Conductors tend to be about 1 to 3 km long.

Finally at the peak of the mast is the *top piece*, which indicates the end of the tower. This part is usually shaped according to the wishes of the architect. While it may seem insignificant, the top piece has a very important function, namely preventing lightning from striking the mast. Since the top of the mast is most susceptible to lightning strikes, usually earth wires are placed there and travelling above the conductors as well. These earth wires then travel along the mast to the bottom and are grounded.

These main parts can be found in most transmission towers. There are variations of course, where some parts are added and others left out, but the basics are as described above.

The big difference in form between steel lattice masts and conical tube masts is the bottom part. Steel lattice towers generally have four legs, while conical tube masts are singular ‘poles’. Furthermore, steel lattice masts are see through because of the various lattices, while the tube masts are closed. Both are hollow, although this effect is somewhat difficult to visualize, because the steel lattice masts are square and see through, while the tube masts are round and closed.

Because of the open form of steel lattice masts, they are less susceptible to wind loads than tube masts and for towers of the same height they use a lot less material. Of course this does not mean that steel lattice towers are the better choice, since for reasons explained later (mainly the high magnetic fields), they have to be built much higher, thus eliminating both of these advantages.

2.6 Mast functions

Based on their function, transmission towers can be divided into three categories:

1. Suspension mast (“steunmast”)
2. Dead-end mast
    a) Angle mast (“hoekmast”)
    b) End mast (“eindmast”)
3. Transposition mast (“wisselmast”)

Suspension mast
The only purpose of these masts is to support the conductors. When the conductors are traveling in straight lines, these suspension (or support) masts are used. The towers only support the conductors vertically. Horizontally the tensions in the conductors cancel each other out. Because the horizontal aspect of the conductor does not have to be taken into account, suspension masts can be designed much lighter and cheaper than other types of masts. Suspension masts are usually placed about 100 to 200 meters from each other and have vertical (or in case of two insulators a v-form) insulators perpendicular to the conductors.

Dead-end mast
Dead-end masts are used when the conductors need to change direction. Since it is not possible for the conductors to curve horizontally, the only option is to change directions at the masts, which means that there’s an angle between the conductors on either side of the mast. Because of this angle, dead-end masts have to support both the vertical and the horizontal force caused by the conductors. Dead-end masts are built more heavily than suspension masts. The masts are usually designed so that the crossbars divide the angle in two even angles (meaning straight through the middle of the angle), since this has proven to be the most effective way to place the

mast. Another function of these masts is to prevent a domino effect when conductors or an entire mast fail. To avoid this problem, they are placed at regular intervals. Dead-end masts are recognizable by their horizontal insulators (in the same direction as the conductors).

End mast
An end mast simply signifies the end (or beginning) of a high voltage transmission line. Since the conductors are only on one side of the mast, they have to be designed for horizontal and vertical forces from the conductors. The insulators for end masts are in the same direction as the conductors.

Transposition mast
Transposition masts are used to change the relative physical positions of the conductors. This is usually done to prevent asymmetry in the three-phase system. The important part of the transposition mast is that it connects the different conductors with each other, while maintaining enough clearance for the conductors. Thus the electrical impedance between the phases of the circuit is minimized.

2.7 Types of masts

Transmission towers can be designed in very different ways. The reason for this variety is complex and has several causes ranging from mast function to transport possibilities. Masts can be divided in three categories based on the type of construction [2]:

1. Truss masts
2. Tube masts
3. Guyed masts

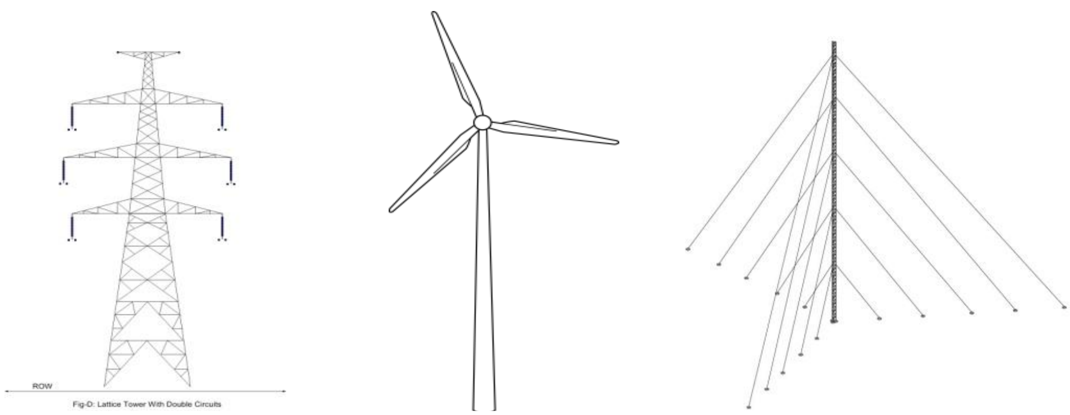


Figure 4: Truss mast (L), wind turbine tube mast (M), guyed mast (R) [VI][VII][VIII]

2.7.1 Truss masts

Truss masts are the most applied type of transmission towers. A couple of these types are described in more detail below [IX].

2.7.1.1 Asymenkelmast

The Asymenkelmast is defined by an asymmetric tower with three ‘half’ crossbars and usually a single circuit system. On one side the conductors are carried at the top and bottom levels, while on the other side one conductor is carried at the middle level. This give the circuit a triangular form. These masts are usually used for low voltages (70 kV).

Figure 5: Asymenkelmast schematic (L) & practice (R) [IX]

2.7.1.2 Halfsymenkelmast

The Halfsymenkelmast is defined by an asymmetric tower with one “complete” crossbar and one ‘half’ crossbar, usually a single circuit system. It consists of two levels with on one level a crossbar carrying two conductors (one on each side) and on the other level a “half” crossbar carrying the third conductor. This third conductor can either be on the left or right of the mast, as well as above or below the “complete” crossbar.

Figure 6: Halfsymenkelmast schematic (L) & practice (R) [IX]

2.7.1.3. Donaumast

In the Netherlands, Donaumasts are by far the most applied type of transmission tower (about 50% of all transmission towers). The Donaumast is defined by a tower with two crossbars and a two-circuit system. In general it tends to have a triangular form. The main disadvantage is the high magnetic field.

Figure 7: Donaumast schematic (L) & practice (R) [IX]

2.7.1.4 Vlakmasts

Eénvlakmast ('hamerkop')

The éénvlakmast is defined by a tower with a single crossbar and a two-circuit system. It is characterized by a low mast height and thus has a reduced visual impact. This low height can be a disadvantage as well, as there are many cases where there have been short circuits between the conductors and machines on the ground. It also has a very high magnetic field.

Figure 8: Eénvlakmast schematic (L) & practice (R) [IX]

Tweevlaksmast

The tweevlaksmast is quite large and is basically two éénvlakmasts on top of each other. It is defined by one tower with two crossbars and a four-circuit system, with each crossbar carrying two circuits. Aside from having a similar appearance to the Donau mast, it shares the disadvantage of having a high magnetic field as well.

Figure 9: Tweevlaksmast schematic (L) & practice (R) [IX]

Drievlaksmast ('dennenboommast')

The drievlaksmast is a fairly popular slender mast with a very large height. It is defined by one tower with three crossbars and a two-circuit system, with each crossbar carrying one circuit.

Figure 10: Drievlaksmast schematic (L) & practice (R) [IX]

2.7.1.5. Tonmast

Tonmasts are probably the most commonly known masts. They are defined by one tower with three crossbars and a two-circuit system. Although Tonmasts look very similar to Drievlaksmasts, there is an important difference: The crossbars on the middle level are longer than the top and bottom ones, causing the middle conductors to be carried at a larger distance from the tower. This causes the mast to look somewhat hexagonal giving it a very distinct look.

Figure 11: Tonmast schematic (L) & practice (R) [IX]

2.7.1.6. Deltamast

This mast is defined by a tower that splits into two at a certain height. Above that split there are an undefined number of levels and one or more so called girders or cross girders. The mast can be a single circuit system (see Figure 12L) or a two (even three) circuit system (see Figure 12R).

Figure 12: Deltamast schematic (L), practice single circuit (M), practice three circuit (R) [IX]

2.7.2 Tube masts

Tube masts are usually designed with a circular cross section. Most of the basic types as explained for truss masts can be applied to tube masts as well. A big difference between truss masts and tube masts is the 2nd order effect, which is far more pronounced in tube masts because of their usual very slender design.

Bipole mast

This mast is characterized by a bipole configuration i.e. two towers that are completely separate from each other. There is nothing between the towers that connects them, except in some cases where the same foundation is used for both towers. There are 3 levels with 'half' crossbars carrying one conductor each. Usually a two-circuit system is applied though four-circuit systems are possible as well. One of the main advantages of the bipole mast is that the magnetic field can be reduced drastically, because the conductors are carried much closer to each other.

Figure 13: Bipole mast schematic (L) & practice (R) [IX]

Wintrack

The so called Wintrack mast is a new type of bipole mast, developed by TenneT. This innovative mast can replace the current truss transmission towers and will greatly reduce the magnetic zone as well. By placing the conductors as close as possible to each other, the magnetic field zone can be reduced by more than 60%.

Furthermore the Wintrack mast also provides the ability to combine multiple connections in one single mast (see Figure 14). For example, the existing 150 kV conductors can be combined with the new 380 kV conductors, thus reducing the amount of masts necessary. Thus an optimal balance between supply and spatial integration can be achieved.

Figure 14: Various types of Wintrack masts: Standard mast (2 x 380 kV) (L), four circuit (4 x 380 kV) (M) & combination mast(150 kV + 380 kV) (R)

2.7.3 Guyed masts

Guyed masts are tall thin structures, usually with a high slenderness ratio, that are supported with so called 'guy' lines. One end of the guy-wire is attached to the mast, while the other end is anchored to the ground at some distance from the foot of the mast. These guy-wires are usually tensioned cables designed to add stability to the mast. To ensure that the wires are always in tension, usually an amount of prestressing is applied. This prestressing in the wires will lead to an increased force in the mast, which means it becomes more susceptible to buckling. For guyed masts the 2nd order effects of both the mast and the wires must be taken into account.

Figure 15: Guyed mast [X]

Guy masts are most frequently used for radio towers, sailing masts, wind turbines, etc (see Figure 15).

2.8 Electricity & magnetism

2.8.1 Three-phase electric power

Three-phase electric power is a well-known method of generating, transmitting and distributing alternating-current (AC) electric power [IV]. It is the most common method applied by power grids globally to transmit electricity. Three-phase systems tend to be more cost effective than single-phase or two phase systems at the same voltage, because the amount of conductor material used for transmitting electricity is much lower.

In polyphase systems, three or more circuit conductors carry alternating currents (AC) with a specific time offset between the voltage waves in each conductor. In three-phase systems this time offset is one third of a cycle (see Figure 16). The main advantages of three-phase systems are [IV]:

- The phase currents cancel each other out (completely in case of a linear balanced load). Thus the size of the neutral conductor can be reduced.
- Transfer of power into a linear balanced load is constant, thus the vibration (generator and motor) can be reduced.

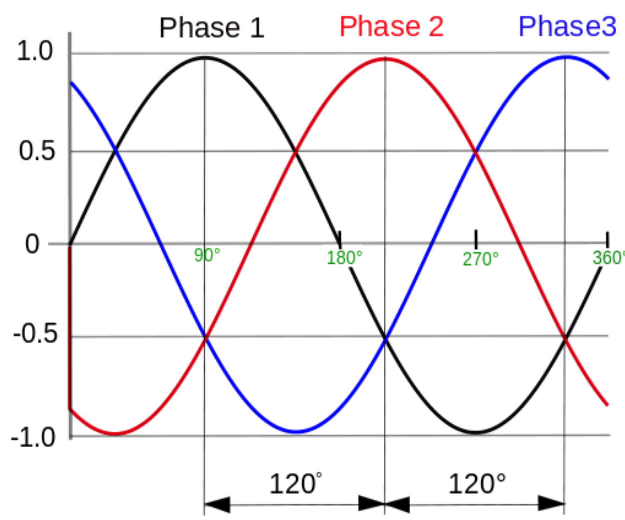


Figure 16: One voltage cycle of a three-phase system [IV]

- Three-phase systems generate a magnetic field that rotates in a specific direction, allowing the design of the electric motors to be simplified.

Most power lines tend to use (high voltage) three-phase AC, while single phase AC is mostly used in railway electric systems.

2.8.2 Circuits

A single-circuit power line implies that only the conductors necessary for one circuit are carried. In case of a three-phase system, this means that a transmission tower will support three conductors. For a double-circuit power line this implied that six conductors are carried, with three conductors per circuit. A drawback of using double-circuit power lines is that maintenance can be difficult, because either both circuits have to be switched off or only one and thus having to work near high voltage. Transmission towers for high voltage lines are usually designed to carry two or more circuits. Sometimes the towers will be overdesigned i.e. designed for more circuits than are actually applied. Multiple circuits do not have to be parallel to each other per se, but can be carried using several levels of crossbars.

2.8.3 Magnetic fields

A very important side effect of overhead power lines are the magnetic fields they emit. The common steel lattice masts have a wide magnetic field causing a large area to be unavailable for use. The enormous advantage of the bipole transmission tower is that the magnetic field can be reduced drastically (see Figure 17). This new generation transmission tower reduces the total maximum 0.4 μT magnetic field zone width to a mere 100 m (compared to the old towers width, larger than 400 m). The demand for the magnetic field reduction is one of the main parameters that decides the mast shape, along with the limitation of “line dancing”.

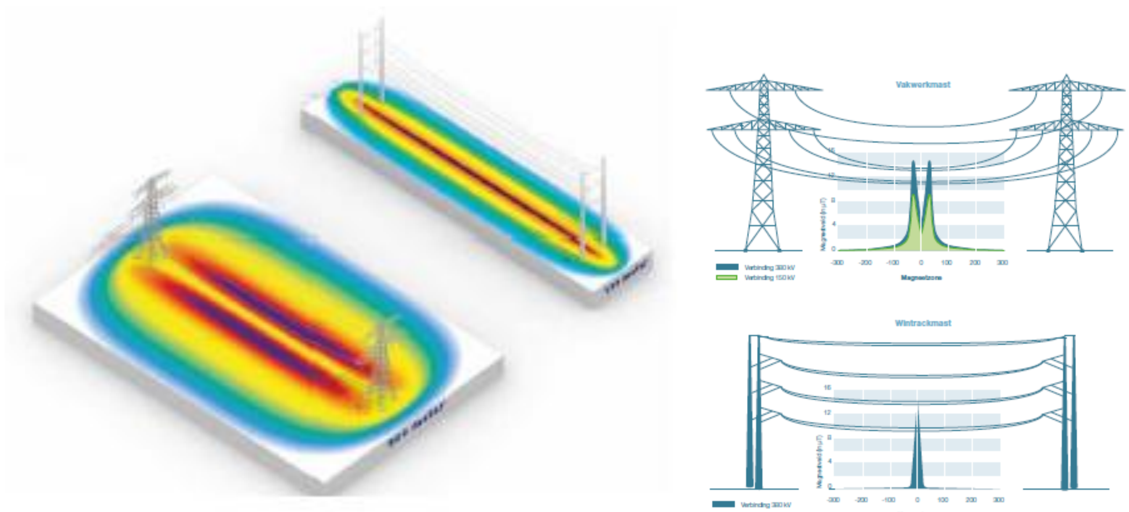


Figure 17: Reduction of magnetic field

The magnetic field around the conductors is caused by the current travelling in the conductor and is inversely proportional to the distance from the center of the conductor i.e. the further away from the conductor, the lower the magnetic field (quadratic relation). The magnitude of the magnetic field is calculated with the 4^{th} law of Maxwell (originally Ampère’s law).

As stated before, for transmission towers mostly double-circuit (or quadruple) conductors are used. These conductors are usually carried in a triangular form i.e. one triangle with three conductors on each side. Because the three-phase system is used, the generated magnetic field is the resultant of current magnitude, phase angle and distance to the conductors. Through practical applications it was noticed that if the left and the right circuit have the same phase sequence, the magnetic field will be very large. Thus it was decided to apply a phase shift on one circuit, by shifting the conductors of that circuit. That way the resulting magnetic field was significantly smaller.

Another method to reduce the magnetic field is reducing the distance between the left and the right circuit. The best way to do this was by applying the conductors in a circular or rectangular form, while still keeping in mind the phase shift. The minimum distance between the conductors (of the same circuit and the other circuit) is decided by the norm NEN-EN 50341 and takes into account the safety aspects when climbing the tower for maintenance while the other circuit is still running.

The general goal when applying the conductors in various ways, is to limit the width of the 0.4 μT magnetic field on ground level as much as possible [XI]. For a more detailed explanation of the workings of the magnetic fields see appendix A.

2.8.4 Lightning wire & passive loop

At the top of transmission towers and travelling above the conductors there is usually another wire which protects the conductors from lightning strikes. These lightning wires are made differently than the conductors, since their only job is to prevent damage to the conductors due to lightning. A problem occurs when applying these lightning wires, because the currents traveling through the conductors will induce currents in the lightning wires. These additional currents will then influence the magnetic field in unfavorable ways. To solve this problem, extra conductors are added beneath the conductors, which are called passive loops (or retour current conductors). The current travelling through the main conductors will then induce a current in the extra retour current conductors. This so called retour current conductor can then actually reduce the magnetic field.

2.9 Environmental impact

In general the area around transmission towers and the area under the overhead power lines are not accessed frequently. There are various reasons for this. For instance during wintertime ice can form on the lines and create hazards for travelers underneath the power lines. Another reason is that because of the magnetic fields, other magnetic based devices brought nearby have trouble functioning (mobiles, radios).

Transmission towers near airfields are usually marked at the top, while the conductors are marked with noticeable plastic reflectors to warn pilots of the danger. Occasionally the towers have bird prevention installations as well, though that is more often done for wind turbines. Some environmental experts argue that transmission towers can disrupt migration routes for animals or chase them from their habitat. These potential effects however can be countered without too much difficulty.

2.9.1 Health concerns

The effect of high voltage electrical transmissions on individuals living in the vicinity, is a subject of debate. Much research has been done in this field, with conflicting results and conclusions. Several studies show no association between exposure to high voltage power lines and the alleged health issues that were measured [3][4][5][6]. Other studies however published results implicating the electrical transmissions as the cause for short or long term health hazards [5][7-13][16][17]. These health effects include, but are not limited to, leukemia [15][XII], amyotrophic lateral sclerosis [6][7][8], Parkinson & Alzheimer's disease [10] and miscarriage [11][12] [13]. Although these studies seem to observe a correlation with various health hazards, the question remains whether this is actually true or just a matter of causation. Another argument against electrical transmission lines is that they attract aerosol pollutants [14][15], although many of these claims are unsubstantiated [4][5][6]. Most of these studies do not reach statistically significant results and conclusions due to a limited sample size. Results also tend to be interpreted differently by researchers with different agendas, allowing the controversy around power lines to continue.

2.9.2 Visual impact

Pollution can take on a wide variety of forms. Visual pollution is the act of polluting the environment through unnatural formations. These formations are unpleasant to look at and are usually seen as a negative change in the environment. Common examples are billboards, litter, transmission towers etc. Although some may be a matter of opinion such as wind turbines. Of course this does not mean that transmission towers are seen as visual pollution indifferent of the location. The effect is most keenly felt in rural areas where there is a clear horizon. In bustling urban areas there is less criticism. For transmission towers, one method of eliminating visual pollution is to simply remove the tower altogether and use underground cables.

3. High & Ultra High Strength Concrete

In this chapter the basic properties of concrete are explained. Because concrete is such a versatile material, the different strength classes all have different production methods, material properties, etc. All of these will be covered and the pros and cons of using each will be made clear.

3.1 Concrete

Concrete is a well-known product all around the world. It comes with a large variety of uses and performances, from bridges and dams to foundations and buildings. To satisfy the demand for various types of structures, concrete as a material has evolved over the years. From the very first discovery of unreinforced concrete to modern self-compacting concrete, concrete has become an essential material.

Concrete is a composite material and is composed of the following materials:

1. Cement
2. Aggregates
3. Water
4. Additives
5. Fillers

Ad 1. Cement

Cement can be seen as the glue of the concrete mix. It binds the aggregates together creating a stable mix. There are various types of cement, distinguished by Roman numbers:

1. CEM I Portland cement
2. CEM II Portland combination cement
3. CEM III Blast Furnace cement
4. CEM IV Puzzolane cement
5. CEM V Composite cement

The most common types of cement used are Portland cement, Blast Furnace cement and Portland fly ash cement. A big disadvantage of cement is the large greenhouse gas emission. Cement production makes up about 5 to 10 % of the world's greenhouse gas emissions.

Ad 2. Aggregates

Aggregates make up the biggest part of the concrete mixture. They consist of large materials (in comparison with cement) and significantly increase the durability of concrete. The aggregates are glued together by the cement matrix. Aggregates are divided in coarse and fine aggregates. Coarse aggregates are large pieces of crushed stone, usually gravel or granite, while fine aggregates are finer materials such as sand.

Ad 3. Water

Water is added to the mixture to react with the cement and create the hardened product commonly known. This chemical process is known as hydration. Water is also necessary for processing/mixing the concrete. The water/cement ratio is a deciding factor in the determination of the concrete strength.

Ad 4. Additives
Additives or admixtures are, just as their name implies, substances that are added to the concrete mix to realize a wide range of properties. They are added to the concrete in small amounts and can influence one or more properties of the concrete. Examples of these properties are better workability, faster (slower) hardening, air bubble formation etc.

Ad 5. Fillers
Fillers are mainly used to improve the packing and workability of the concrete. They can also reduce the amount of cement needed. Fillers consist of very fine particles. Usually inert and natural materials are used such as quartz. Applying fillers improves the properties and microstructure of concrete. Common examples are fly ash, silica fume etc.

By varying the amounts and types of the above ingredients various types of concrete can be realized.

Concrete is generally regarded as a three-component system composed of:

- The aggregates,
- The cement matrix,
- The contact area between the aggregates and the matrix.

This system provides good insight in the transfer of forces inside concrete. The force applied on the concrete, is carried by the contact area between the aggregates. In addition to forces in the main direction, lateral forces emerge as well. These lateral forces are carried by the glue-like cement matrix.

Over the years concrete has slowly developed and adapted to the construction sector. When concrete started exceeding strengths of 100 MPa, it was soon realized that new classification systems were needed. While there are some differences between the various systems that are used, they are all very similar. For concrete with and without steel fibers the following classification can be made based on the compressive strength of concrete:

- Low Strength Concrete LSC $f_{ck} < 25$ MPa
- Ordinary Strength Concrete OSC 25 MPa $\leq f_{ck} < 65$ MPa
- High Strength Concrete HSC 65 MPa $\leq f_{ck} < 115$ MPa
- Very High Strength Concrete VHSC 115 MPa $\leq f_{ck} < 150$ MPa
- Ultra High Strength Concrete UHSC 150 MPa $\leq f_{ck} \leq 250$ MPa
- Super High Strength Concrete SHSC $f_{ck} \geq 260$ MPa

Another popular classification system is one based on High Performance Concrete (HPC). After many developments of concrete, people have started to appreciate the other aspects of concrete beside strength as well, such as workability, durability etc. These criteria can be equally important to strength in some cases. In these cases we talk about High Performance Concrete, where the performance requirements may differ from high strength requirements. This HPC can be applied when for a given load, specific optimized properties are needed based on certain financial, environmental, service and durability requirements. Any concrete mix designed purely to satisfy certain criteria or surpass certain limitations, which ordinary concrete cannot pass, can be called High Performance Concrete. HPC is not based on the use of special machinery or materials, but on a careful design and production process. Some improvements that HPC can induce are improved resistance to the environment, improved durability, less micro cracking, less construction time and a large variety of other benefits.

It is difficult to provide a clear definition for HPC, because the improved performance properties can be so variable. The American Concrete Institute defines High Performance Concrete as: "*A concrete which meets special performance and uniformity requirements that cannot always be achieved routinely by using only conventional materials and normal mixing, placing and curing practices*" [XIII].

Concrete that meets these special performance and uniformity requirements, generally achieves High Strength, but conversely a High Strength concrete is not necessarily a High performance concrete as well. A classification of HPC based on compressive strength is shown in Table 1.

Compressive strength (MPa)	50	75	100	125	150
High Performance class	I	II	III	IV	V

Table 1: Classification of High Performance Concrete [XIII]

High Performance Concrete can be produced by following three vital correlated steps [XIII]:

- Choosing ingredients with appropriate properties (such as strength, durability, workability, etc.) for the specified concrete;
- Deciding the amount of ingredients necessary to achieve the specified properties;
- Thorough quality control of all phases in the concrete production process.

3.1.1 Traditional Concrete

Traditional or ordinary concrete (OSC) is the most basic and well known type of concrete applied for pretty much all small scale projects. It can be produced by simply following mixture instructions on the cement packs. The aggregates most often used are gravel and sand. The aggregates tend to be the strongest component in ordinary concrete. Cracks appear parallel to the applied force on the contact between the aggregates and the cement matrix. These micro cracks propagate through the concrete and form macro cracks, which will eventually lead to failure. Ordinary concrete is usually made with strengths ranging from 20 to 50 MPa.

3.1.2 High Strength Concrete

Over the last two decades High Strength Concrete (HSC) has clearly established its place in the concrete industry. This type of concrete introduced a lot of advantages such as higher compressive strength, earlier demoulding, larger and more spacious high-rise building (reduced column size), larger durability, flexural strength etc. A major drawback however is the reduced ductility, causing unreinforced HSC to be a very brittle material.

The production of HSC requires improvements in the regular production process:

1. Increase of packing density;
2. Increase of strength of cement matrix;
3. Increase of aggregate strength.

HSC is achieved by lowering the water-cement ratio to below 0.35. This can be done by either reducing the amount of water used, by adding more cement or a combination of both. Usually to retain workable concrete, super plasticizers are added to the mixture. Often silica fume is added to the mixture as well. HSC compressive strengths tend to range around 85 to 105 MPa.

3.1.3 Ultra High Strength Concrete

Ultra High Strength Concrete (UHSC) has slowly begun to emerge during the last decade or so. It is characterized as steel fiber reinforced concrete with compressive strengths exceeding 150 MPa. Recently, strengths which were regarded as impossible a decade ago (> 500 MPa), have become possible, although it is then called Super High Strength Concrete in some regions. The concept of UHSC or Ultra High Performance Fiber Reinforced Concrete (UHPFRC) relies on four basic principles:

1. Improvement of the homogeneity
2. Increase of the packing density
3. Improvement of the microstructure
4. Increase of the ductility (steel fibers)

Ad 1. Improvement of the homogeneity

In the concrete mixture, aggregates have higher Young's modulus than the cement matrix. This causes stress peaks to appear on the aggregates, causing them to fail before the cement matrix. By replacing the course aggregates with finer aggregates, the concrete becomes more homogenous and the amount and magnitude of the stress peaks is reduced. To attain optimal homogeneity, the strength of the fine aggregates and the cement matrix should be as close as possible to each other.

Ad 2. Increase of the packing density

To improve the packing density, a careful design is necessary where the aggregate sizes fit perfectly together, only leaving minute open spaces. This can be done by applying a discontinuous sieve analysis, allowing the various aggregate diameters to stay within a certain range and thus easily complementing each other.

Ad 3. Improvement of the microstructure

To improve the microstructure, the hardening of the concrete can take place at high temperatures or high atmospheric pressure, although this is not always necessary. It depends on the required strength and the possibilities of the used materials. For instance when producing C200, no increased temperature or pressure is needed. A temperature of 20 ºC will suffice. Higher temperatures until 90 ºC will accelerate the puzzolane reaction of e.g. silica fume, creating a much compacter and interwoven microstructure. Between 250 ºC and 400 ºC crystalline hydrates are formed, water is extracted from the cement paste, causing the material properties to approximate those of a ceramic material [16].

Ad 4. Increase of the ductility

As the strength of unreinforced concrete increases, the ductility decreases. The ductility of the concrete can be increased by adding steel fibers to the mix. The amount of steel fibers used can range from 100 kg/m^3 up to 700 kg/m^3 or more. Either short fibers (3mm), long fibers (13mm) or both can be used with varying results.

3.2 Material properties

High Strength Concrete has already been included in the Eurocodes (NEN-EN 1992-1-1), alongside Ordinary Concrete. Ultra High Strength Concrete on the other hand has not, because there is still much unknown. Another big reason is that the material properties of UHSC heavily depend on the material composition. Also the utilized steel fibers have a large influence on the material properties. Most of the data on UHSC presented in the following paragraphs is based on the French recommendations [17] and research done at TU Delft [18][19].

3.2.1 Compressive strength

The compressive strength of concrete is one of the most common material properties used by engineers in designing structures. The compressive strength is measured by placing cylindrical or cubical concrete specimens in a compression testing machine usually after 7 or 28 days. The received results are primarily used to determine if the delivered concrete mixture meets the client's requirements.

According to Eurocode 2 for OSC and HSC, the compressive strength of concrete is designated by concrete strength classes which relate to the characteristic (5%) cylinder strength f_{ck}, or the cube strength $f_{ck,cube}$. The strength classes in this code are based on the characteristic cylinder strength f_{ck} determined at 28 days with a maximum value of C90/105. The design value of high strength concrete can be determined by:

$f_{cd} = f_{ck}/\gamma_m$

with:
$f_{ck} = 15 - 105$ N/mm^2
$\gamma_m = 1.5$

For UHSC the same formula will be applied with a possible variation in the material factor:

$f_{cd} = \alpha_{cc} * f_{ck}/\gamma_m$

with:
$f_{ck} = 150\text{-}250$ N/mm^2
$\alpha_{cc} = 1.0$ (for ribs with dimensions of 150x150x150mm)
$\gamma_m = 1.5/\gamma_E$
γ_E = safety coefficient

Note:
The above expressions can be applied for test cubes with ribs of 150 mm. However when testing higher strength class concretes, often cubes with smaller dimensions are used. This is related to the pressure capacity of the cubepressure machine. When using smaller cubes, there is a larger spread of the result as well as a higher strength achieved. Therefore a factor α_{cc} must be applied in order to compare the cubes measurement with the standard ribs of 150 mm (see Figure 18) [20].

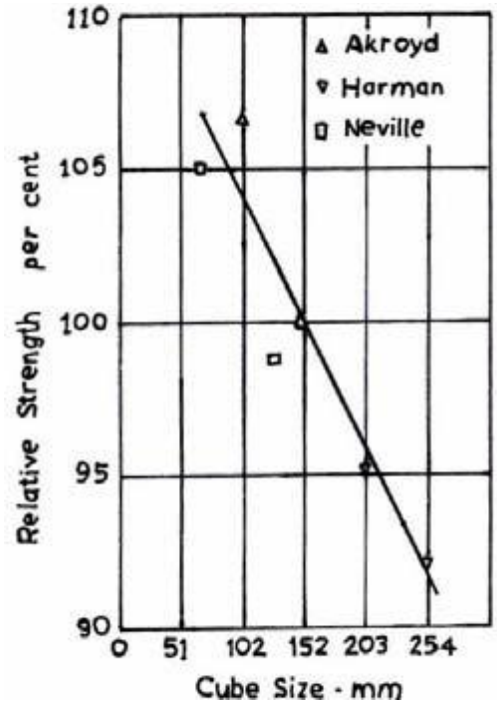


Figure 18: Relative compressive strength dependent on cube dimensions [21]

3.2.2 Young's modulus & Poisson ratio

Young's modulus

The Young's or elasticity modulus is one of the most important characteristics of concrete. It is defined as the ratio between (compressive or tensile) stress and the corresponding strain of the concrete. For the design this parameter is important for predicting the deformations. The Young's modulus of concrete is largely influenced by the properties of the coarse aggregates. By using aggregates with an increased size or stiffness, the Young's modulus increases as well.

For each strength class up till C90/105 the Young's modulus can be found in Eurocode 2. For UHSC it is a bit more complicated. According to the French Recommendations there is no simple useable formula to determine the Young's modulus. Because the Young's modulus depends heavily on material composition the only way to accurately determine it, is either through experimental means or through a complicated model (LCPC). If at the start of the project nothing else is known, a guideline value of 55000 N/mm^2 can be used [17].

Poisson ratio

For OSC and HSC the Poisson's ratio may be taken equal to 0.2 for uncracked concrete and 0 for cracked concrete. Research has shown that for UHSC with cylindrical compressive strength between 150 and 200 N/mm^2, the Poisson ratio is about 0.18 and 0.19 for fine aggregates and 0.21 for coarse basalt aggregates (5 to 8 mm) [21]. These values correspond to the OSC & HSC value of 0.2. Therefore if mix composition and other details are unknown a value of 0.2 can be taken for the Poisson ratio.

3.2.3 (Flexural) tensile strength

Tensile strength

According to Eurocode 2 the tensile strength refers to the highest stress reached under concentric tensile loading. Where the tensile strength is determined as the splitting tensile strength, $f_{ct,sp}$, an approximate value of the axial tensile strength, f_{ct}, may be taken as:

$f_{ct} = 0{,}9f_{ct,sp}$

For UHSC, the tensile strength can be determined by notched prisms, cylinders or sawn flat specimens of plates. The direct tensile-strength procedure is defined in the AFREM recommendations [17]. Previous research [21] has shown that the tensile strength of coarse and fine aggregate UHSC does not differ much. Because the scatter in results is much higher for UHSC, the amount of specimens should be increased to 6 test specimens compared to the usual 3 for OSC. This holds for centric tensile loading tests as well as flexural tensile tests.

Flexural tension

According to Eurocode 2 the mean flexural tensile strength of reinforced concrete members depends on the mean axial tensile strength and the depth of the cross-section. The following relationship may be used:

$f_{ctm,fl} = \max\{(1.6 – h/1000)f_{ctm} ; f_{ctm}\}$

where:

h is the total member depth in mm

f_{ctm} is the mean axial tensile strength

The relation given above also applies for the characteristic tensile strength values.

For UHSC two types of test are proposed: firstly, third-point flexural tests for determining the tensile strength following correction for scale effect; secondly, centrepoint flexural tests using notched prisms, to determine the contribution of fibers as reinforcement of a cracked section, after application of the so-called ‘back-analysis’ method [17]. By means of modeling the tensile strength can be deduced from the flexural tension strength. Additionally a thickness effect can be recognized in the test results. This relation is based on fracture mechanics. Because of the parallel orientation of the fibers to the cross sectional edges, the thickness effect is increased [19]. The flexural tensile strength depends heavily on the applied amount of fibers (volume fraction) in the mixture [22].

3.2.4 Stress-strain relationship

Eurocode 2 defines a couple of stress-strain (σ-ε) relationships for OSC & HSC. Usually a bi-linear stress-strain relation is used (see Figure 19). The analysis for standard sections is carried out with the following fundamental assumptions:

- Plane sections remain plane,
- The strain in bonded reinforcement or bonded prestressing tendons, whether in tension or in compression, is the same as that in the surrounding concrete,
- The tensile strength of the concrete is ignored,
- The stresses in the concrete in compression are derived from the design stress/strain relationships in Eurocode 2.

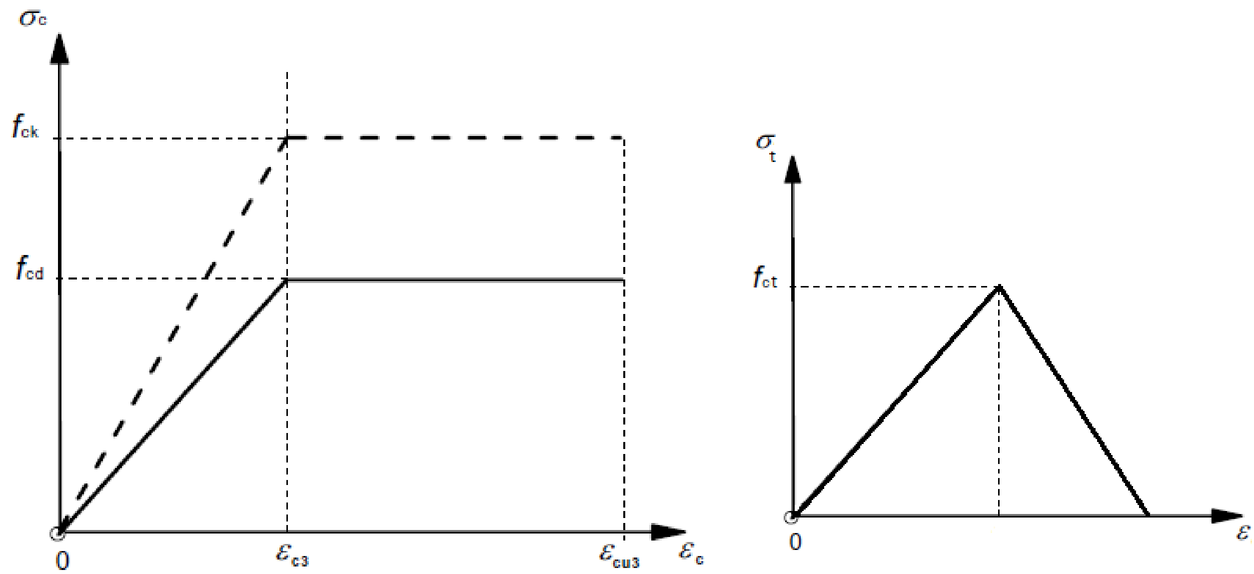


Figure 19: Bi-linear compressive (L) and tensile (R) stress-strain relation

A rectangular stress distribution may be assumed (see Figure 20).

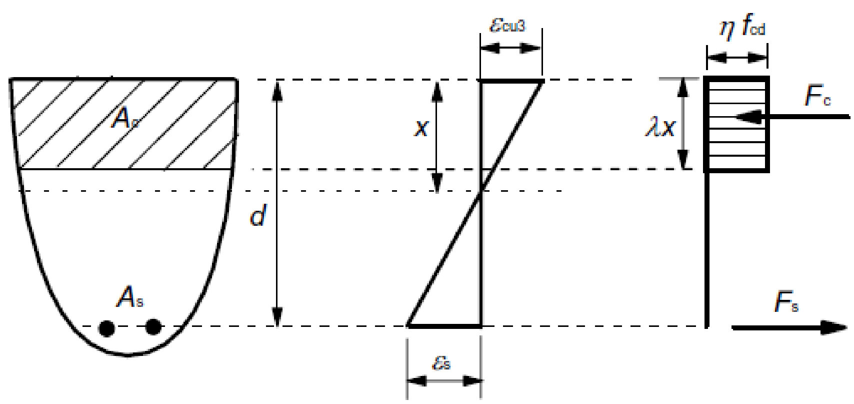


Figure 20: Rectangular stress distribution

For UHSC these relations are a bit different, because the addition of steel fibers in the mix introduces a post-peak behavior (post-cracking constitutive law) in the tension part. Depending on the amount of steel fibers, the occurring loads and, to a lesser extent, the concrete strength, either hardening or softening will occur after crack formation. As the two strain behaviors lead to different stress-strain diagrams, it means that they have different properties and different methods of calculation as well.

The initial design assumptions are different from OSC & HSC as well. The analysis for standard sections for UHSC is carried out with the following two fundamental assumptions:

- Plane sections remain plane,
- Stresses in the uncracked part of the concrete are proportional to strains.

The stress-strain relation of UHSC with strain hardening behavior loaded in compression consists of two stages that are similar to OSC & HSC (see Figure 21):

a) Elastic phase of the compression zone
b) Plastic phase: The concrete matrix starts to fail. The fibers keep the matrix together till the ultimate strain is reached.

The stress-strain relation of UHSC with strain hardening behavior loaded in tension consists of four stages (see Figure 21):

1. Elastic phase: Slope is the same as stage a. The elastic stage is limited by the tensile strength of the cement matrix f_{tj}. The small fibers allow for a greater tensile strength compared to OSC;
2. Hardening behavior: A slight increase in stress still occurs after the first crack. The concrete matrix fails, the micro cracks develop into macro cracks and the long fibers are activated (multiple cracking behavior). Eventually a fully formed crack pattern is reached;
3. Softening behavior: The deformations increase, the crack width increases and the long fibers start to fail or are pulled out of the concrete (fiber pull-out is preferred);
4. Failure: All fibers have failed along with the concrete matrix.

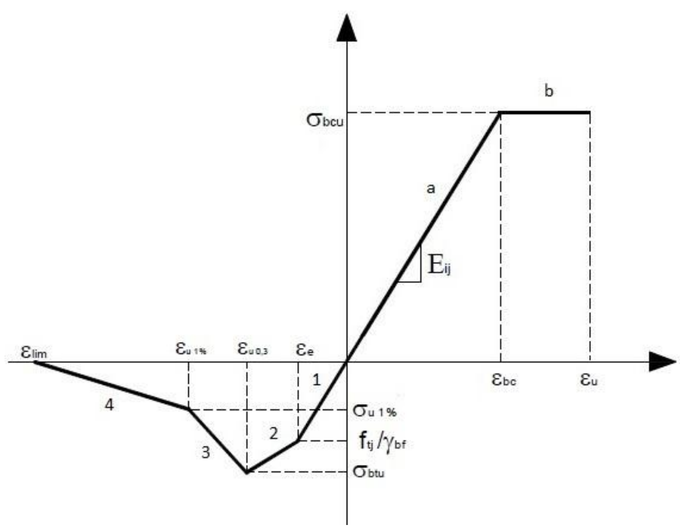


Figure 21: ULS Stress-strain relation with strain hardening law [17]

With:
Strain at plastic deformation: ε_{bc}
Maximum compressive strain: $\varepsilon_u = 3‰$

Strain at first crack occurrence: $\varepsilon_e = f_{tj}/E_{ij}$
Strain at crack width of 0.3 mm: $\varepsilon_{u0.3} = w_{0.3}/l_c + f_{tj}/E_{ij}$ ($w_{0.3} = 0.3$ mm)
Strain at crack width of 1%: $\varepsilon_{u1\%} = w_{1\%}/l_c + f_{tj}/\gamma_{bf}E_{ij}$ ($w_{1\%} = 0.01H$)
Maximum allowable strain: $\varepsilon_{lim} = l_f/4l_c$ (l_f = length of a fiber, l_c = characteristic length in mm)
Tensile stress at first crack occurrence: f_{tj}
Maximum compressive stress: $\sigma_{bc} = 0.85f_{cj}/(\theta*\gamma_b)$
Maximum tensile stress: $\sigma_{btu} = \sigma(w_{0.3})/K\gamma_{bf}$
Characteristic tensile stress at a crack width of 1%: $\sigma_{u1\%} = \sigma(w_{1\%})/K\gamma_{bf}$

Contrary to a strain hardening material, where the hardening occurs in uniaxial tension, the hardening can occur in bending as well, in which case the material is defined as a deflection hardening material. This distinction depends on the fiber content. Because of the probability of stress distribution, a lower fiber content is necessary to realize deflection hardening in bending compared to strain hardening in case of uniaxial tension [22].

Concrete exhibiting strain softening behavior (non-hardening), is characterized by the lacking of phase 2 (see Figure 22).

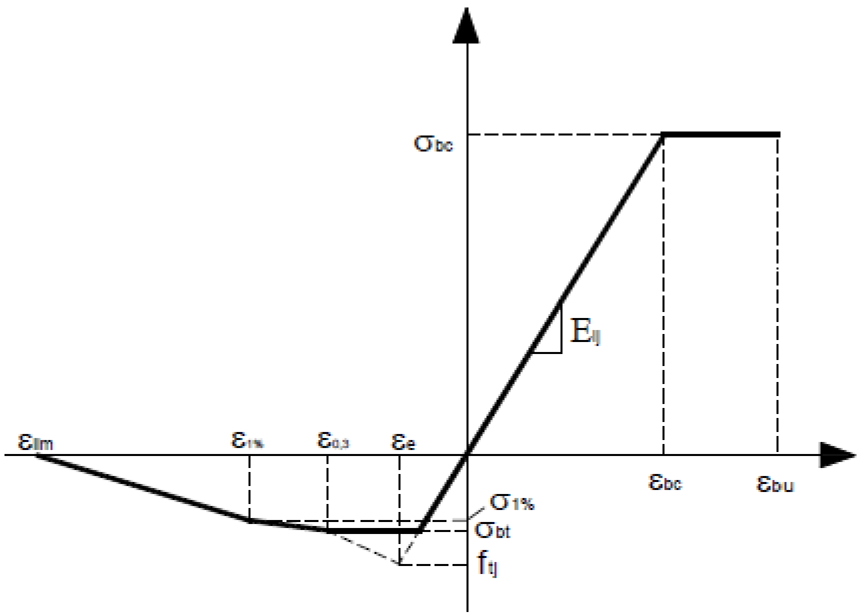


Figure 22: ULS Stress-strain relation with strain softening law [17]

Figure 23 illustrates that while the Young's modulus increases for higher concrete strength classes, the limit for the ultimate concrete strain decreases.

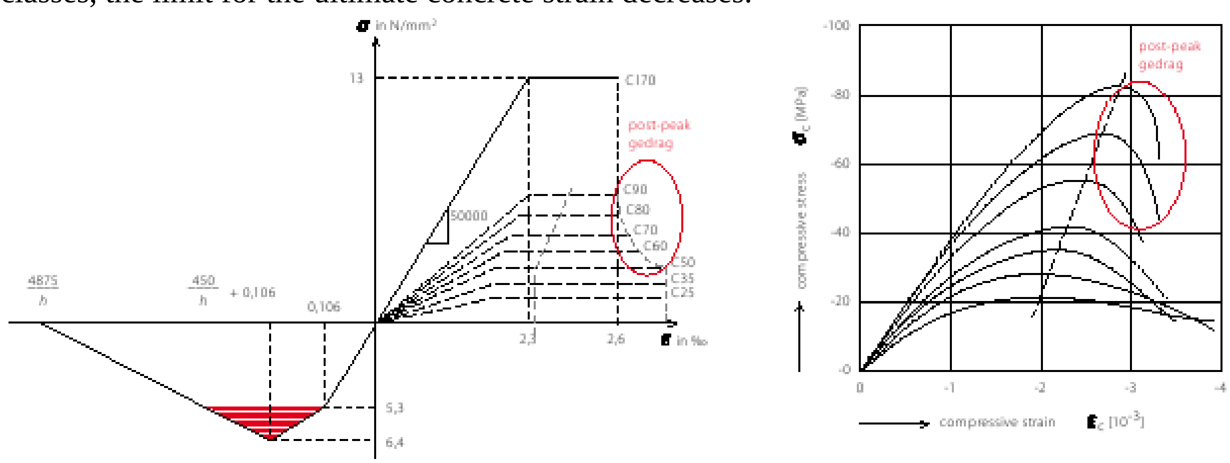


Figure 23: Ideal stress-strain diagram (with l_f = 13 mm & l_c = 2/3h) (L) and actual stress-strain diagram (R) for various strength classes [23]

3.2.5 Creep

Creep is defined as the increasing deformation of concrete under a constant load or stress. The amount of creep deformation depends on the material properties, contact time and magnitude of the load, outside temperature and other factors. Creep is a time-dependent deformation which allows strains to accumulate.

The creep coefficient $\varphi(t,t_0)$ can be obtained through the use of design diagrams given in Eurocode 2 (see Figure 24 & Figure 25). This method is only valid if the concrete is not subjected to a compressive stress greater than 0.45 $f_{ck}(t_0)$ with t_0 being the age of the concrete at the time of loading.

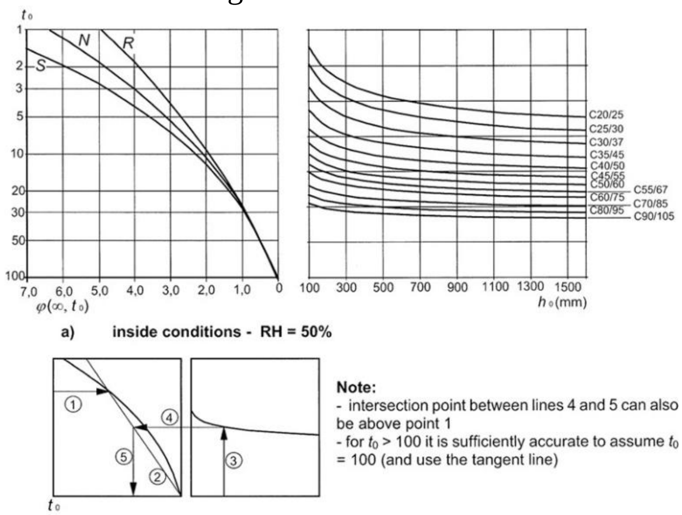


Figure 24: Method for determining the creep coefficient $\varphi(t,t_0)$ for concrete under normal environmental conditions (inside)

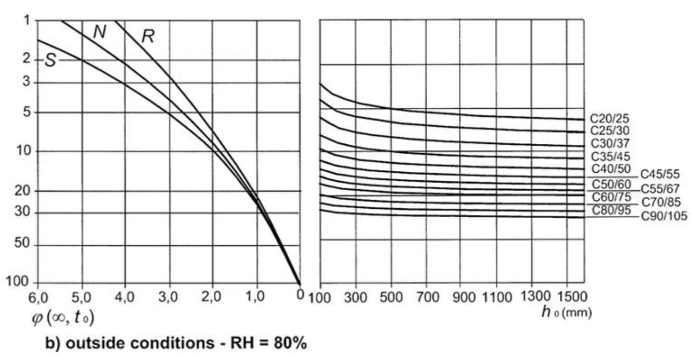


Figure 25: Method for determining the creep coefficient $\varphi(t,t_0)$ for concrete under normal environmental conditions (outside)

The creep deformation of concrete $\varepsilon_{cc}(\infty,t_0)$ at $t=\infty$ for a constant compressive stress σ_c applied at t_0, is given by:

$\varepsilon_{cc}(\infty,t_0) = \varphi(t,t_0)*(\sigma_c/E_c)$ with $E_c = 1.05E_{cm}$

The creep coefficient of UHSC is lower than that of HSC, although the difference is not as big as between HSC and OSC. For OSC usually a creep coefficient of around 2.0 is obtained, while for HSC and UHSC it is respectively 1.0 and 0.8. For UHSC it is important to distinguish between

concrete with and without heat treatment. Without heat treatment UHSC behaves according to the formulas specified below (φ~1.0), whereas with heat treatment it has practically no shrinkage and little creep (φ~0.3).
The expression for specific basic creep for UHSC is given by [17]:
$\varepsilon_s = k(t_0)*f(t-t_0) + h(t_0)$
with:

$$k(t_0) = 19*\exp\sqrt{\frac{0.1}{t_0-2.65}}$$

$$f(t-t_0) = \frac{\sqrt{\frac{t-t_0}{3t_0-5}}}{\sqrt{\frac{t-t_0}{3t_0-5}}+1}$$

$$h(t_0) = 18*\exp\sqrt{\frac{0.2}{t_0+1.2}}$$

3.2.6 Shrinkage

Shrinkage is defined as the volumetric reduction of concrete due to the evaporation of water from the concrete during and after hardening. During shrinkage tensile stresses are generated and usually cracks occur.

The shrinkage strain ε_{cs} is composed of two components:

- Drying shrinkage strain: $\varepsilon_{cd}(t) = \beta_{ds}(t,t_s)*k_h*\varepsilon_{cd,0}$

With:

$$\beta_{ds}(t,t_s) = \frac{(t-t_s)}{(t-t_s)+0.04\sqrt{h_0{}^3}}$$

t = age of concrete at time considered
t_s = age at beginning of drying shrinkage (mostly end of curing)
$h_0 = 2A_c/u$
$\varepsilon_{cd,0}$ = basic drying shrinkage (see Table 2)
k_h = factor that allows for shape and size of the cross-section (see Table 3)

$f_{ck}/f_{ck,cube}$ (MPa)	Relative Humidity (in $^0/_0$)					
	20	40	60	80	90	100
20/25	0.62	0.58	0.49	0.30	0.17	0.00
40/50	0.48	0.46	0.38	0.24	0.13	0.00
60/75	0.38	0.36	0.30	0.19	0.10	0.00
80/95	0.30	0.28	0.24	0.15	0.08	0.00
90/105	0.27	0.25	0.21	0.13	0.07	0.00

Table 2: Nominal unrestrained drying shrinkage values $\varepsilon_{cd,0}$ (in ‰) for concrete cement class N

h_0	k_h
100	1.0
200	0.85
300	0.75
≥ 500	0.70

Table 3: Values for k_h

- Autogenous shrinkage strain: $\varepsilon_{ca}(t) = \beta_{as}(t)*\varepsilon_{ca}(\infty)$

With:
$\beta_{as}(t) = 1 - e^{(-0.2t^{0.5})}$
$\varepsilon_{ca}(\infty) = 2.5(f_{ck}-10)10^{-6}$

Total shrinkage is then: $\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$

For UHSC autogenous shrinkage is governing. Because of the low water-cement factor, all the water is rapidly drawn into the hydration process and as the demand for more water increases, very fine capillaries are created. The surface tension inside the capillaries causes autogenous shrinkage (sometimes called chemical shrinkage) which can cause cracks to appear. UHSC is therefore very sensitive to autogenous shrinkage and measures must be taken to prevent this as much as possible. One such measure is heat treatment. If UHSC has been heat treated, zero to none shrinkage occurs. Without heat treatment a guideline value of 550 µm/m can be considered [17].

If the water-cement factor is known the following values can be used for $\varepsilon_r(\infty)$ [17]:
w/c = 0.09 then ε_r = 250 µm/m
w/c = 0.15 then ε_r = 350 µm/m
w/c = 0.17-0.20 then ε_r = 550 µm/m

The expression for autogenous shrinkage development over time is:
$\varepsilon_r(t) = A\exp\left[\frac{B}{\sqrt{t+C}}\right]$
with:
A = $\varepsilon_r(\infty)$
B = -2.5
C = -0.5

Note: Data on creep and shrinkage of UHPFRC is still quite incomplete. The specific conditions of laboratory tests and the sometimes very evolutionary nature of UHPFRC under early loading, can lead to large imprecisions in the present design methods [17].

3.2.7 Shear

The shear strength of a material is defined as the strength of the material against a shear load i.e. an applied force or deformation parallel to the cross section.

The design value for shear resistance for OSC & HSC is given by Eurocode 2 as:

$V_{Rd,c} = [C_{Rd,c}*k*(100*\rho_1*f_{ck})^{1/3} + k_1*\sigma_{cp}]b_w d \geq (v_{min} + k_1\sigma_{cp})b_w d$

With:
$C_{Rd,c} = 0.18/\gamma_c$
$k = 1+\sqrt{\frac{200}{d}} \leq 2.0$
$\rho_1 = \frac{A_{sl}}{b_w d} = \frac{A_s+A_p}{b_w d} \leq 0.02$
A_{sl} = the area of the tensile reinforcement, which extends ≥(l_{bd}+d) beyond the section considered
$k_1 = 0.15$
$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2f_{cd}$ [MPa]
N_{Ed} = the axial force in the cross-section due to loading or prestressing [N]
b_w = is the smallest width of the cross-section in the tensile area [mm]
$v_{min} = 0.035*k^{3/2}*f_{ck}^{1/2}$

In prestressed members without shear reinforcement, the shear resistance of the regions cracked in bending may be calculated using the previous expression for $V_{Rd,c}$. In regions uncracked in bending (where the flexural tensile stress is smaller than $f_{ctk,0.05}/\gamma_c$) the shear resistance should be limited by the tensile strength of the concrete. In these regions the shear resistance is given by:

$V_{Rd,c} = \frac{I*b_w}{S}\sqrt{f_{ctd}^2 + \alpha_1\sigma_{cp}f_{ctd}}$

With:
I = the second moment of area
b_w = the width of the cross-section at the centroidal axis, allowing for the presence of ducts
S = the first moment of area above and about the centroidal axis
$\alpha_l = l_x/l_{pt2} \leq 1.0$ for pretensioned tendons, = 1.0 for other types of prestressing
l_x = the distance of section considered from the starting point of the transmission length
l_{pt2} = the upper bound value of the transmission length of the prestressing element
σ_{cp} = the concrete compressive stress at the centroidal axis due to axial loading and/or prestressing ($\sigma_{cp} = N_{Ed}/A_c$ in MPa)

For members with vertical shear reinforcement, the shear resistance, V_{Rd} is the smaller value of:
$V_{Rd,s} = \frac{A_{sw}}{s} z*f_{ywd}*\cot\theta$
with:
A_{sw} = the cross-sectional area of the shear reinforcement
s = the spacing of the stirrups
f_{ywd} = the design yield strength of the shear reinforcement
ν_1 = a strength reduction factor for concrete cracked in shear
α_{cw} = a coefficient taking account of the state of the stress in the compression chord

For UHSC the process is a bit different mainly due to the addition of steel fibers to the mix. The fibers increase the shear resistance of the composition. Because the composition of UHSC is different from OSC, aggregate interlock will occur, causing the concrete shear strength to change [24].

The ultimate shear strength V_u is given by [17]:
$V_u = V_{Rb} + V_a + V_f$
With:
V_{Rb} = the term for the contribution of the concrete
V_a = the term for the contribution of the reinforcement
V_f = the term for the contribution of the fibers

The term V_a is the traditional way of calculating the shear resistance for OSC & HSC:
$V_a = \frac{A_{sw}}{s} z * f_{ywd} * \cot\theta$

For the term V_{Rb} we can distinguish between reinforced and prestressed concrete:

- For reinforced concrete:

$$V_{Rb} = \frac{1}{\gamma_E} * \frac{0.21}{\gamma_b} * k * \sqrt{f_{cj}} * b_0 * d$$

$k = 1 + \frac{3 * \sigma_{cm}}{f_{tj}}$ in compression
$k = 1 - \frac{0.7 * \sigma_{tm}}{f_{tj}}$ in tension
$\gamma_E = 1.5/\gamma_b$

- For prestressed concrete:

$$V_{Rb} = \frac{1}{\gamma_E} * \frac{0.24}{\gamma_b} * \sqrt{f_{cj}} * b_0 * z$$

The fibers contribute to the shear strength with the term:
$V_f = \frac{S * \sigma_p}{\gamma_{bf} * \tan\beta_u}$
With:
σ_p = the residual tensile strength: $\frac{1}{K} * \frac{1}{w_{lim}} * \int_0^{w_{lim}} \sigma(w) dw$
$w_{lim} = \max(w_u; 0.3\ \text{mm})$
$w_u = l_c * \varepsilon_u$
$\sigma(w)$ = the experimental characteristic post-cracking stress for crackwidth w
w_u = the ultimate crack width, i.e. the value attained at the ULS for resistance to combined stresses, on the outer fiber, under the moment exerted in the section
S = the area of fiber effect, estimated with: $S = 0.9 * b_0 * d$ or $b_0 * z$ for rectangular or Tee sections, and $S = 0.8 * (0.9 * d)^2$ or $0.8z^2$ for circular sections
K = the orientation coefficient for general effects
γ_{bf} = 1.3 in case of fundamental combinations

γ_{bf} = 1.05 in case of accidental combinations

For the verification of the concrete, the shear stress $\tau_{red,u}$ must be no more than:

$\tau_{red,u} \leq 1.14*\frac{0.85}{\gamma_E*\gamma_b} * f_{cj}^{2/3} * \sin(2\beta_u)$ with $\beta_u \geq 30°$

3.2.8 Crack width

Concrete cracking is an unavoidable phenomenon. It is usually the result of factors such as applied loads, shrinkage, thermal effects, restraints, corrosion of reinforcement, etc. Cracks are generally an unattractive happening for the public and people tend to conclude that a structure has failed when they see a crack, although this is usually not the case. While cracking is almost impossible to prevent, it can be significantly reduced or controlled when taking into account the possible causes in the design.

In Eurocode 2, a limiting crack width w_{max} is introduced to ensure that the structure does not experience detrimental cracking (Table 4)

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XS1, XS2, XS3		Decompression

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

Table 4: Recommended values of w_{max} (mm)

The actual crack width can be calculated as follows:

$w_k = s_{r,max}*(\varepsilon_{sm} - \varepsilon_{cm})$

With:

$s_{r,max}$ = maximum crack spacing = $k_3*c + k_1*k_2*k_4*Ø_s/ \rho_{p,eff}$

k_1 = 0.8 (for plain surface bars) & 1.6 (for prestressing tendons)

k_2 = 0.5 (for bending)

k_3 = 3.4

k_4 = 0.425

c = cover to longitudinal reinforcement

$Ø_s$ = bar diameter

$\rho_{p,eff} = (A_s + \xi_1^2 * A_p')/A_{c,eff}$

A_p' = area of pre or post-tensioned tendons within $A_{c,eff}$
$A_{c,eff}$ = effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth $h_{c,eff}$

ξ_1 = adjusted ratio of bond strength = $\sqrt{\xi * \frac{\O_s}{\O_p}}$

$\O_p = 1.6\sqrt{A_p}$
ε_{sm} = mean strain in the reinforcement under the relevant combination of loads
ε_{cm} = mean strain in the concrete between the cracks

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\left(\sigma_s - \left[\frac{k_t f_{ct,eff}}{\rho_{p,eff}} * \left(1 + \alpha_e \rho_{p,eff}\right)\right]\right)}{E_s}$$

σ_s = stress in the tension reinforcement
k_t = 0.4 (for short term loading) & 0.6 (for long term loading)
$f_{ct,eff}$ = mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur
$\alpha_e = E_s/E_{cm}$

As UHSC has steel fibers in the mix, it is possible to leave traditional reinforcement out of the structure. In the absence of passive reinforcement, the limiting crack width is:
w_{max} = 0.3 mm for normal cracking, i.e. $\varepsilon < 0.003/l_c$
w_{max} = 0.2 mm for detrimental cracking, i.e. $\varepsilon < 0.002/l_c$
w_{max} = 0.1 mm for highly detrimental cracking, i.e. $\varepsilon < 0.001/l_c$

with l_c = the characteristic length in m (generally 2/3 h)

3.2.9 Prestressing

The value of the initial prestress force P_{m0} applied to the concrete immediately after tensioning and anchoring (post-tensioning) or after transfer of prestressing (pre-tensioning) is obtained by subtracting the immediate losses $\Delta P_i(x)$ from the force at tensioning P_{max} and should not exceed the following value:
$P_{m0}(x) = P_{max} - \Delta P_i \leq A_p * \sigma_{Pm0}(x)$
With $\sigma_{Pm0}(x) = \min\{0.75 f_{pk}; 0.85 f_{p0.1k}\}$

The immediate losses $\Delta P_i(x)$ of prestress can be divided into 4 types:

a) Relaxation losses ΔP_r
b) Elastic deformation losses ΔP_{el}
c) Friction losses $\Delta P_\mu(x)$
d) Anchorage slip losses ΔP_{sl}

a. Relaxation losses ΔP_r

The relaxation losses for OSC & HSC are calculated with the following expression:

$$\Delta P_r = A_p * \Delta\sigma_{pr} = A_p * [\sigma_{pi} * 0.66 * \rho_{1000} * e^{9.09\mu} * \left(\frac{t}{1000}\right)^{0.75\,(1-\mu)} * 10^{-5}]$$

With:
$\Delta\sigma_{pr}$ = absolute value of the relaxation losses of the prestress
σ_{pi} = absolute value of the initial prestress = σ_{pm0}

ρ_{1000} = value of relaxation loss at 1000 hours after tensioning at a mean temperature of 20 °C
= 2.5%
$\mu = \sigma_{pi}/f_{pk}$
t = time after tensioning = 500000 hours

b. Elastic deformation losses ΔP_{el}

$\Delta P_{el} = A_p * E_p * \sum[\frac{j*\Delta\sigma_c(t)}{E_{cm}(t)}]$

$\Delta\sigma_c(t)$ = variation of stress at the centre of gravity of the tendons applied at time t

j = (n-1)/2n where n is the number of identical tendons successively prestressed. As an approximation j may be taken as 1/2
1 for the variations due to permanent actions applied after prestressing

c. Friction losses $\Delta P_\mu(x)$

$\Delta P_\mu(x) = P_{max}(1 - e^{-\mu(\theta + kx)})$ or $P_o\, e^{-\mu(\theta + kx)}$

θ = sum of the angular displacements over a distance x (irrespective of direction or sign)
μ = coefficient of friction between the tendon and its duct
k = unintentional angular displacement for internal tendons (per unit length)
x = distance along the tendon from the point where the prestressing force is equal to P_{max} (the force at the active end during tensioning

Note: For external tendons, the losses of prestress due to unintentional angles may be ignored.

d. Anchorage slip losses ΔP_{sl}

$l_{set} = \sqrt{\frac{w*E_p}{\frac{\Delta\sigma_p}{\Delta x}}}$

with w = wedge set in mm

$\Delta P_\mu(l_{set}) = P_{max}(1 - e^{-\mu(\theta + k*lset)})$

Time-dependant losses ΔP_{c+s+r}

In addition to the immediate losses, the time-dependent losses of prestress $\Delta P_{c+s+r}(x)$ as a result of creep and shrinkage of the concrete and the long term relaxation of the prestressing steel should be considered:

$\Delta P_{c+s+r} = A_p * \Delta\sigma_{pr,c+s+r} = A_p \frac{(\varepsilon_{cs} + \varphi\varepsilon_{cc}(\infty))E_p + k\Delta\sigma_{pr} + \alpha_{p*}\varphi*\sigma_{c,qp}}{1 + \alpha_p * \rho_p * f * (1 + k\varphi)}$

ε_{cs} = estimated shrinkage strain in absolute value
ε_{cc} = estimated creep strain in absolute value
$\varphi(t,t_0)$ = creep coefficient at a time t and load application at time t_0
k = age factor = 0.8
$\sigma_{c,QP}$ = stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant
$\alpha_p = E_p/E_{cm}$
$\rho_p = A_p/A_c$

$f = 1 + \frac{A_c * z_{cp}{}^2}{I_c}$

z_{cp} = distance between the centre of gravity of the concrete section and the tendons

The final working prestress force $P_{mt} = P_{m0} - \Delta P_{c+s+r}$

In addition to the fundamental assumptions made in paragraph 3.2.4, UHSC has additional assumptions depending on whether a cracked or uncracked section is considered [17]:

a) For calculation of uncracked sections the following assumptions are given:

- the concrete withstands tensile stress;
- the constituent materials are subject to no relative slippage i.e. normal stresses due to all actions other than permanent actions can be calculated for the entire section made uniform using equivalence ratios n_i and n_v (with $n_i = 4$ and $n_v = 8$ without heat treatment and $n_v = 5$ with heat treatment).

b) For calculation of cracked sections the following assumptions are given:

- the constituent materials are subject to no relative slippage;
- when the strain of the concrete is eliminated at a reinforcement bar, the tension in the reinforcement is:
  0 if it is passive reinforcement,
  $\sigma_{pd} + n_i * \sigma_{bpd}$ (with $n_i = 4$) if it is prestressing reinforcement (with σ_{bpd} representing the concrete stress at the reinforcement considered, under the effect of permanent actions and prestress assumed to be P_d).
- the stress in passive reinforcement and the variation of overstress in the prestressing reinforcement which appear after decompression of the concrete are evaluated from the equivalence coefficient nv = 8 without heat treatment and nv = 5 with heat treatment.
- behavior of concrete under tension is as shown in Figure 21 & Figure 22.

Making use of these additional assumptions, the previous expressions for OSC & HSC can be applied for UHSC as well.

3.2.10 Fatigue

Fatigue failure is defined as the failure of a material when subjected to repeated loading, such as wind loading or traffic loads, before the static loading strength of the material is reached. The term is mostly used in the steel industry. In metals, fatigue causes a distinctive crack pattern to appear, thus enabling inspection and repairs. In the last decennia fatigue has been recognized as a possible failure mode for concrete as well. The problem with concrete fatigue however is that it is not possible to differentiate between fatigue cracks and cracks caused by other failure modes.

Fatigue loading or cyclic loading, can be divided into three different classes, depending on the number of load cycles [25]:

- Low-cycle fatigue ($<10^3$ load cycles);
- High-cycle fatigue (10^3<load cycles<10^7);
- Super high-cycle fatigue ($>10^7$ load cycles).

Table 5 shows a few examples of various type of structures subjected to different classes of fatigue.

Low-cycle fatigue < 10^3	High-cycle fatigue 10^3 - 10^7	Super high-cycle fatigue > 10^7
Runways Bridges on airports Structures subjected to earthquakes Structures subjected to storms	Railroads Concrete roads Railway sleeper Motorway bridges Wind power plants Airport pavement	Metro viaducts Offshore structures Mass rapid transit structures

0 10^1 10^2 10^3 10^4 10^5 10^6 10^7 10^8 10^9

Number of cycles

Table 5: Examples of structures subjected to different classes of fatigue loading

The behavior of concrete fatigue is influenced by a large number of factors such as [26]:

- Internal: Dimensions, concrete composition, type and amount of curing, age, reinforcement;
- External: Type, duration, frequency and maximum level of loading;
- Environmental conditions: Humidity, temperature, milieu.

When analyzing the fatigue resistance of structures there are two basic methods that can be used. The first method considers an analysis of crack propagation at the point considered and is based on linear elastic fracture mechanics. The second method, utilizes a curve that shows the relation between the cyclic stress range and the number of cycles to fatigue failure in logarithmic scales. This is known as a Wöhler diagram (see Figure 26) or an S-N curve (see Figure 27) (with S as the stress range and N is the number of cycles). These S-N curves are obtained from the experimental results of fatigue tests. For safety reasons the used S-N curve tends to be lower than the S-N curve obtained from test results [25].

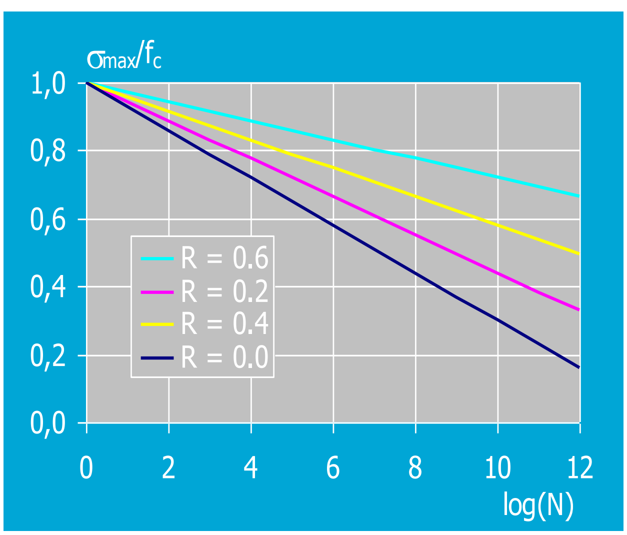


Figure 26: Wöhler diagram [31]

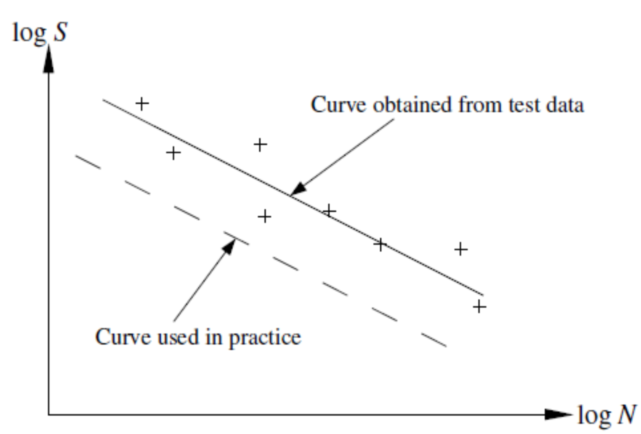


Figure 27: S-N curve [25]

For concrete in compression the Wöhler diagram is defined as [27]:
$S = \sigma_{max}/f_{cm} = 1 - \beta(1\text{-}R)*\log N$

With:
S = applied stress ratio of the upper load level
β = material constant
$R = \sigma_{min}/\sigma_{max}$ = ratio between the applied maximum and minimum stress level
N = total number of loading cycles

In fatigue failure three different stages can be distinguished:

1. Crack initiation: Initial crack formation;
2. Crack propagation: Stable crack growth;
3. Failure: Unstable crack growth and eventual failure.

The three stages can be observed when looking at the strain rate of concrete under repeated loading (see Figure 28).

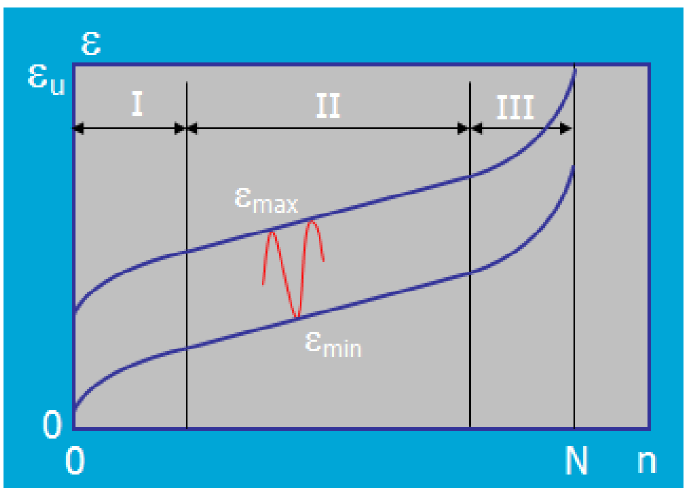


Figure 28: Strain rate under repeated loading [27]

The fatigue limit is defined as the stress level below which no failure occurs for an unlimited number of cycles. While this fatigue limit is clearly defined in metals, it has not yet been proven for concrete. The presence of this fatigue limit is incredibly difficult to verify, mostly because of the high scatter in the mechanical properties of concrete. Although the fatigue limit for concrete is thus fairly unknown, a practical limit is often used in codes and regulations. Generally for plain concrete, a limit of 50-60% is used. This limit also seems reasonable for higher strength concretes [22].

There are several hypotheses of damage development, most popular among them being Miners hypothesis. When the number of load cycles until fatigue failure is known the Palmgren-Miner damage summation may be applied to estimate the fatigue life.

Miners hypothesis: $M_s = \sum_{i=1}^{k} \frac{n_i}{N_i}$ or $\sum_{i=1}^{c} \frac{1}{N_i} = \frac{1}{N_1} + \frac{1}{N_2} + \frac{1}{N_3} + \ldots + \frac{1}{N_c}$

With:
n_i = the acting number of stress cycles at a given stress level and stress range
N_i = the number of cycles causing failure at the same given stress level and stress range
c = the number of stress cycles during the service life

Miners hypothesis says that each single cycle contributes in damage with size $1/N_i$. Each single contribution to damage may be summated randomly. Total damage after c cycles is the sum of damage of each single cycle and is called miners sum M_s. If the sum is 1, failure due to fatigue will occur. Other methods to calculate the fatigue include, but are not limited to, peak count, range count, rain-flow count, cycle counting method, etc.

According to Eurocode 2 a satisfactory fatigue resistance may be assumed for OSC & HSC under compression or compression struts subjected to shear, if the following condition is fulfilled:
$E_{cd,max,equ} + 0.43\sqrt{1 - R_{equ}} \leq 1$

With:
Stress ratio: $R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}}$ with $0 < R_{equ} < 1$, else $R_{equ} = 0$
Minimum compressive stress level: $E_{cd,min,equ} = \frac{\sigma_{cd,min,equ}}{f_{cd,fat}}$
Maximum compressive stress level $E_{cd,max,equ} = \frac{\sigma_{cd,max,equ}}{f_{cd,fat}}$
Design fatigue strength of concrete: $f_{cd,fat} = k_1 * \beta_{cc}(t_0) * f_{cd}{}^{*}(1 - f_{ck}/250)$
Coefficient depending on reference number of cycles till failure: $k_1 = 1$
Coefficient for concrete strength at first load application: $\beta_{cc} = 1$ for t_0 is 28 days
Time of the start of the cyclic loading on concrete in days: t_0
Upper stress of the ultimate amplitude for N cycles: $\sigma_{cd,max,equ}$
Lower stress of the ultimate amplitude for N cycles: $\sigma_{cd,min,equ}$

The fatigue verification for concrete under compression may be assumed, if the following condition is satisfied:
$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.5+0.45\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.9$ for $f_{ck} \leq 50$MPa
$\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.5+0.45\frac{\sigma_{c,max}}{f_{cd,fat}} \leq 0.8$ for $f_{ck} > 50$MPa

Maximum compressive stress at a fiber under the frequent load combination (compression measured positive): $\sigma_{c,max}$
Minimum compressive stress at the same fiber where $\sigma_{c,max}$ occurs: $\sigma_{c,min}$
If $\sigma_{c,min}$ is a tensile stress, then $\sigma_{c,min}$ should be taken as 0.

These methods however only take the compressive stresses in the concrete into consideration and not the number of load cycles until fatigue failure occurs. Instead of the actual maximum amount of load cycles, a reference value of 10^6 load cycles is applied. The method uses a comparison for the stresses caused by the cyclic load and a reference concrete strength for a static load $f_{cd,fat}$. For a spectrum load, like the wind acting on a transmission tower, these methods were found to be unviable [25].

The previous equation is then rewritten to:

$$E_{cd,max,equ} + \frac{\log N}{14}\sqrt{1 - R_{equ}} \leq 1$$

$$N \leq 10^{14 * \frac{1-E_{cd,max,equ}}{\sqrt{1-R_{equ}}}}$$

With N = number of load cycles until fatigue failure

As the number of cycles till fatigue failure is now known, the Palmgren-Miner rule can be used.

Higher strength concretes tend to be designed thinner and lighter than OSC, which could mean that they are more susceptible to fatigue. Although higher strength concretes have mostly superior material properties compared to OSC, this is not the case for fatigue properties. Instead the fatigue behavior tends to be somewhat similar. This proves that increased strength alone does not necessary result in improved fatigue performance. A general conclusion resulting from fatigue tests of higher strength concretes is therefore, that the fatigue codes used for OSC & HSC as defined in Eurocode 2, remain suitable for UHSC fatigue design [22].

A common problem with fatigue experiments is the large scatter, which often makes it difficult to interpret the results. Nevertheless many conclusions have still been drawn. For example research showed that a better workability in the early stages of the concrete, leads to a smaller scatter in the mechanical properties and thus also in the fatigue experiment results. A good workability also improves the homogeneity of the steel fiber alignment, thus improving the mechanical properties as well.

In the case of UHSC subject to fatigue, the French norms advise that the tensile stress should be restricted to [17]:

- min(σ_{bt} ; f_{t28}) for frequent combinations;
- min(σ_{bt} ; f_{tj}) during construction, in areas subsequently under tension in service.

If these criterions are complied with, there is no need to carry out a check for fatigue.
Note: These limitations should be applied for bridges in particular. For buildings, these limitations do not apply, unless stated otherwise by the designer [17].

3.2.11 Durability & sustainability

Durability

Durability is defined as the ability to resist an applied force, stress, attack or any other process of deterioration. The durability of concrete is usually tested by the environment, through the following means:

- Temperature changes;
- Moisture (penetration, permeability);
- Applied loading (abrasion, fatigue, cracking);
- Chemical attack (chlorides, sulfates, nitrates);
- Biological processes (carbonation, corrosion).

To ensure that concrete is durable enough certain requirements have to be met. These requirements generally relate to the concrete strength class, concrete cover, maximum allowable crack width, water-cement-factor and cement content. As the concrete strength class increases from OSC to HSC and eventually UHSC, the material composition becomes more and more compact and durable.

UHSC has always boasted an outstanding durability and recently these claims have finally be substantiated. The first tests with UHSC were performed around 20 years ago, while the first structures built with it are now around 15 years old. These structures are still standing strong and natural-ageing results have confirmed good durability [17]. This increased durability allows for a whole lot of advantages such as:

- Decreased concrete cover;
- Thinner structural elements;
- Use in aggressive or nuclear environments;
- Increased fire resistance;
- Long lifetime with close to no maintenance or repair.

Table 6 illustrates the superiority of higher strength concretes with respect to lower types of concrete for a couple of durability indicators [17].

	OSC	HPC	UHPFRC
Water porosity (%)	12 - 16	9 - 12	1.5 - 6
Oxygen permeability (m^2)	10^{-15} á 10^{-16}	10^{-17}	< 10^{-19}
Tritium-ion diffusion factor (m^2/s)	$2*10^{-11}$	$2*10^{-12}$	$2*10^{-14}$
Carbonation depth (mm)	10	2	0.02
Portlandite content (kg/m^3)	76	66	0
Rate of reinforcement corrosion (µm/year)	1.2	0.25	< 0.01[1]
Resistivity (kW/cm)	16	96	11331

Table 6: Durability properties OSC, HSC & UHSC

While UHSC performs better in most aspects compared to OSC, there is another characteristic of UHSC that must be considered, namely the corrosion of the steel fibers. Research has shown that even micro cracked UHSC is very effective at maintaining the pH level necessary for the passivation of the steel fibers as well as eventual prestressing tendons. This passivation can be maintained even in drastic exposure conditions due to the formation of hydrates that gradually fill the micro cracks (self-healing). The exceptional resistance of UHSC to carbonation (<0.2 mm after 4 years) also guarantees good protection of the steel fibers and prestressing tendons. The only instance when corrosion of steel fibers becomes a problem, is when the steel fibers are located at the surface (either due to incorrect mixing, extremely aggressive environment or other unforeseen circumstances). In such cases using polymer fibers might provide an alternative.

[1] Ductal® values based on French recommendations

With regards to fire safety UHSC does not differ much from OC. During fire it experiences changes in mechanical properties (strength and Young's modulus), usually loss of strength. Like most types of concrete, UHSC has the following properties with regards to fire:

- Non-combustible;
- No contribution to the development of fire;
- Low thermal conductivity, at about 1.6 W/m.K;
- Risk of spalling.

Sustainability

Sustainability generally means: "*Having no net negative impact on the environment"*[28]. Sustainability can be divided in three components, namely: people, profit and planet. To meet its goal, sustainable development must ensure that these three components are balanced. For engineers sustainability usually means to design 'green' or eco-friendly structures. This is generally realized by ensuring that no damage to the environment is caused as well as trying to achieve high recyclability. This recyclability is important mainly because of the impending exhaustion of raw materials. Therefore nowadays there is huge demand for renewable (sustainable) materials.

The impact of concrete on sustainability throughout the life of the structures depends mainly on the amount of emitted and stored energy (i.e. insulating properties). As concrete is not a very good heat conductor or insulator, a high porosity is needed to provide good insulating properties (at the cost of strength). Concrete does have a large thermal mass though, enabling it to store and release energy at a later date. When concrete has reached the end of its service life it is usually demolished. This demolition process is usually done with heavy machinery or blasting equipment, which use a moderate amount of energy. During demolition a lot of fine particles are released such as dust, cement powder, etc. After it has been demolished, concrete is usually recycled as much as possible.

UHSC uses about twice as much cement as OSC and thus produces twice as much CO_2 and consumes twice as much energy during production. Still past experiences with UHSC show that if utilized correctly, the amount of material used in a structures can be two or three times less than OSC. Consequently a structure built with UHSC scores better in terms of initial CO_2 footprint and energy consumption [29]. While the cement content maybe higher compared with OSC, UHSC uses a higher percentage of silica fume as well. This byproduct of the ferrous silica industry, is recognized as a recyclable material and thus allows the structure to be recognized as using recyclable materials, under certain sustainability programs and certifications. UHSC is also superior in terms of lightness (construction speed), durability (long life expectancy and less maintenance) and the reduced amount of necessary traditional reinforcement. Therefore when looking at the costs and global economy of a structure in UHSC, it is important to incorporate an anticipation of sustainability earnings. This is especially relevant now that long-life or evolutive structures are desired and definitely when comparing it with the costs, operating limitations, maintenance and repair efforts of OSC.

3.2.12 Summary of material & durability properties

Table 7 illustrates the differences in concrete composition of OSC, HSC and UHSC [30].

Units in kg/m^3	OSC	HSC	UHSC
Cement	360	475	950
Silica fume	-	25	235
Portlandite content (kg/m^3)	76	66	0
Sand	790	785	995
Gravel	1100	960	-
Steel fibers 13 mm	-	-	145

Table 7: Concrete composition [30]

Table 8 shows a summary of all the previously covered material & durability properties of OSC, HSC and UHSC [17][30].

	OSC C45/55	HSC C90/105	UHSC C170/200 (Ductal®)	Units
Char. compressive strength f_{ck}	45	90	170	MPa
Design compressive strength f_{cd}	30	60	113	MPa
Mean axial tensile strength f_{ctm}	3.8	5.0	10.27	MPa
Char. axial tensile strength f_{ctk}	2.7	3.5	8.2	MPa
Mean flexural tension strength $f_{ctm,fl}$	5.7	7.6	41.8	MPa
Young's modulus E_{cm}	36	44	55	GPa
Ultimate pure compression strain ε_{c3}	1.75	2.3	2.35	‰
Ultimate compression strain ε_{cu3}	3.5	2.6	3.0	‰
Material factor γ_c	1.5	1.5	1.5	-
Density ρ_c	2405	2410	2500	kg/m^3
Water porosity	12 - 16	9 - 12	1.5 - 6	%
Air permeability				
5 days at 50 °C	$30 \cdot 10^{-18}$	$0.3 \cdot 10^{-18}$	-	
30 days at 80 °C	-	$120 \cdot 10^{-18}$	$2.5 \cdot 10^{-18}$	
Chloride diffusion coeff.	$1.1 \cdot 10^{-12}$	$0.6 \cdot 10^{-12}$	$0.02 \cdot 10^{-12}$	m^2/s
Tritium-ion diffusion coeff.	$2 \cdot 10^{-11}$	$2 \cdot 10^{-12}$	$2 \cdot 10^{-14}$	m^2/s
Carbonation depth	10	2	0.02	mm
Rate of reinforcement corrosion	1.2	0.25	< 0.01[1]	µm/yr
Electrical resistance (resistivity)	16	98	137-1133	kW/cm
Erosion	4	2.8	1.3	V/V_{ref}
Freeze/thaw scaling	>1000	900	7	g/cm^2

Table 8: Summary of durability properties OSC, HSC & UHSC

[1] Ductal® values based on French recommendations

3.3 Production

In a previous report published by Rijkswaterstaat [31], a comprehensive overview of the specific execution aspects related to higher strength concrete (strength class C53/65 and higher) is given. This report was based on earlier projects implemented in the Netherlands that utilized higher strength concrete. In the following paragraphs the major points of this report are summarized [28].

3.3.1 Mixing procedure

During the manufacturing of higher strength concrete, the mixing procedure warrants extra attention. The combination of the low water content and a high proportion of additives has a dry mixture as a result. In such a dry mixture the additives are difficult to integrate in the mix. From various tests results, the conclusion was drawn that the mixture is to be dosed into the mixer in phases. The moment when the fillers and additives are dosed should thus be determined accurately. Partly because of this phased process, the total mixing time is longer than usual. A separate place is occupied by concrete compositions that contain silica fume. When working with silica fume in slurry form, the mixer must always be filled so that the slurry does not come into contact with the (still dry) cement. A clot formation which is incredibly difficult to "break" is the result. In contrast, for powdered silica fumes it is recommended to have some dry premixing.

3.3.2 Production capacity

Manufacturing of higher strength concrete requires special attention at the concrete plant. The various concrete plants all utilize somewhat different mixing procedures, resulting in different capacities for the plants. Important factors that influence the production capacity are [31]:

- The reduced capacity of the concrete plant should be taken into account (for C53/65 the capacity can be reduced till 75% to 90%, for C70/85 it is around 25 to 50% and for higher concrete strengths it is close to zero). This reduction is strongly dependent on the mixer type. The total production time will therefore increase and a second concrete plant might have to be used;
- During production rinsing times are to be included with regard to cleansing of the mixers, machines etc. This can cause a significant discontinuity in the supply to the building site;
- When supplying the concrete to the building site, proper arrangements have to be made between both parties since waiting times can be critical. Long waiting times for truck mixers on the building site can lead to a reduced workability of the concrete. When the workability has reduced past a certain point the only remaining option is to reject and send back the unprocessable concrete. This can also result in a shortage of concrete at the building site for a while, which is unacceptable as well. Especially higher strength concrete demands a continuous pouring process. Each interruption runs the risk that the pouring front becomes clearly visible on the concrete surface and that bonding is not sufficient. The supply of concrete from the concrete plant must therefore be perfectly in sync with the pouring capacity of the building site and if necessary must (continuously) be adjusted;
- Because of the longer mixing times, the production capacity of the concrete plant is reduced. It is therefore of great importance that the various parties have intensive contact during the design and implementation phase, so that proper arrangements can be made in the event of large concrete demands. In view of the continuous process of pouring

concrete, which is required for good constructions, it is therefore always useful to have a backup concrete plant.

3.3.3 Processing

The concrete producer and the contractor must maintain close relations to allow the concrete to be discharged rapidly after arrival at the construction site. Delays in delivery and placing must be prevented as much as possible e.g. by reducing the batch sizes if the placing process is slower than expected. Improvements in workability should not be achieved by anything other than superplasticizers [32].

At first glance the transported slurry looks fluid, but this is not always the case. The slurry contains a lot of cement paste and fine fillers so that a thixotropic behavior occurs. To realize the full potential of HSC, consolidation is a vital course of action. After placement in the moulds or forms the concrete has to be vibrated as quickly as possible. For OSC over-vibration is a very important issue since it results in segregation, loss of entrapped air or both. In contrast for HSC under-vibration is more important, because of the relative stiffness as well as the minimal amount of air entrapped in the material. Therefore under-vibration should be prevented to a higher degree than over-vibration [31]. For UHSC even without adding extra energy, the material is much stiffer than expected. By applying vibration energy, the slurry can immediately regain the previous high fluidity.

Because of the low water-cement ratio, the concrete composition of higher strength concretes is sensitive to small variations in raw materials. This is caused by applying a lot of fine material (e.g. cement, silica fumes and fine fillers) and significant amounts of superplasticizer in the concrete mix. Changes in weather conditions play an important role as well. For all the above reasons, additional attention is required when applying the higher strength concretes to ensure the quality of the final construction. In factories, where the conditions are controlled to a much higher degree than outside, this aspect is less important. It is therefore reasonable to say that nowadays higher strength concretes are more eligible for prefabrication than for in-situ casting. This does not however mean that in-situ casting of higher strength concrete is impossible.

Another important aspect for higher strength concretes is the steel fiber alignment and orientation. The main factors influencing the steel fiber alignment are the formwork boundaries, the casting method, gravity (segregation) and vibration. Nowadays most higher strength concretes tend to be self-compacting, thus reducing the amount of vibration energy needed. The formwork boundary effect is most relevant for mixtures with large steel fiber lengths. And lastly the direction of the concrete flow (both primary and secondary) heavily depends on the type of casting method chosen. The falling height of the concrete poured in the moulds depends on the casting method as well.

The finishing sequence for higher strength concretes does not differ that much from normal strength concretes, allowing the same finishing techniques to be used for higher strength concretes albeit somewhat modified. A difficult aspect of higher strength concrete is the sticky nature, which causes problems during finishing. The concrete tends to stick to moulds, trowels and other finishing equipment thus hindering the finishing activities. For this reason finishing activities should be minimized.

Aside from finishing, curing is an important activity as well, even more so for higher strength concretes. Supplying a sufficient amount of moisture and creating favorable temperature settings is recommended for a long period.

For horizontal surfaces with a very low water cement factor and especially when silica fume is used in the mixture, there will be a small amount of bleeding either before or after the finishing activities. During these conditions it is vital that fog curing or evaporation retarders are applied to the concrete immediately after the surface has been struck off. This is essential to avoid plastic shrinkage cracking on the horizontal surfaces and to prevent crusting. Fog curing, followed by seven days of wet curing, has been proven to be a very effective method [31].

Of course it can be difficult to cure vertical surfaces, such as columns, effectively. Columns are regularly stripped at an early age to allow the raising of self-climbing form systems and are therefore exposed to early drying. Because access to these columns is limited, additional curing of these columns is usually very difficult and impractical. For that reason columns require the best possible initial curing process.

3.4 Pros and cons

Higher strength concrete offers great advantages compared to traditional concrete. The advantages that makes these higher strength concretes so attractive to use will be listed in the following paragraph. There are however drawbacks to using them as well. The disadvantages will be summed along with how these disadvantages can be reduced or eliminated altogether by improving design or production aspects.

3.4.1 Advantages

- Improved material properties such as high compressive and tensile strength, elasticity modulus, durability etc;
- Very thin or slender structures can be achieved;
- Higher strength concretes with steel fibers do not have to be reinforced with regular steel bars (no traditional, shear or punching shear reinforcement is needed). Prestressing however can still be needed depending on the design. The removal of the traditional steel reinforcement reduces production costs;
- A high degree of prestressing is possible;
- Lower transportation and maintenance costs, easier to handle on site as well as longer service life;
- Creep is smaller compared to traditional concrete. This in turn causes less prestressing losses [32];
- Highly homogenous concrete microstructure throughout the concrete member can be achieved [34];
- Because of the high density the concrete is very durable. Penetration of harmful substances is reduced immensely (for chloride ions it is about 50 times smaller than for traditional concrete). Because of this improved resistance, the concrete cover can be reduced, thus achieving thinner (slender) structural elements;
- Good post-fracture behavior if fibers are distributed evenly [34];
- High strengths are reached much earlier, so prestressing or applying loads can be done at an earlier time. This in turn results in shorter construction times;

- For high rise buildings higher strength concretes mean more than just higher buildings. Because the high strength columns can be constructed more slender than before, while retaining the same load bearing capacity, the net office space can be increased, creating more available area to be used [34].

3.4.2 Disadvantages

- Higher strength concrete is very susceptible to crack formation during finishing and curing;
- In comparison with traditional concrete, higher strength concrete is more sensitive to (autogenous) shrinkage without heat treatment [29]. This shrinkage mostly occurs in the first few days. In case post-tensioned prestressing is used, this shrinkage loss can be reduced by applying the prestressing at a later time. In case of pre-tensioned prestressing, this loss can be reduced by over tensioning the prestressing steel (when allowed by the code);
- The hydration process in higher strength concrete occurs at a very fast pace, which results in a very large warmth production. Uneven temperatures during hardening of the concrete, result in eigenstresses, which cause cracks to appear. This is especially relevant for thick structural elements. For thin elements this problem is of less importance. However since use of higher strength concrete usually results in thinner structural elements, this large warmth production problem is not relevant in most cases [19]. This problem is generally tackled by cooling the hardening concrete;
- Placement of fibers must be carefully considered, to prevent fibers from being oriented in all directions, thus reducing the overall effectiveness;
- High strength concrete is very brittle compared to traditional concrete. This problem can be solved by adding steel fibers to the mix (standard procedure for UHSC). With the addition of these steel fibers it becomes more ductile, attains a higher tensile strength which in turn leads to a lower minimum reinforcement percentage (in the case of strain hardening);
- One of the biggest obstacles for applying UHSC on a large scale is that current design criteria do not clearly present the properties and possibilities of UHSC. Much research on this subject is still required. Because there is still much that is disputed about the properties of UHSC, there are only a few norms and codes available.
- For higher strength concrete the aggregate strength is of great importance to the overall strength of the concrete. When cracks arise they propagate through the cement matrix as well as the aggregates, unlike traditional concrete which only has cracks propagating through the cement matrix. Because of this intense propagation the pathway for aggressive substances to the reinforcement can be shorter for traditional concrete types despite the higher density. Also the hook resistance of the cracks is close to zero [33];
- Higher strength concrete is more expensive than traditional concrete per m^3, because of the large amount of aggregates, cement, required stronger gravel, as well as the more complex mixing procedures. However because of the higher strength, less m^3 concrete is needed for structures as well as a shorter construction height, which results in less overall material used and lighter (cheaper) foundations;
- Production capacity is much lower compared to traditional concrete which has a very basic well-known process. On the other hand the needed amount of higher strength

concrete is smaller compared to traditional concrete, because of the superior material properties. This in turn saves production time;

- The demolition of higher strength concrete is more difficult than traditional concrete (can be seen as a positive point as well). However because less material is used (thinner, slender structures) this reduces the demolition costs.
- Higher strength concretes reinforced with steel fibers are very difficult to recycle because of the steel fibers. The fibers are bend and broken in various manners, making it very difficult to remove them.

3.5 Applications

While HSC has been applied in quite a lot of projects and is starting to become more known and regular, people still hesitate to apply UHSC. Most of the application of UHSC so far have been in bridges, plates or joints. To better understand the possibilities of Ultra high strength concrete a number of implemented projects and applications will be covered in the next paragraphs.

3.5.1 Bridges

Sherbrooke Bridge

The Sherbrooke bridge, located in southern province Quebec, Canada, was the first structure ever to be designed with Ultra High Performance Concrete (UHPFRC). It was built in the year 1997 in Sherbrook, spanning a total of 60 m (see Figure 29). This precast, (post-tensioned) prestressed foot bridge is an open-web space Reactive Powder Concrete (RPC) truss, with 4 access spans made of HPC. The main span consists of six 10 m prefabricated parts, which were connected together in one day [34]. The bridge utilizes no traditional steel reinforcement.

Figure 29: Sherbrooke bridge, Canada [XIV]

Bourg-lès-Valence overpasses

These overpasses, located in the in France’s Drôme region (southeast), are the world’s first road bridges made from UHPFRC, constructed in the year 2001 (see Figure 30). Each bridge spans 20 m and has 12 m wide decks. The beams are precast, (pre-tensioned) prestressed and contain large amounts of steel fibers. There is no passive reinforcements, except for the joints. These inventive road bridges are already serving as a reference for new projects in UHPFRC [34].


Figure 30: Bourg-lès-Valence overpasses [33]

Bridge decks for Kaagbruggen

In 2002 world's first Compact Reinforced Composite (CRC) application was realized in Sassenheim, The Netherlands. They were the replacement for an existing azobé wood bridge deck. Each bridge deck consists of 4 concrete panels with weight equal to the previous wooden deck per m^3. Because the concrete structure had to be very light, CRC was chosen as the only possibility. CRC has very dense reinforcements (up to 20%) and high volume percentages of short, stiff and strong steel fibers (up to 6%). The strong matrix, including the steel fibers, cooperates very well with the steel reinforcement enabling strengths of over 200 MPa. Because of the large content of steel fibers an extreme ductility can be observed as well [XV]. The time it took to replace the decks was only one week, compared to the wooden alternative of 2-3 weeks (see Figure 31) [34].


Figure 31: Placing CRC panels of the Kaagbrug [33]

The Gärtnerplatzbrücke

This hybrid UHPFRC-steel bridge is a pedestrian and cycle track bridge across the Fulda river that was realized in 2007 in Kassel, Germany. This project is the first bridge application using UHPFRC in Germany. It is also the first time worldwide that a hybrid UHPFRC-steel structure is applied. The bridge spans 132 m and consists of 6 spans. The bridge decks consist of precast (post-tensioned) prestressed UHPFRC plates, while the girder is formed by a steel truss system. The hybrid bridge had to replace the existing wood bridge, which had 7 spans and a total length of 147 m, while still making use of the available pillars and their foundations. Bolted as well as glued (epoxy resin) connections were used [35].

Figure 32: The Gärtnerplatzbrücke [XVI]

3.5.2 Joints

CRC JointCast

Compact Reinforced Composite (CRC) can be applied for joints with great results. Because of the good bond properties, full anchorage of ribbed Ø8 bars is possible with embedment lengths of only 5-10 times the diameter. The CRC JointCast, as it is called, is most often used for repairs or for connections that are complicated to design without compromising the strength. JointCast has been used in connections for spans to spans, columns to columns, columns to foundations, walls to walls, bridges and countless others [XV]. People tend to be much less hesitant to applying UHSC in joints, most likely because the joint strength in general has to be larger than the connecting parts, thus UHSC provides an attractive alternative. The CRC JointCast makes it further attractive by being easy to use. It is sold as dry mortar, which contains the binder, sand and the fibers. The only thing that must be done on the building site is adding water [XV].

3.5.3 Other applications

Ultra Thin Hi-Con Balconies

These very thin cantilevered balconies made from CRC were designed by Pieters Bouwtechniek in cooperation with the Danish producer Hi-Con. They were applied for housing project Amber in Delft, The Netherlands and they are the very first Dutch project to be realized with Hi-Con Balconies. The 65 mm thin balconies were prefabricated in Denmark and connected to the main structure in the Netherlands, with a typical Dutch tunnel formwork method. Because there are no codes available for designing with UHSC in the Netherlands, the balconies were calculated according to the Eurocode including specific rules form the Dutch National Annex (with some exceptions), while the entire structure was calculated according to NEN 6720 [36].

Figure 33: Test Hi-Con Balcony loaded with 5 times the design load [35]

The Atrium

The “Atrium Project” in Victoria, North America is the first building in North America to have a façade made entirely of UHPFRC. The façades of this seven-storey building consist of ultrathin, precast, curved UHPFRC panels that are part of a curtain wall system. These cladding panels with varying widths (10-20 mm) were precast using a displacement casting method in order to produce a grand total of 690 panels both flat and curved shapes. This innovative project is expected to be a reference for future projects utilizing similar panels in building façades [36].

Figure 34: The Atrium in Victoria, North America [XVII]

Staircases

Staircases in UHPFRC have been applied as early as 1997. They were one of the first possible applications where architects could take advantage of the high strength and durability of UHPFRC to design light and thin steps for the stairs. Nowadays staircases constructed in UHPFRC have been applied on a wide scale, ranging from simple basic stairs to complex, architecturally impressive staircases.

Figure 35: Spiral staircase at CBS [XVIII]

4. Preliminary design

This chapter will describe the design process. At the start of the project, the first predictions and calculations can already be made. The mast that will be considered is a W6S400 support mast. Initially nine variants were thought of . Not all of these variant proved to be satisfactory and some were thus eliminated quite early in the design process. As UHSC is the most important variant in this thesis, this variant will mostly be used in the shown calculations. For an overview of the used calculations see appendix E – G.

4.1 Design method

The design process can broken down into five steps:

1. Decide the variants;
2. Calculate the reinforced variants with a fixed initial wall thickness;
3. Apply prestressing to the reinforced variants;
4. Optimize the wall thickness for all variants
5. Optimize prestressing.

Ad 1. Variants

As mentioned before there are nine variants that will be looked at in detail. The main difference of these variants will be in the concrete strength class. The three concrete strength classes that will be looked at are:

1. C45/55 (OSC)
2. C90/105 (HSC)
3. C170/200 (UHSC)

When designing the mast, eventually a decision will be made whether the mast should be prestressed or not. Therefore both a non-prestressed and a prestressed mast will be calculated. An overview of all the variants can be seen in Table 9.

	No fibers		Steel fibers
Concrete classes	OSC	HSC	UHSC
Reinforcement only	√	√	√
Reinforcement + prestressing	√	√	√
Prestressing only	√	√	X

Table 9: Considered variants

Note: The UHSC variant is always reinforced with steel fibers. Therefore usually traditional reinforcement is not necessary anymore. Thus when reinforcement is mentioned for UHSC it always refers to the steel fibers and not traditional reinforcement. For this reason the ninth variant is eliminated as it is very unpractical to apply UHSC without fibers.

Ad 2. Fixed initial wall thickness

For all the variants initially a wall thickness of 250 mm will be chosen. This value has no real substantiation. It is only chosen so a first impression of the mast can be gained. Later on the wall thickness can then be varied and eventually optimized. But first with this value all the necessary requirements for the mast are checked i.e. the axial, shear & moment resistance, as well as deformation aspects. This is done for all three concrete strength classes.

Ad 3. Apply prestressing

After the first variants have been checked, the process is repeated, but this time prestressing is added into the mix. This prestressing will allow the entire mast to be in compression, completely eliminating the tension stresses.

Ad 4. Optimize wall thickness

Now that both the reinforced and the prestressed variants have been checked, the initial assumed wall thickness of 250 mm can be scrutinized. Based on the overcapacity of the initially designed structure, the wall thickness can be reduced. After some iterations in a calculation sheet, the optimal wall thickness can then be found.

Ad 5. Optimize prestressing

After the optimal wall thickness is found, the prestressing can then looked at in more detail. First it should be checked if traditional reinforcement can be reduced by increasing the prestressing. After that it should be checked if a constant prestressing force is necessary over the entire height of the mast, or if a variable prestressing can be applied i.e. if the prestressing can be reduced in certain sections.

Premises

At the start of the project, certain restrictions or limitations are always known. The most important premises are summarized in Table 10.

Component	Value	Unit
Mast height	57	m
Mast diameter at foot	2.20	m
Mast diameter at top	0.50	m
Field length	400	m
Cultivation	Uncultivated	-
Wind class	II	-
Environment class	XC3	-
Crack width criteria	0.3	mm
Deflection criteria (at top)	2.5	%

Table 10: Premises of the W6S400 transmission tower

4.2 Dimensions

The dimensions of the mast have already been decided in the premises. These initial dimensions will be kept as best as possible. The only mast dimension that is variable is the wall thickness. Furthermore the conductor and other cable heights have to conform to NEN 50341. In Figure 36 & Table 11: an overview of the mentioned dimensions is given.

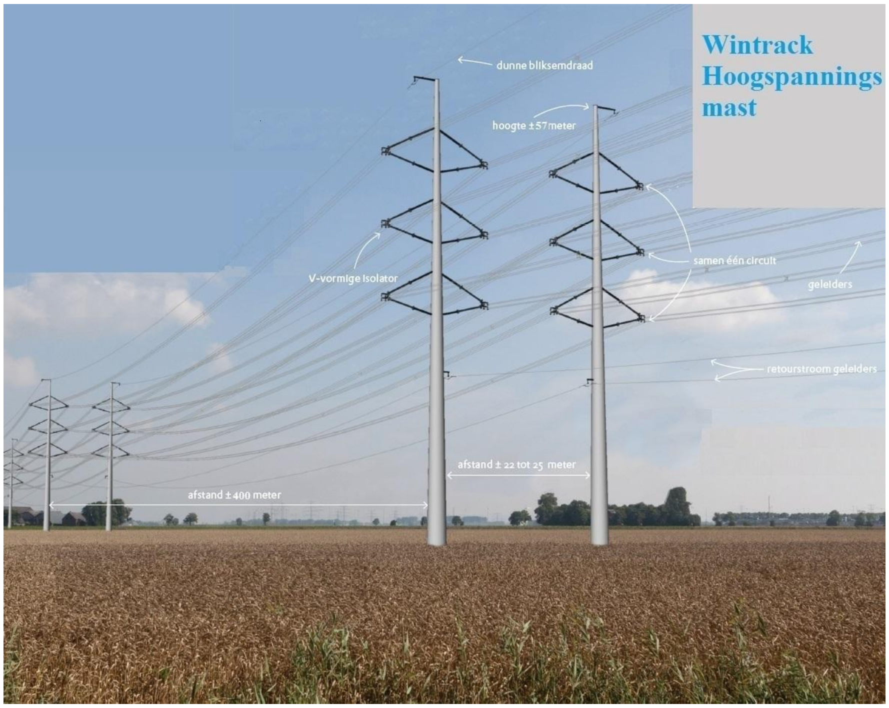


Figure 36: W6S400 transmission tower specifications

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	250	mm
Height passive loop	h_{pl}	57	m
Height 1^{st} level of conductors	h_1	47	m
Height 2^{nd} level of conductors	h_2	37	m
Height 3^{rd} level of conductors	h_3	27	m
Height lightning wire	h_{lw}	22	m
Field distance	L	400	m

Table 11: Mast dimensions

Material properties

Before the analysis can start, the various material properties have to be known. Most of the properties for UHSC where gained from the French norms (AFGC) and the Ductal® material properties, while the steel and prestressing values where obtained from Eurocode 2. Table 12 – Table 15 give an overview.

Component	Symbol	Value	Unit
Concrete quality	C	170/200	N/mm^2
Compressive strength, characteristic	f_{ck}	170	N/mm^2
Compressive strength, design	f_{cd}	113	N/mm^2
Material density	ρ_c	2500	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$2.35*10^{-3}$	-
Ultimate strain	ε_{cu3}	$3.00*10^{-3}$	-
Young's Modulus, mean	E_{cm}	55000	N/mm^2
Young's Modulus, design	E_{cd}	45833	N/mm^2
Tensile strength, characteristic	f_{ctk}	10.27	N/mm^2
Tensile strength, design	f_{ctd}	6.85	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	41.8	N/mm^2

Table 12: Material properties UHSC

Component	Symbol	Value	Unit
Reinforcement type	B	500	N/mm^2
Yield stress	f_{yd}	435	N/mm^2
Material density	ρ_s	7800	kg/m^3
Material factor steel	γ_s	1.0	-
Young's Modulus	E_s	210000	N/mm^2
Yield strain	ε_s	$2.07*10^{-3}$	

Table 13: Material properties reinforcement steel

Component	Symbol	Value	Unit
Prestressing type	ASTM	1860	N/mm^2
Nominal tensile strength	f_{pk}	1860	N/mm^2
Prestressing stress	σ_{po}	1395	N/mm^2
Number of wires per strand	n_{wire}	7	-
Number of strands per tendon	n_{str}	19..21	-
Area strands	A_{str}	150	mm^2
Young's Modulus	E_p	195000	N/mm^2
Material factor steel	γ_p	1,1	-
Ratio of bond strength	ξ	0,5	-

Table 14: Material properties prestressing steel

Component	Symbol	Value	Unit
Steel fiber type	Ductal®	S 235	N/mm^2
Yield stress	f_{yd}	235	N/mm^2
Tensile strength fibers	f_u	1250	N/mm^2
Material density	ρ_s	7900	kg/m^3
Fiber fraction (Strain hardening)	V_f	2,15	%
Volume of fibers used	V_f	55	kg/m^3
Fiber length	l_f	20	mm
Fiber diameter	d_f	0,3	mm
Area fibers	A_f	$5.96*10^6$	mm^2

Table 15: Material properties steel fibers

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1100	mm
Area mast at top	A_{top}	196350	mm^2
Area mast at foot	A_{bot}	1531526	mm^2
Area mast average	A_{av}	863938	mm^2
Volume	V	$4.92*10^{10}$	mm^3
Section modulus average	W_{av}	$2.04*10^8$	mm^3
Moment of inertia average	I_{av}	$1.37*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 16: Section properties

For the entire slenderness calculation see appendix B.

4.3 Loads

After the material properties are known the next important step is to determine the acting loads. All of the conductors and other cables will exert both vertical and horizontal forces on the mast. The vertical force is due to the self weight of the cables, while the horizontal force is due to the wind working on the entire length of the cable. Aside from theses forces, the self weight of the concrete mast must be taken into account, as well as the wind working directly on the mast (For the entire wind load calculation see appendix C). Table 17 & Table 18 show the acting loads in the Service and the Ultimate Limit State.

Loads SLS	Vertical (self weight)	Horizontal (due to wind)	Units
Retour current conductor	8	5	kN
Conductors 1st level	95	65	kN
Conductors 2nd level	95	76	kN
Conductors 3rd level	95	82	kN
Lighting wire	16	7	kN
Self weight	1231	-	kN
Wind	-	1.4	kN/m
Total normal/ shear force	1539	316	kN

Table 17: Acting loads SLS

Loads ULS	Vertical (self weight)	Horizontal (due to wind)	Units
Retour current conductor	10	7	kN
Conductors 1st level	114	97	kN
Conductors 2nd level	114	114	kN
Conductors 3rd level	114	123	kN
Lighting wire	19	11	kN
Self weight	1477	-	kN
Wind load	-	2.1	kN/m
Total normal/ shear force	1847	474	kN

Table 18: Acting loads ULS

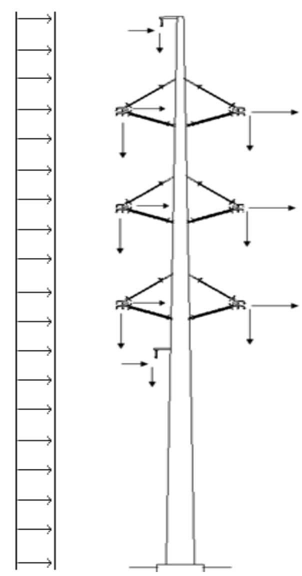


Figure 37: Acting loads

After the loads have been decided the axial force, shear force and moments can be determined over the entire mast height (Figure 38 – Figure 40).

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	1750	2624	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	3860	5790	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2308	3462	kNm
Total	13540	16881	kNm

Table 19: Moments in SLS & ULS

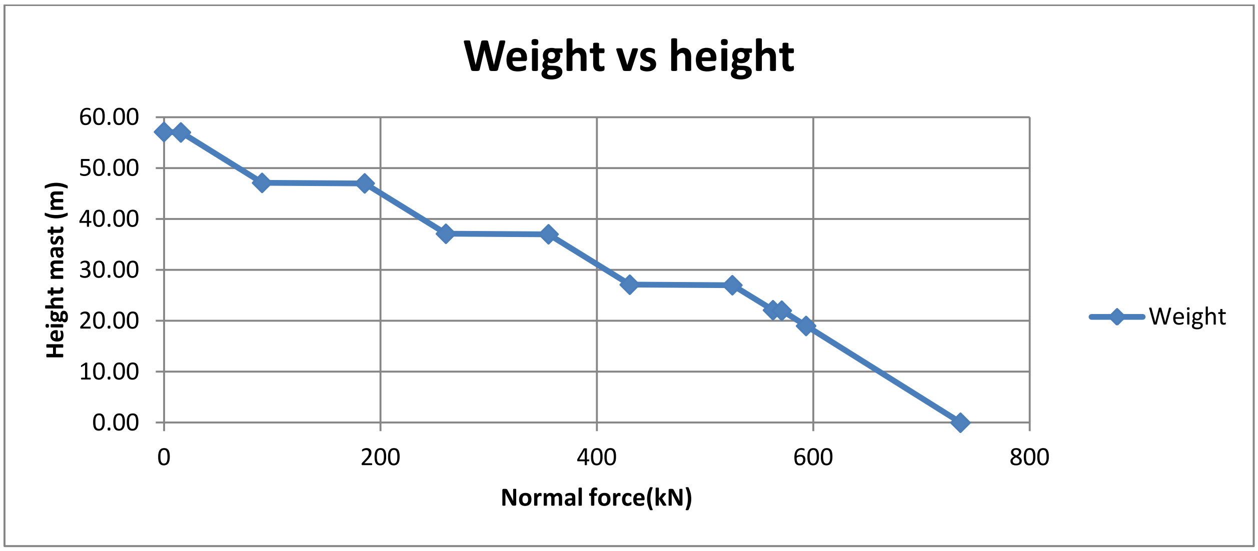


Figure 38: Axial force vs. height

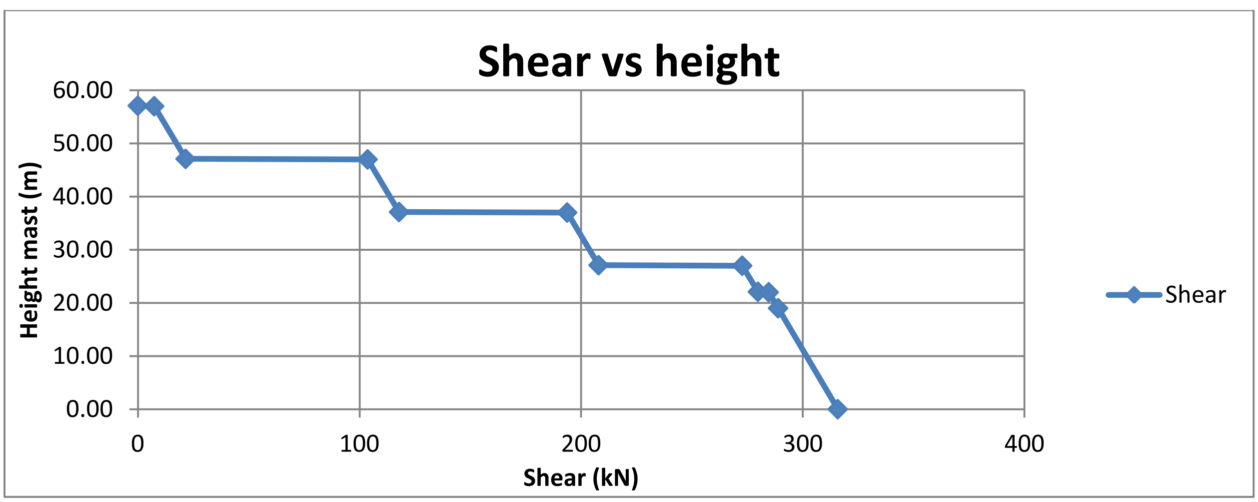


Figure 39: Shear force vs. height

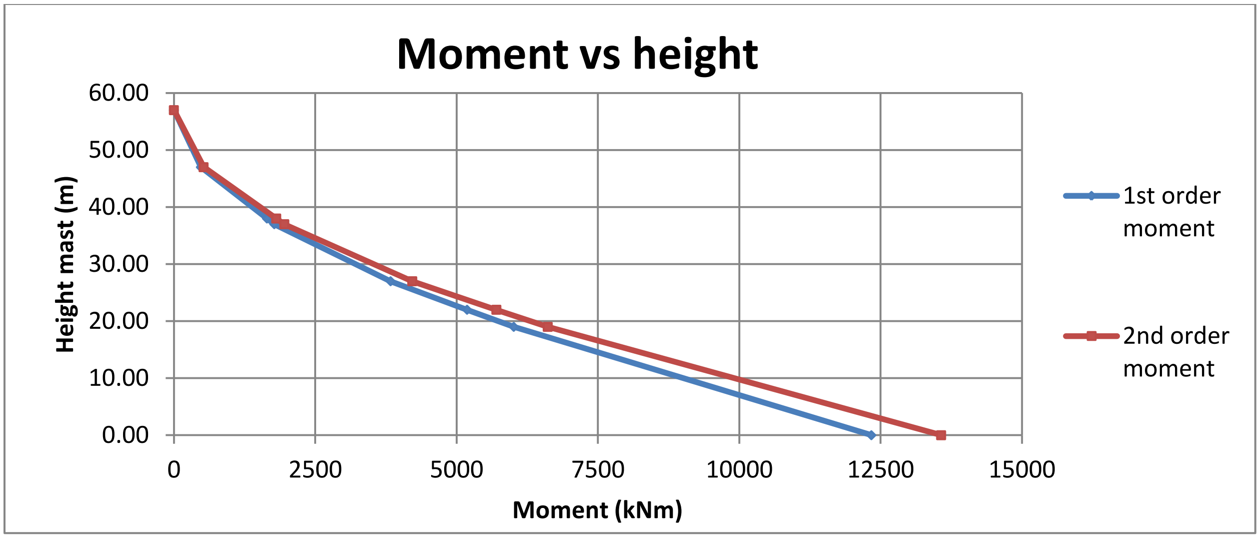


Figure 40: 1st en 2nd order moments vs. height

Now that all the preliminary values and properties have been decided, the actual analysis can begin.

Note: In the direction orthogonal to the conductors, the acting loads are as illustrated in Figure 37. In the direction parallel to the conductors, only wind loading as well as a possible cable break load should be taken into account (conductors). Although this cable break load has a very large value, compared to the other loads, it was found to be not governing (due to combination 5a of NEN 50341-3). Thus the mast was designed based on the governing loads, that work in the direction orthogonal to the conductors. The only impact the cable break load has on the design, is that instead of designing the mast with an oval shape, a circular shape is chosen instead, to guarantee that loads in all directions can be taken by the mast.

4.4 Prestressing

Stresses without prestressing

Stress due to axial load: $$\sigma_{Ns;r} = \frac{N_{s;r}}{A_c} = \frac{1231}{1531526} = -1.01\,\text{N/mm}^2 \tag{4.1}$$

Stress due to moment: $$\sigma_{Ms;r} = \frac{M_{s;r}}{A_c} = \frac{13270}{1531526} = 19.73\,\text{N/mm}^2 \tag{4.2}$$

Initial stress at t=0: $\sigma_{c;0} = -3.83$ N/mm^2 $\leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -20.73$ N/mm^2 $\leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 18.72$ N/mm^2 $> f_{ctd}$ NOT OK

4.4.1 Prestressing force

To remove the tensile stresses in the cross section, prestressing must be applied.

Maximum prestressing force at t = 0:

$$-\frac{P_{m0}}{A_c}-\frac{N_0}{A_c}+\frac{M_p}{W_c}-\frac{M_G}{W_c}\geq 0.6 f_{ck} \tag{4.3}$$

$$P_{m0}\leq\ 149730\text{ kN}$$

Minimum prestressing force at t = 0:

$$-\frac{P_{m0}}{A_c}-\frac{N_0}{A_c}-\frac{M_p}{W_c}+\frac{M_G}{W_c}\leq 0 \tag{4.4}$$

$$P_{m0}\geq\ 4024\text{ kN}$$

Maximum prestressing force at t = ∞:

$$-\frac{P_{m\infty}}{A_c}-\frac{N_\infty}{A_c}+\frac{M_p}{W_c}-\frac{M_{G+Q}}{W_c}\geq\ f_{cd} \tag{4.5}$$

$$P_{m0}\leq\ 156896\text{ kN}$$

Minimum prestressing force at t = ∞:

$$-\frac{P_{m\infty}}{A_c}-\frac{N_\infty}{A_c}-\frac{M_p}{W_c}+\frac{M_{G+Q}}{W_c}\leq 0 \tag{4.6}$$

$$P_{m0}\geq\ 32543\text{ kN}$$

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	32543	kN
Governing prestressing force	$P_{min,\infty}$	29288	kN
Required prestressing steel	$A_{p,req}$	23328	mm^2
Number of tendons required	n_{req}	9	-
Number of tendons applied	n_{apl}	10	-
Applied prestressing steel	$A_{p,apl}$	28500	mm^2
Applied prestressing force	$P_{min,0}$	39578	kN
Working prestressing force	$P_{min,\infty}$	35782	kN

Table 20: Prestressing tendons

Stress due to axial load: $$\sigma_{Ns;r} = \frac{N_{s;r}}{A_c} = \frac{1231}{1531526} = -1.01\,\text{N/mm}^2 \tag{4.7}$$

Stress due to prestressing: $$\sigma_{Pm} = \frac{P_{m\infty}}{A_c} = \frac{35782}{1531526} = -23.36\ \text{N/mm}^2 \tag{4.8}$$

Stress due to moment: $$\sigma_{Ms;r} = \frac{M_{s;r}}{A_c} = \frac{13540}{1531526} = 20.13\,\text{N/mm}^2 \tag{4.9}$$

Stress due to prestressing moment: $$\sigma_{Mp} = \frac{M_p}{A_c} = \frac{0}{1531526} = 0\,\text{N/mm}^2 \tag{4.10}$$

Initial stress at t=0: $\sigma_{c;0}$ = –30.19 N/mm^2 ≤ 0.6f_{cd} OK

Compressive stress at t = ∞: $\sigma_{c;\infty}$ = –44.50 N/mm^2 ≤ f_{cd} OK

Tensile stress at t = ∞: $\sigma_{t;\infty}$ = -4.24 N/mm^2 ≤ f_{ctd} OK

4.4.2 Losses

The losses that occur after prestressing can be divided in two groups:

- Friction losses
- Time-dependent losses (creep, shrinkage, relaxation)

After these losses are calculated, the actual value of the prestressing at t = ∞ can be found.

Friction

$$\begin{aligned} \mu &= 0.16 \\ k &= 0.01\,\text{rad/m} \\ \theta &= 0.00\,\text{rad} \\ x &= 29\,\text{m} \end{aligned} \tag{4.11}$$

$$\begin{aligned} \Delta P_u &= P_{m0}(1 - e^{(-\mu(\theta + kx))}) \\ &= 39758(1 - e^{(-0.16(0.010+0.01*13))}) = 1863\,\text{kN} \end{aligned} \tag{4.12}$$

Shrinkage

$f_{ck}/f_{ck,cube}$ (MPa)	Relative Humidity (in $^0/_0$)					
	20	40	60	80	90	100
20/25	0.62	0.58	0.49	0.30	0.17	0.00
40/50	0.48	0.46	0.38	0.24	0.13	0.00
60/75	0.38	0.36	0.30	0.19	0.10	0.00
80/95	0.30	0.28	0.24	0.15	0.08	0.00
90/105	0.27	0.25	0.21	0.13	0.07	0.00

Table 21: Nominal unrestrained drying shrinkage values $\varepsilon_{cd,0}$ for concrete with cement CEM Class N

h_0	k_h
100	1.0
200	0.85
300	0.75
≥ 500	0.70

Table 22: Values for k_h in equation (4.13)

$$u = \pi d = 6.91 \text{ m}$$
$$h_o = \frac{2A_c}{u} = 443 \text{ mm}$$
$$\varepsilon_{cd,\infty} = k_h * \varepsilon_{cd,0} = 3.50*10^{-5}$$
$$\varepsilon_{ca} = 2.5*(f_{ck} - 10)*10^{-6} = 4*10^{-4} \qquad (4.13)$$
$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 4.35*10^{-4}$$
$$\Delta\sigma_{pcs} = E_p * \varepsilon_{cs} = 84.83 \text{ N/mm}^2$$
$$\Delta P_{cs} = A_p * \Delta\sigma_{pcs} = 2418 \text{ kN}$$

Creep

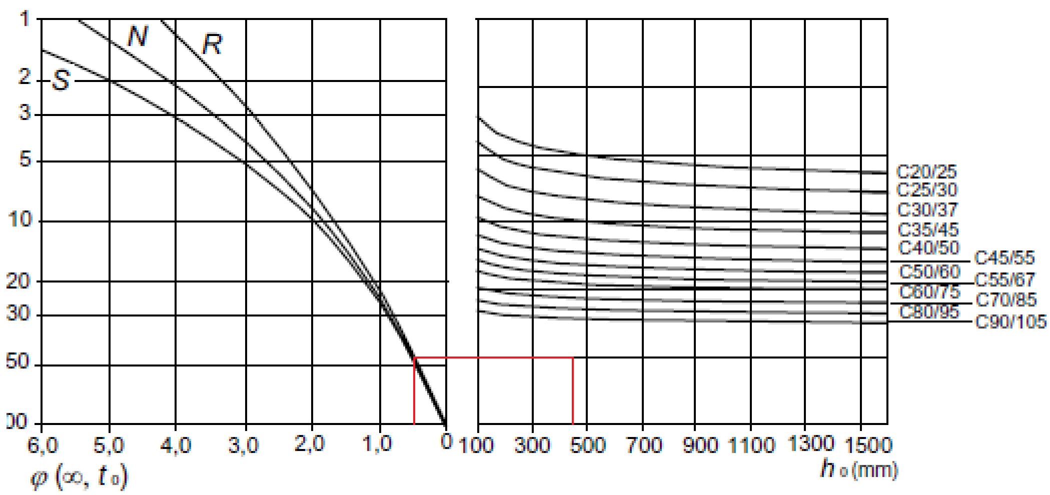


Figure 41: Method for determining creep coefficient $\varphi(\infty, t_0)$ for concrete under normal environmental conditions

$$f_{cm} = f_{ck} + 8 \geq 35$$
$$\sigma_c = \frac{N_{sd}}{A_c} = 1.01 \leq 0.45 f_{ck}$$
$$\alpha_1 = \left(\frac{35}{f_{cm}}\right)^{0.7} = 0.32 \qquad (4.14)$$
$$\alpha_2 = \left(\frac{35}{f_{cm}}\right)^{0.2} = 0.72$$
$$\varphi_{RH} = \left[1 + \frac{1 - \frac{RH}{100}}{0.1 * h_0^{1/3}} * \alpha_1\right] * \alpha_2 = 0.73$$

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{cm}}} = 1.26$$

$$\beta(t_0) = \frac{1}{(0.1 + t_0^{0.20})} = 0.49 \qquad (4.15)$$

$$\beta_c(t,t_0) = \left[\frac{t - t_0}{\beta_H + t - t_0}\right]^{0.3} \approx 1$$

$$\begin{aligned} \varphi(t,t_0)_{analysis} &= \varphi_0 * \beta_c(t,t_0) = \varphi_{RH} * \beta(f_{cm}) * \beta(t_0) * \beta_c(t,t_0) \\ &= 0.73 * 1.26 * 0.49 * 1.00 = 0.45 \\ \varphi(t,t_0)_{graph} &= 0.50 \\ \varphi(t,t_0)_{chosen} &= 0.47 \end{aligned} \qquad (4.16)$$

$$\begin{aligned} \varepsilon_{cc}(\infty) &= \varphi \frac{\sigma_c}{E_c} = 9.88 * 10^{-6} \\ \Delta\sigma_{pcs} &= E_p * \varepsilon_{cc} = 1.93 \text{ N/mm}^2 \\ \Delta P_{cc} &= A_p * \Delta\sigma_{pcc} = 54.90 \text{ kN} \end{aligned} \qquad (4.17)$$

Relaxation

$$\begin{aligned} t &= 500000 \\ \mu &= 0.75 \\ \rho_{1000} &= 2.5\% \\ \Delta\sigma_{pr} &= \sigma_{po} * 0.66 * \rho_{1000} * e^{9.1\mu} \left(\frac{t}{1000}\right)^{0.75(1-\mu)} 10^{-5} \\ &= 67.95 \text{ N/mm}^2 \end{aligned} \qquad (4.18)$$

Total c+s+r

$$\begin{aligned} z_{cp} &= e_p = 285 \text{ mm} \\ \alpha_e &= \frac{E_p}{E_{cm}} = 3.55 \\ \rho_p &= \frac{A_p}{A_c} = 1.86\% \end{aligned} \qquad (4.19)$$

$$\Delta\sigma_{p,c+s+r} = \sigma_{p0}\frac{\varepsilon_{cs}E_p + 0.8\Delta\sigma_{pr} + \alpha_p\varphi(t,t_0)\sigma_{c,QP}}{1+\alpha_p\rho_p\left(1+\frac{A_c}{I_c}z_{cp}{}^2\right)\left[1+0.8\varphi(t,t_0)\right]} \quad (4.20)$$

$$= 118.72 \text{ N/mm}^2$$

Total losses:

$$(\Delta P_u / P_{m0}) + (\Delta\sigma_{p,c+s+r} / \sigma_{p0}) = 5.21 + 8.51 = 13.72\% \quad (4.21)$$

4.5 Reinforcement

After the correct prestressing values have been found, the necessary amount of reinforcement (in SLS) must be determined next. This fictitious reinforcement[3] can be determined with the help of the two main equilibrium equations.

Horizontal equilibrium:

$$\sum H = N_{sc} - N_{st} + \Delta N_{pc} - \Delta N_{pt} + N_c - P_{m\infty} = N$$
$$N_{rep} + P_{m\infty} + (0.5 * \mathrm{A}_s * [\sigma_{st} - \sigma_{sc}]) + (0.5 * \mathrm{A}_p * [\Delta\sigma_{pt} - \Delta\sigma_{pc}]) - (\mathrm{E}_c * \varepsilon_c * b * x_u) = 0 \quad (4.22)$$

$$N_s = A_s * \sigma_s = A_s * E_s * \varepsilon_s$$
$$N_c = A_c * \sigma_c = b * x_u * E_c * \varepsilon_c \quad (4.23)$$
$$\Delta N_p = A_p * \Delta\sigma_p - P_{m\infty} = A_p * \xi * E_p * \varepsilon_p - P_{m\infty}$$

$$\varepsilon_{st} = \frac{\sigma_{st}}{E_s}$$

$$\varepsilon_c = \frac{x_u}{d_s - x_u}\varepsilon_s$$

$$\Delta\varepsilon_p = \frac{x_u}{d_s - x_u}\varepsilon_s \quad (4.24)$$

$$\varepsilon_p = \frac{P_{m\infty}}{E_p A_p} + \Delta\varepsilon_p$$

Moment equilibrium:

$$\sum M = \left(N_{st} + N_{sc}\right) * d_s + \left(\Delta N_{pt} + \Delta N_{pc}\right) * e_p + N_c * e_c = M$$
$$M_{s;r} - (0.5 * \mathrm{A}_s * [\sigma_{st} + \sigma_{sc}]) * \mathrm{d}_s - (0.5 * \mathrm{A}_p * [\Delta\sigma_{pt} + \Delta\sigma_{pc}]) * \mathrm{e}_p - (\mathrm{E}_c * \varepsilon_c * b * x_u) * e_c = 0 \quad (4.25)$$

From (4.22) & (4.25) the necessary amount of reinforcement can be calculated. After some reshuffling the following equations are attained:

[3] For UHSC this is the fictitious amount of reinforcement that is necessary, which will be supplied by the steel fibers. For OSC & HSC it is the traditional reinforcement.

$$A_{s,req,N} = 2\left[N_{rep} + P_{m\infty} + (0.5 * A_p * [\Delta\sigma_{pt} - \Delta\sigma_{pc}]) - (E_c * \varepsilon_c * b * x_u)\right] / (\sigma_{sc} - \sigma_{st}) \quad (4.26)$$

$$A_{s,req,M} = 2\left[M - N_c * e_c - \left(\Delta N_{pc} + \Delta N_{pc}\right) * e_p\right] / \left[\left(\sigma_{st} + \sigma_{sc}\right) * d_s\right] \quad (4.27)$$

After an iteration process the following values can be obtained:

$A_{s,req}$[4] = 60059 mm^2
x_u = 1263.13 mm

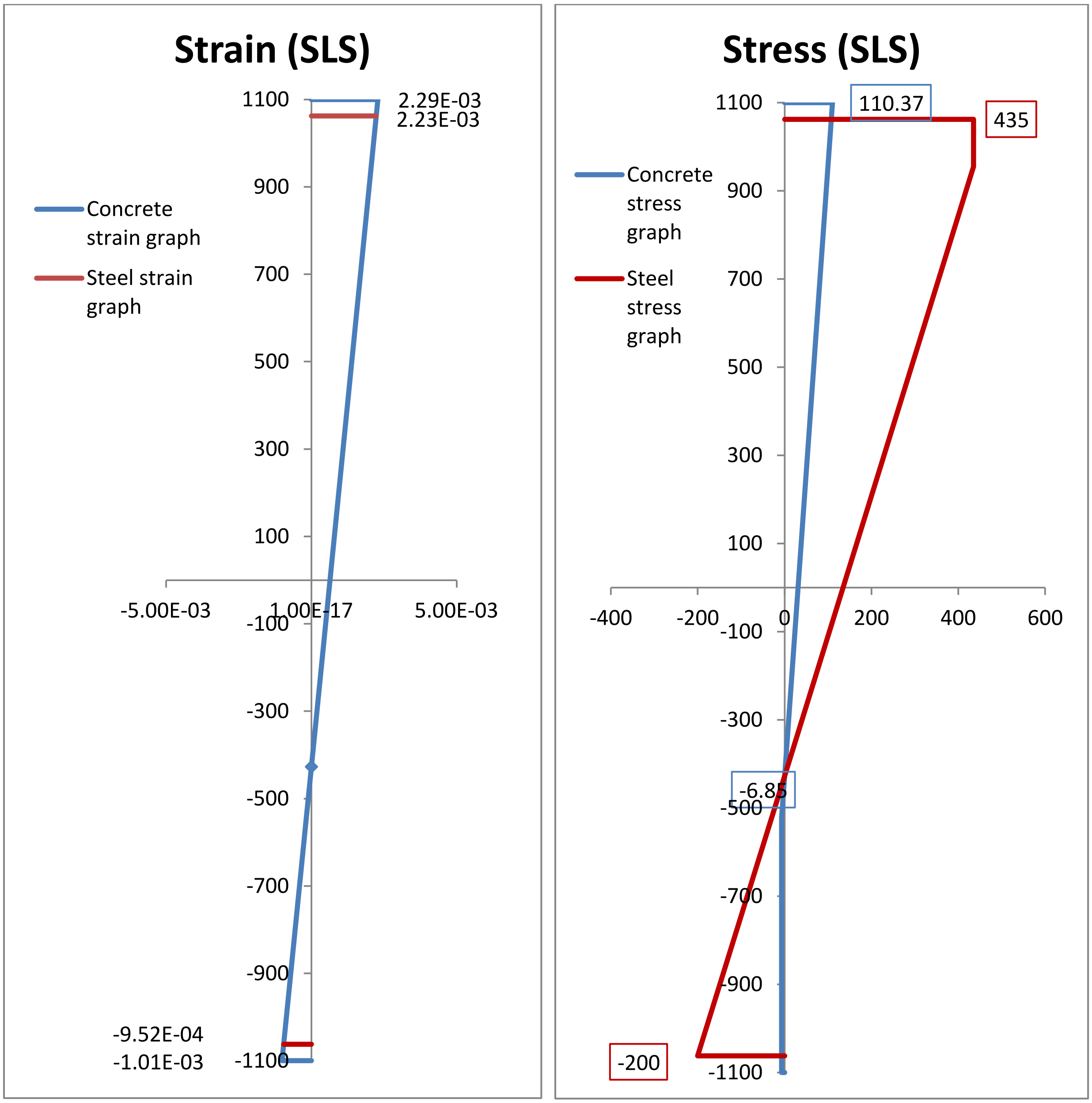


Figure 42: Stress & strain graphs for UHSC (SLS)

[4] For UHSC this is the fictitious amount of reinforcement that is necessary.

4.6 Moment capacity

Now that the structure satisfies the strength criteria in the SLS, the capacity of the structure in the ULS should be checked. This is done by calculating the bending moment capacity.

$$\sum H = N_{sc} - N_{st} + \Delta N_{pc} - \Delta N_{pt} + N_c - P_{m\infty} = N$$
$$N_{sd} + P_{m\infty} + (\mathrm{A}_s * [\sigma_{st} - \sigma_{sc}]) + (\mathrm{A}_p * [\Delta\sigma_{pt} - \Delta\sigma_{pc}]) - (\mathrm{E}_c * \varepsilon_c * b * x_u) = 0 \tag{4.28}$$

$$N_s = A_s * \sigma_s = A_s * f_{yd}$$
$$N_c = A_c * \sigma_c = b * x_u * 0.75 f_c d$$
$$\Delta N_p = A_p * \Delta\sigma_p - P_{m\infty} \tag{4.29}$$
$$\Delta\sigma_p = f_{pd} + \frac{(\varepsilon_p - \frac{f_{pd}}{E_p})}{0.0035 - \frac{f_{pd}}{E_p}} \left(\frac{f_{pk}}{\gamma_s} - f_{pd}\right)$$

$$\varepsilon_{st} = \frac{f_{yd}}{E_s}$$
$$\varepsilon_c = \frac{x_u}{d_s - x_u} \varepsilon_s$$
$$\Delta\varepsilon_p = \frac{x_u}{d_s - x_u} \varepsilon_s \tag{4.30}$$
$$\varepsilon_p = \frac{P_{m\infty}}{E_p A_p} + \Delta\varepsilon_p$$

From equations (4.28) to (4.30) we can find the value of x_u:

$$x_u = 858 \text{ mm}$$

Now that the value of the compressive zone height has been found, it can be used to calculate the moment capacity.

$$M_{Rd} = (N_{st} + N_{sc}) * d_s + (\Delta N_{pt} + \Delta N_{pc}) * e_p + N_c * e_c$$
$$= (0.5 * \mathrm{A}_s * [\sigma_{st} + \sigma_{sc}]) * \mathrm{d}_s + (0.5 * \mathrm{A}_p * [\Delta\sigma_{pt} + \Delta\sigma_{pc}]) * \mathrm{e}_p + (\mathrm{E}_c * \varepsilon_c * b * x_u) * e_c \tag{4.31}$$
$$= 85547 \text{ kNm}$$

$M_{Rd} > M_{sd}$ => 85547 kN > 16881kN OK
$M_{Rd} > M_{cr}$ => 85547 kN > 21132 kN OK

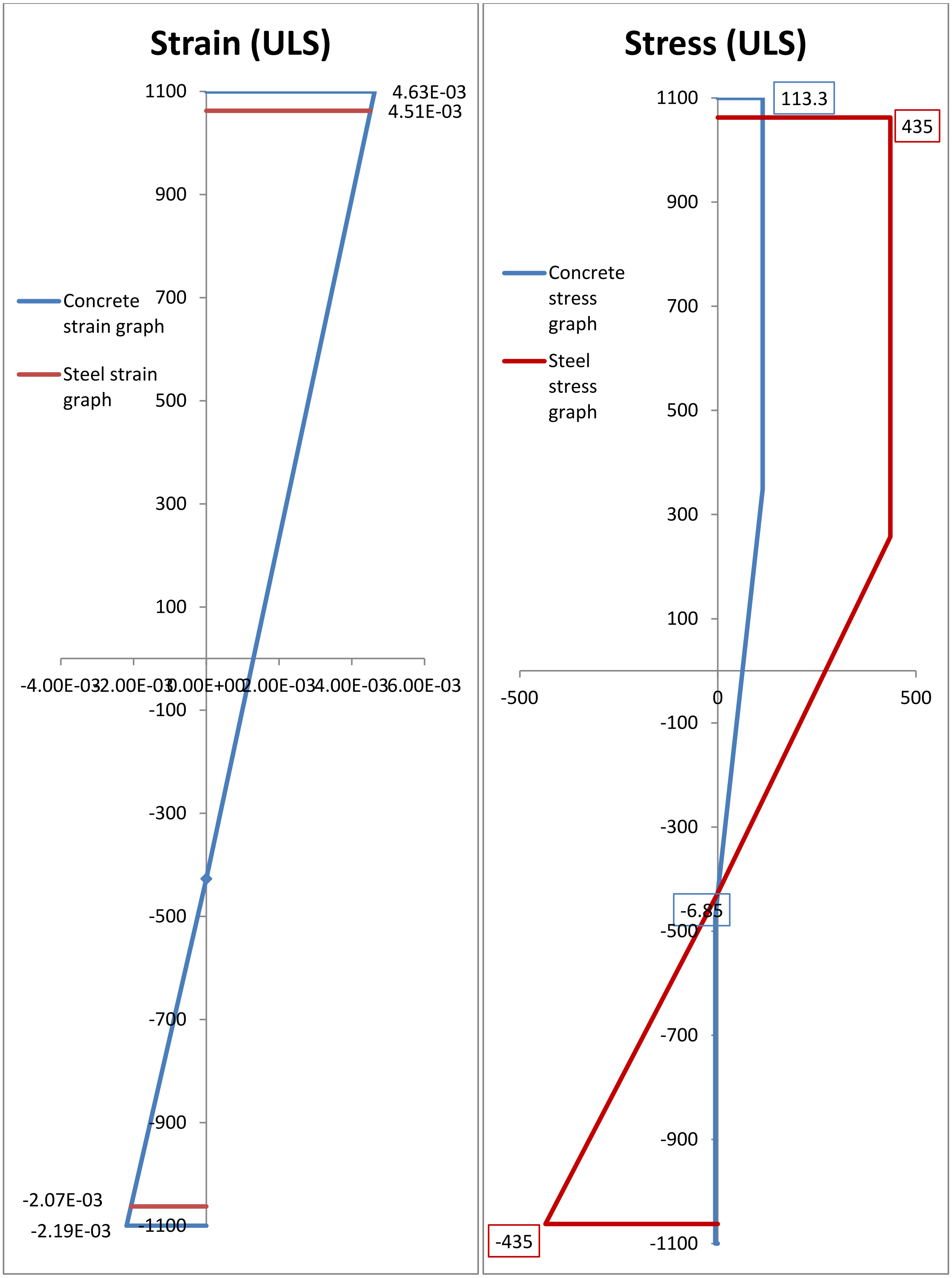


Figure 43: Strain & stress graph for UHSC (ULS)

4.7 Foundation

Although the foundation does not exactly fall within the scope of the thesis, it will still be studied.

4.7.1 Dimensions

First the shape of the foundation block should be decided on. There are several options:

- Circular for one mast
- Oval for both masts
- Rectangular for one mast
- Rectangular for both masts

As the shape of the mast was chosen circular to enable the mast to resist forces in all directions, it makes sense to apply the same principle to the foundation. Therefore the rectangular variants can be eliminated. When considering one large foundation block, it is found that because of the large distance between the two masts, the dimensions of the foundation block will be very large, resulting in large material costs. Therefore it is chosen to construct two separate foundations for each mast.

The dimensions of the foundation block can be seen in Table 23Table 23: & Figure 44.

Component	Symbol	Value	Unit
Inner length	d_1	2.20	m
Outer length	l_2	2.50	m
Total diameter	d	7.20	m
Thickness	t	1.50	m

Table 23: Foundation block dimensions

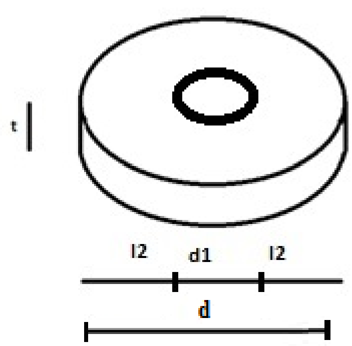


Figure 44: Circular foundation with dimensions

4.7.2 Foundation piles

As the shape of the foundation block has been established as circular, the pile configuration should now be decided. There are two configurations that can be considered:

- Orthogonal vs. concentric
- Distributed piles vs. piles only at edge

Ad 1. Orthogonal vs. concentric
The pile pattern under a circular foundation block could either be orthogonal or concentric. In the case of an orthogonal configuration, the force distribution is easy to calculate, but the edge is problematic. In the second case, the edge is simple, but the determination of the force is complex.

Ad 2. Distributed piles vs. piles only at edge
The choice here depends on the governing load. If the axial force is distributed over the foundation block, then the first option is very attractive. If the axial force is a single load and if the moment is very large, than the second option can provide a better alternative.

In this case a concentric pile configuration was chosen, to be applied only at the edges of the foundation block.

Component	Symbol	Value	Unit
Pile length	l_p	20	m
Pile diameter	d_p	200	mm
Pile cross section	A_p	0.04	m^2
Grid diameter	d_{grid}	400	mm
Grid cross section	A_{grid}	0.13	m^2
Configuration		Concentric	-
Pile strength	C	45/55	N/mm^2
Young's modulus	E_c	36000	N/mm^2
Bearing capacity	P_p	200	kN
Axial force at foot	N_{ed}	3217	kN
Moment at foot	M_{ed}	12000	kNm

Table 24: Foundation pile properties

$$n_{\min} = \frac{N_{sd}}{P_p} = 11$$
$$n_{\max} = \frac{0.25 * \pi * d^2}{A_{grid}} = 68 \quad (4.32)$$

Piles at the edge: $n = 21$

Pile spacing: $l_{r,\tan} = \frac{\pi * d}{n-1} = 1.13$ m > 0.400 m OK

Axial resistance: $N_{Rd} = n * P_p = 10500$ kN > 2263 kN OK (4.33)

Pile eccentricity: $n * e_p = 22.73$ m

Moment resistance: $M_{Rd} = 11365$ kNm > 11247 kNm OK

So a total of 42 piles will be needed at mast location (for both masts).

4.7.3 Rotational stiffness

$$\xi_{rigid} = \frac{3.43 * E_c * I_c}{\gamma_c} = 2.89 * 10^{10} \text{ Nm/rad} \tag{4.34}$$

$$\theta = M / \xi = 3.89 * 10^{-4} \text{ rad} \tag{4.35}$$

$$\begin{aligned} \delta &= \theta * h \leq 0.001h \\ &= 22.15 \text{ mm} < 57 \text{ mm} \end{aligned} \tag{4.36}$$

4.7.4 Shear

$$\begin{gathered} V_{Ed} = 2263 \text{ kN} \\ d = 0.9 * t = 1.35 \text{ m} \\ k = 1 + \sqrt{\frac{200}{d}} = 1.37 \\ c_1 / c_2 = 1.0 \\ k_c = 0.60 \\ u_1 = \pi * (d_{mast} + 2d) = 16.34 \text{ m} \\ e = 0.5 * d_{mast} \\ W_1 = e * u_1 = 17.97 \text{ m}^2 \\ \beta = 1 + k * \frac{M_{Ed}}{V_{Ed}} * \frac{u_1}{W_1} \\ v_{Ed} = \frac{\beta * V_{Ed}}{u_1 * d} \\ A_s = 30000 \text{ mm}^2 \\ \rho = \frac{A_s}{1 * t} = 2.00\% \end{gathered} \tag{4.37}$$

$$\text{v}_{\min} = 0.035 * k^{1.5} * f_{ck}^{\ 0.5} = 0.31 \text{ N/mm}^2 < 0.12 * k * (100 * \rho * f_{ck})^{1/3} \tag{4.38}$$

$$\sigma_{cp} = \frac{V_{Ed}}{0.25 * \pi * d^2} = 0.06 \text{ N/mm}^2$$

$$v_{Rd,c} = 0.12 * k * (100 * \rho * f_{ck})^{1/3} + \sigma_{cp} = 0.65 \text{ N/mm}^2 > \text{v}_{Ed} \qquad \text{OK}$$

For the complete foundation calculation and drawings, see appendix H.

4.8 Deformations

Aside from the strength criterion, the stiffness criterion must be checked as well. The deformations that must be checked are:

- Crack width
- Deflection (top and relative)

4.8.1 Crack width

The theoretical crack width can be calculated with the following formulas:

$$
\begin{aligned}
c &= 25 \text{ mm} \\
\varnothing_s &= 20 \text{ mm} \\
s &\le 5*(c+0.5*\varnothing_s) = 175 \text{ mm} \\
\mathrm{k}_1 &= 1.6 \\
k_2 &= 0.5 \\
k_3 &= 3.4 \\
k_4 &= 0.425 \\
\varnothing_p &= 1.6*\sqrt{n_{str}*A_p} = 86 \text{ mm} \\
\xi_1 &= \sqrt{\xi\tfrac{\varnothing_s}{\varnothing_p}} = 0.341
\end{aligned}
\tag{4.39}
$$

$$
h_{c,eff} = MIN\begin{cases} 2.5(d-d_s) = 312.50 \text{ mm} \\ \dfrac{h-x_u}{3} = 312.32 \text{ mm} \\ \dfrac{d}{2} = 1100.00 \text{ mm} \end{cases}
$$

$$
A_{c,eff} = b*h_{c,eff} = 303342 \text{ mm}^2 \tag{4.40}
$$

$$
\rho_{p,eff} = \frac{A_s+\xi^2 A_p^{'}}{A_{c,eff}} = 0.571
$$

$$
s_{r,\max} = k_3 c + \frac{k_1 k_2 k_4 \varnothing}{\rho_{p,eff}} = 96.92 \text{ mm} \tag{4.41}
$$

$$
\begin{aligned}
\sigma_s &= 200 \text{ N/mm}^2 \\
k_t &= 0.6 \\
f_{ct,eff} &= 10.27 \text{ N/mm}^2 \\
\alpha_e &= 3.82 \\
\varepsilon_{sm}-\varepsilon_{cm} &= \frac{\sigma_s - (\frac{k_t f_{ct,eff}}{\rho_{p,eff}}(1+\alpha_c\rho_{p,eff}))}{E_s} \ge 0.6\frac{\sigma_s}{E_s} \\
&= 7.89*10^{-4} > 5.71*10^{-4}
\end{aligned}
\tag{4.42}
$$

$$
\begin{aligned}
w_k &= s_{r,\max}*(\varepsilon_{sm}-\varepsilon_{cm}) \\
&= 0.076 \text{ mm}
\end{aligned}
\tag{4.43}
$$

4.8.2 Deflection

There are two deflection criteria that must be met:

- Deflection at the top must be smaller than 2.5% of the mast height;
- Relative deflection at 0.6h must be smaller than 1% of the mast height.

$$\delta_q = \frac{q_{wind} * h^4}{8EI} \tag{4.44}$$

$$\begin{aligned}\delta_F = \; & \frac{F_{lw} * h_{lw}^{\;3}}{3EI} + \frac{F_{h;3} * h_3^{\;3}}{3EI} + \frac{F_{h;3} * h_3^{\;2}}{2EI} * (h - h_3) + \frac{F_{h;2} * h_2^{\;3}}{3EI} + \frac{F_{h;2} * h_2^{\;2}}{2EI} * (h - h_2) + \\ & \frac{F_{h;1} * h_1^{\;3}}{3EI} + \frac{F_{h;1} * h_1^{\;2}}{2EI} * (h - h_1) + \frac{F_{pl} * h_{pl}^{\;3}}{3EI} + \frac{F_{pl} * h_1^{\;2}}{2EI} * (h - h_{pl})\end{aligned} \tag{4.45}$$

$$\begin{aligned}&\delta_{top} = \delta_q + \delta_F \leq 0.025h \\ &= 141.45 + 582.30 = 724 \text{ mm} < 1425 \text{ mm} \qquad \text{OK}\end{aligned} \tag{4.46}$$

$$\delta_{hor,x,q} = \frac{q_{wind} * h^4}{8EI} - \frac{q_{wind} * h^3}{6EI}(h - 0.6h) \tag{4.47}$$

$$\begin{aligned}\delta_{hor,x,F} = \; & \frac{F_{lw} * h_{lw}^{\;3}}{3EI} - \frac{F_{h;3} * h_3^{\;2}}{2EI} * (h_{lw} - 0.6\,\mathrm{h}) + \frac{F_{h;3} * h_3^{\;3}}{3EI} - \frac{F_{h;3} * h_3^{\;2}}{2EI} * (h_3 - 0.6\,\mathrm{h}) \\ & + \frac{F_{h;2} * h_2^{\;3}}{3EI} - \frac{F_{h;2} * h_2^{\;2}}{2EI} * (h_2 - 0.6\,\mathrm{h}) + \frac{F_{h;1} * h_1^{\;3}}{3EI} - \frac{F_{h;1} * h_1^{\;2}}{2EI} * (h_1 - 0.6\,\mathrm{h}) \\ & + \frac{F_{pl} * h_{pl}^{\;3}}{3EI} + \frac{F_{pl} * h_1^{\;2}}{2EI} * (h_{pl} - 0.6\,\mathrm{h})\end{aligned} \tag{4.48}$$

$$\begin{aligned}&\delta_{hor,x} = \delta_{hor,x,q} + \delta_{hor,x,F} \\ &\qquad = 66.01 + 270.89 = 337 \text{ mm} \\ &\delta_{rel} = 0.6\delta_{top} - \delta_{hor,x} \leq 0.01h \\ &\qquad = 434 - 337 = 97 \text{ mm} < 570 \text{ mm} \qquad \text{OK}\end{aligned} \tag{4.49}$$

4.9 Stability

4.9.1 Folding (“Plooi”)

Although folding is usually not an issue for concrete structures, it was checked as well due to the high slenderness of the mast and because the mast, especially the UHSC variant, has a small wall thickness.

$$\varepsilon = \sqrt{\frac{235}{f_{yd}}} = 0.735$$

$$\rho_A = \begin{cases} 1 & for\ \frac{d}{t} < 90\varepsilon^2 \\ 0.3 + \frac{63\varepsilon^2}{d/t} & for\ \frac{d}{t} \geq 90\varepsilon^2 \end{cases} \tag{4.50}$$

$$A_{eff} = \rho_A * A = 863938 \text{ mm}$$

$$\rho_W = \begin{cases} 1 & for\ \frac{d}{t} < 157.5\varepsilon^2 \\ 0.6 + \frac{63\varepsilon^2}{d/t} & for\ \frac{d}{t} \geq 157.5\varepsilon^2 \end{cases} \tag{4.51}$$

$$W_{eff} = \rho_W * W = 2.04 * 10^8 \text{ mm}$$

$$\sigma_N = \frac{N + P_{m0}}{A_{eff}} = 26.96 \text{ N/mm}^2$$

$$\sigma_M = \frac{M}{W_{eff}} = 16.73 \text{ N/mm}^2$$

$$\sigma_{total} = \sigma_N + \sigma_M \leq \frac{f_{yd}}{\gamma_s} \tag{4.52}$$

$$= 43.70 \text{ N/mm}^2$$

$$\text{UC}: \frac{\sigma_{total}}{f_{cd}} = 0.39 < 1 \qquad \text{OK}$$

4.9.2 Buckling

$$N_{cr} = \frac{\pi^2 EI}{l_0^{\,2}} = \frac{\pi^2 * 55000 * 6.13 * 10^{10}}{(2*57)^2} = 5740 \text{ kN} \tag{4.53}$$

$N_{s;top} = N_{s;lw} = 16$ kN < 5740 kN OK

4.10 Optimize wall thickness

A summary of the design result can be seen in Table 25: & Table 26:. After the amount of required prestressing tendons was calculated, it was checked to see how much more prestressing could be applied, before the structure would inevitably fail in compression. From the ratio between the maximum amount of prestressing and the minimum required prestressing, a statement could be made about the overcapacity of the structure. Based on these ratios, the wall thickness could undergo an initial reduction. After that initial reduction the wall thickness could then be further optimized through iterations and checks.

Reinforced variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength class	C	45	90	170	N/mm^2
Wall thickness	t	250	250	250	mm
Compressive zone height	x_u	710	664	626	mm
Reinforcement	A_s	63	63	66*	10^3 mm^2

Table 25: Summary of reinforced design results for t = 250 mm

Prestressed variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength class	C	45	90	170	N/mm^2
Wall thickness	t	250	250	250	mm
Compressive zone height	x_u	1351	1247	1263	mm
Reinforcement	A_s	63	57	60*	10^3 mm^2
Number of prestressing tendons applied	n	7	7	8	-
Max number of prestressing tendons	n	5	12	24	-
Initial reduction factor	-	1.40	0.58	0.33	-

Table 26: Summary of prestressed design results for t = 250 mm

It can easily be seen that the amount of reinforcement does not differ much between the reinforced and the prestressed variants, while the height of the compressive zone of the prestressed variants is nearly double the height of the reinforced variants. It can also be seen that for the prestressed OSC mast, the initial dimensions are not sufficient for the mast. The wall thickness has to be increased beyond 250 mm ($0.5*d_{top}$), which means that the initial diameters at bottom and top should be increased. Thus already a conclusion can be made that it is not possible for the OSC variant to be prestressed if the initial dimensions are absolute.

The results in Table 25& Table 26, are based on strength criteria. Of course the mast should satisfy the stiffness criteria as well. The governing stiffness criteria turned out to be the deflection at the top of the mast. Based on this maximum allowable deflection, the minimum necessary wall thickness to satisfy this criteria, could be decided. Table 27: shows the optimal wall thickness based on strength as well as stiffness.

Component	Symbol	OSC	HSC	UHSC	Units
Concrete strength	C	45	90	170	N/mm^2
Concrete stiffness	EI	5.31	6.69	6.85	10^{15} Nmm^2
Based on strength	t_{str}	420	145	75	mm
Based on stiffness	t_{stiff}	340	125	90	mm

Table 27: Optimal wall thickness based on strength or stiffness

* This is the fictitious amount of reinforcement necessary for the UHSC variant.

Figure 45: Wall thickness based on strength or stiffness

Figure 45 illustrates the decrease of wall thickness with increasing concrete strength classes. The decrease seems to stabilize at UHSC. This is due to the minimal dimensions necessary for the structure to still comply to the criteria. Thus even though the material properties will improve even more beyond UHSC, the wall thickness will likely barely decrease.

It can also easily be seen, that at a certain point (C130 – C140) the governing criteria changes. For lower concrete classes the strength criteria governs the wall thickness, while for higher strength classes, the stiffness criteria becomes more important. This is because for lower concrete strength classes, more material is used, resulting in a higher moment of inertia (I). This increase of moment of inertia is higher than the decrease of elasticity modulus (E). Thus for decreasing concrete classes, the stiffness increases (see Table 27:).

After the wall thickness has been optimized, the new design results can be calculated (see Table 28:). This will be looked at in more detail in chapter 7.

Prestressed variants	**Symbol**	**OSC**	**HSC**	**UHSC**	**Units**
Concrete strength class	C	45	90	170	N/mm^2
Wall thickness	t	415	145	75	mm
Compressive zone height	x_u	1263	1399	1527	mm
Reinforcement	A_s	39	66	72*	10^3 mm^2
Number of prestressing tendons applied	n	10	8	8	-

Table 28: Summary of design results for t = 75 mm

With this the preliminary design is complete. The values in Table 28: can now be used, when considering the production and execution process. For the complete design results of all variants see appendix E – G. For the design drawings see appendix I.

* This is the fictitious amount of reinforcement necessary for the UHSC variant.

5. Production process

For the construction of a new high voltage transmission route, it is obvious that a large number of masts will be needed. These masts will most likely vary in length and diameter, because of the different types used, as well as the changes in terrain. To ensure a smooth execution process at the building site later on, an efficient production method must be devised.

5.1 General

As the mast has a variable cross section, the production process is more complicated than in the case of a straight tubular column. The dimensions for the dead-end mast are larger than for the suspension mast to take up the larger loads. This implies that it is a good idea to devise two separate moulds for the suspension and the dead-end mast.

The production process of the masts should include the following aspects:

- Moulds can be used numerous times;
- Moulds have high adaptability to deal with changing cross section;
- Moulds can be used efficiently to reduce total production time;
- Moulds should be as cheap as possible.

Material

The material choice for the moulds is mostly determined by the number of repetitions and is in essence an economical assessment. Generally for the mould material either timber, steel or plastics are used. Below the advantages and disadvantages of the three materials are discussed.

Timber

Timber moulds are usually applied because of their high versatility. For simple forms they are by far the cheapest alternative. However they are much more susceptible to wear than steel moulds and are not as accurate.

Steel

Steel moulds are able to produce very accurate results. However in exchange for this high accuracy, they have a higher cost and take longer to construct. Because they are also not easily adjustable, they are usually applied when a large number of standard units have to be produced.

Plastics/polymers

Plastic or polymer moulds offer an even greater degree of accuracy than steel moulds, but in return are more brittle and easy to break. Because they damage so easily, they regularly need repair. This places them between timber and steel moulds in terms of mould life span. They often tend to be used in combination with timber.

An overview of the three options can be seen in Table 29.

	Timber	Steel	Plastic
Costs	–	– –	0
Maintenance	– –	+	0
Repetition	0	+	++
Accuracy	+	0	+
Adaptability	++	0	– –

Table 29: Mould material comparison

As can be seen from Table 29, each alternative has advantages and disadvantages that balance each other out. None of the materials are superior to the others and the choice must be made, based on the situation. In this case steel moulds are a reasonable choice, because of their high accuracy, dimensional stability, low maintenance and their 100% recyclability rate.

5.2 Moulds

Because prefab elements are usually produced in heavy stiff moulds, that are used multiple times, the elements must be shaped in such a way that easy demoulding is possible. Furthermore the moulds should be shaped in such a way that reinforcement of the elements, as well as notches or recesses can be applied in an efficient way.

The entire mould of a prefab element can be broken down in the following parts [37]:

a) A stiff supporting structure, which allows the mould to stay rigid during pouring and compaction of the concrete, as well as during demoulding.
b) The base, on top of which the element is actually poured.
c) The side walls: These resist the horizontal pressure caused by the fluid concrete mix and should thus be stiff and strong to prevent deformations from occurring.
d) The top side: Via the top side the mould is filled with concrete (as well as sometimes compacted). This top side should allow for easy pouring of the concrete and should be as spacious as possible.

Another important aspect of production is the finishing side. Usually after the concrete has been poured, the top side still needs finishing. Therefore the concrete gets a side with a rougher surface than the other smooth sides. For concrete columns that will be in view on all sides, this is very undesirable. Therefore these type of columns are usually produced in vertical moulds. This is especially the case for round columns.

Because the number of segments for each concrete class differs, only the HSC alternative will be worked out, as it provides a good indication for all the variants. As previously seen the HSC mast consists of six segments. To produce these six segments either a full length mould can be created or six separate moulds for the segments. Below the pros and cons of both these options, as well as a variation are discussed.

Full-length mould

+ Fast production time
+ Low costs
– Difficult production process
– Low flexibility
– Low alteration ability

One mould per segment

- \+ Simple production process
- \+ Fast production time
- \+ Easy alteration ability
- – Low flexibility
- – High costs

One mould per two segments

- \+ Moderate production process
- \+ Low costs
- \+ Moderate alteration ability
- – Low flexibility
- – Slow production time

As can be seen the alternatives all have fairly balanced pros and cons. Considering that the ability to optimize and alter each segment for connections later on, is of the utmost importance, the best alternative is to make one mould per segment.

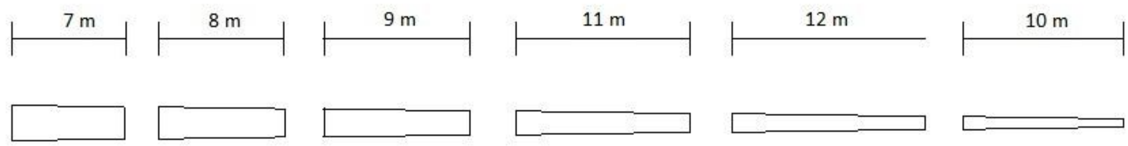


Figure 46: Moulds per segment

5.3 Process

Now that the amount of moulds and the material type have been determined, the actual process has to be established. For the segment production process, two methods can be distinguished. One for concrete tubes with small diameters and the other for concrete tubes with large diameters.

Small diameter

As the concrete mix is being made, simultaneously the mould is being build. A special machine is used to build what's called the "cage". This cage is a circular steel frame that will form the pipe's internal structure. First 2.5 m long steel rods are inserted in a circular configuration, pushing each rod partway through the machine to the other side. There an automated spot welder fuses a steel cable to one rod. Then the machine begins turning, winding the cable tightly around the rods at high speed. As the rods pass through the machine, the welder fuses the cable to them in one continuous spiral.

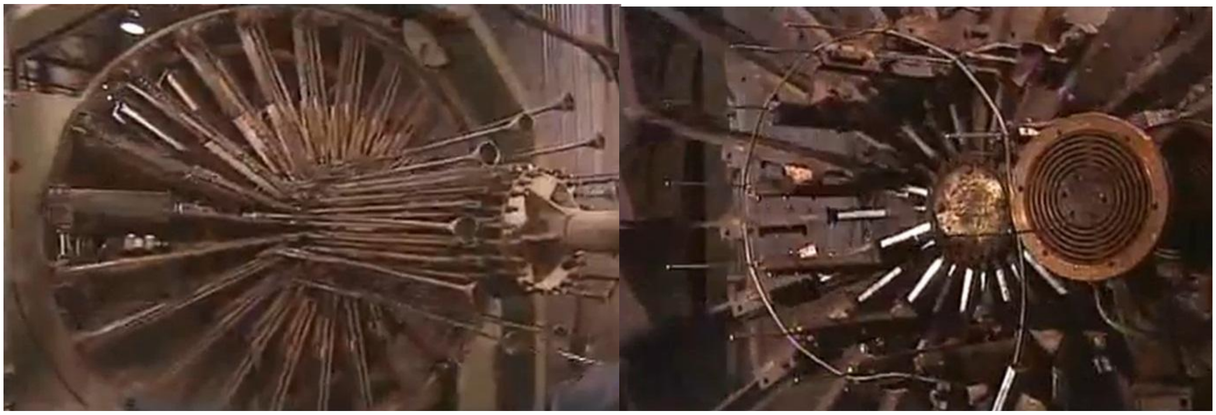


Figure 47: The "cage" (L) & automated spot welder (R) [XIX]

Figure 48: Spiraled cable with bell section [XIX]

As the spiraled cable reaches the end of the rods, the machine's claws spread outward, forming a wider section on the end. This is called the bell section. It is then positioned on a base ring designed to hold the cage in place. Subsequently a hinged steel mould is closed over it. Now the mould is ready for the adding of the concrete. A fork lift transports the mould to a machine called the packer-head. The mould is positioned directly under it. A long drill like screw descends into the mould. As concrete pours in, the screw turns at high speed, while moving up and down. Its blades propel the concrete outward against the mould walls forming the pipe.

Figure 49: Hinged steel mould (L) & drill like screw (R) [XIX]

The high speed at which the concrete is spun causes centrifugal acceleration. For that reason this process is called *centrifugal projection forming*.

This method has proven to be viable for OC & HSC. For UHSC which has steel fibers, the method is somewhat controversial. For fiber reinforced materials, the spinning may change fiber distribution and lead to a certain orientation along the rotation axis. This effect could have an advantageous (or disadvantageous) influence on the material properties. As this method has not yet been applied much for FRC, mainly theoretical predictions have been used. Some researchers say that the spinning will lead to optimal fiber distribution. Others argue that due to their high density, steel fibers are not the best choice for application in centrifugal techniques, because segregation occurs. Therefore they suggest using relatively expensive high strength and modulus fibers (carbon, PVA, etc.) instead. It is clear that much more research must be done to come to a clear conclusion.

Large diameter
The process for making large diameter pipes is slightly different. A welder fuses spacers to the cage both inside and out. These will center the cage inside the mould. The cage is then positioned onto a base ring, before the outer part of the mould, called the outside form, is lowered over it. After securing it to the ring, the entire unit is lowered over the smaller inner part of the mould called the inside form. An overhead funnel pours concrete into the cavity between the two moulds.

Figure 50: Outer form (L) & concrete pouring (R) [XIX]

During the concrete pouring, powerful electric vibrators shake the mould. This forces the thick concrete downward filling the cavity. This process is called *vibration forming*.

With either concrete tube forming technique the moulded concrete is initially quite fragile, so the moulds are carried to the curing warehouse, were they harden for about 12 hours (depending on concrete strength class). After reaching sufficient strength, the moulds are removed, leaving the pipes standing upright.

5.4 Adjustments
In the previously discussed methods, it was assumed that the shape of the segments will be simple formed pipes. However as already seen in chapter 4, prestressing will be applied. These prestressing tendons must be connected to the concrete at certain places, most likely at segment ends. Furthermore as will be seen in chapter 6, there are a number of horizontal connections that

must be applied as well. These connections will have an influence on the production process, because depending on the chosen type of connection, the segments will have to be formed differently. Therefore the productions process should be able to shape unnaturally formed segments.

The difficulty with unnatural shapes is not so much the forming of the shape, but more demoulding of the concrete. Therefore flexible solutions must be thought of to enable easy demoulding. A couple of these options are described below. The options consider necessary connections at 4 locations in a section.

Note: Although the figures below depict a straight, constant wall thickness it should be remembered that the mast has a tapered form as well as a variable wall thickness. However this form can easily be achieved by utilizing slanted outer moulds.

If a segment has extra protrusions at the top or bottom of the segment it complicates the demoulding process. If there is only one protrusion at either the top or the bottom of the segment, the demoulding process is still relatively simple. First the outer mould is removed and then the concrete segment can be pulled out in the direction of the protrusion. Or alternatively, the inner mould can be pulled away in the direction opposite of the protrusion.

When there are protrusions at both segment ends however, more creative solutions have to be thought up.

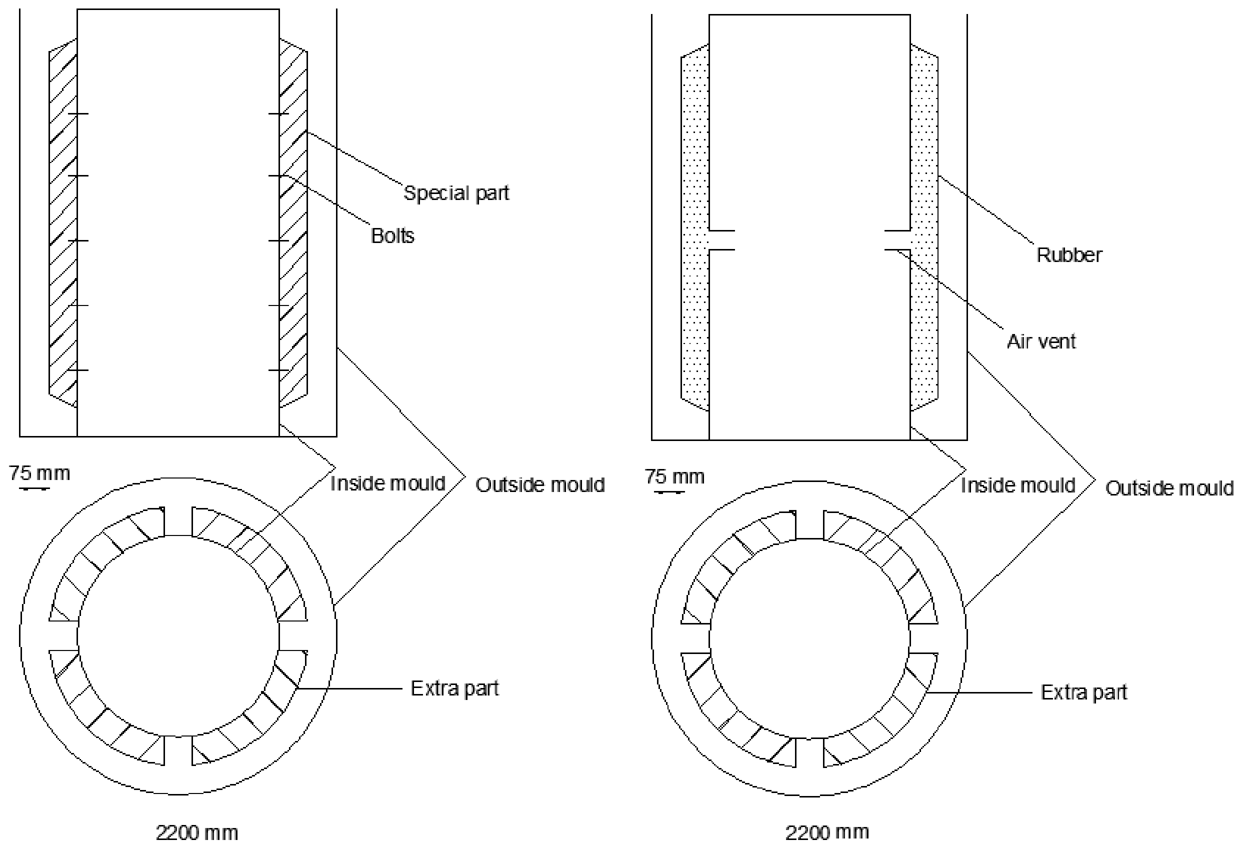


Figure 51: Bolted part (L) & blown rubber part (R)

In Figure 51L the special parts can be welded or bolted to the inner mould. The extra parts are only present so the rest of the segment can be made with a normal shape. To demould the segment, first the extra parts must be removed. Afterwards the inside mould can be rotated and consequently removed. Of course to accomplish this all the surfaces must be coated with a demoulding layer to ensure easy demoulding. If rotating turns out to be a difficult solution, then the bolted option has an additional method of demoulding. Once again the extra parts must be removed first. Afterwards the bolts are screwed loose, so the special part is not connected to the inner mould anymore. Then either the inside mould or outside mould with special parts can be lifted. The special and extra parts can be made from a wide range of materials ranging from steel or timber to plastics or foam.
The same principle can be applied for Figure 51R. First the "rubber balloon" is blown to its specific proportions. After the concrete has been cast, the air is released through the inside mould. After the extra parts have been removed, the inside mould can either be rotated and then removed or can simply be lifted, depending on the simplicity of the air vent.

With these two methods a large variety of forms can be created at the top and bottom of a segment, but also in between (e.g. for the isolator connection).

6. Execution

The execution of transmission towers in concrete, is a vital aspect of the entire process. After the initial design has been carried out, the entire execution process must be looked at in detail. While examining this process, factors that influence the initial design should be scrutinized. One such an important aspect are the connections. In tubular steel masts, the connections are simple to establish. In concrete tube masts however, this is a lot more complicated. Depending on the type of connections, the initial design assumptions might have to be changed.

6.1 Building process

As seen in the previous chapter, the segments are produced in a factory, after which they can be transported to the building site. This is however just one of the many steps in the building process. The building process can be broken down in six steps [XX] (see Figure 52):

1. Building site preparation
2. Foundation activities
3. Transport of segments to building site
4. Assembly of mast
5. Assembly of conductors and other cables
6. Landscape restoration

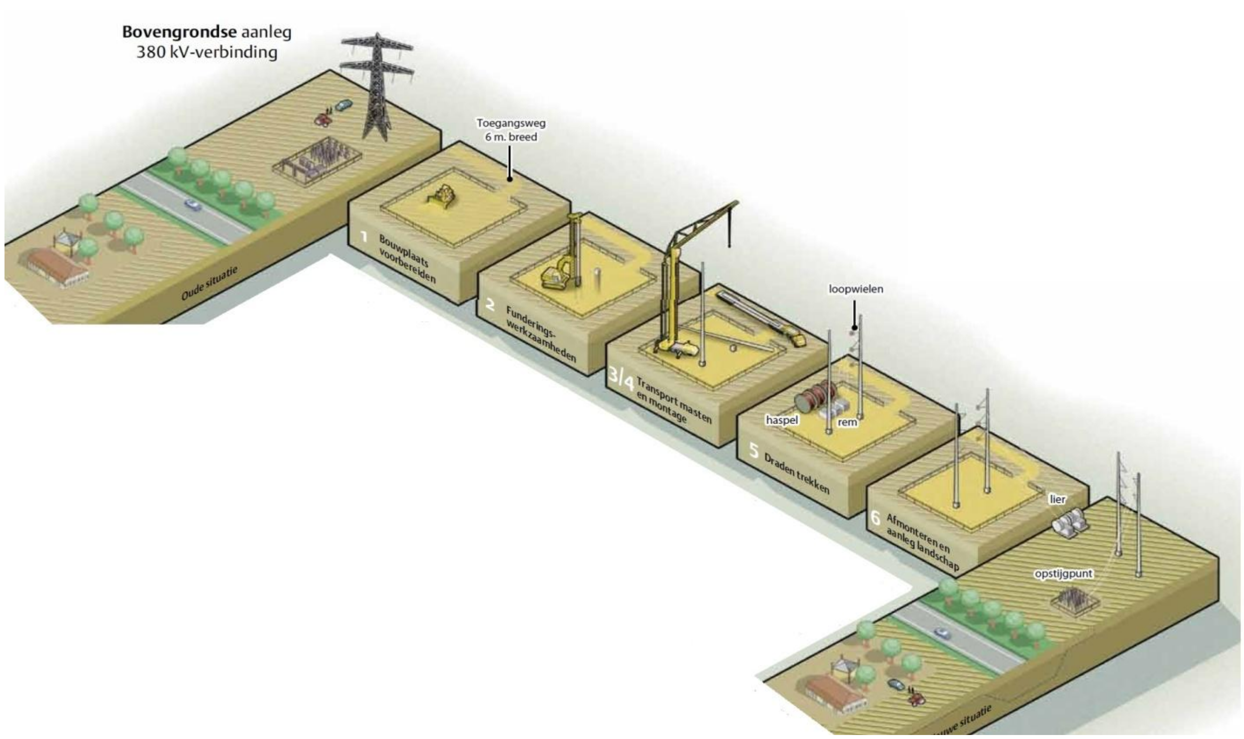


Figure 52: Schematized building process [XX]

Note: Some of the representations in this chapter are from previously executed wind turbines [XXIII][XXV][XXVII]. While they do not give an exact representation of the considered mast, they provide a sufficient indication of the actual process.

6.2 Building site preparation

Figure 53: Access road (L) & field research (R) [XX]

6.2.1 Access road

To get to the actual building site, access roads must be constructed. This is one of the first activities in the building process. These access roads are mostly built up out of granulate or sand. The use of this material, leaves the natural ground fairly unaffected. The thickness of the access road is around 40 to 80 cm, while the width is around 5 m, depending on the type of traffic that will use this road. In case very heavy machinery is utilized, concrete panels can be placed on top of the granulate layer (see Figure 53). The access road can be constructed in approximately a week. After the construction is finished, this road can easily be removed, without leaving any residue.

6.2.2 Work platform

At the mast location a work platform is constructed. The work platform is made of the same material as the access road. The length of the edges ranges from 50 to 80 m, occasionally even 80 to 100 m. It encompasses an area of about 3000 to 5000 m^2. Usually around this work platform, a fence is build to keep onlookers at a safe distance and only qualified workers can enter the premises.

6.3 Foundation activities

Depending on the type and size of the structure and the ground conditions, different types of foundations are possible. Foundations are commonly divided into two categories:

- Shallow foundations
- Piled (deep) foundations

Shallow foundations transfer the vertical loads to the upper layers of the ground, near the surface. They are usually embedded about a meter into the soil.

Deep foundations transfer the vertical loads through the upper layers of the ground to the stronger subsurface. They are usually embedded more than 10 m into the ground. The most common type of deep foundation is the piled foundation. When thick layers of soft soil are present, piling provides good results.

Before the choice of foundation is made and the form and dimensions are decided, the soil composition below the surface down to a depth of at least the effective width of the foundation, and in the case of a piled foundation, greater than the pile tip depth, must be known in sufficient detail. Geotechnical investigations are required to determine the necessary parameters for foundation design. These investigations must include all soil layers which influence the foundation strength [NEN-EN 50341-1].

6.3.1 Piling

As most of the masts will be applied in rural areas, where thick undisturbed layers of soft soil are present, piling is the best option. This takes about 1-2 weeks per mast location.

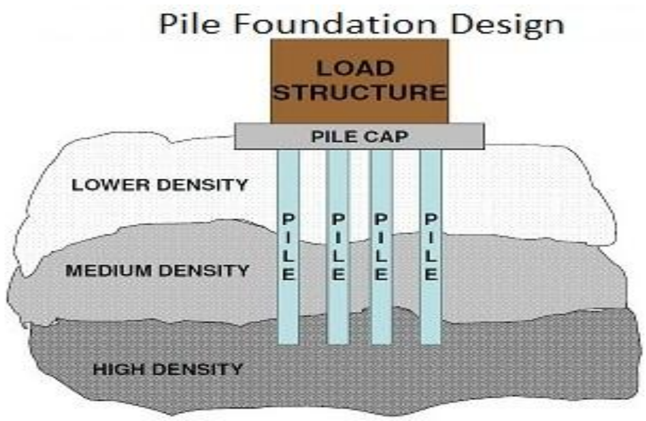


Figure 54: Schematic pile foundation [XXI]

Based on the material from which they are made, the following types of piles can be distinguished:

- Timber (treated or untreated)
- Concrete (cast in-situ or precast, reinforced or prestressed)
- Steel
- Composite

Factors influencing the choice of material are the type, size and weight of the structure, as well as the soil properties. For the pile form a wide variety of options exist as well (see Figure 55).

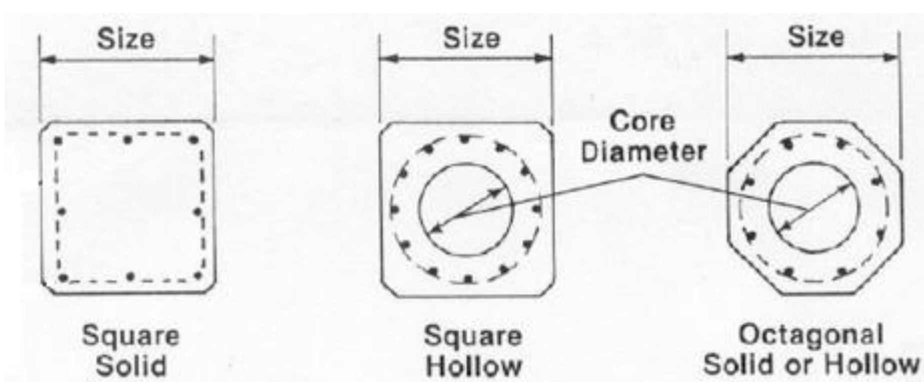


Figure 55: Different cross sections of piles [XXII]

Another important factor that can vary is the pile configuration. The pile configuration depends on the form of the structure and the form of the foundation used (see chapter 4).

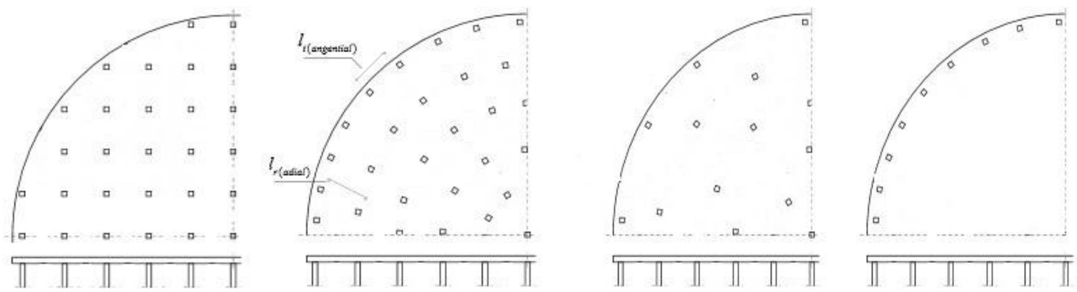


Figure 56: Pile configuration: Orthogonal vs. concentric (L) & distributed vs. at edge (R)

As was seen in chapter 4, because of the high acting moment and (in comparison) low axial load on the foundation, it is more practical to place the piles near the edges of the foundation, as the middle piles do not contribute to the moment resistance (see Figure 57).

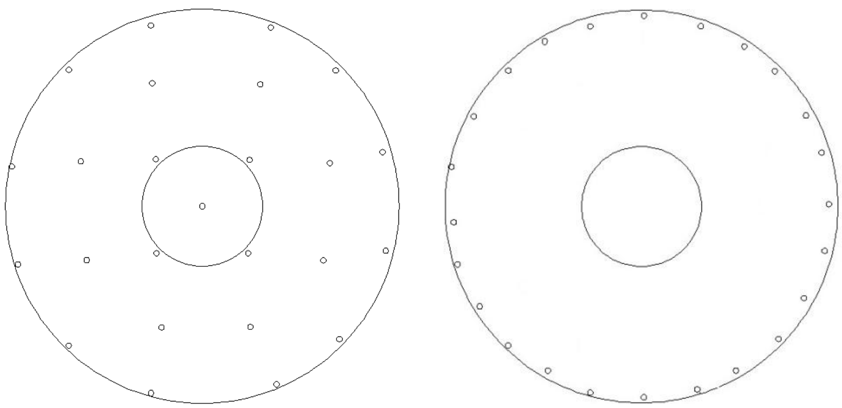


Figure 57: Applied pile configuration per mast: Initial (L) & reconfigured (R)

Execution

First a cone penetration test (“*sondering”*) is done at general mast locations, to determine the necessary pile length. Afterwards the exact location of all the masts are marked with numbered pickets. The pile locations are then marked with pickets as well (see Figure 58).

Figure 58: Marking of pile locations [XXIII]

Now the actual piling process can start. The chosen pile material will most likely be concrete (though steel is a reasonable option as well). It could either be reinforced cast in-situ concrete (vibro piles) or prestressed prefab. Variables to look at are the adhesion between pile and ground, tension in the piles, crack formation, etc. As this is not a governing aspect of the thesis, it will not be discussed any further. In Figure 59, a schematic overview of the piling process is given.

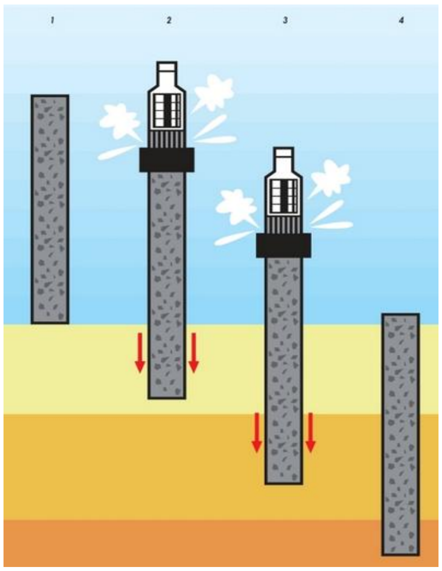


Figure 59: Piling process [XXIV]

6.3.2 Foundation block

When a piled foundation is utilized, the base foundation slab is supported by a number of point supports. The main design aspect is the punching shear resistance. The dimensioning of the slab is heavily depended on this shear force (induced by pile heads), sometimes more so than the working normal loads and moments. When the punching shear resistance is unsatisfactory, local thickening of the slab is possible. Usually the piled foundation is assumed to be rigid in the force distribution analysis, although in reality it behaves more like a slab on elastic point supports. The entire foundation process takes about 3-5 weeks.

As mentioned in the previous paragraph, the pile configuration depends on the form of the foundation. There are a number of possible foundation blocks to be considered. The four main possibilities considered are (see Figure 60):

1. Circular or oval foundation (per mast);
2. Ellipse foundation (for both masts)
3. Square or rectangular foundation (per mast);
4. Rectangular foundation for both masts (for both masts).

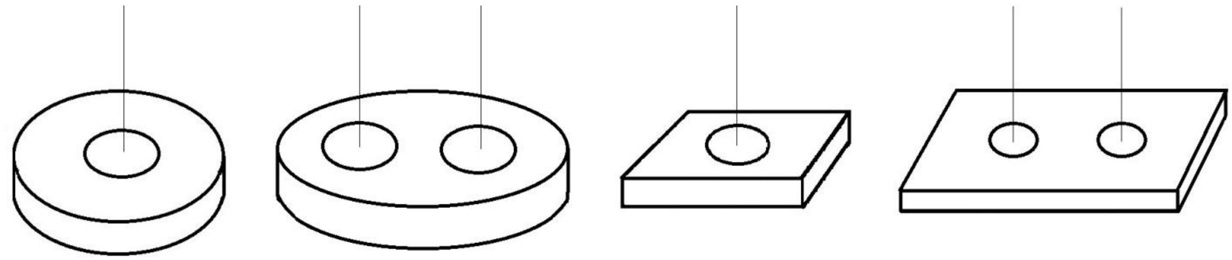


Figure 60: Foundation block options: circular (L), ellipse (ML), square (MR) & rectangular (R)

The results from the preliminary design, indicated that applying one foundation block for both masts is not as effective as applying separate foundation blocks per mast. Therefore the application of one foundation per mast is recommended. With regards to the form, circular forms are superior when the acting loads are equal in all directions. In the case of the mast, the acting moment in the main direction (orthogonal to the conductors), is about 5 times larger than in the secondary direction (parallel to the conductors). Thus it seems rational to conclude that a rectangular or oval form would be best for the foundation block. However as the circular mast form is a boundary condition (due to cable break being a sensitive subject), it seems practical to apply this boundary condition to the foundation as well. Therefore a circular foundation will be applied.

Execution

After the piling process is finished, a crane arrives to excavate the ground about 2 m deep. The ground floor is then prepared for the impending foundation. The visible part of the piles are subsequently stripped of the surrounding concrete, leaving only the reinforcement protruding from the ground (see Figure 61). This way the reinforcement of the piles can be integrated with the foundation block. Now preparations for the actual foundation are prepared. Reinforcement is placed all over the work floor (see Figure 62).

Figure 61: Stripping of concrete at the pile tops [XXV]

Figure 62: Placing of the reinforcement [XXV]

In the middle usually a plinth ("opstort") is placed that will end up about 30 cm above ground level. This is done to reduce the damage done by eventual collisions by e.g. tractors or other agricultural machines. This way the safety of the mast is ensured by letting the foundation take all the potential damage. Of course this does not prevent major damage, such as a truck driving in the mast at full speed, or an explosion. But the chances of such an event occurring are incredibly low and thus does not need to be taken into account.

When all the reinforcement has been placed, moulds are placed at the outside of the foundation block and the concrete is then poured (alternatively the mould can be applied first and then the reinforcement is placed inside).

Figure 63: Moulds on the outside [XXV]

Figure 64: Higher middle part [XXIII]

Finally the foundation is covered up with the previously excavated ground, with only the plinth sticking out from the ground.


Figure 65: Foundation with middle part sticking out [XXIII]

6.4 Transport

Transport encompasses the loading of the vehicle at the production factory, the transportation from the production factory to the building site and the unloading of the vehicle at the building site.

The type of transport utilized mainly depends on the length and weight of the segments. There is a large variety of transport available, ranging from simple trucks to specialized helicopters. A good alternative for transporting the mast segments are the standard long trucks (also known as B-trains or Ecocombies). These Ecocombies are capable of transporting a wide variety of products. They have lower transport costs compared to normal truck combinations as well as a lower environmental impact. Since 2012 these Ecocombies are allowed in the Netherlands and they have shown good results so far [XXVI]. In Europe the trucking combinations are allowed a maximum length of 18.75 m (carriage values) and weight of 40 ton (including the truck weight), though national exceptions are allowed. Thus the mast segment's length and weight will be limited by these values.

To ensure minimal hindrance to normal traffic, the segments will usually be transported in the night/early morning. Usually they will arrive at the building site at about 6-7 o’clock. For every site location, a minimum of 8 trucks will be used (2 masts x 4 segments). Of course it is unproductive for the trucks to simply be left waiting on the building site or to store the segments for a long duration on the building site. Therefore systematic planning of logistics is imperative.

Figure 66: Transport of segment (wide and short) [XXIII]

Figure 67: Transport of segment (narrow and long) [XX]

6.5 Assembly of mast

Usually a day before the segments are scheduled to arrive at location, the cranes are assembled at the building site. A main crane, as well as a support crane is utilized. When the segments arrive, both the main and the support crane lift the segment horizontally from the truck (see Figure 68). The main crane than slowly rises, so the segment is put in a vertical position. At this point the support crane can be released. The segment is then lifted to its position, so that the mast can be assembled. When the segments arrive, the cranes hoist them directly from the trucks and lift them to their position, so that the mast can be assembled (see Figure 69).

Figure 68: Lifting of the segments from the truck [XXVII]

Figure 69: Assembly of the mast

6.5.1 Segments

As previously mentioned, the segment length is limited by transport restrictions. Aside from these limitations, other restrictions such as the occurring stresses during transport and lifting, as well as the maximum capacity of the chosen lifting crane should be considered. These stresses are not allowed to cause cracks and are thus limited to the tensile stress of the applied concrete. Below an overview of the segment sizes and the occurring stresses during transport for HSC and UHSC.

HSC	Segment length [m]	Weight [ton]	Stress [N/mm^2]
0 – 7	7	15.6	0.32
7 – 15	8	15.7	0.48
15 – 24	9	15.1	0.72
24 – 35	11	14.7	1.37
35 – 47	12	11.2	2.38
47 – 57	10	5.7	2.84

Table 30: HSC segments

UHSC	Segment length [m]	Weight [ton]	Stress [N/mm^2]
0 – 13	13	14.8	1.13
13 – 29	16	14.1	2.24
29 – 44	15	9.2	2.89
44 – 57	13	4.7	3.77

Table 31: UHSC segments

As can be seen from Table 30 & Table 31, all HSC and UHSC segments are well below the length restriction of 18.75 m. This means that for both concrete classes, the weight (G = 19.3 ton) and stress restrictions ($f_{ct,HSC}$ = 3.33 N/mm^2, $f_{ct,UHSC}$ = 6.85 N/mm^2) are governing. The segments were designed taking into account the most unfavorable stress occurrence.

Aside from all the above, the segments were checked for one additional aspect, namely if instead of whole ring segments, it was possible to more efficiently utilize half ring segments. This would mean that there is an additional vertical connection in the structure, but on the other hand it might reduce the total amount of segments needed. Thus it was checked to see whether a good balance could be found. The results can be seen in Table 32. For an overview see appendix J & K.

6.5.2 Prestressing

Background

Concrete prestressing is defined as the application of an compressive force to a concrete element, with the purpose of introducing an initial compressive force in members that are required to carry tensile stresses, under working load conditions. There are several prestressing methods based on different premises. Based on the stage at which the prestress is applied, two categories can be distinguished:

- Pre-tensioning (before the concrete has set);
- Post-tensioning (after the concrete has set).

Pre-tensioning is the application of a tensile force via high tensile steel tendons before casting. This tensile force in the tendons, will ultimately introduce a compressive force in the concrete element. First the tensile force is applied to the prestressing tendons, before the concrete is cast. Then the concrete is cast around the prestressing tendons. When the concrete has matured and has developed sufficient compressive strength, the tendons are released thus imparting a compressive stress to the concrete member. For the concrete element to be in a permanent state of prestress, the tensile force in the tendons must be carefully maintained before concrete casting. The tensile force is introduced in the prestressing steel trough one of the following methods:

- Abutment method: an anchor block is cast in the ground;
- Strut method: the bed is designed in such a way, that when tensioning forces are applied it acts as a strut without deformation;
- Mould method : tensioning forces are resisted by strong steel moulds.

After the concrete has been cast, it is usually cured immediately so that the required strength and bond between the concrete and steel can develop in 8-20 hours. After the necessary strength has been achieved, the steel tendons can be released from the ends and the units are cut to the specified length. The prestressing force is introduced to the concrete by bond**.**

Post-tensioning is the application of a tensile force via high tensile steel tendons after the concrete has been cast. This applied tensile force in the tendons, will introduce a compressive force in the already cast concrete element. The method is based on the direct longitudinal tensioning of steel tendons from either one or both ends of the concrete element. First the ducts for the steel tendons are positioned along with the traditional reinforcement before concrete casting. Then the concrete is poured and starts to harden. After the concrete has developed sufficient strength, the tendons are placed in the ducts and the prestressing is introduced by tensioning the tendons with the help of mechanical (hydraulic) jacks, specifically designed for post-tensioning. The prestressed tendons are then anchored (locked) with mechanical anchors. The ducts prevent contact between the concrete and the steel tendons, thus eliminating the bond that would have occurred if pre-tensioning was used. To still allow for bond between the concrete and the steel, the ducts are thoroughly grouted with cement grout. Unlike pre-tensioning the prestressing force is not transferred via bond, but via the anchorage at the ends.

General Advantages

Prestressing can be utilized to achieve a wide variety of advantages over traditional reinforced concrete. Some of these advantages are listed below [XXIX]:

- Effect of concrete cracking is minimized because of the induced compression;
- Reduced beam depths are possible for equivalent design strengths;
- Lighter elements allow for longer spans with a higher strength-to-weight ratio (as well as a high span-to-depth ratio);
- Prestressed concrete is more durable than reinforced concrete and can recover from the effects caused by overloading;
- If cracks occur due to overloading, they can close upon removal of the overload;
- Prestressing enables a higher degree of precasting (prefab);
- Deflections are more easily controlled resulting in the possibility of longer spans;
- Prestressing enables a more efficient use of traditional reinforcement;
- In terms of total costs prestressed concrete can be more attractive.

Another important distinction that can be made is between internal and external prestressing. This classification is based on the location of the prestressing tendon with respect to the concrete section.

Internal prestressing is the most commonly known and used method of prestressing. For this method the prestressing tendons are simply placed inside of the concrete member either with or without ducts (embedded tendons). Most prestressed structures utilize internal prestressing. Some advantages are listed below:

- Method is simple and well-known;
- Tendons do not need additional protection from corrosion if grouting is done properly;
- Tendons can be draped in smooth curves, thus balancing the applied loads better;

External prestressing is a recent development that is quickly gaining popularity because of its fast construction and minimal disruption to traffic flows. Unlike internal prestressing, where the tendons are placed inside the concrete, external prestressing is realized by placing the tendons outside of the concrete section. The tendons can lie outside the concrete member (beams, walls) or inside the hollow space of the member (e.g. box girders, hollow columns). Currently this technique is mostly applied in the bridge industry and for strengthening of buildings [XXX]. Because the prestressing tendons are placed outside, there is no continuous bond between the concrete and the steel. To still allow for interaction between the concrete and the steel, end anchorages, deviators and saddles are used, which transfer the prestressing force to the concrete. Some advantages are listed below:

- Due to the absence of bond between concrete and steel, the external prestressing tendons can be removed or replaced one or two at a time so that the structure can be repaired or the capacity can be increased;
- Due to absence of ducts the concrete can be poured more easily;
- Allows for easier access to the anchorages as well as inspection in general;
- Enables optimal control or modification of the prestressing force;
- More freedom in design of the structure shape as well as thinner structures;
- Friction losses are reduced (no wobble).

External prestressing is an interesting alternative, because it allows for more durable structures with a wide variety of other advantages. However as it is still relatively new and few applications exist, the cost is very high (approximately three times higher than internal prestressing) [38]. Because of this high cost it is necessary to design simple constructions elements with as little prestressing steel as possible. Globally external prestressing has proven to be very cost effective and technically attractive if used efficiently, as it allows for large cost savings due to faster construction.

Previous research has shown that for shallow cross sections (less than 3.0 m), the internal tendon alternative will lead to a smaller amount of reinforcement, both traditional and prestressed. For deeper cross sections the external tendons are the better option [1].

The choice of using either internal or external prestressing has an influence on the design aspect as well, namely on the variation of tendon eccentricity. The deflection shape of the external tendons does not always coincide with the concrete member deflection (e.g. beam), because the displacement of the external tendons is determined by the deviators. Because of these different deflection shapes, a second order effect can occur which is important for the SLS but even more so for the ULS [XXXI].

From previous experience, it has been found that the amount of prestressing necessary, should not be used as a criteria for the choice between internal or external prestressing. Instead, the selection should be based on criteria related to the advantages and disadvantages of the two kinds of prestressing.

Prestressing type

For normal internally prestressed structures, such as box girders, the thickness tends to be at least 500 mm. In the case of the mast however, the biggest wall thickness is 420 mm for OSC. Therefore applying internal prestressing is quite difficult, because it would require applying many small tendons, which is very labor intensive. For UHSC with a wall thickness of 75 mm, it is nearly impossible. Therefore, external prestressing is the best alternative.

After the choice for external prestressing tendons has been made, it is obvious that pre-tensioning is not an option anymore, as that method requires the concrete to be cast around the tendons. Thus post-tensioning must be applied. The only remaining problem is how exactly these external tendons can be applied with the post-tensioning method. This will be described briefly in the next paragraph.

Execution

Methods of prestressing [XXXII]

- *Mechanical jacking of tendon*
  This is the most simple and well-known type of prestressing, enabling considerable prestressing forces. For the tensioning of the steel tendons, a hydraulic jack is used, which is comprised of calibrated pressure gauges which directly illustrate the magnitude of force developed during the tensioning;
- *Thermal or electrical prestressing*
  This method of prestressing utilizes electric heat to prestress steel wires, after which the tendons are anchored and the concrete is cast;
- *Pre-bending high strength steel beam* and encasing its tensile flange with concrete;
- *Chemical prestressing* is realized by using expansive cement which expands chemically after setting and during hardening. It is also known as self-stressing.

As mentioned previously, it is difficult to apply the prestressing in the foundation. Usually to achieve anchorage of the tendons in the foundation, an unstressed anchor is placed in the foundation (“blind anchor”). After the concrete has been cast, the anchor is now anchored to the cast in-situ concrete, but still unstressed. The tendon is then mechanically stressed from the top and anchored in the mast as well at key positions.

One method of executing the prestressing is to install the prestressing tendons before the segments are placed. This implies that the segments will have to be lowered over the present tendons. This can be achieved by utilizing either the support crane or falsework, to support the prestressing in the air. The tendons can then be kept in one place in the air, while the segments are lowered over the tendons. As more segments are lowered over the prestressing, eventually the falsework or the support crane can be removed, as most of the tendons will be anchored by then and the remaining ones will be supported by the segments. However this method is very difficult to apply in practice for prestressing bars and nearly impossible for prestressing cables. Therefore this method is ruled out.

A much more viable alternative is to apply the prestressing tendons, after each segment is placed. The tendon can then be fixed to the blind anchor with the help of couplers. This way, after each segment is placed, a number of tendons can be placed and fixed to the foundation. This method is possible due to the large maintenance space available at the foot of the mast (see Figure 70).

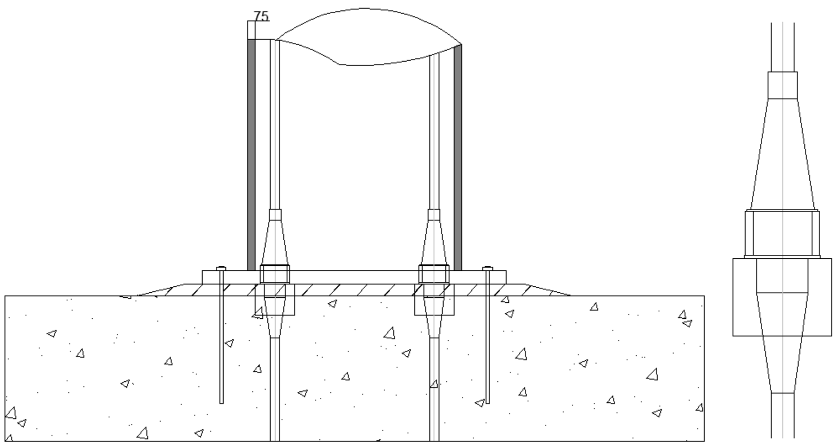


Figure 70: Prestressing in foundation (L) & couplers detail (R)

Anchorage locations

From the design calculations it was found that 8 prestressing tendons are necessary. The governing section that was considered was the bottom of the mast. This does not mean that it is necessary to apply the 8 tendons over the whole length of the mast. In fact the top segment does not need prestressing at all. So a practical prestressing configuration has to be applied. Ideally this configuration will utilize the already available horizontal connections as anchorage locations. Figure 71 shows the different positions for anchorage of the tendons of each segment, while Figure 72 shows the alignment of the prestressing tendons.

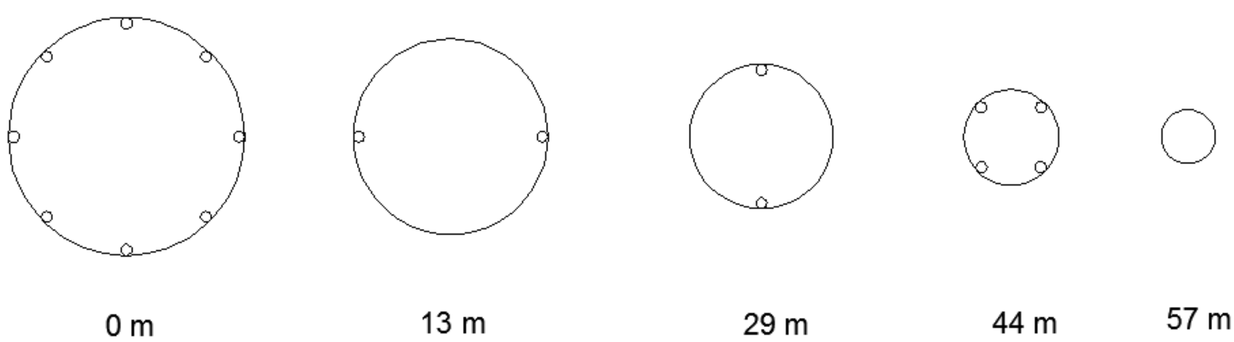


Figure 71: Prestressing anchorage locations for UHSC

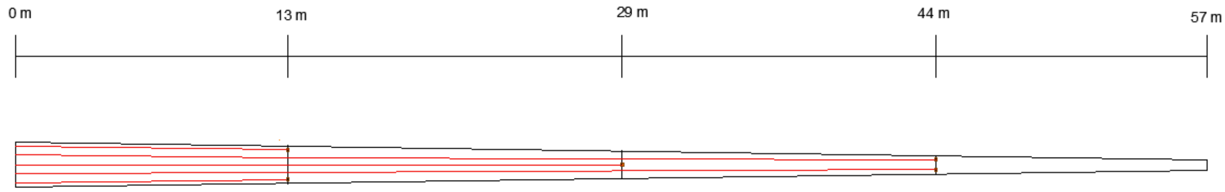


Figure 72: Alignment of the prestressing tendons for UHSC (side view)

As the mast assembly progresses, the prestressing will be increasingly difficult to apply. Reason for this is as the mast height increases, the internal diameter decreases. In case of the UHSC segments, the prestressing will thus be most difficult to apply at a height off 44 m, where the internal diameter of the mast is 888 mm. For workers to freely maneuver inside the mast, an area of 0.50x0.50 m^2 is needed (see Figure 73). This leaves a length of 388/2 =194 mm for the connections on both sides (including the wall thickness). The connection width is limited to 734 mm for both sides (including wall thickness).

Note: The top segment does not have any prestressing at all. This has the additional advantage that lightning strikes, at the top of the mast, are less likely to affect the prestressing cables. The chances of tendon failure due to lightning is thus significantly decreased.

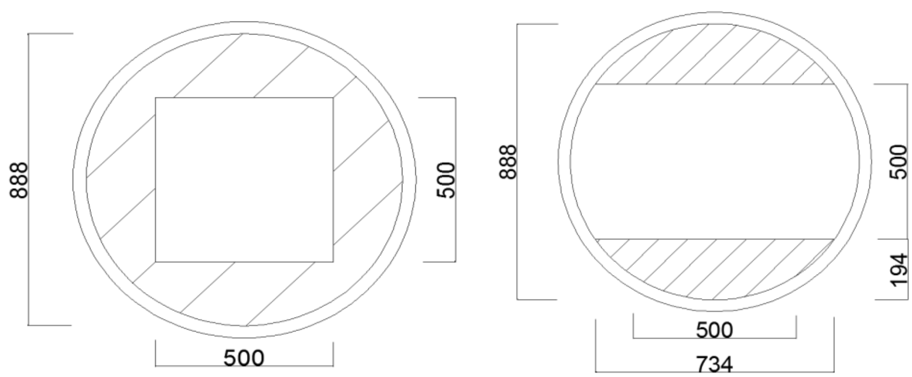


Figure 73: Free space inside mast (L) & connection limitations

To prestress the first and the second segment a large jack called type III (ZPE-7A) is used. This jack is lifted using the same crane used for the segments. Because the third segment has less internal space than the previous segments, this jack might be too large to utilize if the workers do not have previous experience with it. In that case a type I (ZPE-23FJ) jack can be used for the last two prestressing tendons. The type I jack can only stress one strand at a time, which means that for the two tendons it will have to be used 14 times. Therefore the type III jack is heavily preferred.

6.5.3 Connections

Connections or joints are one of the most important aspects of a structure, as they are basically responsible for keeping all of the loose segments together. They must also be able to transfer all forces and moments from one segment to the other and must be able to resist imposed deformations.

Before the connections are looked at in more detail, first a consideration is made whether whole or half segments should be used (see Figure 74). There are a whole lot of advantages and disadvantages with utilizing each option, such as whole segments might be too heavy to be transported or by utilizing half segments less horizontal connections will be necessary.

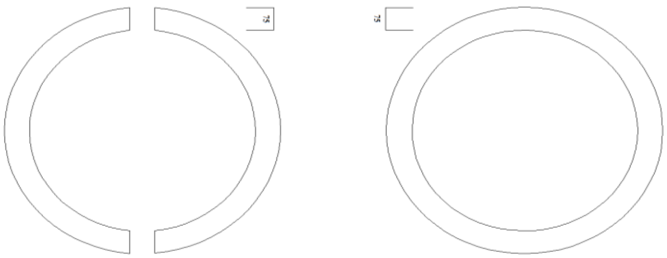


Figure 74: Whole (L) vs. half (R) segments

The results of this comparison can be seen in Table 32 & Table 33.

Whole segments	**OSC**	**HSC**	**UHSC**
Number of segments	12	6	4
Number of horizontal connections	11	5	3
Number of vertical connections	0	0	0

Table 32: Number of connections with whole segments

Whole + half segments	**OSC**	**HSC**	**UHSC**
Number of whole segments	8	4	3
Number of half segments	4	2	2
Number of horizontal connections	9	4	3
Number of vertical connections	2	1	1

Table 33: Number of connections with whole and half segments

It can clearly be seen, that the introduction of half segments, does not reduce the total amount of segments (in fact for UHSC it increases), but it does reduce the amount of necessary horizontal connections (stays constant for UHSC). This is an attractive prospect, because the horizontal connections are more expensive than the vertical connections, because they have to resist the large occurring moments. In comparison the vertical connection only has to resist a low shear force. Of course a consideration must be made, whether the saved material costs are worth the labor costs for the extra connection. However it is obvious that this consideration only needs to be made for OSC & HSC, as the introduction of half segment for UHSC has no benefits.

For the mast there are 3 definite connections that need to be worked out:

1. Foundation-column connection
2. Segment-segment connection
    - Horizontal connection
    - Vertical connection
3. Column-isolator connection

6.5.3.1 Foundation-column connection

There are a number of possible column-foundation connections for prefab systems. However most of these solutions are for solid columns. For concrete tube-foundation connections, more specialized connections must be utilized. In this paragraph some of the basic possible connections for solid column connections will be discussed, how they can be modified for tubular columns, as well as solutions specific for the tubular columns.

Pocket foundations

A pocket foundation can be seen as a box without a top and the bottom being comprised of the base foundation slab. The column is inserted in this “pocket” and is restrained by the four surrounding walls. Instead of having direct contact with the base slab it is usually placed on a bearing pad. After the column has been placed in the correct position, with the help of temporary bracings, the space between the bottom slab and the column, as well as the gaps between the column and walls, are filled with no-shrinkage mortar. To ensure proper compacting of the mortar, the pocket must be spacious enough around the column. To increase the bond between the mortar and the column and pocket, the surfaces of both can be made course or indented. At the bottom side of the pocket there are protruding stirrups, which accomplish the connection with the base foundation slab. Common pocket foundation cross sections range from 700x700 mm to 1400x1400 mm.

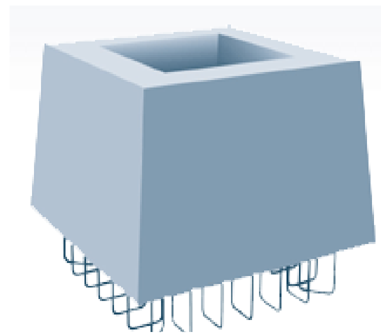


Figure 75: Pocket foundation [[XXVIII]]

From the range of cross sections utilized for the pocket foundation, it can already be seen that this type of connection is mostly unsuitable for the tube mast. However a variation, in which a sort of “double” pocket foundation is utilized, could provide some merit (see Figure 76). This double pocket foundation would essentially be the same as the normal pocket foundation, with the only difference being an extra wall around the initial pocket foundation. The tube mast can then be lowered between these two walls. The initial pocket foundation can then either be left as seen in Figure 77L, or a simple block could be used (though this would mean more use of material).

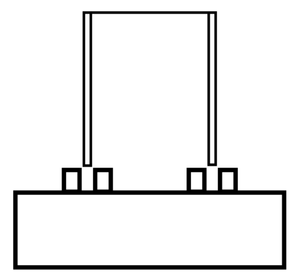


Figure 76: Side view of double pocket foundation

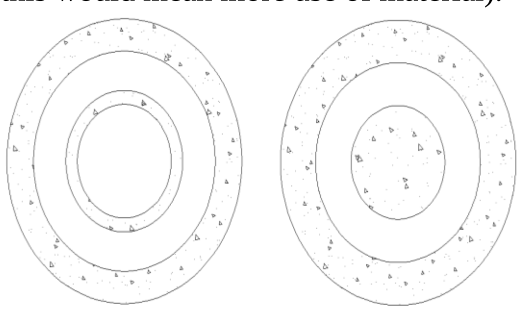
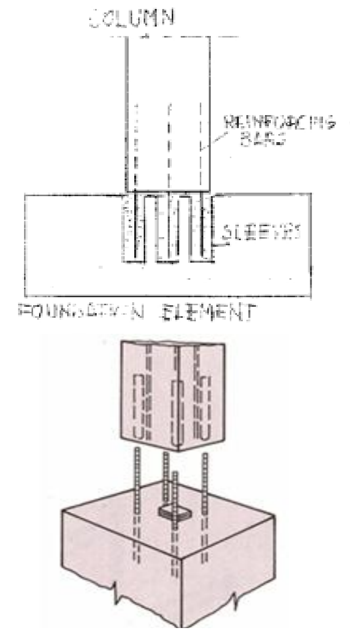


Figure 77: Top view of double pocket foundation: Hollow internal block (L) & massive internal block (R)



Foundations with protruding bars

This foundation is established by inserting corrugated sleeves in the foundation. The longitudinal reinforcement bars (starter bars) from the column are then placed in the sleeves, which are then filled with no-shrinkage mortar. The column itself rests on the bed of mortar, that is created by filling the sleeves and the surrounding column area with mortar (see Figure 78T). To prevent pulling out of the longitudinal reinforcement, careful attention must be paid to the anchorage of the bars in the column. Alternatively the bars can protrude from the foundation and can be inserted in sleeves in the column. The mortar is then injected into special openings in the columns.

For the tube mast again this method seems inconvenient. While this method is a very successful method for massive columns, it would be very difficult and time consuming for a tubular column.

Figure 78: Foundations with protruding bars: Normal (top) & reverse (bottom) [39]

Foundations with bolted steel plate

This solution is an adapted alternative from the steel industry. From the base foundation slab, protruding anchor bolts with threaded ends are prepared (see Figure 79). Alternatively the anchor bolts can be placed in prepared sleeves, after which they are filled with mortar (see Figure 80). These anchor bolts fit directly in the bolt holes in the steel plate attached to the column. The bolts are then screwed loosely with nuts forming a pinned base connection. The steel plate can be indirectly or directly jointed to the longitudinal reinforcement bars of the columns, by welding or anchoring during manufacturing.

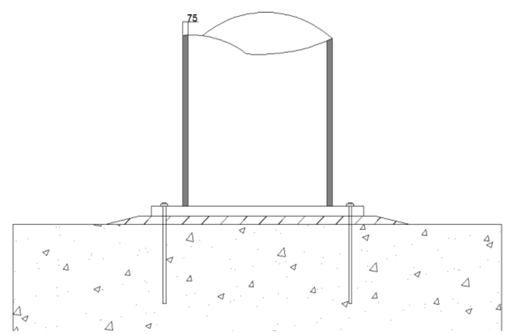


Figure 79: Foundations with bolted steel plate

Another option is to cast-in grade 8.8 threaded bars and bolt the steel plate to the column. After positioning of the column, the gap between the steel plate and the base slab, is filled with no-shrinkage mortar. The process is completed with the tightening of the nuts.

These options seem to be quite possible for the tube mast. The main difficulty of this solution is the attachment of the steel plate to the bottom of the segment. The fact that the tube mast is hollow has no real influence on the solution. While for massive columns the bolts are applied on the outside, for the tubular mast they could be applied both inside and outside of the mast, which could be advantageous visually.

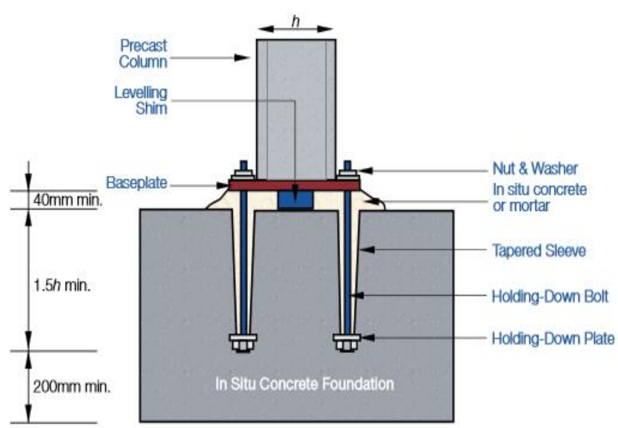


Figure 80: Foundation with bolted steel plate (alternative) [39]

Instead of the usually applied bolts at the edge of the steel plate, it would be more convenient to apply the bolts in a circle around the mast. Additionally these bolts must be preloaded. Whenever steel is used for the connection, the connecting area must be protected against fire and corrosion.

Foundations with bolts
This solution is essentially the same as the previous solution, with the exception that the steel plate is omitted. Instead the concrete tube has protrusions or notches at the bottom of the segment, which can be bolted in the same manner as before. Of course the local loads on these notches must be analyzed very carefully to prevent local failure.

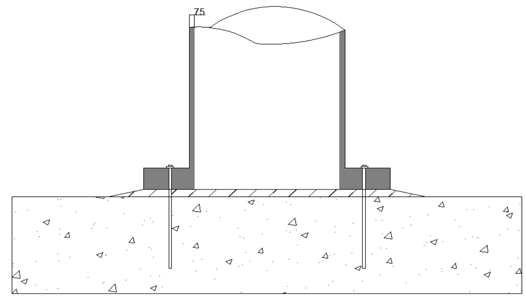


Figure 81: Bolted foundation

Prestressed foundation
By means of prestressing, a connection can be established between the foundation and column. In theory this is a very attractive solution. However, in practice the construction faces complex difficulties due to the following reasons:

- The execution of the joint is incredibly difficult;
- Detailing of the joint is very complicated
  - Anchorage elements usually have to be installed in the underlying foundation and are not easily accessible;
  - Anchorage elements have to be installed at the top of the column as well and will thus hinder the column-column joint.
- The prestressing is applied against the mast wall instead of in the middle.

As previously seen, prestressing will be applied in the mast to eliminate the tensile stresses. However this does not mean that a prestressing connection is automatically achieved. The main goal of the tendons is not to provide an adequate column-foundation connection, as they have not been applied with that purpose. Therefore it is not really possible to speak of a prestressed connection. Of course a possible solution could eventually be researched, how to combine both of these functions (eliminate tensile stresses and provide adequate connection).

6.5.3.2 Segment-segment connection

When connecting structural elements a wide variety of parameters have to be considered. The main parameters are load transmission, deformations, assembling & disassembling. These parameters result in the following basic requirements [40]:

- Force distribution has to be as simple as possible;
- The least amount of joints should be used;
- The joint should be functional in every occurring condition;
- There must be a possibility to adjust for construction tolerances.

The most important condition for joints between UHSC segments, is that the load bearing capacity of the structure, is not determined by the load bearing capacity of the joints, as this would strongly reduce the proper use of the material or require extensive strengthening of the structure at the joints. Another factor to consider is that the prefabricated parts are often exposed to treatment temperature, while temperature treatment of the in-situ joints is very difficult and expensive. Additional requirements include the liquid tightness of the joint, no temperature treatment, preferably no reinforcing bars due to their large dimensions and the stress concentrations, and the free form in plan [40].

Horizontal connection

Concrete column-column connections are normally fairly easy to make. However because of the mast's tubular form, the process becomes more complex. In this paragraph a number of possible column-column connections for prefab systems are summarized and explained.

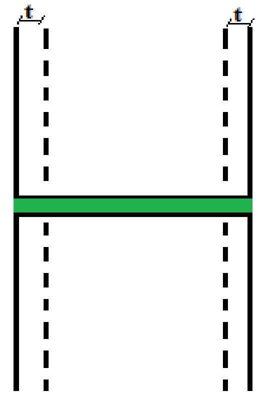


Figure 82: Glued connection

Glued connection

Glued connections are only applied for very thin joints between wide elements. This means that dimensional deviations are almost impossible to deal with. That's why glued connections can only be applied in structures where the segments are produced in a good position relative to one another. They can then be assembled in the same way during execution, allowing the glue to only handle small deviations. The segments are then pressed together by the own weight of the structure as well as the prestressing. Some disadvantages are that the continuous adhesion is difficult to guarantee, constructions tolerances are complex and it can only be used for a small temperature range.

When a glued connection is utilized, the structure acts like a homogenous body. This allows the compressive forces and the moments to be transferred from segment to segment without a problem. The shear force however is a bit more troublesome. Usually notches ("schuifnokken") are used to solve this problem.

The glued connection seems like a very attractive solution, but it has the disadvantage that the production of the segments must be very controlled. The best way to prevent dimensional deviations from occurring is by using contra moulds. However as the mast height is almost sixty meters, this is not a very adequate alternative.

Connection with protruding bars

This is exactly the same as the previously discussed foundation-column connection, with the only difference that the foundation is replaced with the top of a column. Just as for the foundation connection, this connection is incredibly difficult and time consuming to apply for concrete tubular columns.

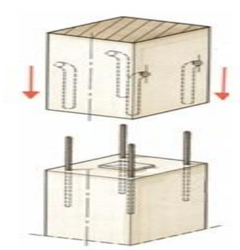


Figure 83: Column-column connection with protruding bars

Bolted connection

A bolted connection is established by applying the columns in the form of a console (corbel). The "consoles" are then bolted to each other through high strength, preloaded bolts. These bolts will be preloaded as well to prevent loosening of the connection. The advantage of this method is that the consoles can be used as the deviators for the prestressing as well.

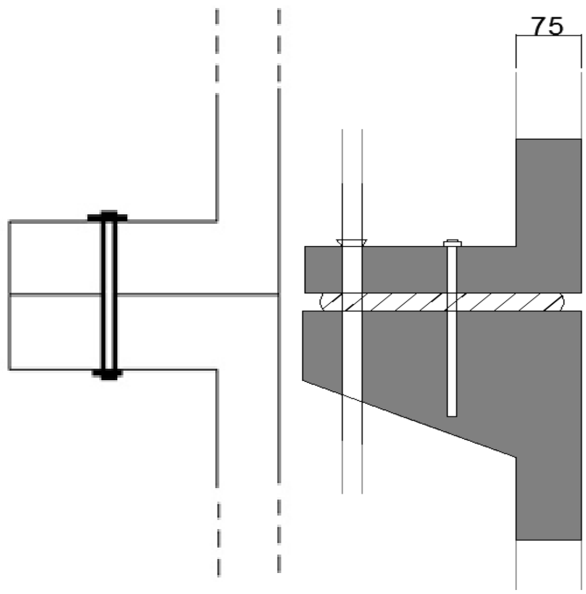


Figure 84: Horizontal segment-segment connection with bolts: Simple (L) & optimized (R)

An important aspect to take into account, is that only the local prestressing can be considered for the compressive force in the connection. The normal axial force and global prestressing are taken by the edges of the tube (see Figure 85). For that reason, the forces in the connection can only be taken by the bolts and the locally applied prestressing. Local reinforcement will almost certainly need to be applied, to compensate for the forces induced by the local prestressing.

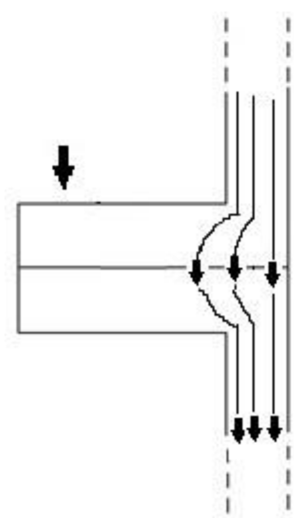


Figure 85: Force distribution in horizontal connection

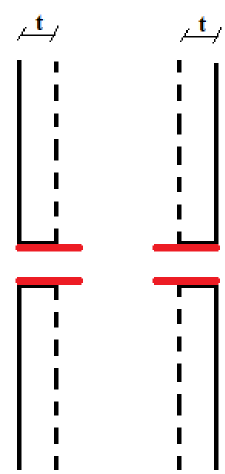


Figure 86: Bolted steel plate connection

Connection with bolted steel plate

This solution is similar to the previously seen column-foundation solution. The steel plates are welded to the top and bottom of the segments. The segments can then simple be placed on each other, after which the steel plates are bolted together.

A different solution with the steel plate can be achieved by utilizing curved steel plates around the mast. This curved plate can then be bolted on the outside on two sides (see Figure 87). Mortar can be applied between the segments to resist the compression forces, while the steel plate resists the shear. Of course the major disadvantage of this solution is that the connection can be seen from the outside. To rectify this, a possible solution could be thought of, where the steel plate is applied on the inside instead. However this will be quite complicated, as inside already the prestressing, climbing facilities and space for maintenance is necessary.

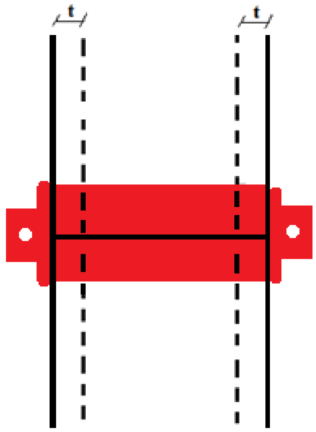


Figure 87: Curved steel plate connection

<ins>Prestressing connection</ins>
This is exactly the same as the previously discussed foundation-column connection, with the only difference being that the foundation is replaced with the top of a column. For column-column connections this solution is much easier in practice. However the main problem here as well is that the prestressing is applied at the the mast wall instead of in the middle of the structure. Connections by prestressing require a minimum thickness of the element, which in the case of UHSC is not attained. The prestressing can however provide a contribution to the shear resistance.

Vertical connection
For the vertical connection, a number of possible connections for prefab systems can be considered as well, such as the glued or toothed connection and the bolted and looped connection. However as seen at the start of this chapter, utilizing half segments does not yield better results for UHSC. Thus further elaboration was deemed redundant.

6.5.3.3 Column-isolator connection

Conductors
On each support mast, a total of six 380 kV conductors will be applied. Each conductor is supported by V-isolators, which are inserted into the mast at two positions. This means that per mast a total of 12 connections are made for the isolators. The top diagonal of the V-brace is the tension part and the bottom part is the compression part (see Figure 88).

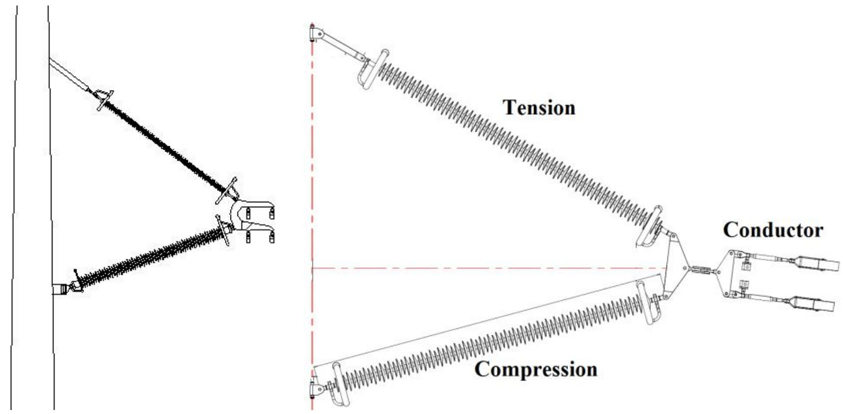


Figure 88: Column-isolator connection support mast

The V-isolators will be attached to the mast, via certain clips. These so called clips will have been already inserted in the segment during production. The necessary wall thickness, as well as the necessary local reinforcement should be calculated to ensure that the isolators do not cause local failure.

For the dead-end masts the isolators are a bit different. As can be seen from Figure 89, a small ring is attached to the mast, to which the isolators are then connected. The isolator can take both tension and bending.

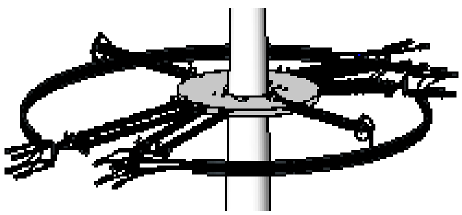


Figure 89: Column-isolator connection dead-end mast

Lightning wire
The lightning wire of each mast is placed on the outer side of the mast i.e. away from the other mast, compliant with the norms.

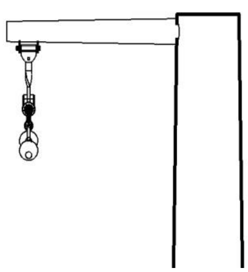


Figure 90: Column-lightning wire connection

Retour current conductor (passive loop)
The same principle applies here as for the lightning wire.

6.5.4 Equipment

In this paragraph a short overview of the material used during assembly is given.

Pile driver

The type of pile driver depends on the chosen pile type. In Figure 91 a commonly used pile driver for prefab piles can be seen.

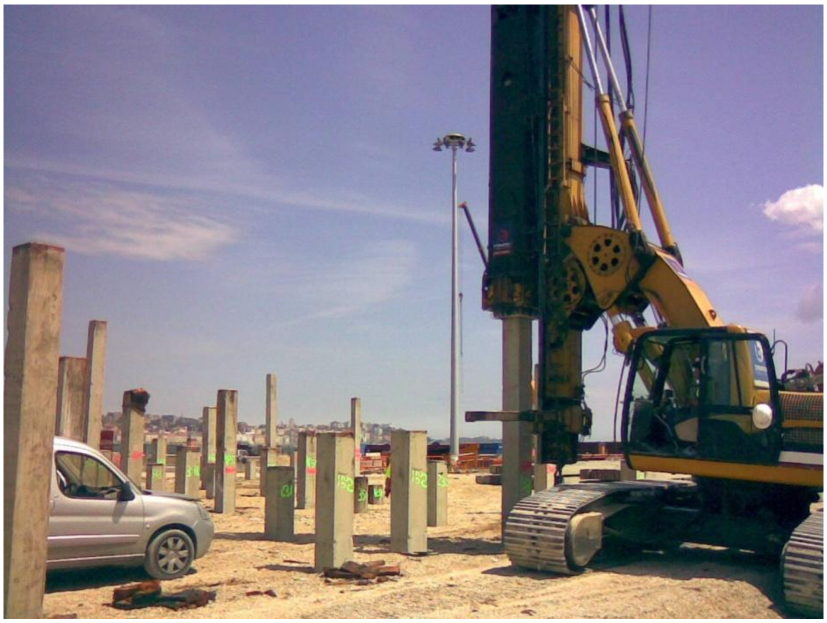


Figure 91: Pile driver [XXXIII]

Crane

The crane used for the lifting of the segments is a 70-tons crawler crane. Some specifications are given in Table 34.

	Value	Unit
Self weight	69	ton
Min. vertical clearance	3.56	m
Width extended	4.47	m
Width retracted	3.50	m
Length undercarriage	6.30	m
Swing radius ballast	4.83	m
Crawler width	0.87	m
Number of lift winches	2	-
Ground pressure	0.65	kg/cm^2
Max. height	62.0	m
Max .weight over 10 m	19.3	ton

Table 34: Crawler crane specifications (Kobelco Cranes Co.)

Figure 92: Crawler crane 70-ton (Kobleco Cranes Co.)

The segments will be lifted in a horizontal position with the crane and will be fastened at a preferred distance of 0.2L (with L=segment length) from each end if one main crane is used. After the designated height is reached, one side is carefully released and the segment is lowered vertically onto the segment below. If a support crane is used as well, the segments can simple be lifted from the truck at the segment ends by both cranes, before lifting the segment in the vertical position.

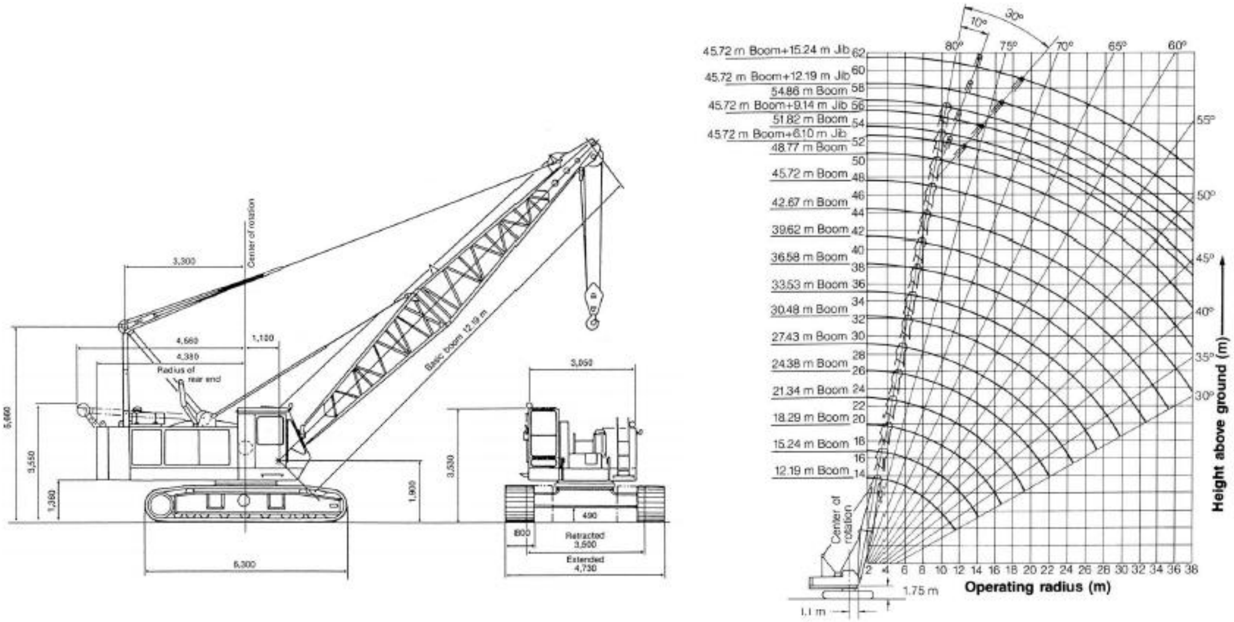


Figure 93: Schematic overview of the crane dimensions and reach

Prestressing jacks

Type III ZPE-7A	Value	Units
Length	690.12	mm
Diameter	279.91	mm
Pressure	52.30	N/mm^2
Weight	115.21	kg
Used for 13 mm (0.5”)	5-7	-

Table 35: Jack type ZPE-7A specifications [XXXIV]

Figure 94: Jack type III ZPE-500[7] [XXXIV]

Type I ZPE-23FJ	Value	Units
Length	798.94	mm
Diameter	116.08	mm
Pressure	48.80	N/mm^2
Weight	23.13	kg
Used for 13 mm (0.5”)	5-1	-

Table 36: Jack type ZPE-23FJ specifications [XXXIV]

Figure 95: Monojack type I (ZPE-23FJ) [XXXIV]

Reels (including braking machine)

As mentioned before, for the montage of the conductors and other cables, reels are utilized. These reels can be skid, trailer or truck mounted. These reels can hold more than 800 m of cable. Figure 96 shows two possibilities for reels.

[7] The picture shown is of type III ZPE-500 instead of the chosen type III ZPE-7a. However the two jacks look similar so the ZPE-500 provides a good impression..

Figure 96: Reels for the conductors. Skid mounted (L) and truck mounted (R) [XX, XXXV]

6.6 Assembly of conductors and other cables

6.6.1 Conductors

The installation of the conductors is more complicated than one might imagine. Once a number of masts have been installed (5-10), the conductor installation can start. This is done with the help of large reels, that are transported to the mast location by trucks (see Figure 97). To keep the conductors in the air, isolators must be placed first. These isolators can either be vertical or in a V-form (i.e. V-brace) and are usually installed with the help of winches and other tools.

The amount of conductor stringing systems currently utilized in the power industry is numerous. One of the most common methods is the tension method. As the name implies, the conductor is kept under tension during the entire process. This method is especially attractive when the conductor must be kept of the ground and away from other obstacles, that could cause conductor surface damage. It is one of the most economical methods of installing conductors.

Using this method, first a light pilot line is pulled into the isolators with the help of a quad or tractor. This pilot line is then used to pull in a heavier pulling line. Subsequently the heavier pulling line will then pull in the conductors from the reels. For light conductors the heavier pulling line can be omitted, while the pilot line is replaced by a light pulling line. The reels are usually combined with a braking machine to prevent overrunning and backlash. It is common to install temporary wheels in the mast, to facilitate the pulling lines and conductors.

The conductor is pulled from dead-end mast to dead-end mast. In between, the suspension masts are simply there to support the conductors. Only at the dead-end masts the conductors are actually pulled. Also to keep the conductors and the mast at a distance from each other, spacers are used.

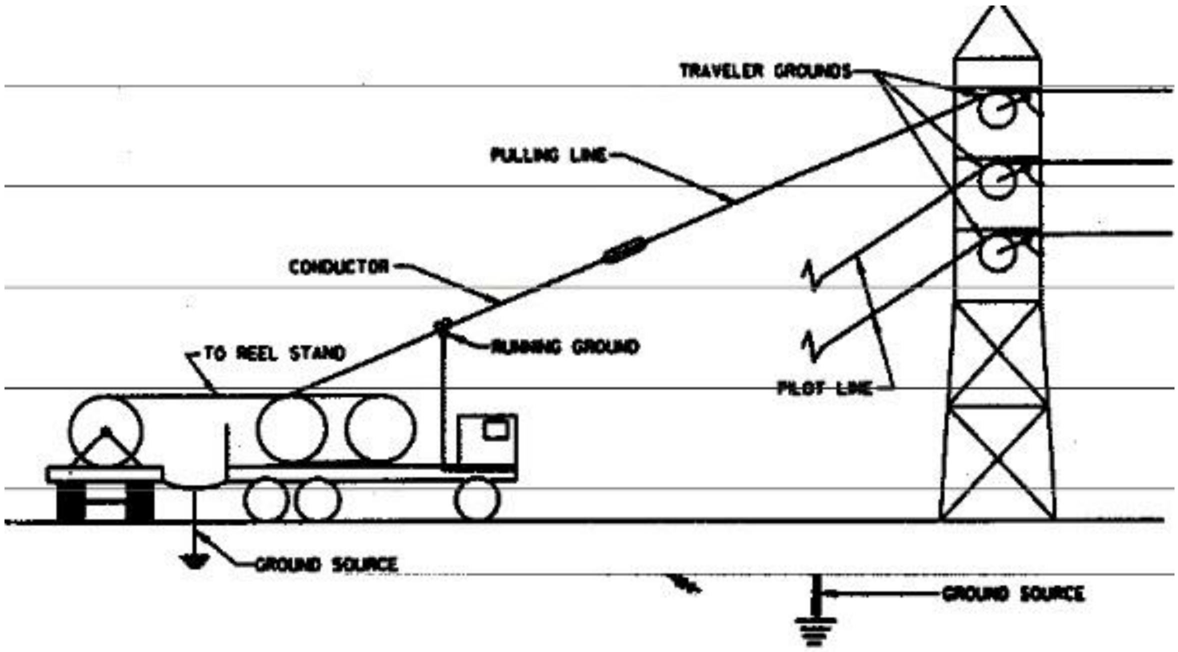


Figure 97: Conductor connected to pulling line [41]

6.6.2 Other cables

Lightning wire

For the lightning wires the process is exactly the same as for the conductors. They are usually installed after the conductors and the retour current conductor, because of their higher location. Because lightning wires tend to be lighter than conductors, they can usually be installed without a pilot line, using only a light pulling line.

Retour current conductor (passive loop)

See lightning wire.

6.6.3 Miscellaneous

Door

Since the mast has a closed, tubular form, it should be possible to enter the mast somewhere for maintenance etc. This entry is usually established by making a door in the bottom segment. The inclusion of this door however, should by no means compromise the strength, stability or safety of the mast in any way.

Figure 98: Entrance for the mast [XXV]

Climbing system

According to NEN-EN 50341, “Every tower must be provided with a climbing facility”. For tubular poles this is adjusted to two climbing facilities. These two climbing facilities must be opposite of each other and are used to allow personnel safe access. The climbing facilities are at an angle of 45° with the conductor direction. The distance between the steps is around 250 – 300 mm.

Electrical installation

Inside or near the mast an electrical installation must be placed, which controls most of the electrical aspects of the mast. This installation can either be inside the mast, provided there is enough space, or outside of it (underground is possible as well).

Figure 99: Electrical installation [XXV]

6.7 Landscape restoration

After all the necessary cables have been placed and a sufficient amount of tests has been done to verify that everything functions properly, the last phase is entered. The final step in the building process is the restoration of the landscape to its original state. This includes the direct area surrounding the mast as well as the access road. The entire surrounding area should look just as it did before or potentially even better (by adding plants etc.). To guarantee that the restoration runs smoothly, a base measurement is done at the start of the building process. This entails an overview of the undisturbed surroundings and can thus be used to monitor the restoration activities.

6.8 Construction time

In the table below an overview of the duration of the total building process is given. The assumptive start date is taken as March 2014. The total process to construct two masts at a mast location takes about 34 days.

Task	**Duration**	**Start**	**Finish**
Phase 1: Production	17	03-3-14	25-3-14
Segment production	15	03-3-14	21-3-14
Temporary supports	2	24-3-14	25-3-14
Phase 2: Site preparation	20	03-3-14	28-3-14
Construction of access road	3	03-3-14	05-3-14
Preparing mast location	2	06-3-14	07-3-14
Piling	5	10-3-14	14-3-14
Foundation block	10	17-3-14	28-3-14
Phase 3: Transport & assembly	4	31-3-14	03-4-14
Transport	1	31-3-14	31-3-14
Assembly of mast	3	01-4-14	03-4-14
Joint coupling	3	01-4-14	03-4-14
Prestressing of segments	3	01-4-14	03-4-14
Phase 4: Completion	4	04-4-14	09-4-14
Placing of conductors and wires	1	04-4-14	04-4-14
Removal of supporting structure	1	07-4-14	07-4-14
Removal of assembly system	1	08-4-14	08-4-14
Landscape restoration	1	09-4-14	09-4-14
Total construction time	**34**	**03-3-14**	**09-4-14**

Table 37: Construction time

A better overview can be achieved by using a project planning program. A more organized result is given in Figure 100.

Name	Start date	End date	Work duration
Phase 1: Production	03-03-2014...	25-03-2014...	17 Days
Segment production	03-03-2014...	21-03-2014...	15 Days
Temporary supports	24-03-2014...	25-03-2014...	2 Days
Phase 2: Site preparation	03-03-2014...	28-03-2014...	20 Days
Construction of access...	03-03-2014...	05-03-2014...	3 Days
Preparing mast location	06-03-2014...	07-03-2014...	2 Days
Piling	10-03-2014...	14-03-2014...	5 Days
Foundation block	17-03-2014...	28-03-2014...	10 Days
Phase 3: Transport & a...	31-03-2014...	03-04-2014...	4 Days
Transport	31-03-2014...	31-03-2014...	1 Day
Assembly of mast	01-04-2014...	03-04-2014...	3 Days
Joint coupling	01-04-2014...	03-04-2014...	3 Days
Prestressing of segmen...	01-04-2014...	03-04-2014...	3 Days
Phase 4: Completion	04-04-2014...	09-04-2014...	4 Days
Placing of conductors...	04-04-2014...	04-04-2014...	1 Day
Removal of supporting...	07-04-2014...	07-04-2014...	1 Day
Removal of assembly s...	08-04-2014...	08-04-2014...	1 Day
Landscape restoration	09-04-2014...	09-04-2014...	1 Day

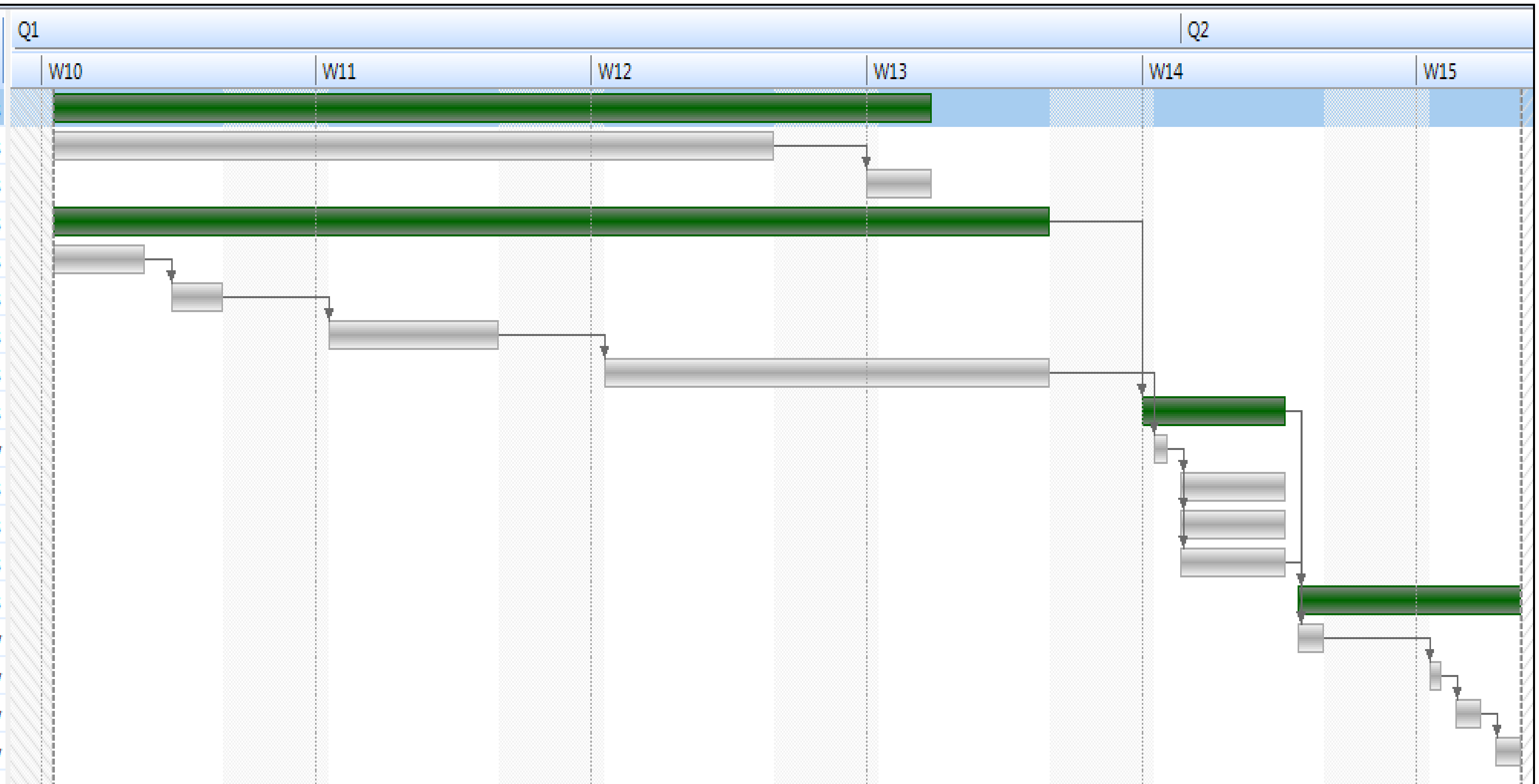


Figure 100: Project planning of transmission tower

7. Final Design

After the production and execution processes are clear, the initial assumptions made for the preliminary design can be reevaluated. As already stated in chapter 4, it is possible to optimize the wall thickness over the height of the mast. Furthermore it is reasonable to assume that the prestressing can be optimized as well. This chapter will discuss the final calculations for the mast, as well look at all of the connections considered in chapter 6, in more detail.

7.1 Optimize mast wall thickness

As already mentioned in chapter 4, after calculating the reinforced and the prestressed variants, step 4 in the design process was to optimize the wall thickness. Although this was already done to some extent in the preliminary design, it was done with certain assumptions, that may or may not have changed while considering the production and execution processes. The final reduction of the mast wall thickness will now be looked at in more detail.

7.1.1 Wall thickness reduction

As was previously seen in the preliminary design, the initial wall thickness of the mast could be greatly reduced from 250 mm to 75 mm. Now that the production and execution aspects have been decided, the final design can be made. For the full calculation see appendix L – N.

Dimensions

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	75	mm

Table 38: Mast dimensions for UHSC

Prestressing

Minimum prestressing force at t = ∞: $P_{m\infty} \geq 25869$ kN

Maximum prestressing force at t = ∞: $P_{m\infty} \leq 35545$ kN

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	25869	kN
Governing prestressing force	$P_{min,\infty}$	23282	kN
Required prestressing steel	$A_{p,req}$	18544	mm^2
Number of tendons required	n_{req}	7	-
Number of tendons applied	n_{apl}	8	-
Applied prestressing steel	$A_{p,apl}$	22800	mm^2
Applied prestressing force	$P_{min,0}$	31806	kN
Working prestressing force	$P_{min,\infty}$	28625	kN

Table 39: Prestressing tendons

Losses

Friction

$$\Delta P_u = P_{m0}(1-\mathrm{e}^{(-\mu(\theta+kx))})$$
$$=1491\,\mathrm{kN}$$

Shrinkage

$$\varepsilon_{cd,\infty} = k_h * \varepsilon_{cd,0} = 4.65*10^{-5}$$
$$\varepsilon_{ca} = 2.5*(f_{ck}-10)*10^{-6} = 4*10^{-4}$$
$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 4.47*10^{-4}$$
$$\Delta P_{cs} = A_p * \Delta\sigma_{pcs} = 1985\ \mathrm{kN}$$

Creep

$$\varphi(t,t_0)_{analysis} = \varphi_0 * \beta_c(t,t_0)$$
$$=0.45$$
$$\varphi(t,t_0)_{graph} = 0.40$$
$$\varphi(t,t_0)_{chosen} = 0.43$$

Relaxation

$$\Delta\sigma_{pr} = \sigma_{po} * 0.66 * \rho_{1000} * e^{9.1\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)} 10^{-5}$$
$$=67.95\ \mathrm{N/mm^2}$$

Total c+s+r

$$\Delta\sigma_{p,c+s+r} = \sigma_{p0}\frac{\varepsilon_{cs}E_p + 0.8\Delta\sigma_{pr} + \alpha_p\varphi(t,t_0)\sigma_{c,QP}}{1+\alpha_p\rho_p\left(1+\frac{A_c}{I_c}z_{cp}{}^2\right)\left[1+0.8\varphi(t,t_0)\right]}$$
$$=90.51\ \mathrm{N/mm^2}$$

Total losses

$$(\Delta P_u / P_{m0}) + (\Delta\sigma_{p,c+s+r} / \sigma_{p0}) = 6.49 + 5.21 = 11.70\%$$

Reinforcement

SLS

Horizontal equilibrium:

$$\sum H = N_{sc} - N_{st} + \Delta N_{pc} - \Delta N_{pt} + N_c - P_{m\infty} = N$$
$$N_{rep} + P_{m\infty} + (0.5*\mathrm{A}_s*[\sigma_{st}-\sigma_{sc}]) + (0.5*\mathrm{A}_p*[\Delta\sigma_{pt}-\Delta\sigma_{pc}]) - (\mathrm{E}_c*\varepsilon_c*b*x_u) = 0$$

Moment equilibrium:

$$\sum M = \left(N_{st}+N_{sc}\right)*d_s + \left(\Delta N_{pt}+\Delta N_{pc}\right)*e_p + N_c*e_c = M$$
$$M_{s;r} - (0.5*\mathrm{A}_s*[\sigma_{st}+\sigma_{sc}])*\mathrm{d}_s - (0.5*\mathrm{A}_p*[\Delta\sigma_{pt}+\Delta\sigma_{pc}])*\mathrm{e}_p - (\mathrm{E}_c*\varepsilon_c*b*x_u)*e_c = 0$$

From the above equations the necessary amount of reinforcement can be calculated. After an iteration process the following values can be obtained:

$$A_{s,req}{}^{8} = 72201 \text{ mm}^2$$
$$x_u = 1527.04 \text{ mm}$$

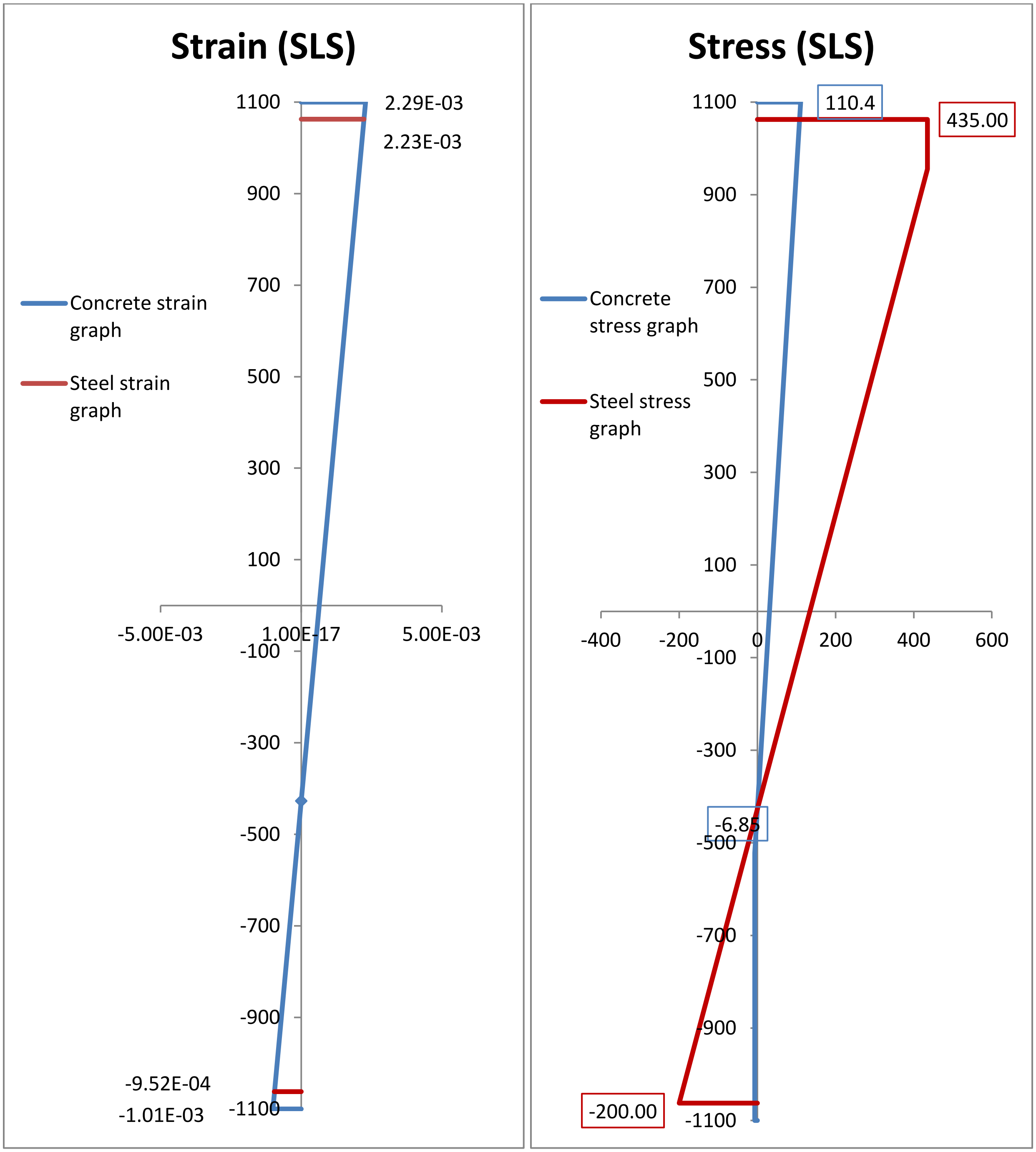


Figure 101: Stress & strain graphs for UHSC (SLS)

[8] For UHSC this is the fictitious amount of reinforcement that is necessary.

Moment capacity

ULS

$$\sum H = N_{sc} - N_{st} + \Delta N_{pc} - \Delta N_{pt} + N_c - P_{m\infty} = N$$

$$N_{sd} + P_{m\infty} + (\mathrm{A}_s * [\sigma_{st} - \sigma_{sc}]) + (\mathrm{A}_p * [\Delta\sigma_{pt} - \Delta\sigma_{pc}]) - (\mathrm{E}_c * \varepsilon_c * b * x_u) = 0$$

From this equation we can find the value of x_u:

$$x_u = 1493.59 \text{ mm}$$

$$M_{Rd} = \left(N_{st} + N_{sc}\right) * d_s + \left(\Delta N_{pt} + \Delta N_{pc}\right) * e_p + N_c * e_c$$
$$= (0.5 * \mathrm{A}_s * [\sigma_{st} + \sigma_{sc}]) * \mathrm{d}_s + (0.5 * \mathrm{A}_p * [\Delta\sigma_{pt} + \Delta\sigma_{pc}]) * \mathrm{e}_p + (\mathrm{E}_c * \varepsilon_c * b * x_u) * e_c$$
$$= 85547 \text{ kNm}$$

$M_{Rd} > M_{sd}$ => 26681 kN > 16870 kN OK

$M_{Rd} > M_{cr}$ => 26681 kN > 16922 kN OK

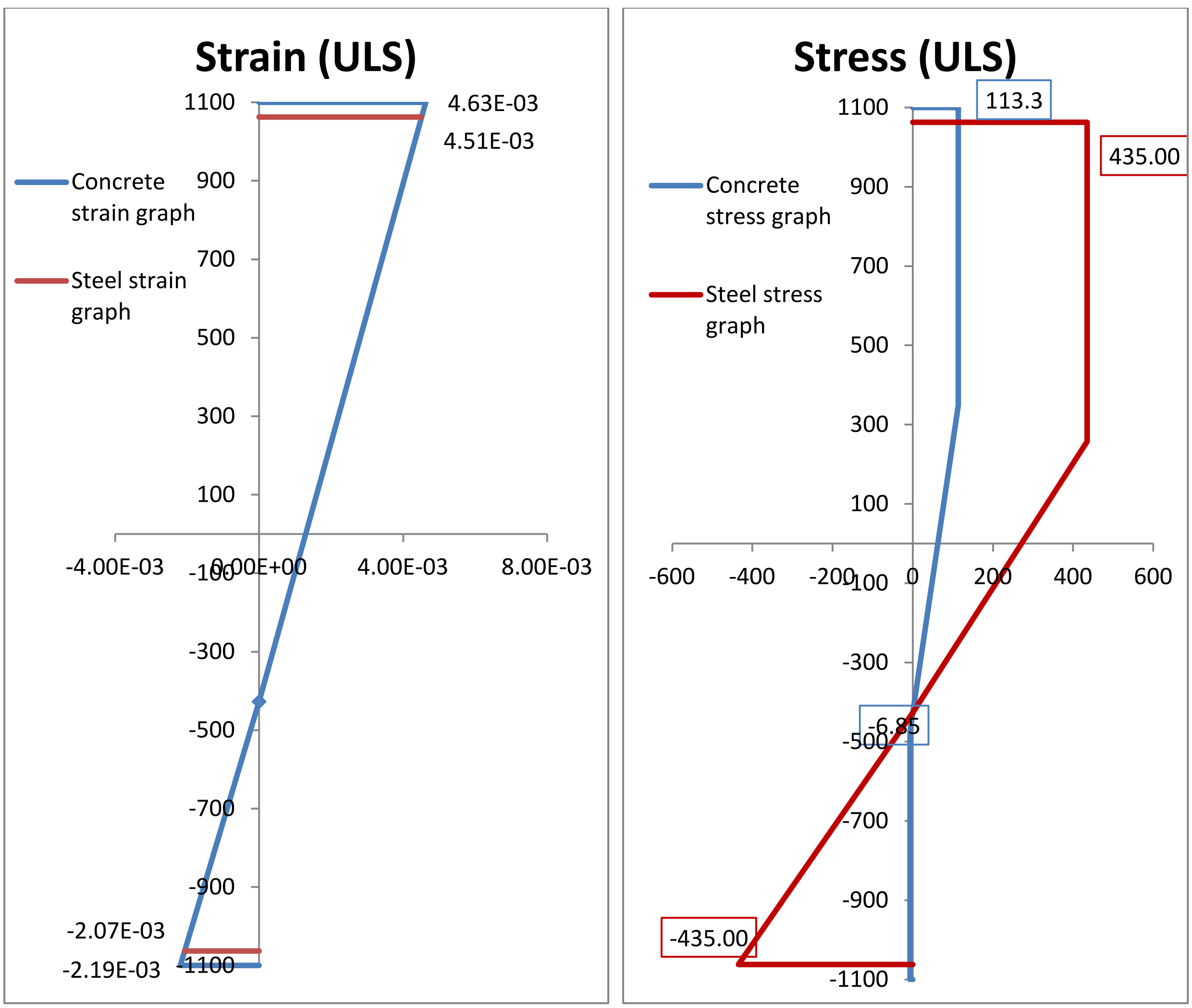


Figure 102: Strain & stress graph for UHSC (ULS)

Foundation
Same as in chapter 4.

Deformations
Crack width

$$w_k = s_{r,\max} * (\varepsilon_{sm} - \varepsilon_{cm})$$
$$= 0.074 \text{ mm}$$

Deflection

$$\delta_{top} = \delta_q + \delta_F \leq 0.025h$$
$$= 316.30 + 1306.31 = 1623 \text{ mm} > 1425 \text{ mm} \qquad \text{NOT OK}$$

$$\delta_{hor,x} = \delta_{hor,x,q} + \delta_{hor,x,F}$$
$$= 147.60 + 607.69 = 775 \text{ mm}$$
$$\delta_{rel} = 0.6\delta_{top} - \delta_{hor,x} \leq 0.01h$$
$$= 973 - 755 = 218 \text{ mm} < 570 \text{ mm} \qquad \text{OK}$$

Stability
Folding ("plooi")

$$\sigma_{total} = \sigma_N + \sigma_M \leq f_{cd}$$
$$= 108.71 \text{ N/mm}^2$$
$$\text{UC}: \frac{\sigma_{total}}{f_{cd}} = 0.96 < 1 \qquad \text{OK}$$

$$N_{cr} = \frac{\pi^2 EI}{l_0^2} = 2559$$

Buckling

$$N_{s;top} = N_{s;lw} = 19 \text{ kN} < 2559 \text{ kN} \qquad \text{OK}$$

7.1.2 Wall thickness refinement

In chapter 4 the optimal thickness was already found for the UHSC mast (t = 75 mm). However this value was found by considering the governing section of the mast, namely the foot. It makes sense then, that this value does not have to be constant over the entire mast height, as the total loads are much lower at the top of the mast. By considering the mast at several sections, an optimal wall thickness can be found for each of these sections. By doing this a so called wall thickness refinement can be accomplished, i.e. the wall thickness can gradually be decreased from the foot of the mast to the top. The results of the top segment section are shown in Table 40.

Prestressed variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength class	C	45	90	170	N/mm^2
Wall thickness at foot	t	420	145	75	mm
Wall thickness at top	t	170	95	45	mm
Wall thickness refinement	-	60%	35%	40%	-
Compressive zone height	x_u	473	419	446	mm
Reinforcement	A_s	46	51	52[*]	10^3 mm^2
Number of prestressing tendons applied	n	2	2	2	-

Table 40: Prestressed design results at the top of the UHC mast

It is seen that of all the variants, the OSC variant has the largest reduction in wall thickness. The UHSC variant has a sizeable reduction as well in terms of percentage, but in actual values the wall thickness does not reduce by too much. Furthermore only a minimal amount of prestressing is necessary to prevent stresses in the cross section.

7.1.3 Summary of final design results

The main results from the calculations are summarized in Table 41 – Table 44.

Reinforced variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength class	C	45	90	170	N/mm^2
Wall thickness	t	270	95	45	mm
Compressive zone height	x_u	705	808	890	mm
Reinforcement	A_s	65	57	57[*]	10^3 mm^2

Table 41: Summary of reinforced design results for t = 75 mm

Reinforced variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength	C	45	90	170	N/mm^2
Concrete stiffness	EI	5.12	4.01	4.95	10^{15} Nmm^2
Based on strength	t_{str}	270	95	45	mm
Based on stiffness	t_{stiff}	355[10]	300	180	mm

Table 42: Wall thickness based on strength or stiffness for reinforced variants

Prestressed variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength class	C	45	90	170	N/mm^2
Wall thickness	t	420	145	75	mm
Compressive zone height	x_u	1262	1399	1527	mm
Reinforcement	A_s	39	65	72[*]	10^3 mm^2
Number of prestressing tendons applied	n	10	8	8	-

Table 43: Summary of prestressed design results for t = 75 mm

[*] This is the fictitious amount of reinforcement necessary for the UHSC variant.
[10] Different initial dimensions

Reinforced variants	Symbol	OSC	HSC	UHSC	Units
Concrete strength	C	45	90	170	N/mm^2
Concrete stiffness	EI	5.31	6.69	6.85	10^{15} Nmm^2
Based on strength	t_{str}	420	145	75	mm
Based on stiffness	t_{stiff}	340[11]	125	90	mm

Table 44 Wall thickness based on strength or stiffness for prestressed variants

The reason why the concrete stiffness, reduces with lower concrete classes, is due to the higher creep. As the creep coefficient increases, the elasticity modulus reduces. The same is true for cracked concrete. The more cracks, the lower the elasticity modulus. For this reason the stiffness of the prestressed variants is slightly higher than the reinforced variants.

When comparing the reinforcement for the prestressed variants with the reinforced variants, it is seen that for OSC it reduces, while for HSC and UHSC it increases slightly. From this it is already clear, that applying the prestressing does not have any significant reduction on the amount of reinforcement. The prestressing only causes the structure to be in compression i.e. eliminates the tension in the cross section, which has a positive effect on the joints. The prestressing also positively influences the crack width as well as the elasticity modulus. Furthermore the prestressing has a negative influence on the buckling.

7.2 Optimize prestressing

Similar to how the wall thickness was refined ,the prestressing can be optimized as well. In the previous paragraph it was already seen that a lower wall thickness was needed at higher levels of the mast. The same is true for the prestressing. From the calculations it is found that the prestressing can be lowered gradually for each segment. In chapter 6 it was already covered that this gradual decrease of the prestressing will be done by anchoring only a couple of tendons in one segment, while allowing the rest of the tendons to continue upwards Table 45 shows the amount of tendons necessary for each segment, while Figure 103 shows the locations for the anchors at each segment end.

Height [m]	Number of tendons necessary
0	8
13	6
29	4
44	2
57	0

Table 45: Prestressing refinement

[11] Different initial dimensions

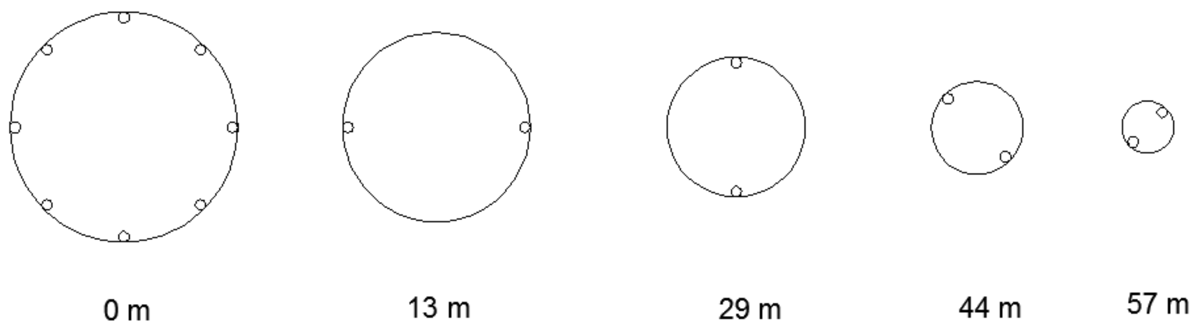


Figure 103: Initial prestressing configuration for UHSC

It was already shown, that the main advantage of the prestressing is that it removes the tension in the joints. Thus if the prestressing is solely applied for that reason, the top segment would not need prestressing as there are no more joints in this segments. The prestressing configuration could then be changed as in Figure 104 & Figure 105.

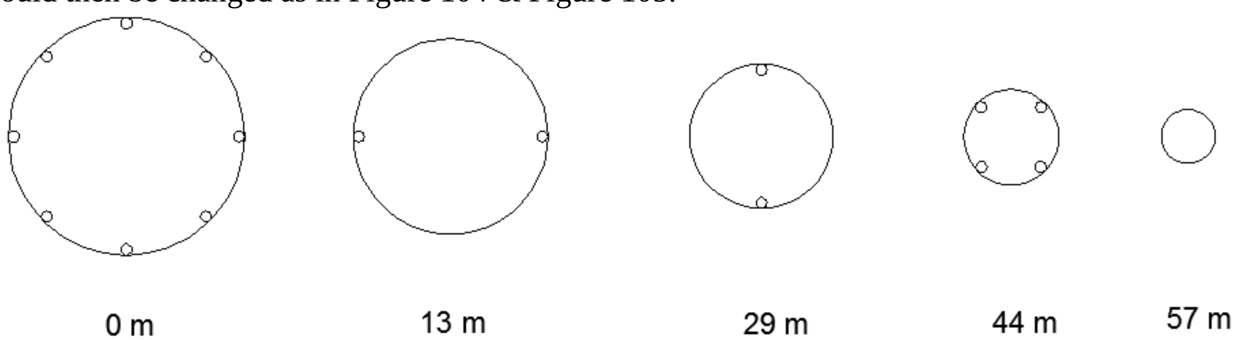


Figure 104: Prestressing anchorage locations for UHSC

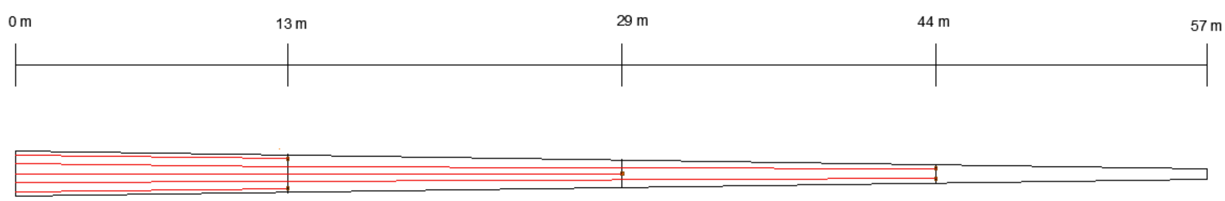


Figure 105: Alignment of the prestressing tendons for UHSC (side view)

7.3 Connections

In the design of prefab concrete structure, the connection between the elements is of paramount importance. The joints should be designed while keeping several boundary conditions in mind. These conditions can relate to the acting loads, the execution and the integral costs.

Acting loads

The forces must be able to be transferred with enough safety and minimal deformation. On the other hand imposed deformations are not allowed to induce large stresses or forces. The entire load scheme should remain simple and clear, so that the many joints can be carried out with statically determined elements.

Execution

The elements must be producible. It is useless to design the perfect joint, if it cannot be executed in practice. Additionally execution of the joint should be simple and fast on the building site.

This way crane times can be lowered significantly. The tolerances of the elements that come together in a joint, need to be aligned i.e. it should be possible to accommodate deviations. Last, but not least, the joints should be accessible for inspection and eventual maintenance.

Costs

Aside from the structural aspects of the joints themselves, the effect of the joint on the entire structure should be looked at as well. If less joints are utilized, the segments can be longer, less transport or storage room has to be used and the structure has fewer critical points. This could affect the total costs of the structure.

Furthermore connections need to be durable, fire resistant and ductile.

As already seen in chapter 6, there are three connections that have been worked out. Their constructive aspect will now be looked at in more detail. As the connections will be the most challenging for the thin UHSC variant, only this variant will be worked out.

7.3.1 Segment-segment connection

In every joint there are a number of loads at work, that must be transferred from one segment to the other. The loads that have to be considered for the segment-segment connection are (see Figure 106):

- Axial force
- Moments
- Shear force
- Prestressing force
- Moment due to prestressing
- Shear due to prestressing

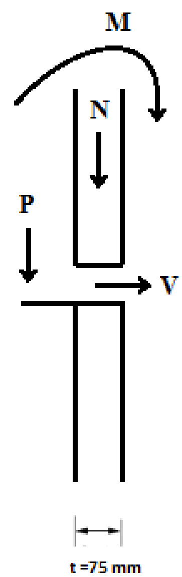


Figure 106: Acting loads on the segment-segment

Ad 1.Axial forces

Compressive forces can be transmitted between adjacent precast components by direct bearing or through intermediate medium such as in-situ mortar, concrete, bearing pads or other bearing elements.

Ad 2. Moments

Moments acting in a flexural joint can always be resolved into tension and compression force couples.

Ad 3. Shear forces

Shear stress between adjacent precast concrete elements can be transferred through bond, interface joint friction, interlocking by shear keys, dowel action of transverse steel bars or rods, welding or by other mechanical means.

Ad 4.Prestressing forces

The prestressing force is a local axial force, applied at a certain distance from the wall, that must be introduced into the structure via the connections.

Ad 5.Moment due to prestressing
Because the prestressing is applied at a certain distance from the wall thickness, it will cause an additional moment to appear in the joint (prestressing force x distance to wall).

As already seen in chapter 6, because the mast is in the form of a concrete tube, there will be a number of connections at a section, instead of one connection over the full section. Depending on the amount of these connections per section, the acting loads can be distributed among them. Table 46 illustrates all the five loads, as well as the necessary amount of connections per section, for the different segments.

Height	Diameter	Area	Minimum number of connections	Axial force	Prestress force	Shear force	Moment	Moment due to prestress
h [m]	d [m]	A [mm^2]	-	N [kN]	P [kN]	V [kN]	M [kNm]	M_p [kNm]
44	0.89	191492	2	208	7952	108	867	1193
29	1.34	296901	4	416	23855	205	3423	4175
13	1.81	409337	2	639	31806	297	7666	6361

Table 46: Acting loads on segment-segment interface

As previously stated, on the inside of the mast a maintenance area of 500x500 mm^2 is necessary. Because this area is known, the maximum joint dimensions can be decided as well.

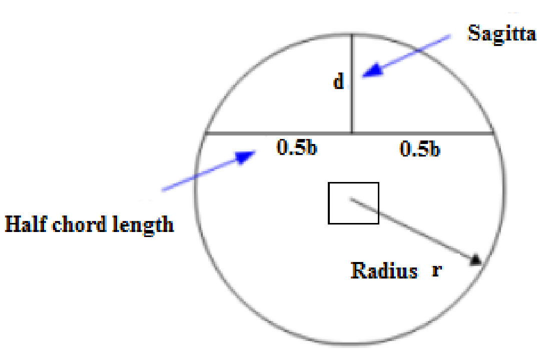


Figure 107: Schematic of inner space of mast

$$d = r - 250$$
$$d = r \pm \sqrt{r^2 - (0.5b)^2} \qquad (7.1)$$
$$b = 2\sqrt{(r^2 - 250^2)}$$

Table 47 gives an overview of the maximum joint dimensions decided with the help of Figure 107 & Equation (7.1).

Connection number	Height	Diameter	Maximum connection (incl. wall thickness) width	Maximum connection length (incl. wall thickness)
n	h [m]	d [m]	b_c [mm]	l_c [mm]
1	44	0.89	734	270
2	29	1.34	1237	492
3	13	1.81	1741	731

Table 47: Maximum dimensions of segment-segment joint

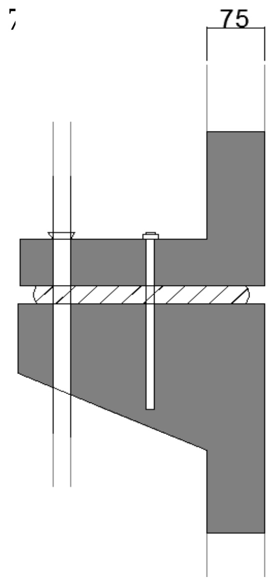


Figure 108: Segment-segment connection

Now that these limitations for the joint dimensions are known, the actual design process can begin.

Chapter 6 has already discussed the various type of possible connections. It was then decided that for the segment-segment connection, a console (or corbel) like connection was the best option (see Figure 108). Of course the connection is more like two separate consoles as there is mortar (or concrete) between the top and bottom segment. Nonetheless, the connection will be calculated as a modified console (Figure 109) , while the shear resistance in the interface between the segments (taken by the bolts) will be calculated as well.

Note: The shear resistance of the prestressing ducts was not taken into account. If the ducts have a significant impact on the shear resistance, then the amount of necessary bolts will reduce or might even become obsolete.

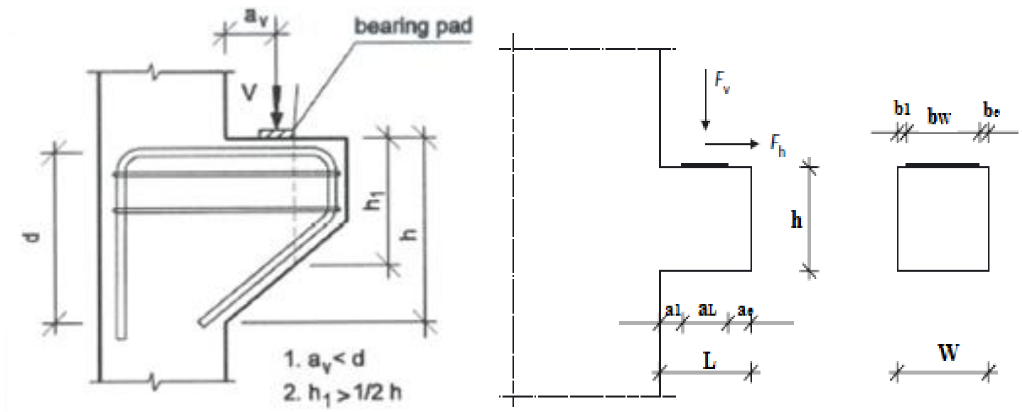


Figure 109: Console schematic: Actual (L) & approximation (R)

The considered connection will be the one between the top segment and the third segment. First the prestressing will be checked.

$$
\begin{aligned}
n &= 1 \\
a_1 &= 50 \text{ mm } \geq 50 \text{ mm} \\
a_L &= 100 \text{ mm } \geq 0.5L \\
a_e &= 20 \text{ mm} \\
L = a_1 + a_L + a_e &= 170 \text{ mm} && \text{OK} \\
b_1 &= 50 \text{ mm} \\
b_W &= 100 \text{ mm } \geq 0.5L \\
b_e &= 50 \text{ mm} \\
W = b_1 + b_W + b_e &= 200 \text{ mm} && \text{OK} \\
h &= 400 \text{ mm}
\end{aligned}
\tag{7.2}
$$

$$
\begin{gathered}
a = 200 \\
l_{ov} = 400 \\
l_{ov}/h = 1 \qquad \text{Deep beam} \\
z = 0.2 * l + 0.4 * h = 240 \text{ mm} < 0.8l
\end{gathered}
\tag{7.3}
$$

$$
\begin{gathered}
F_v = 3976 \text{ kN} \\
\mathrm{F}_h = 0 \text{ kN} \\
M_d = F_v * a = 795 \text{ kNm} \\
A_{s,Fv} = \frac{M_{sd}}{f_y * z} = 7616 \text{ mm}^2 \\
A_{s,Fh} = 0 \text{ mm}^2 \\
\phi = 25 \text{ mm} \\
n = \frac{A_s}{\phi} = 16
\end{gathered}
\tag{7.4}
$$

Now that the amount of reinforcement necessary to introduce the prestressing into the wall is known, the shear resistance of the joint can be calculated.

$$
\begin{gathered}
V = 54 \text{ kN} \\
n_{bolt} = 2 \text{ bolts} \\
F_{v,Ed} = \frac{V}{n} = 27 \text{ kN} \\
a_{bolt,l} = 20 \text{ mm} \qquad \text{OK} \\
a_{bolt,r} = L - a_{bolt,l} = 150 \text{ mm} \\
b_{bolt,l} = 20 \text{ mm} \\
b_{bolt,m} = W - b_{bolt,l} - b_{bolt,r} = 160 \text{ mm} \qquad \text{OK} \\
b_{bolt,r} = 20 \text{ mm}
\end{gathered}
\tag{7.5}
$$

$$
\begin{gathered}
\text{Bolt quality: 5.6} \\
f_{yb} = 300 \text{ N/mm}^2 \\
f_{ub} = 500 \text{ N/mm}^2 \\
\alpha_v = 0.6 \\
\gamma_m = 1.25 \\
M = 16 \text{ mm} \\
F_{v,Rd} = \frac{\alpha_v * f_{ub} * A_{b,s}}{\gamma_m} = 35 \text{ kN} \qquad \text{OK}
\end{gathered}
\tag{7.6}
$$

Finally it is checked if the moment and compression force can be transferred from the top segment, to the segment below it.

$$\begin{aligned} f_{mk} &= 45 \text{ N/mm}^2 \\ f_{md} &= 30 \text{ N/mm}^2 \\ f_{mm} &= 27 \text{ N/mm}^2 \\ v_o &= 20 \text{ mm} \\ v &= v_o + 20 = 40 \text{ mm} \end{aligned} \tag{7.7}$$

$$\begin{aligned} d &= 0.89 \text{ m} \\ \text{x}_u &= 460 \text{ mm} \\ \text{bx}_u &= 98167 \text{ mm} \\ \text{k}_1 &= 0.9 \\ k_2 &= 0.98 \\ k_3 &= 0.12 \\ k_4 &= 38.18 \\ k_5 &= 0.5 \end{aligned} \tag{7.8}$$

$$\begin{aligned} f'_v &= k_1 * k_2 * f_{md} = 26.41 \text{ N/mm}^2 \\ A_{bv} &= \pi * d * t = 209164 \text{ mm}^2 \\ N'_{uv} &= A_{bv} * f'_v = 5524 \text{ kN} \\ M_{Ed} &= 2060 \text{ kNm} \\ N_c &= 0.5 * bx_u * f'_v = 1277 \text{ kN} \\ e_c &= 301.5 \text{ mm} \\ M_{Rd,\text{As}} &= M_{Ed} * N_c * e_c = 1675 \text{ kNm} \\ d_s &= d - x_u - c = 418 \text{ mm} \\ d_s' &= x_u - c = 430 \text{ mm} \\ A_s &= \frac{M_{Rd,As}}{f_y (d_s + d_s')} = 4543 \text{ mm}^2 \\ M'_{uv} &= N_c * e_c + f_y * A_s * (d_s + d_s') = 2070 \text{ kNm} \qquad \text{OK} \end{aligned} \tag{7.9}$$

For a complete overview of all the segment-segment connection calculations, see appendix O.

7.3.2 Segment-foundation connection

The second most important connection to consider is the segment foundation-connection. As this connections is at the bottom the acting loads on this connection will be the largest (see Table 48).

Height	Diameter	Area	Axial force	Prestress force	Shear force	Moment
h [m]	d [m]	A [mm^2]	N [kN]	P [kN]	V [kN]	M [kNm]
0	2.20	500691	736	31806	316	11247

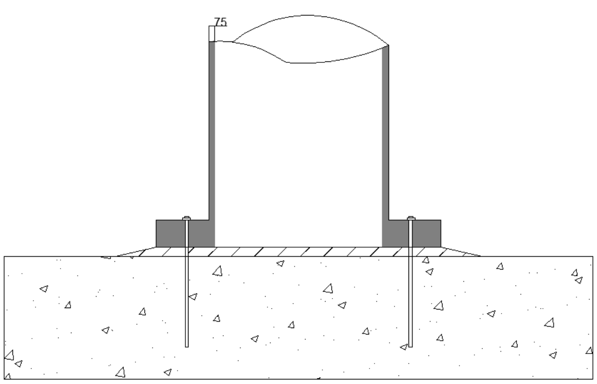
Table 48: Acting loads at interface segment-foundation

As was done for the segment-segment connection, the joint dimensions can be limited here as well (Table 49). However depending on what type of connection is used, this might not applicable. For instance if a steel plate connection is applied underneath the bottom segment, any occurring maintenance will simply happen on top of this plate. Thus in several cases the interface between the foundation and segment will simple be used as the maintenance floor. However as it was determined in chapter 6 that a large variety of options exist, the maximum dimensions were determined anyway.

Wall thickness	Maximum connection width (incl. wall thickness)	Maximum connection length (incl. wall thickness)
t [mm]	b_c [mm]	l_c [mm]
75	2142	850

Table 49: Maximum dimensions of foundation joint

Now that the limitations are clear, the interface resistance of the segment-foundation can be calculated. The calculation method is valid for both connections as shown in Figure 110.



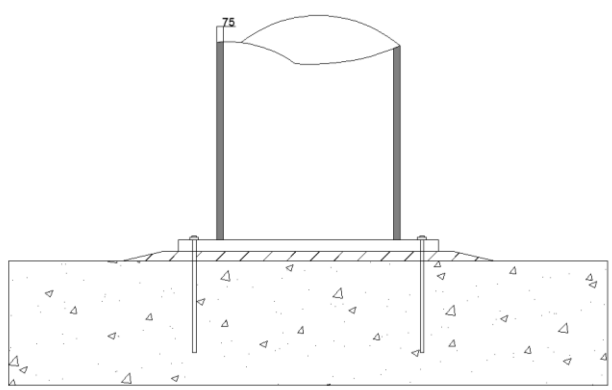


Figure 110: Segment-foundation connection: Bolted concrete (L) & bolted steel plate (R)

Figure 111 shows a schematic of the acting loads on the segment-foundation interface.

Note: As with the segment-segment connection, the shear resistance of the prestressing ducts was not taken into account. If the ducts have a significant impact on the shear resistance, then the amount of necessary bolts will reduce.

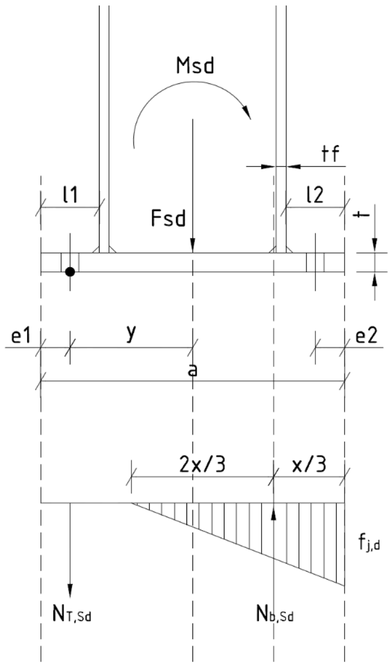


Figure 111: Schematic of segment-foundation connection

$$a = 2500 \text{ mm}$$
$$b = 2500 \text{ mm}$$
$$e_1 = 75 \text{ mm}$$
$$e_2 = 75 \text{ mm}$$
$$y = 0.5a - e_1 = 1175 \text{ mm} \quad (7.10)$$
$$l_1 = 2 * e_1 = 150 \text{ mm}$$
$$l_2 = 2 * e_2 = 150 \text{ mm}$$

$$f_{ck} = 30 \text{ N/mm}^2$$
$$f_{cd} = 20 \text{ N/mm}^2$$
$$\beta_j = 0.67 \quad (7.11)$$
$$k_j = \frac{0.25 * \pi * d_{found}{}^2}{a * b} = 2.55$$
$$f_{jd} = \beta_j * k_j * \text{f}_{cd} = 34 \text{ N/mm}^2$$

$$f_{jb} = 900 \text{ N/mm}^2$$
$$f_{ub} = 1000 \text{ N/mm}^2$$
$$\alpha_v = 0.5 \quad (7.12)$$
$$\gamma_m = 1.25$$
$$k_2 = 0.9$$
$$N_{b,\text{sd}} = \tfrac{1}{2} b * x * f_{m,d}$$
$$\sum V = N_{c,sd} - N_{t,sd} = F_{sd}$$
$$\sum M_{left} = F_{sd} * y - N_{c,sd}(a - e - \frac{x}{3}) = M_{sd}$$
$$x = 483 \text{ mm} \quad (7.13)$$
$$t_{plate} = \sqrt{\frac{6M_{sd}}{b * f_{yd}}} = 82 \text{ mm}$$

OR

$$\text{t}_{concrete} = \sqrt{\frac{6M_{sd}}{b * f_{cd}}} = 135 \text{ mm}$$

Bolt quality:10.9

Number of bolts: $n = 8$ bolts (7.14)

Bolt diameter: M45

$$F_{t,Ed} = \frac{N_{t,sd}}{0.5 * n} = 1177 \text{ kN}$$

$$F_{v,Ed} = \frac{V_{sd}}{n} = 40 \text{ kN}$$

$$\mathrm{F}_{t,Rd} = \frac{k_2 * f_{ub} * A_{b,s}}{\gamma_m} = 1028 \text{ kN} \tag{7.15}$$

$$\mathrm{F}_{v,Rd} = \frac{\alpha_v * f_{ub} * A_{b,s}}{\gamma_m} = 571 \text{ kN}$$

$$\text{UC: } \frac{F_{t,Ed}/1.4}{F_{t,Rd}} + \frac{F_{v,\mathrm{Ed}}}{F_{v,\mathrm{Rd}}} = 0.89 \qquad \text{OK}$$

7.3.3 Segment-isolator connection

The last type of connection that must be considered is the segment-isolator connection. In Table 50 the local acting loads, at all three conductor heights, are illustrated.

Connection number	Height	Height	Axial force	Prestress force	Shear force	Moment	Vertical conductor force	Horizontal conductor force
n	h [m]	h [m]	N [kN]	P [kN]	V [kN]	M [kNm]	F_V [kN]	F_H [kN]
1_t	47	49.63	71	0	18	137	57	62
1_c		46.13	97	0	23	591		
2_t	37	39.63	241	7952	114	1236	57	57
2_c		36.13	267	7952	119	1957		
3_t	27	29.63	411	23855	204	3124	57	49
3_c		26.13	437	23855	279	4071		

Table 50: Acting loads at isolator locations

As seen for the segment-segment connection, the connection dimensions will be limited by the necessary maintenance area inside the mast. Table 51 shows the maximum dimensions of any of the connections.

Connection number	Height	Diameter	Wall thickness	Maximum connection width	Maximum connection length (incl. wall thickness)	Maximum increase of connection length
n	h [m]	d [m]	t [mm]	b_c [mm]	l_c [mm]	l_c [mm]
1_t	47	0.72	45	518	110	65
1_c		0.82		655	162	117
2_t	37	1.02	55	887	259	204
2_c		1.12		1005	311	256
3_t	27	1.32	65	1218	408	343
3_c		1.42		1330	460	395

Table 51: Maximum dimensions of isolator joint

The segment-isolator connection can be divided in (see Figure 112):

- Tension isolator connection
- Compression isolator connection

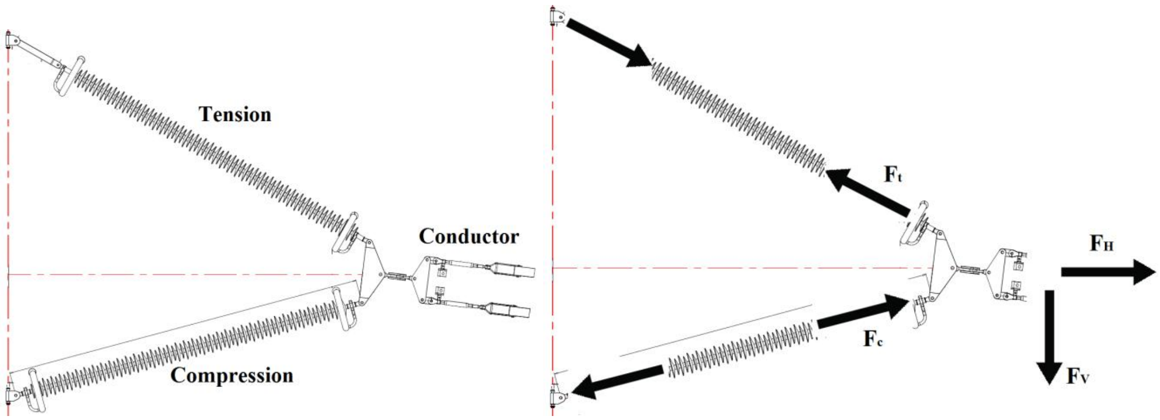


Figure 112: Isolator schematic (L) & acting loads (R)

Aside from dividing the isolator in a tension and a compression part, anther distinction must be made, namely of the isolator on the left side of the mast and the isolator to the right of the mast. For both the left and the right isolator, the top part remains in tension, while the lower part is in compressions. The difference lies in the fact that on the right side, the tension part is more heavily loaded than on the left side, while on the left side the same is true for the compression part. This can more easily be seen by looking at the resultant of the forces. From Figure 113 it is clear that the resultant of the forces on the left side works directly in the line of the compression isolator, while on the right side it works on the tension isolator.

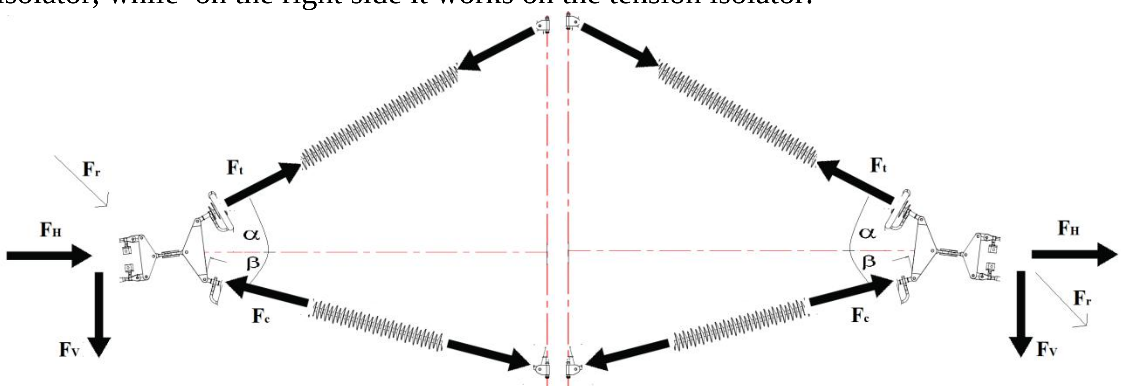


Figure 113: Left and right isolators

7.3.3.1 Tension isolator

The bond of the isolator to the concrete is much stronger than the bond of concrete itself. Typically the concrete will separate next to the bond line of the isolator with the segment. Therefore the weakest link of the connection is the concrete itself. The force require to pull the concrete must be calculated, to see if it can resist the acting loads. Table 52 shows an overview of the acting loads on the tension part of the isolators.

		Left isolator			Right isolator		
Connection number	Height	Tension force [kN]			Tension force [kN]		
n	h [m]	F_t	$F_{t,V}$	$F_{t,H}$	F_t	$F_{t,V}$	$F_{t,H}$
1_t	49.63	35	20	29	90	52	74
2_t	39.63	37	22	31	88	51	72
3_t	29.63	41	24	34	85	49	69

Table 52: Acting loads on tension isolators

The pullout force of concrete can be calculated with a shear force calculation. The top isolator is the most critical connection, as the largest force occur at that location. Furthermore it has the smallest cross section (diameter and thickness) as well. Therefore this connection will be worked out in detail. Appendix P gives an overview of all the segment-isolator connections.

$$n = 1_t$$
$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$
$$f_{ck} = 170 \text{ N/mm}^2$$
$$b_w = 50 \text{ mm} \qquad \text{OK}$$
$$d = t + x = 45 + 60 = 105 \text{ mm} \qquad \text{OK}$$
$$\text{k=1+}\sqrt{\frac{200}{d}} \geq 2$$
$$A_{sl} = 1000 \text{ mm}^2$$
$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0.23$$
$$k_1 = 0.15$$
$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} = 0.47 \text{ N/mm}^2 > 0.2 f_{ck}$$
$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$
$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 7 \text{ kN}$$
$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 20 \text{ kN}$$
$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 3.84 \text{ N/mm}^2 \tag{7.16}$$
$$D = 60 \text{ mm}$$
$$\text{F}_p = \pi * D * d * v_{Rd,c} = 76 \text{ kN} \qquad \text{OK}$$

7.3.3.2 Compression isolator

The compression part of the isolator can be regarded as a point load working on a thin plate. Thus punching shear will be the governing criteria. Table 53 shows an overview of the acting loads on the tension part of the isolators.

		Left isolator			Right isolator		
Connection number	Height	Compression force [kN]			Compression force [kN]		
n	h [m]	F_t	$F_{t,V}$	$F_{t,H}$	F_t	$F_{t,V}$	$F_{t,H}$
1_c	46.13	98	37	91	13	5	12
2_c	36.13	94	35	88	17	6	15
3_c	26.13	89	33	82	22	8	21

Table 53: Acting loads on compression isolators

$$n = 1_c$$

$$V_{Ed} = F_{c,H} = 91 \text{ kN}$$

$$b_w = 90 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 45 + 75 = 120 \text{ mm} \qquad \text{OK} \qquad (7.17)$$

$$d^* = 2b = 180 \text{ mm}$$

$$\mathrm{u}_0 = \pi * d^* = 565 \text{ mm}$$

$$v_{Ed} = \frac{\beta * V_{Ed}}{u_0 * d} = 1.33 \text{ Nmm}^2$$

$$C_{Rd,c} = 0.18 / \gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$\text{k=1+}\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 0 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} = 0.55 \text{ N/mm}^2 > 0.2 f_{ck}$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$

$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 7 \text{ kN}$$

$$\mathrm{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 0 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 1.37 \text{ N/mm}^2 \qquad \text{OK} \qquad (7.18)$$

8. Sustainability

As previously stated in chapter 3, sustainability is a major topic nowadays. The sustainability of the transmission tower depends mainly on the amount of emitted and stored energy (i.e. insulating properties), throughout the entire life of the structure. UHSC uses about twice as much cement as OSC and thus produces twice as much CO_2 and consumes twice as much energy during production. Still past experiences with UHSC show that if utilized correctly, the amount of material used in structures can be two or three times less than OSC. This chapter will compare the steel mast (both traditional truss and new tubular form) with the concrete masts, as well as the concrete masts individually, in terms of energy consumption and emission. To get a complete picture, the total energy of a bipole mast will be used i.e. the energy of both separate masts.

8.1 Embodied energy

Embodied energy is defined as: "*the sum of all the energy required to produce any goods or services, considered as if that energy was incorporated or 'embodied' in the product itself*" [XXXVI]. In other words, the energy that is consumed by all of the processes associated with the construction of a structure, from the extraction and processing of raw materials to manufacturing, transport and delivery. Embodied energy does not include the operation and demolition of the structure. These parts are only included in a life cycle approach.

Although the definition simply mentions the sum of all energy, this is in fact a rather complex combination of various materials, each of which has a different contribution to the total embodied energy of the structure. As examining the entire energy process accurately is too complex, this thesis will only look at the main ingredients of the embodied energy which differ for the variants[12]. For the concrete masts this comprises of concrete, reinforcement and prestressing, while for the steel masts it consists of structural steel and a coating layer. Furthermore for all variants the foundation will be included as well. Similar components for the variants, such as conductors, transport of workers to site, worker facilities, etc., were not taken into account because they are not relevant for the comparison.

The embodied energy and emissions for the steel truss mast is derived by assuming a standard thickness value of 12 mm for the steel sections (see appendix Q).

The embodied energy and emissions for the steel tube masts are derived by assuming a standard thickness value of 17 mm for the wall thickness (see appendix R).

[12] This is usually called Process Energy Requirement (PER).

Embodied energy	**Steel truss**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Galvanized steel S235	21.63	MJ/kg	21.828	472129	MJ
Foundation block C30/37	1.80	MJ/kg	424.800	764640	MJ
Foundation reinforcement B500B	10088	MJ/kg	7.488	81469	MJ
Total energy				**1.318.239**	MJ

Table 54: Embodied energy steel truss mast

Embodied energy	**Steel tube**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Construction steel S355	12.42	MJ/kg	73.011	900664	MJ
Coating	97,00	MJ/kg	738	71543	MJ
Foundation block C30/37	1,80	MJ/kg	420.616	757109	MJ
Foundation reinforcement B500B	10,88	MJ/kg	8.405	91447	MJ
Total energy				**1.820.762**	MJ

Table 55: Embodied energy steel tube masts

As can be seen from Table 54 & Table 55, the steel and coating layer have large energy unit rates, but are only utilized in small quantities (in comparison with concrete). In contrast, concrete has fairly low energy unit rates, but requires large amounts for construction (see Table 56).

Embodied energy	**UHSC**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C170/200	5,39	MJ/kg	74.894	403677	MJ
Reinforcing steel B500B	10,88	MJ/kg	2.337	25423	MJ
Prestressing steel Y1860S	36,00	MJ/kg	8.579	308844	MJ
Foundation block C30/37	1,80	MJ/kg	455.148	819267	MJ
Foundation reinforcement B500B	10,88	MJ/kg	9.527	103657	MJ
Total energy				**1.660.868**	MJ

Table 56: Embodied energy UHSC masts

The energy unit rates depicted in Table 54 -Table 56, are attained by considering the total energy that is consumed from extraction of raw materials, processing, transport and assembly (for an overview see appendix S). A stated before, the embodied energy does not take into account operational aspects (maintenance) and demolition. For a complete overview of the embodied energy for all variants see appendix T.

Note:

- The actual embodied energy of a product might differ from country to country;
- Products made from recycled material will contain less embodied energy than products made directly from raw materials;

- Materials with high energy unit rates (such as steel), will almost certainly be recycled many times over the course of their life, which reduces their lifecycle impact.
- Estimates of embodied energy are usually varied and should not be taken as correct. What is of importance, is the relative relationships and comparison of the values.

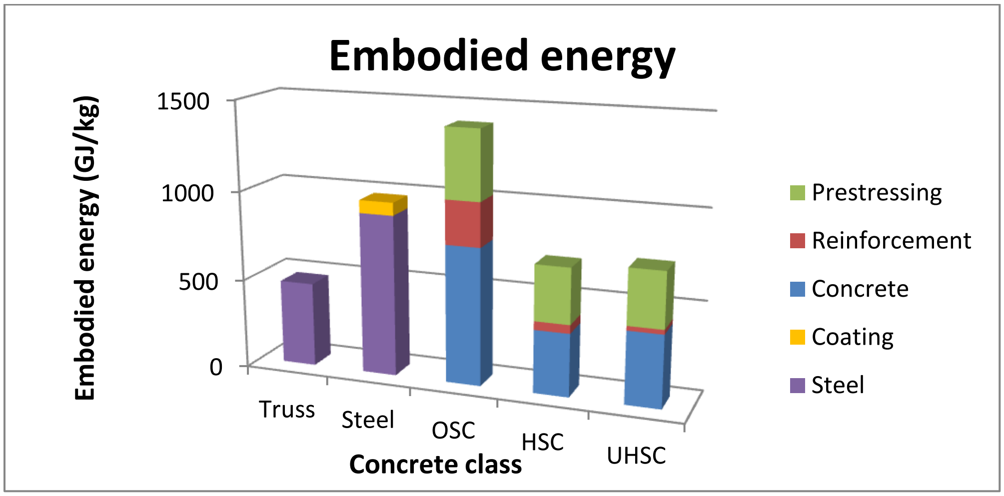


Figure 114: Embodied energy comparison (excluding foundation)

As can be seen from Figure 114, steel truss mast has the least amount of embodied energy. This is credited to the low amount of material used to construct the mast. When the concrete alternatives are considered, it is obvious that the OSC variant scores a lot worse compared to the rest. This high amount of energy is clearly due to the high amount of concrete used. The difference between HSC & UHSC seem to be minimal with both alternatives scoring slightly better than the steel tube variant.

An important aspect that is not taken into account in Figure 114 is the foundation. Although the foundation for the variant will be fairly similar, including the foundation in the analysis does provide a more complete picture.

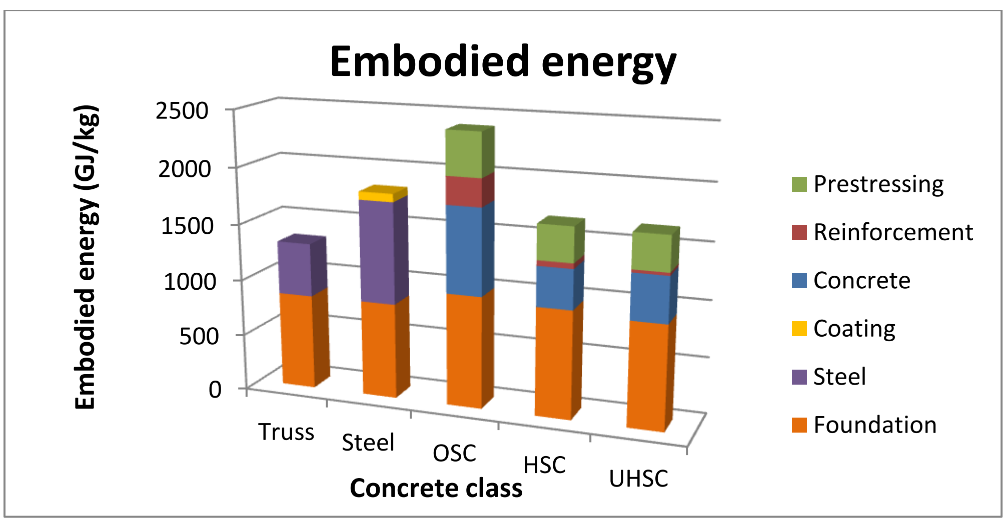


Figure 115: Embodied energy comparison (including foundation)

From Figure 115 it can be gathered that initial conclusions remain the same, although the differences between the variants reduce in a relative sense.

8.2 Embodied emissions

Another way of comparing the energy between the variants is the embodied emissions method. In this method the emission of CO_2 is compared. This embodied emission is the amount of CO_2 that is released from raw materials during processing, transport to building site, application in a structure and demolition. Unfortunately the amount of energy as previously calculated in the embodied energy method, cannot simply be converted to amount of CO_2 emitted. This is due to the fact that the different type of materials all have different CO_2 emission rates.

Embodied emissions	**Steel truss**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Galvanized steel S235	1,350	kg CO_2/kg	21.828	29467	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	424.800	50976	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	7.488	5391	kg CO_2
Total energy				**85.835**	kg CO_2

Table 57: Embodied emissions steel truss mast

Embodied emissions	**Steel tube**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Construction steel S355	0,686	kg CO_2/kg	73.011	50056	kg CO_2
Coating	3,130	kg CO_2/kg	738	2309	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	420.616	50474	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	8.405	6052	kg CO_2
Total energy				**108.890**	kg CO_2

Table 58: Embodied emission steel tube masts

As with the embodied energy method, the energy unit rates for the embodied emissions are larger for steel and the coating than for concrete.

Embodied emissions	**UHSC**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C170/200	0,329	kg CO_2/kg	74.894	24640	kg CO_2
Reinforcing steel B500B	0,720	kg CO_2/kg	2.337	1682	kg CO_2
Prestressing steel Y1860S	1,250	kg CO_2/kg	8.579	10724	kg CO_2
Foundation C30/37	0,120	kg CO_2/kg	455.148	54618	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	9.527	6860	kg CO_2
Total energy				**98.524**	kg CO_2

Table 59: Embodied emissions UHSC masts

For a complete overview of the embodied emissions for all variants see appendix U.

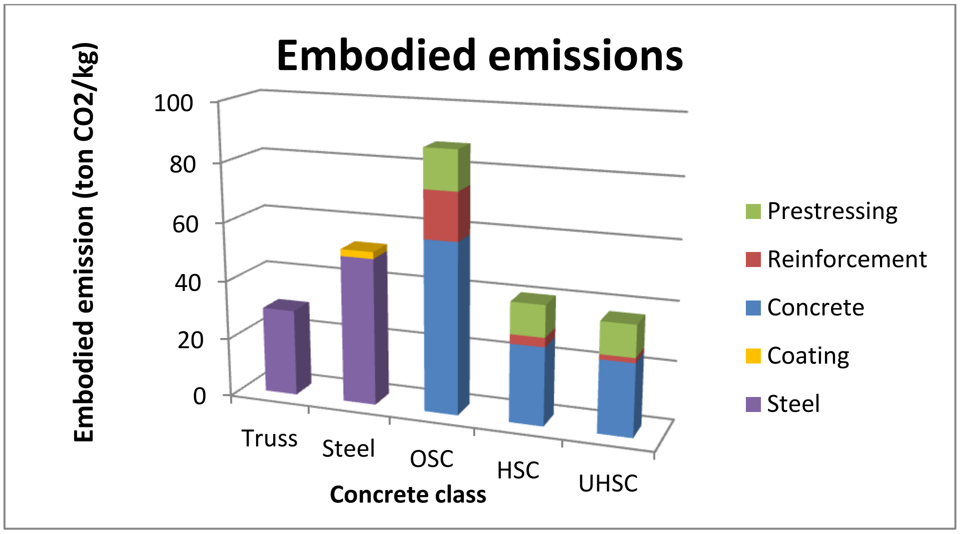


Figure 116: Embodied emissions comparison (excluding foundation)

As can be seen from Figure 116, the embodied emission method seems to agree with the embodied energy method. Once again HSC & UHSC show minimal differences and a slight superiority over the steel tube variant. In this method the steel truss variant remains the most environment friendly alternative.

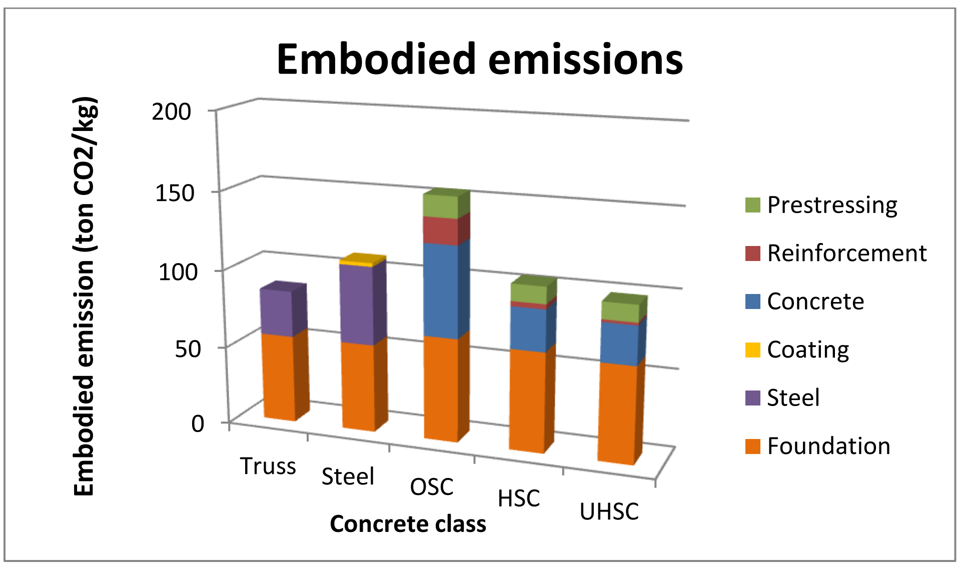


Figure 117: Embodied emissions (including foundation)

By once again including the foundation in the analysis it can be seen that the difference between the variants is reduced. This is once again credited to the high amount of concrete that is used in the foundation for all alternatives.

8.3 Self weight

It can be observed that, especially in the case of the concrete variants, both the embodied energy and the embodied emissions of the mast, heavily depend on the weight of the construction.

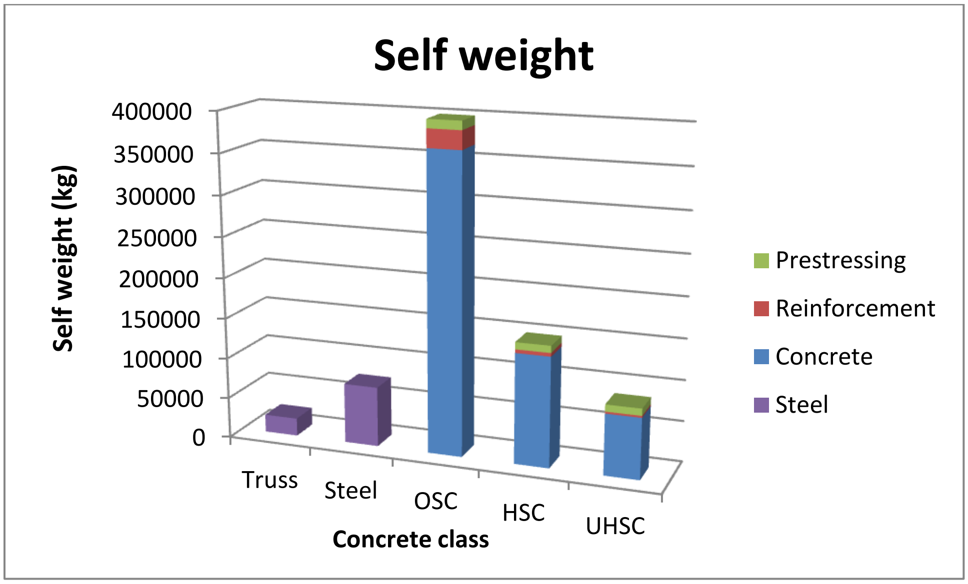


Figure 118: Self weight

If we compare Figure 118 with Figure 114 & Figure 116, we can see a clear similarity. This reaffirms that the embodied energy and emission are heavily dependent on the self weight or volume of the structure. The self weight of the structure, in turn, is heavily dependent on the wall thickness (see Figure 119), for the concrete variants more so than for the steel variants.

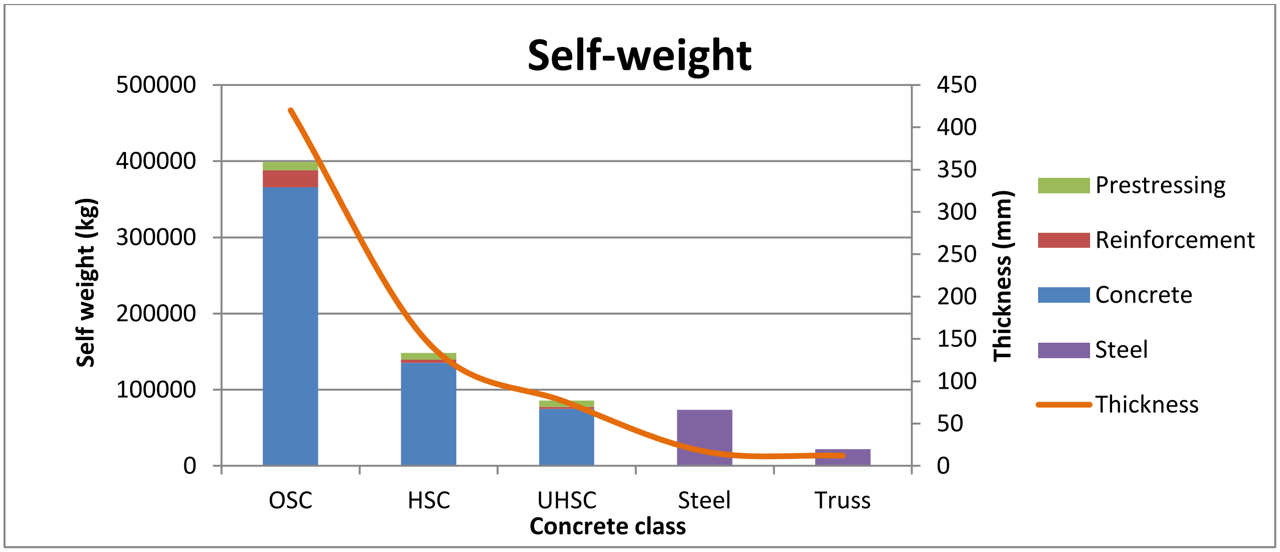


Figure 119: Self weight vs. thickness

8.4 Visual impact

Although the subject of visual pollution does not directly relate to energy considerations, it does relate to sustainability in general. The transmission tower has a lifespan of 100 or more years. It will thus be part of the scenery for quite a while.

As the Netherlands has a small surface area, good area planning is essential. People generally tend to dislike large obstacles in their view, especial in rural areas. The old bulky lattice masts stood out in a negative manner. For this reason a revolutionary new type of mast was invented by architects. These conical tube masts are less prominent in the environment compared to the old lattice towers. The best alternative would of course be to use one mast instead of two, but as explained in chapter 2, these two masts are necessary to reduce the magnetic fields and to allow easier maintenance of the masts as well. These two tapered conical masts, were decided to have a significantly lower amount of visual impact compared to the old lattice masts. This reduction of pollution can eventually be used as an important factor in the choice between the old lattice masts or the new conical masts.

9. Costs

One of the most important aspects to consider when utilizing a new material is the cost. The lower the total costs (material, construction, maintenance, etc.), the more attractive it is to use. Currently, Ultra High Strength Concrete is an expensive material. Therefore initially it might not seem very attractive. But one must take into account the amount of material that can be saved when using higher strength concretes. In some cases this can compensate for the high price or even prove to be the better alternative. Aside from comparing the concrete variants with each other, they will be compared with both the traditional steel truss masts and the new steel tube masts as well.

The costs covered in this chapter give a good indication of the real costs, but it should be noted that prices can fluctuate, as well as differ from manufacturer to manufacturer. For the applied price list see appendix V.

9.1 Material costs

The material costs of the concrete transmission tower can be divided into four main parts:

1. Concrete segment production (including production costs)
2. Foundation (block & piles)
3. Reinforcement (mast, foundation block and piles)
4. Prestressing steel

The costs for all these parts were calculated by either multiplying the total amount of volume or weight of the material with their unit price (Table 62). For a complete overview of the costs see appendix W-AA.

The costs for the steel truss mast are derived by assuming a standard thickness value of 12 mm for the steel sections (see appendix Z). They make up about 50% of the primary direct costs.

Material costs: Steel truss mast				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Steel sections S235	€ 2.50	/kg	23786	€ 59.465
Galvanizing process	€ 80	/m^2	190,59	€ 15.247
Foundation block(s) C30/37	€ 80	/m^3	96,00	€ 7.680
Foundation piles C45/55	€ 35	/p	60,00	€ 2.100
Reinforcing steel B500B	€ 1	/kg	7488,00	€ 7.488
Total material costs				*€ 91.981*

Table 60: Material costs for steel truss mast

The costs for the steel tube masts are derived by assuming a standard thickness value of 17 mm for the wall thickness (see appendix AA). They make up about 65% of the primary direct costs.

Material costs: Steel tube mast				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Steel segments S355	€ 2,00	/kg	56075	€ 112.149
Rolling process	€ 40	/m²	483,49	€ 19.340
Coating	€ 50	/m²	483,49	€ 24.175
Foundation block(s) C30/37	€ 80	/m³	107,76	€ 8.621
Foundation piles C45/55	€ 35	/p	50,00	€ 1.750
Reinforcing steel B500B	€ 1	/kg	8405,02	€ 8.405
Total material costs				*€ 174.439*

Table 61: Material costs for steel tube masts

Table 62 shows the material costs for the UHSC variant. For the OSC and HSC variant see appendix W & X. They make up about 50% of the primary direct costs.

Material costs: UHSC mast				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Concrete segments C170/200[13]	€ 1.110	/m³	29,96	€ 33.253
Steel moulds for production	€ 250	/m²	253,40	€ 63.351
Foundation block(s) C30/37	€ 80	/m³	122,15	€ 9.772
Foundation piles C45/55	€ 35	/p	50,00	€ 1.750
Reinforcing steel B500B	€ 1	/kg	9527,32	€ 9.527
Prestressing steel Y1860S	€ 4	/kg	8579,00	€ 34.316
Total material costs				*€ 149.587*

Table 62: Material costs for UHSC masts

As can be seen from Table 60, the material costs for the steel truss mast are relatively low. For the steel tube masts, the material costs are mostly dependent on the raw steel costs, while for the concrete masts the production costs have the biggest impact. Furthermore, when comparing HSC and UHSC with the steel tube mast, it can be seen that the material costs for concrete are lower than for steel. Figure 120 gives an overview.

[13] Material costs of the UHSC segments consist of the concrete costs and the steel fibers costs.

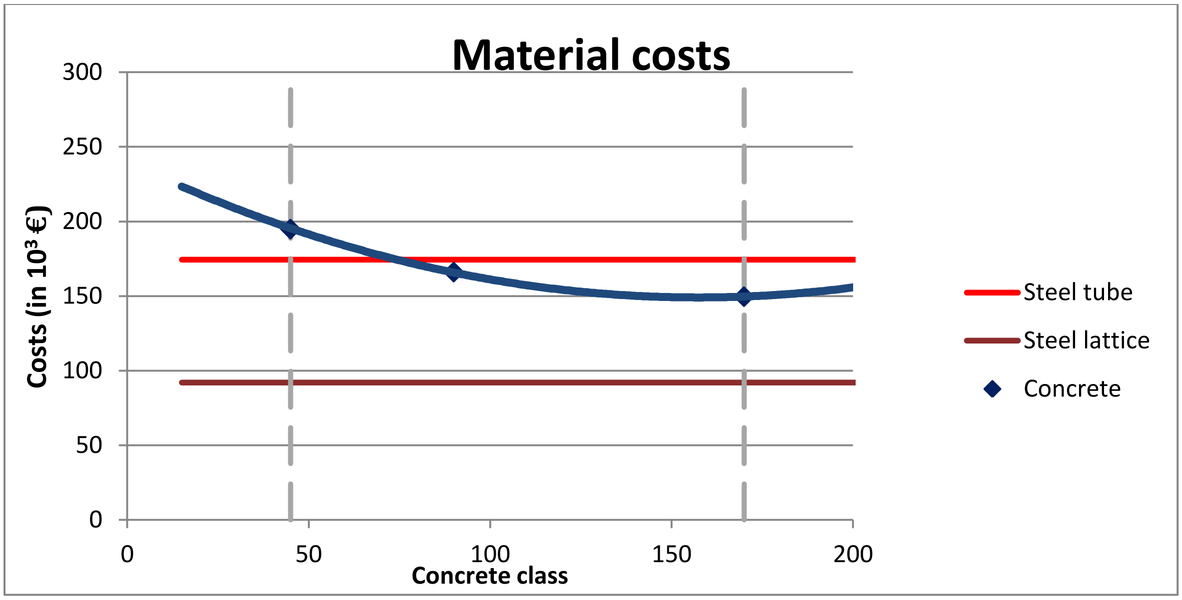


Figure 120: Material costs comparison

9.2 Building costs

For the building costs of the transmission tower two main parts can be distinguished:

1. Building site activities
2. Equipment

Ad 1: Building site activities
The building site activities include preparing the working site by: Removing weeds, applying a work floor, building an access road, etc. The activities include the landscape restoration as well.

Ad 2: Equipment
This part includes all machines or devices used outside of the fabrication process.

The building costs for the steel truss mast make up about 40% of the primary direct costs.

Building costs: Steel truss mast				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Building site preparation	€ 5	/m^3	1640	€ 8.200
Work floor	€ 25	/m^2	200	€ 5.000
Foundation activities	€ 350	/m^3	63,62	€ 22.266
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 300	/unit	3	€ 900
Main crane	€ 2.500	/day	2	€ 5.000
Support crane	€ 1.500	/day	2	€ 3.000
Connections[14]	20%	perc.	€ 59.465	€ 11.893

[14] Percentage of material

Landscape restoration	€ 5	/m^3	1640	€ 8.200
Total building costs				*€ 71.959*

Table 63: Building costs for steel truss mast

The building costs for the steel tube mast make up about 25% of the primary direct costs.

Building costs: Steel tube mast				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Building site preparation	€ 5	/m^3	1640	€ 8.200
Work floor	€ 25	/m^2	200	€ 5.000
Foundation activities	€ 350	/m^3	53,01	€ 18.555
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 500	/unit	4	€ 2.000
Main crane	€ 2.500	/day	1	€ 2.500
Support crane	€ 1.500	/day	1	€ 1.500
Connections[1]	20%	perc.	€ 112.149	€ 22.430
Landscape restoration	€ 5	/m^3	1640	€ 8.200
Total building costs				*€ 75.885*

Table 64: Building costs for steel tube masts

The building costs for the UHSC variant makes up about 40% of the primary direct costs.

Building costs: UHSC mast				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Building site preparation	€ 5	/m^3	1640	€ 8.200
Work floor	€ 25	/m^2	200	€ 5.000
Foundation activities	€ 350	/m^3	53,01	€ 18.555
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 300	/unit	8	€ 2.400
Main crane	€ 2.500	/day	3	€ 7.500
Support crane	€ 1.500	/day	3	€ 4.500
Connections[15]	20%	perc.	€ 67.569	€ 40.541
Prestressing per layer	€ 5.000	/layer	3	€ 15.000
Landscape restoration	€ 5	/m^3	1640	€ 8.200
Total building costs				*€ 132.396*

Table 65: Building costs for UHSC masts

[15] Percentage of material + prestressing

The building costs for the steel truss mast are the lowest, with the steel tube mast having slightly higher costs. In comparison the concrete variants are much higher. Figure 121 gives an overview.

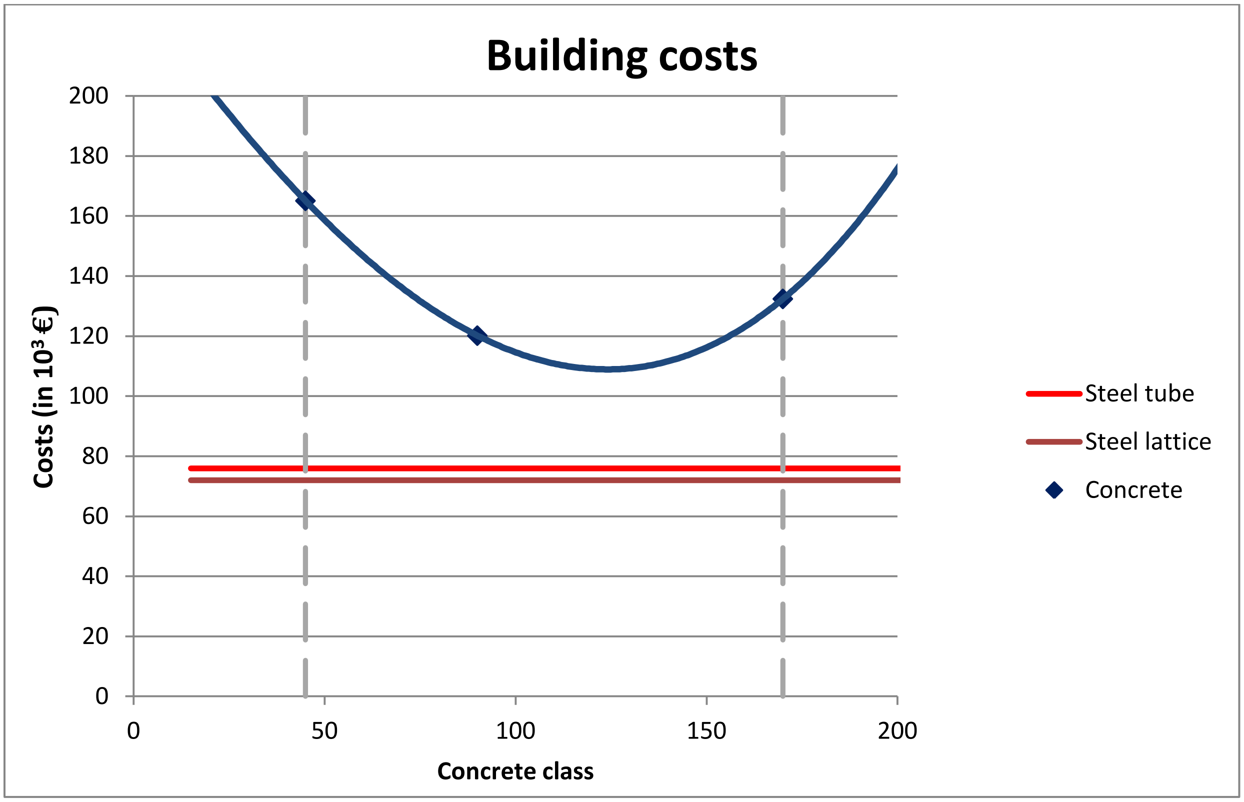


Figure 121: Building costs comparison

The building costs are heavily influenced by delays in construction. These undesirable delays combined with the high rental prices of equipment, can result in high building costs, surpassing even the material costs. Therefore the construction time or speed should be carefully monitored.

9.3 Labor costs

Labor costs are exactly what the name implies namely, the costs of the labor done by the workers who actually construct the product. In other words the amount of funds that must be kept aside to pay the workers. It is usually quantified by looking at the effort hours for each labor. The total amount of these so called man-hours can then be converted into monetary values. In the past labor costs tended to be fairly low and had almost no influence on the choice between alternatives, but nowadays labor is more expensive and must be monitored carefully to ensure that the total costs do not substantially increase. The costs found in the following tables were calculated with a value of € 40 per man-hour.

For the steel truss masts the labor costs make about 10% of the primary direct costs.

Labor costs				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Labor man-hours steel sections S235	0.006	mh/kg	23786,02	€ 5.709
Labor man-hours foundation	0.1	mh/m³	156,00	€ 624
Labor man-hours reinforcement	0.01	mh/kg	7488,00	€ 2.995
Labor man-hours assembly	0.01	mh/kg	23786,02	€ 9.514
Total labor costs				€ 18.842

Table 66: Labor costs for steel truss mast

For the steel tube masts the labor costs make about 10% of the primary direct costs.

Labor costs				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Labor man-hours steel segments S355	0.003	mh/kg	56074,51	€ 5.607
Labor man-hours coating	0.025	mh/m²	483,49	€ 483
Labor man-hours foundation	0.1	mh/m³	157,76	€ 631
Labor man-hours reinforcement	0.01	mh/kg	8405,02	€ 3.362
Labor man-hours assembly	0.005	mh/kg	56074,51	€ 11.215
Total labor costs				€ 21.299

Table 67: Labor costs for steel tube mast

For the concrete masts the labor costs make about 10% of the primary direct costs.

Labor costs				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Labor man-hours concreting segments	0.13	mh/m³	29,96	€ 150
Labor man-hours foundation	0.1	mh/m³	172,15	€ 689
Labor man-hours reinforcement	0.01	mh/kg	9527,32	€ 3.811
Labor man-hours assembly	€ 4	mh/m³	29,96	€ 4.793
Labor man-hours prestressing	0.05	mh/kg	8579,00	€ 17.158
Total labor costs				€ 26.600

Table 68: Labor costs for UHSC mast

As expected the labor costs for all variants are fairly low. For the concrete variants they seem to be slightly higher, though not significantly so.

For an overview of how the material, building and labor costs impact the total direct costs for each variant, see Figure 122.

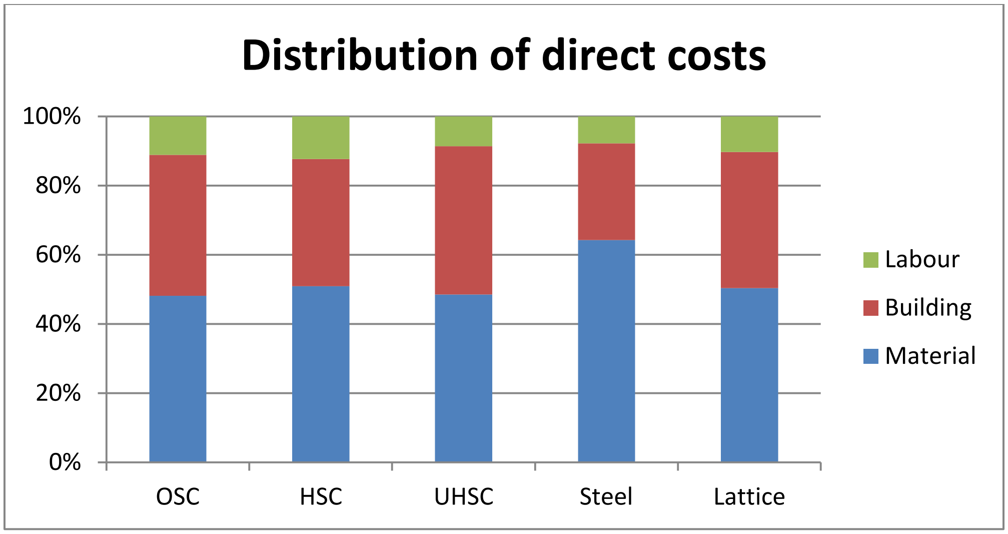


Figure 122: Distribution of direct costs

9.4 Maintenance costs

Maintenance costs are part of the indirect costs and are thus not directly attributable to the structure. Moreover they are not a onetime investment, but costs that continue for the entire lifespan of the structure.

For the concrete masts, the need for maintenance decreases as the concrete strength class increases. For UHSC the necessary unforeseen maintenance is practically zero. Because of the tight packing of UHSC, the permeability is extremely low. The amount of carbonation that occurs is negligible as well. This results in an extremely low maintenance of the concrete mast itself. The only real maintenance necessary is of the tendons inside the mast as well as the anchorages. Because the tendons are applied externally, they can simply be checked e.g. on corrosion. However in case a tendon must be replaced it will be somewhat difficult, especially at higher levels. In this case it would be best to utilize a monojack, because of lack of internal space.

In comparison the steel variants will need quite a lot of maintenance. The coating layer on the steel masts will have to be replaced periodically every 10-20 years. If a life span of about 100 years is expected for the masts, then this means that the coating will need to be replaced at least 5-10 times. The same principal applies for the connections. And in case a part of the steel mast succumbs to corrosion, the maintenance costs will be immense. Although when comparing the steel tube with the steel truss, the steel tube needs much less maintenance, due to the smooth surface of the mast.

To accurately quantify the maintenance cost, it is best to make use of present value calculation. Present (discounted) value is defined as the actual worth of a future amount of funds, that has been discounted over a specific time duration, to reflect its current worth. To calculate the present value the following basic formula can be used:

$$PV = \frac{FV}{(1+i)^t}$$

with:
PV = Present Value
FV = Future Value
i = effective annual interest rate
t = time period

It is clear from the equation that the interest rate (i) has a large influence on the result. The calculation of this value is a complex, economic problem. To find the correct value goes beyond the scope of this thesis. Therefore a value of 2.5% will be used, which has been known to provide adequate results.

The maintenance costs are illustrated in Table 69 - Table 71 and Figure 123 gives an overview.

Maintenance costs				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Conductors[16] 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 10x/100 years	€ 25	/m^2	190,59	€ 22.716
Platforms etc. 10x/100 years	€ 216	/day	15	€ 15.446
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Attachments (% of material) 10x/100 years	20%	perc.	59465	€ 34.602
Total maintenance costs				€ 102.731

Table 69: Maintenance costs for steel truss mast[17]

Maintenance costs				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Conductors[3] 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 5x/100 years	€ 5	/m^2	483,49	€ 4.502
Recoating 5x/100 years	€ 25	/m^2	483,49	€ 22.508
Platforms etc. 5x/100 years	€ 216	/day	5	€ 2.011
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Attachments (% of material) 10x/100 years	20%	perc.	22430	€ 13.052
Total maintenance costs				€ 72.040

Table 70: Maintenance costs for steel tube masts[18]

[16] Maintenance, replacement, etc.
[17] Note that all values have already been discounted.

Maintenance costs				
Material/Activity	Cost unit rate	Unit	Quantity	Total
Conductors[3] 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 2x/100 years	€ 5	/m^2	519,31	€ 922
Surface treatment 2x/100 years	€ 5	/m^2	519,31	€ 922
Platforms etc. 1x/100 years	€ 216	/day	5	€ 384
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Connections (% of material) 10x/100 years	10%	perc.	33253	€ 11.795
Prestressing 1x/10 years	€ 50	/tendon	16	€ 2.328
Total maintenance costs				€ 46.317

Table 71: Maintenance costs for UHSC masts[19]

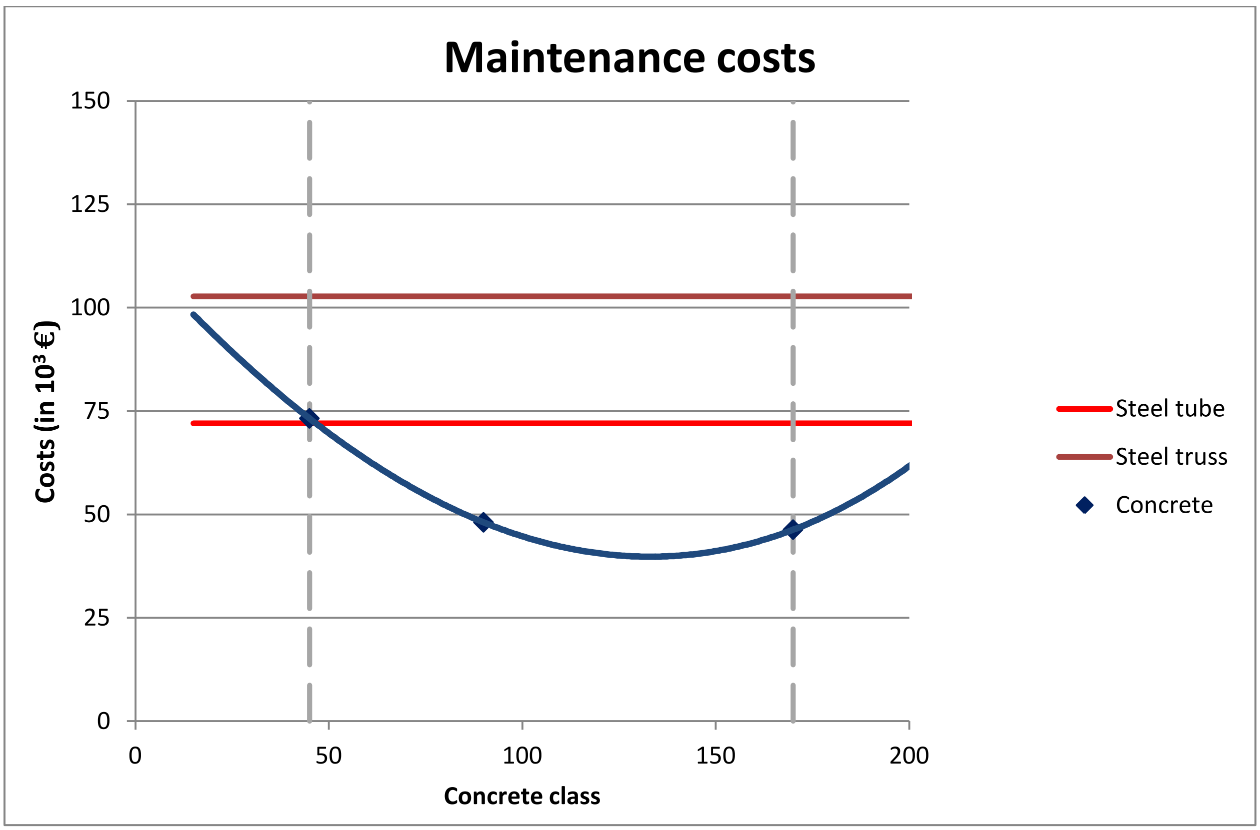


Figure 123: Maintenance costs comparison

An overview for the concrete variants of all the costs discussed previously is given in Figure 124.

18

[19] Note that all values have already been discounted.

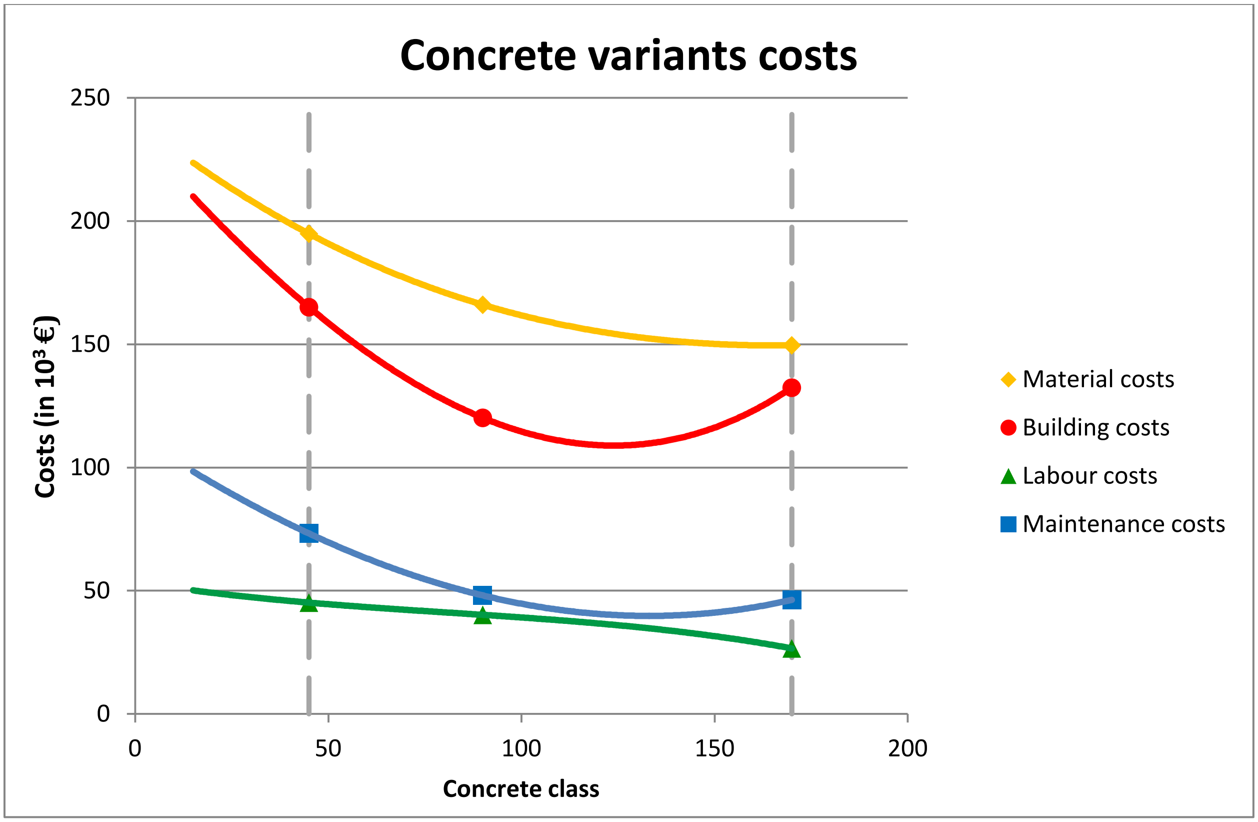


Figure 124: Concrete variants cost comparison

9.5 Cost comparison

As stated before, two comparisons will be made. First the total costs of the concrete variants will be compared to see how the different strength classes influence the costs. Afterwards the total costs of the concrete and the steel variants will be compared. For a complete overview of all the costs see appendix W-AA.

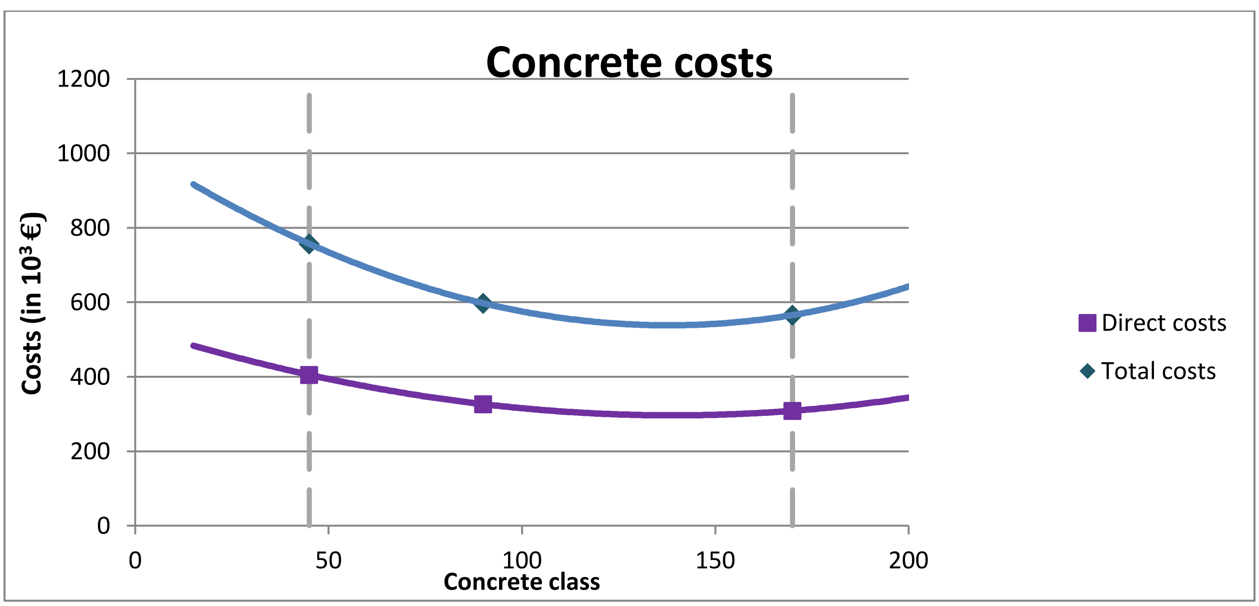


Figure 125: Total cost comparison for concrete variants

As can be seen from Figure 125, the total costs decrease as the concrete strength class increases. There is a large reduction from OSC to HSC, while the range between HSC and UHSC is shown to be more constant. Furthermore it can be seen that around C130 or C140 an optimum is reached, after which the cost increases again.

Note : As the concrete strength increase to values above C170, it is expected that the costs will increase significantly, because the advantages of the stronger material cannot be fully utilized in the design anymore, resulting in high initial material costs (high price per m^3).

When comparing the concrete variants with the steel variants, it is clear that as previously seen the steel variants score better on some aspects, such as building costs, while the concrete variants come out superior in terms of other aspects such as maintenance.

When observing the total direct costs (Figure 126), it can be seen that concrete is more expensive than the steel variants and that the steel truss variant comes out very cheap.

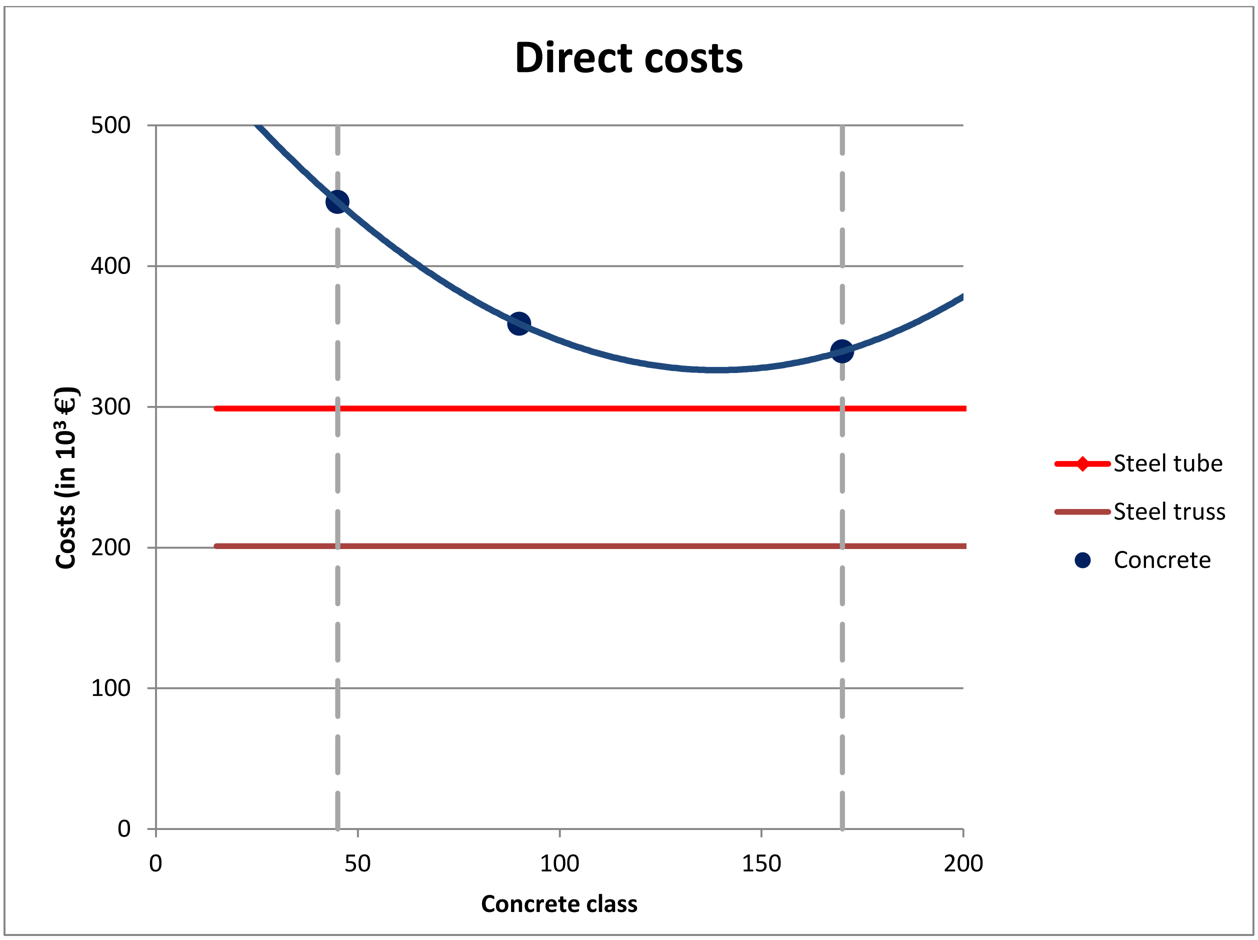


Figure 126: Direct costs comparison

However when we include the indirect costs (maintenance) and look at the complete picture, the result changes somewhat (see Figure 127).

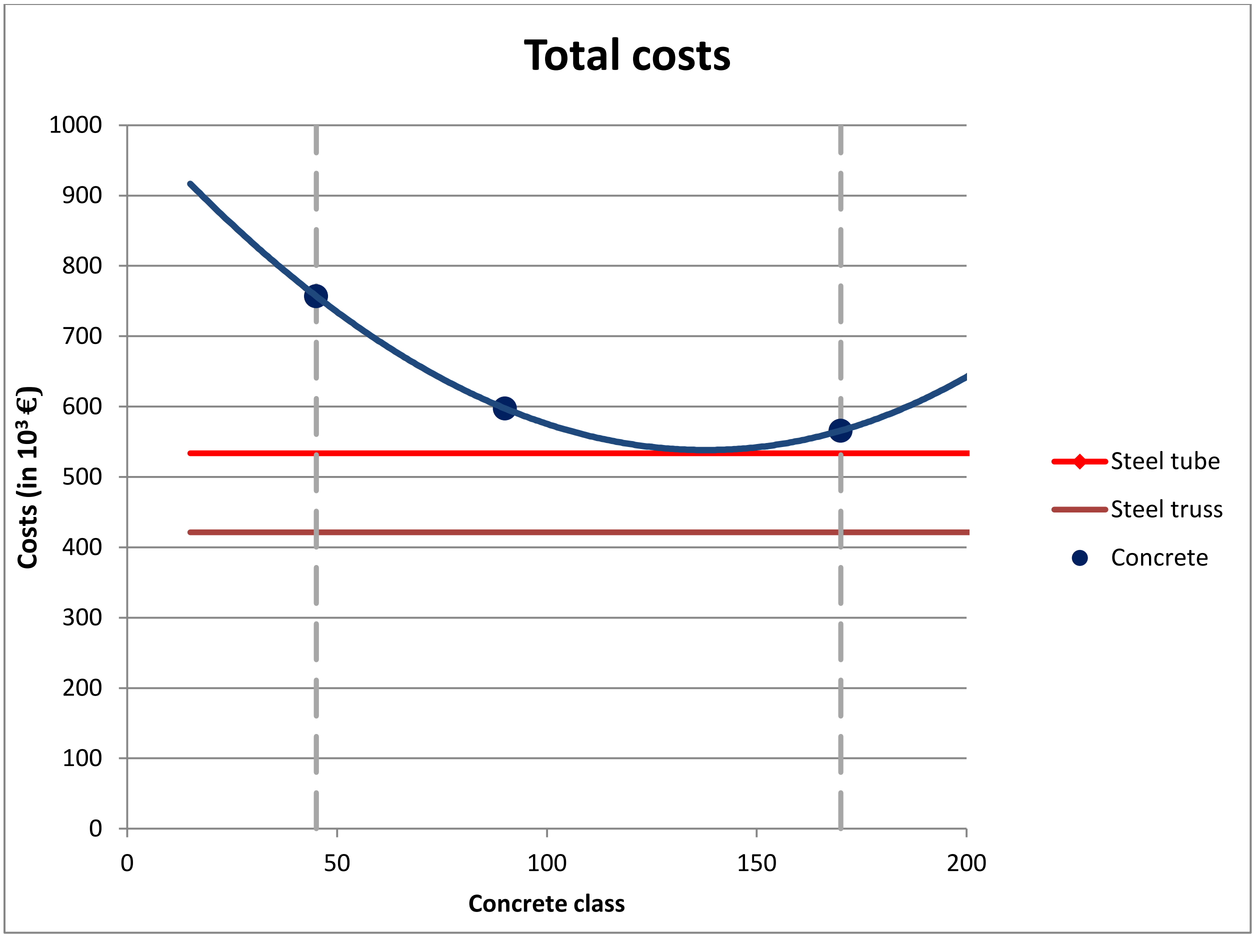


Figure 127: Total costs comparison

It can be observed that the variants start to close in on each other. Because of the high maintenance necessary for the steel truss, the total costs increase quite significantly. However even with this significant increase in costs, it remains the most economic result. When comparing the steel tube and the concrete variant, it can be seen that the concrete variants result has improved. The costs of the steel tube and concrete close in on each other, with the optimum of concrete being nearly equal with the steel costs.

Note: The cost comparison is not without faults, as the alternatives are compared using the same (or similar) initial dimensions. However in reality each alternative will have its own optimized dimensions.

10. Comparison

In order to compare all the variants with each other the previous chapters have already discussed aspects such as design, energy emissions, costs etc. Now all those results must be compared with each other to get a complete picture of the advantages and disadvantages of each variant. To do so a Multi Criteria Decision Analysis (MCDA) as well as a Value Engineering (VE) will be applied.

10.1 Performance criteria

Choosing the criteria

In the previous chapters the variants have already been compared on aspects such as sustainability and costs. These are however just a few of many possible criteria. In large projects such as these, the client always has certain demands. These demands are then translated in certain criteria that have to be met in a certain degree. Thus before the comparison of the variants can begin, the criteria by which they will be compared should be decided upon. These criteria can be anything ranging from safety to necessary construction time.

We can distinguish nine main criteria:

1. Technical feasibility
2. Magnetic field influence
3. Execution feasibility
4. Nuisance
5. Sustainability
6. Unforeseen maintenance
7. Landscape integration
8. Marketability
9. Costs

Most of these criteria are composed of several sub criteria.

Ad 1. Technical feasibility

<u>Strength</u>

The strength of a structure is the ability of a structure to adequately support and resist the acting loads (axial or shear stress, bending, torsion, etc.). This strength depends mostly on the material properties. The structure fails when the occurring stress (or strain) exceeds the capacity of the materials. As previously seen in chapter 4, the governing criteria for the concrete variants will most likely be either strength or stiffness. If strength is governing, that means that the stiffness of the structure is adequate and that the strength criterion is the one limiting the optimal dimensions. Thus strength would score worse when compared with stiffness.

<u>Stiffness</u>

The stiffness of a structure can be defined as the extent to which the structure can resist deformations caused by an external force. Other common names for it are flexibility or rigidity. It is mostly dependent on the material properties and geometry. For a cantilevered column, the deflection at the top will most likely be the governing deflection. The masts must thus be able to accommodate or limit this deflection. Depending on how well this deflection criterion is satisfied, the variants can be scored.

<u>Stability</u>
The definition of stability is the ability of a system to recover an equilibrium state upon being disturbed by any allowed perturbation. Some factors that influence column stability are initial curvature, initial bending moments and load eccentricity. Buckling can occasionally occur as well. If stability is not taken into account, the structure might collapse.

Ad 2. Magnetic field influence
As already mentioned in chapter 2, the influence of magnetic fields on health is an important subject. Therefore reducing the magnetic fields is of the highest importance. Masts that have a lower magnetic field influence, are safer for the public and will thus receive less resistance upon application. It is generally known that the steel truss masts have a large magnetic field, which means that in terms of health concerns, they are not very attractive to implement. This is especially true, when considering that the new norms prescribe a much lower limit for the magnetic fields than in the past. In this respect the tube masts will score significantly better.

Ad 3. Execution feasibility
<u>Transport</u>
The transport of materials, segments, equipment, etc., from the factory to the building site is an aspect that must be taken into account as well. Depending on the distance to be travelled and the product to be transported, a variety of options ranging from trucks to helicopters exist. If segments have unnatural shapes, transport will be more expensive as well since special transport has to be used, which might need police escort etc. The variants are all built up out of a different amount of segments as well, which will result in more or less necessary transport.

<u>Labor</u>
Labor is usually defined as the necessary amount of hours that workers need to produce a component or structure. The more man-hours are necessary, the larger the amount of workers need to be or the longer a project will take.

<u>Difficulty</u>
The difficulty of assembling a component can be expressed as the product of the total cost of the component and the required effort to construct that component. The higher the product of cost and effort, the more difficult it is to construct the component. Thus the more difficult a variant is to construct, the lower the score.

<u>Construction time</u>
The element time plays a large role in construction. Any delays costs money, and especially cases of bottlenecks are extremely expensive. Longer construction times means longer renting of equipment, land, labor, etc. Thus variants that can be erected quickly are very attractive.

<u>Safety/Risk</u>
Construction site safety is of great importance in the construction industry. Fatal and non-fatal accidents tend to happen at astonishing rates. Therefore it is of the utmost importance that the safety of the structure, during and after construction, is guaranteed and that the workers are not exposed to hazardous environments.

Ad 4. Nuisance
Often during construction, surrounding residents complain about the noise generated. Therefore structures with a minimal impact on the surrounding environment are often preferred. To prevent the noise disruption a couple of measures can be taken, such as limiting the hours of noisy work, use sound barriers, etc. Aside from noise other frequent problems are the dust generation, burning or air pollution, deformations of the surrounding land. As the variants have differences in erection methods, this might lead to differences in how the nuisance is experienced by surrounding residents.

Ad 5. Sustainability
<u>Durability</u>
Durability of structures is increasingly important for structures with a long life span. The structures must be able to resist all kinds of accidental loads, chemical attacks, dehydration or fire hazards. The more durable a structure, the less maintenance is required.

<u>Sustainability</u>
The impact of the structure on the climate is becoming a more important topic. Many companies like to boast of green products nowadays. Thus sustainable structures are becoming more and more attractive. As previously seen in chapter 8, the variants were compared in terms of embodied energy and embodied emissions. Based on those results, the variants can be scored on sustainability.

Ad 6. Unforeseen maintenance
Maintenance is a crucial aspect of structures. Maintenance and repairs tend to be very costly. Thus care should be taken that the structure needs the least amount of maintenance as possible. Furthermore, the need for maintenance is a critical point in a comparison between concrete and steel. In general steel tends to have more maintenance than concrete, due to the high risk of corrosion. However that does not simply mean that the steel variants should be scored worse, as other factors play a role as well. For example the concrete variants need less surface maintenance, but have prestressing tendons which need to be checked regularly.

Ad 7. Landscape integration
This subject deals with how people conceive the structure. As the definition of beauty differs from person to person, the aesthetics of a structure are a very subjective topic, but are nonetheless increasingly important for structures with a long lifespan. It is a well-known fact that the current steel truss masts are experienced as an irritation in the landscape by many people. Thus they will most certainly score worse that the innovative, sleek tube masts.

Ad 8. Marketability
The marketability of the product is the measure of whether a product is in demand by the market i.e. is appealing to buyers. It is directly related to the benefits that will be gained by utilizing that product, as well as the risk that comes with it. Marketability is also related to how much of an innovation the product is. If a product is adequate in terms of technical and execution aspects, but is not attractive for clients, than chances of implementation are very low.

Ad 9. Costs
<u>Direct costs</u>
The direct costs are the costs that can most easily be identified at the start of a project. They are therefore the first indication of the amount of money that must be invested into the structure. High direct costs will tend to discourage clients, because of the substantial initial investment..

<u>Total costs</u>
The total costs of a structure are comprised of direct (material, building costs, etc.) and indirect costs (maintenance, profit, risk, etc.). In other words the total amount of money that will go into the production (and maintenance) of a structure. The total costs are therefore essential and usually a deal breaker, to decide if building the structure is worth it or not.

Weight of the criteria
Now that the criteria have been decided, the importance of each criteria should be decided, as some criteria will be more important than others. To attain the weights of the criteria, each criterion is compared with the others and given a grade that marks it either as superior, equal or inferior to the other criteria in terms of relevance (respectively 2, 1 or 0). This method of applying weights is called the Paired Comparison method. Because some criteria might end up with a total score of zero, a correction factor can be applied to account for this. In this case, one of the criteria (nuisance) ends up with a so called zero-score, so a correction factor is necessary. An overview of the process can be seen in appendix AB. A summary of the weights is given in Table 72.

Criteria	**Weight**
Technical feasibility	5%
Magnetic field influence	17%
Execution feasibility	11%
Nuisance	2%
Sustainability	9%
Unforeseen maintenance	15%
Landscape integration	11%
Marketability	10%
Costs	20%

Table 72: Summary of the resulting weight of the criteria

These values can then be utilized for further analysis.

Note: The values as specified in Table 72 and appendix Z, are normally the result of a brainstorming session with various parties, so an objective result can be obtained. As holding such a session falls outside the thesis parameters, the values were obtained by the graduate after discussion with some people knowledgeable in the field.

10.2 MCDA I

Multiple-criteria decision analysis (MCDA) is, just as the name implies, an analysis that considers several criteria in a decision making process. MCDA's are an incredibly handy tool to properly structure the problem and clearly evaluate multiple criteria. The main parameters needed to carry out a MCDA are:

- Variants;
- Criteria;
- Weight of the criteria;
- Scores of the variants.

Now that the criteria and the weights of the criteria have been decided, the variants can actually be compared with each other. The variants are scored with a point system ranging from one to nine, with:
1 = very bad
3 = bad
5 = neutral
7 = good
9 = very good
This score is then multiplied with the weight of the specific criterion, which gives an end result. By doing this for all criteria, a total result can be found for each variant, simplifying the comparative process.

For the first MCDA, the concrete alternatives, based on different strength classes, will be compared. The result can be seen in Table 73 & Figure 128.

		Traditional reinforcement		Steel fibers
Component	**Weight**	**OSC C45/55**	**HSC C90/105**	**UHSC C170/200**
Technical feasibility	5	3	7	7
Magnetic field influence	17	9	9	9
Execution feasibility	11	3	7	7
Nuisance	2	7	7	7
Sustainability	9	3	5	5
Unforeseen maintenance	15	5	5	7
Landscape integration	11	7	7	7
Marketability	10	5	7	9
Costs	20	3	7	7
	Total	**504**	**686**	**736**

Table 73: MCDA I: Concrete alternatives

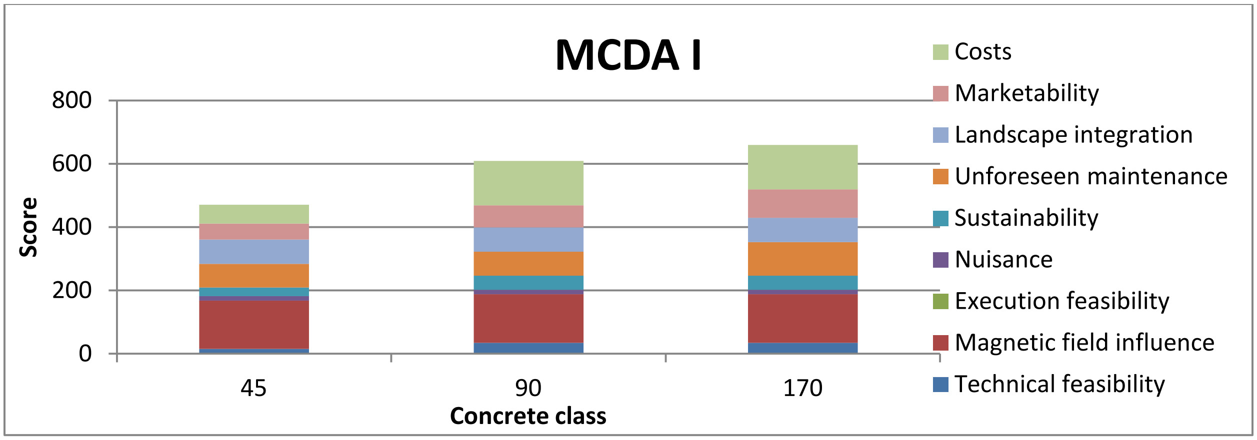


Figure 128: MCDA I - Concrete alternatives

It is clear from the illustrations above, that increasing concrete strength classes seem to have better overall performance.

Note: The scores as specified in Table 73, are normally the result of a brainstorming session with various stakeholders, to obtain an objective result As holding such a session falls outside the thesis parameters, the scores were obtained by the graduate after discussion with some people knowledgeable in the field.

10.3 MCDA II

Now that the result of varying concrete classes are clear, a comparison can be made between the different mast variants, namely the steel truss, steel tube and concrete tube masts. The result can be seen in Table 74 & Figure 129.

		Steel		Concrete		
Component	**Weight**	**Truss**	**Tube**	**OSC**	**HSC**	**UHSC**
Technical feasibility	5	9	7	3	7	7
Magnetic field influence	17	5	9	9	9	9
Execution feasibility	11	7	7	3	7	7
Nuisance	2	5	7	7	7	7
Sustainability	9	9	5	3	5	5
Unforeseen maintenance	15	3	3	5	5	7
Landscape integration	11	5	7	7	7	7
Marketability	10	5	7	5	7	9
Costs	20	9	7	3	7	7
	Total	**628**	**656**	**504**	**686**	**736**

Table 74: MCDA II - Comparison of variants

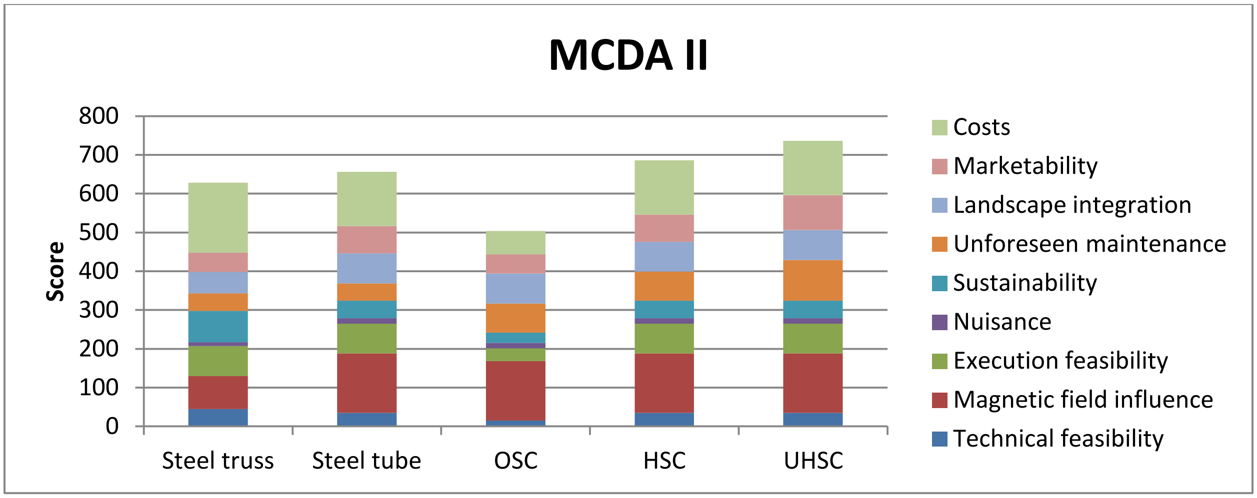


Figure 129: MCDA II - Comparison of variants

As can be seen from Figure 129, the differences between the steel truss, steel tube and HSC are minimal. They all have different aspects on which they score better than the others, but it seems to balance out in the total score. OSC seems to score the worst, while UHSC has the best score. It is clear that by only using a MCDA it is difficult to come to a final conclusion. Therefore the analysis will be taken on step further with the help of Value Engineering.

10.4 Value Metrics

The aim of Value Engineering (VE) is to find the optimal function or performance of a product, at the lowest cost. The "value" (or "worth") of a product is defined as the ratio between function and cost. The value of a product can thus be increased by either improving the performance of the product or by reducing the necessary costs. After this ratio has been determined, the different type of variants can more easily be compared with each other, thus making it easier to choose the best alternative. A Value Engineering study is a large process carried out with a multidisciplinary team.

One of the most frequently used tools in value engineering are the Value Metrics (VM). Value Metrics are techniques or methods that can be used to quantify the value. These VM basically go one step further than the MCDA. The criteria and their weights are decided in the same way and the total performance score is determined exactly the same way. However where the MCDA ends, VM continues by determining the ratio between performance i.e. quality of the product and the total costs that are needed to establish the product. The process is shown in Table 75. To illustrate the relative values, the HSC variant was taken as the baseline i.e. all the other variants are compared with HSC.

Note: Although the MCDA uses the costs as one of the criteria, a VM does not. A VM only uses the costs as one of the criteria, when the costs cannot be quantified.

Alternatives	Performance	Variation quality	Total costs (10^3 €)	Value P/C	Variation value (P/C)
OSC	552	81%	757	0.73	64%
HSC	684	100%	597	1.14	100%
UHSC	746	109%	566	1.32	115%

Table 75: Performance vs. total costs I

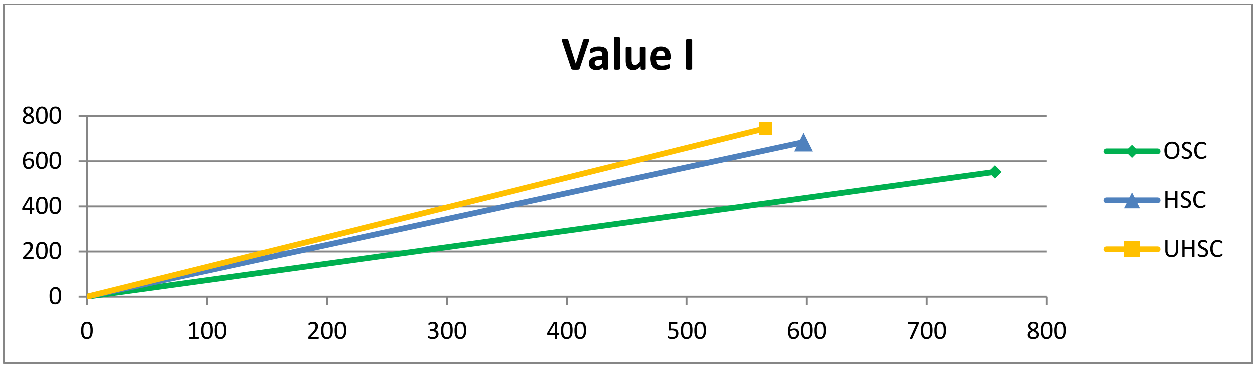


Figure 130: Quality-price ratio I

As can easily be seen from Figure 130, UHSC has the best quality-price ratio followed by HSC. It seems that if UHSC is utilized you can achieve a slightly better result than HSC for a similar (or even cheaper) price.

Note: The conclusion that higher concrete classes are better cannot simply be made for concrete strength classes higher than C170. This is because it is expected that the (production) costs will increase, because the advantages of the stronger material cannot be fully utilized in the design anymore, resulting in higher initial material costs. This higher total cost will then reduce the quality-price ratio, thus offering a similar quality for a much more expensive price.

The same principle can be applied for the second comparison (Table 76). As the steel truss variant is currently in use (and has been the traditional choice for a long time) and the steel tube and concrete tube mast variants are both relatively new ideas, the steel truss variant was taken as the baseline.

Alternatives	Quality	Variation quality	Total costs (10^3 €)	Value P/C	Variation value (P/C)
Steel truss	564	100%	421	1.34	100%
Steel tube	648	115%	534	1.21	91%
OSC	552	98%	757	0.73	54%
HSC	684	121%	597	1.14	86%
UHSC	746	132%	566	1.32	99%

Table 76: Performance vs. total costs II

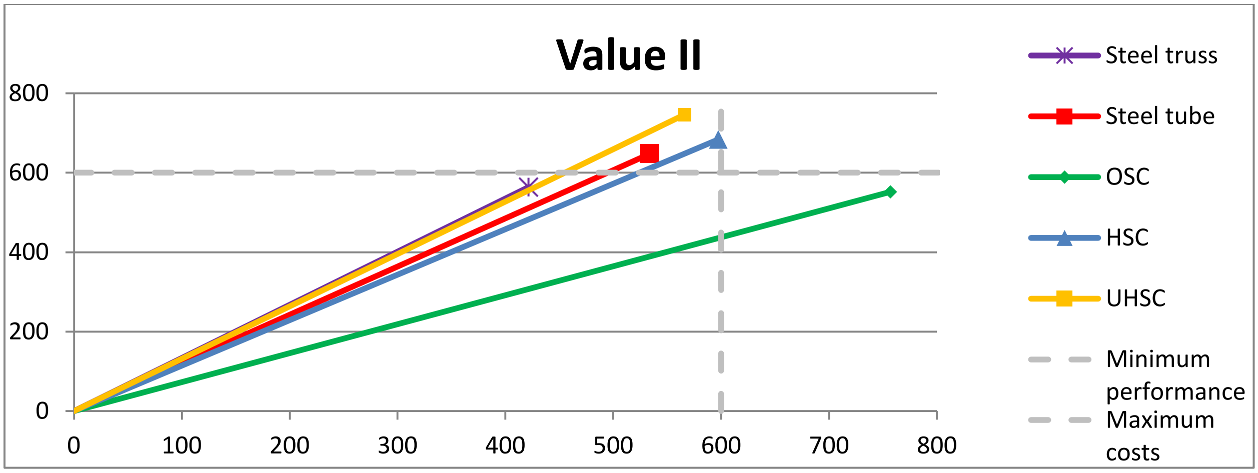


Figure 131: Performance-cost ratio II

To further illustrate the usefulness of VE, a minimum performance of 600 and a maximum cost of €600.000 was introduced. With these boundaries, it is obvious that only variants which fall into the second quadrant of the graph are considered suitable options.

As can be seen from Figure 131, OSC falls into the fourth quadrant and is thus unsuitable both in terms of quality and in terms of costs. The steel truss variant fall just outside of the second quadrant. All other variants fall into the second quadrant. The steel truss mast has the best price-quality ratio (easily seen due to having the largest gradient). However if a higher quality product is desired, the steel tube, HSC or UHSC masts provide the necessary quality for an increase in costs. The price for these three variant seems incredibly similar with the steel tube variant falling directly in between the HSC and the UHSC variant. It can easily be seen from the graph, that the steel tube offers a good performance for a low price, while the both HSC and UHSC offer better performances for higher prices.

Thus with the help of Value Engineering and Figure 131, the client can make an informed decision on which alternative is preferred. If the funds are limited, the steel truss is recommended. However if the client is willing to spend some more, he can receive a better product, with a higher quality than the steel truss mast.

Note1: Although Figure 131 shows a low performance for the steel truss mast (mainly due to high magnetic fields), it should be remembered that these masts score very good when it comes to sustainability and costs. So in case the masts need to be built in places where the magnetic fields are of less importance, this variant is the best(/cheapest) alternative. However as magnetic fields are a sensitive issue nowadays, it is most likely that this variant will be eliminated.

Note2: The boundary values of minimum performance and maximum cost, have no substantiation and are simply applied to illustrate the principle of VE. For instance if the minimum performance was set to 450 and the maximum costs to €750.000, then all variants would be eligible.

Note 3: As previously stated the weights of the various criteria where mostly decided by the graduate with input from others. If a client has different wishes and considers other criteria more important for a project with different premises and restrictions, than the results of Figure 131 might change.

10.5 Rankings

As previously seen in chapter 9, the variants were already compared in terms of costs. In this chapter two more comparisons are made, namely based on performance and based on performance/costs ratio. Thus a ranking could be introduced for the variants based on:

1. Lowest total costs
2. Best total performance
3. Best performance/costs ratio

Based on which ranking the client finds more important or wishes to utilize, he can then easily choose the best variant.

11. Conclusions & recommendations

11.1 Conclusions

The objective of this study is to assess whether UHSC as new material could be used in conical tube masts and the investigation focused on the following five key aspects:

- *Design*
- *Production*
- *Execution*
- *Sustainability*
- *Cost*

Design

It was demonstrated that OSC could not be constructed with its initial dimensions, in contrast to HSC and UHSC, where a small wall thickness was feasible as well.

Lowering the concrete strength class, lessened the importance of the stiffness criterion and for OSC and HSC, the governing norm is thus strength. On the other hand, the governing criterion for UHSC was stiffness and deflection is thus a leading criterion for the increasingly thin walled UHSC variant.

It was revealed that wall thickness refinement could be achieved over the height of the mast. This wall thickness reduction was the largest for OSC followed by HSC and finally UHSC.

Furthermore it was shown that prestressing in tube masts made of higher strength concrete should mainly be applied to eliminate tensile stresses in the concrete section, which will have a positive influence on the crack width as well. However, the application of prestressing has no positive influence on the amount of reinforcement for higher concrete strength classes.

Production

The best option from a production perspective was to use one mould per segment. For UHSC this equals only four moulds, which is a satisfactory amount, while for HSC it is six, which is also still an acceptable amount.

For the production process of OSC and HSC, two known methods were considered, while for the UHSC segments the possible options were mostly derived through theoretical conjectures, due to the complicating factor of steel fibers and the scarce research in this field.

Execution

The factors which influence the segment length are the maximal length and weight of the transport truck, the maximum stress possible during transport and maximum capacity of the crane.

Although the amount of segments (4) required for UHSC is twice the amount used for the steel mast (2), this is not a substantial increase.

It was demonstrated that the wall thickness especially for UHSC was too thin for internal prestressing and external prestressing was therefore the logical option. The external prestressing was also chosen to be reduced gradually over the mast height, by anchoring two tendons every segment.

In terms of assembly, prestressing will face the difficulty that the tendons should be held in the air with the help of a support crane, while the segments are lowered over them. After the segments are placed, anchoring the prestressing becomes increasingly difficult at higher levels, due to the decreasing working space. Although this practice somewhat complicates the assembly process it is not impossible.

Since most normal joint solutions for the segments could not be applied for a tube mast, solutions were searched applicable for this case. These solutions were adjusted and worked out and the one deemed best was the “console” connection.

The connection with the foundation could be realized by either applying a steel plate to the bottom segment and bolting this to the foundation, or the concrete itself could simply be bolted.

Sustainability
In order to assess the sustainability, both the embodied energy and embodied emissions method were utilized.
The steel truss mast is the superior option in both methods, followed by UHSC which scores better than both the steel tube and the HSC variant.
The UHSC tube mast offers its high durability and low maintenance as an additional advantage over the steel variants which have considerable maintenance.

Costs
It was revealed that the material costs for UHSC are high but this can be compensated because UHSC requires a low amount of material.
The steel truss mast turned out to be the cheapest alternative followed by steel tube masts. UHSC and HSC have similar costs just above the steel tube mast. The cost for higher strength concrete exhibits an optimum somewhere between the range of C130 – C160.

Comparison
The results from this study clearly substantiate that the construction of conical tube masts with UHSC is feasible and a comparative analysis for UHSC shows that concrete requires less maintenance and offers most durability.

UHSC is more innovative than the HSC variant, but neither HSC, nor UHSC proved to be the best possible alternative. Instead the optimum was found to be somewhere between the range of C130 – C160.

Overall, the steel truss proved to be quite a good alternative, with the high magnetic fields as the only significant drawback.

Based on aspects as technical feasibility and costs, the steel tube mast, the HSC and the UHSC seem fairly equal, while aspects as sustainability and durability favor the higher strength concrete, especially UHSC.

It is quite obvious, that the selection of the best alternative from all the options is not straightforward due to the many elements involved. Depending on the specific requirements or preferences, different choices are possible.

Based on the investigated variables the following ranking was presented in this study:

- In terms of costs, the steel truss is the best alternative, followed by the steel tube mast and finally the UHSC and HSC masts.
- In terms of performance the UHSC variant is the best alternative, followed by the HSC and steel tube masts and finally the steel truss mast.
- Based on the value of performance per cost, the steel truss mast has the best ratio, followed by the UHSC mast and finally the steel tube mast and the HSC variant.

11.2 Recommendations

- ➢ It is recommended that alternative shapes of the mast should be researched and worked out, especially focusing on optimization of the mast shape, prompted by the current imbalance. The load in the direction orthogonal to the conductors is considerably larger than the loads parallel to the conductors. A possible option is an elliptical shape for the outer diameter and a circular shape for the inner shape. In this alternative shape the wall thickness will be thicker in the main direction and thinner in the non governing direction. If for some reason the circular shape becomes a boundary condition, the reverse can be applied i.e. a circular shape for the outer diameter and an ellipse for the inside diameter. Besides the optimal shape for the mast, the corresponding optimal shape of the foundation should be investigated as well.

- ➢ The favorable aspects of the bipole configuration should be investigated in other possible configurations, which not only have the same architectural impression, but comply with the design criteria as well. A possibility could be a single mast, that branches out in 2 masts at a certain level or a portal configuration connecting the two masts, which would improve the deflection criterion.

- ➢ The limited scientific data on UHSC in production processes, due to its novelty as material, impedes its application. The execution of a couple small scale projects can generate vital information for future projects.

- ➢ It is recommended that options are explored to increase the initial dimensions (diameter at foot and top of the mast). Positive results will contribute to the possibility for dimensioning purely based on strength, since stiffness will no longer pose a problem. UHSC could subsequently be applied with even lower thickness values.

- ➢ The comparison of the concrete variants exhibited a certain point where the strength and stiffness criteria coincide, which was also the case in the costs comparison. Follow up research is recommended to search for this possible optimum C value of concrete strength between HSC and UHSC.

- The position as frontrunner for the steel truss mast is weakened by the high magnetic fields and its ghastly landscape integration. It is therefore recommended that more research is dedicated to discover solutions to lower the magnetic fields and to the remodeling of the bulky form.

- The lack of preset criteria, weight of the criteria and variant scores leads to potentially subjective set values. It would be beneficial to determine performance/costs values with all relevant stakeholders in a complete Value Engineering process.

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Appendices

Appendix A: Magnetic fields

Magnetic field zone

The magnetic field around the conductors is caused by the current travelling in the conductor and is inversely proportional to the distance from the center of the conductor i.e. the further away from the conductor, the lower the magnetic field (quadratic relation). The magnitude of the magnetic field is calculated with the 4th law of Maxwell (originally Ampère's law).

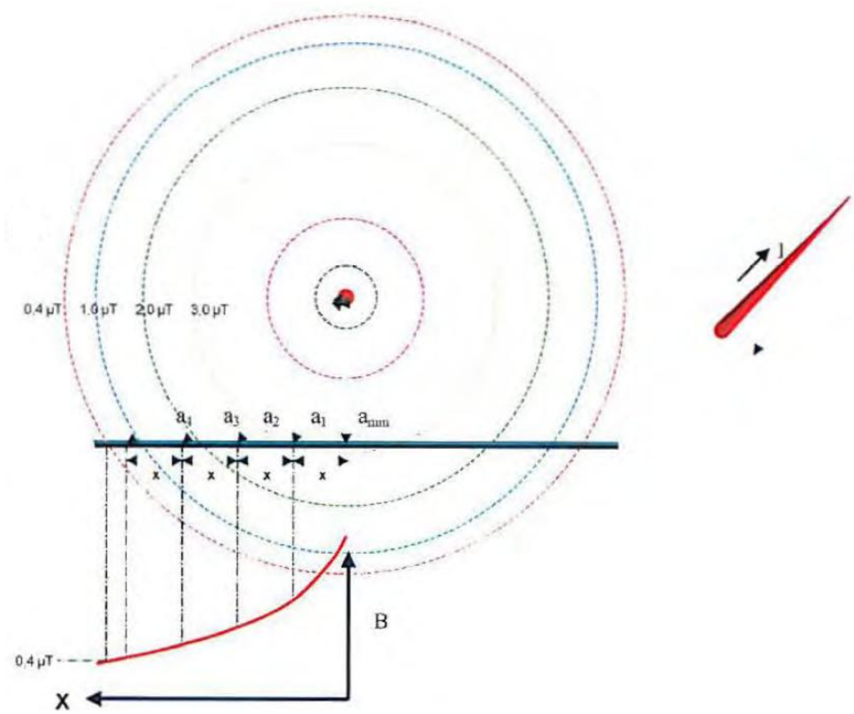


Figure 132: Magnetic field zone

Phase order and clock numbers

In the traditional transmission tower, the conductors are usually applied in a triangular form. These masts are usually suitable for two circuits, which means that at least six conductors (2 circuits x 3 conductors) contribute to the magnetic field.

The contribution of each phase to the magnetic field in a random point P differs, because the conductors are all applied at different distance from point P (see Figure 133).

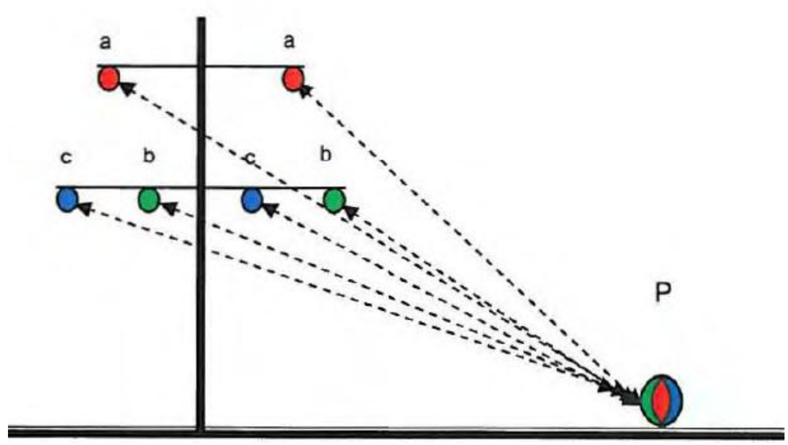


Figure 133: Symmetrically applied conductors

Because the phases of the three-phase current system are shifted 120 degrees with each other, the resulting magnetic field will be the vector result of the current magnitude, phase shift and the distance. As can be seen from Figure 134, phase b (green) has the largest influence, while phase c (blue) has the least influence.

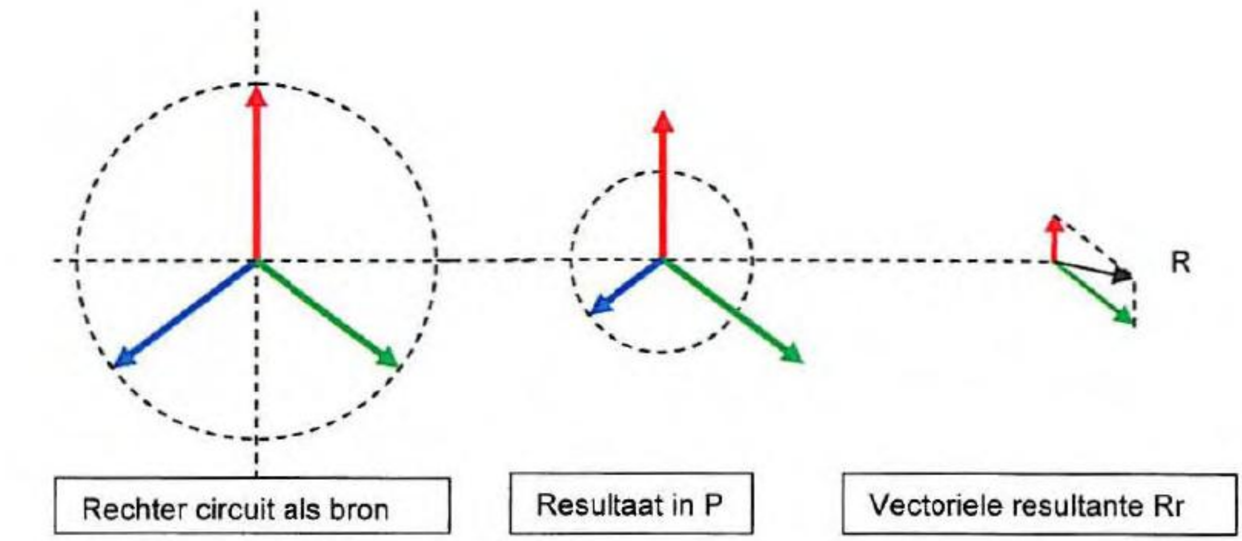


Figure 134: Phase shift of the conductors

In case the left circuit would utilize the same phase order as the right circuit, it is clear that the vector result, in point P, of the two circuits would be twice as big. If however the phases in the left circuit are shifted, by applying the conductors in different places in the mast, then the vector result will change. Now Phase a (red) has the largest influence, while phase b (green) has the lowest influence. The resultant of the left circuit now has a different phase angle when compared with the right circuit. This cause the total result to be smaller than in case the conductors where applied symmetrically (see Figure 136).

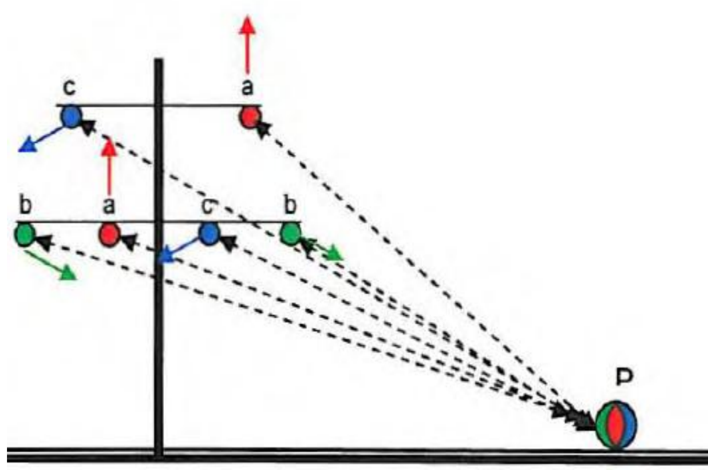


Figure 135: Optimal position of conductors

Figure 136: Total vector result: Symmetric (L) & optimal (R)

This asymmetric placing of the conductors to cause phase shift is also known as clock number modification. For masts with a triangular phase formation as shown before, the width of the 0.4 μT zone is in the order 200-300 m. Thus choosing an optimal configuration of the clock numbers can reduce the magnetic field zone with about 35%. This reduced magnetic field can be seen in Figure 137. It can clearly be seen that the magnetic field varies with the distance x from the points P_x.

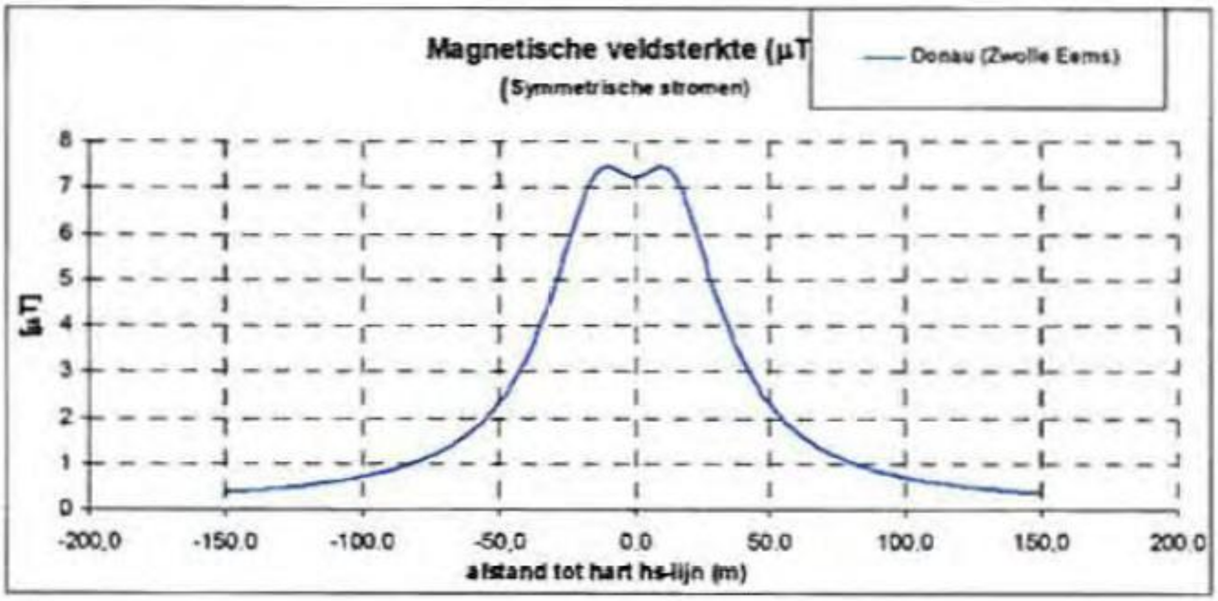


Figure 137: Reduced magnetic field due to clock number modification

Mast head

Based on the previous chapter, it can be concluded that smaller configurations of the phases are the most ideal possible configurations. Thus a circular formation of the phases could be the most optimal solution, as the distances to a point P would almost be the same for all conductors. The resultant vectors will most likely be similar in magnitude, causing the total resultant to reduce drastically. (see Figure 138).

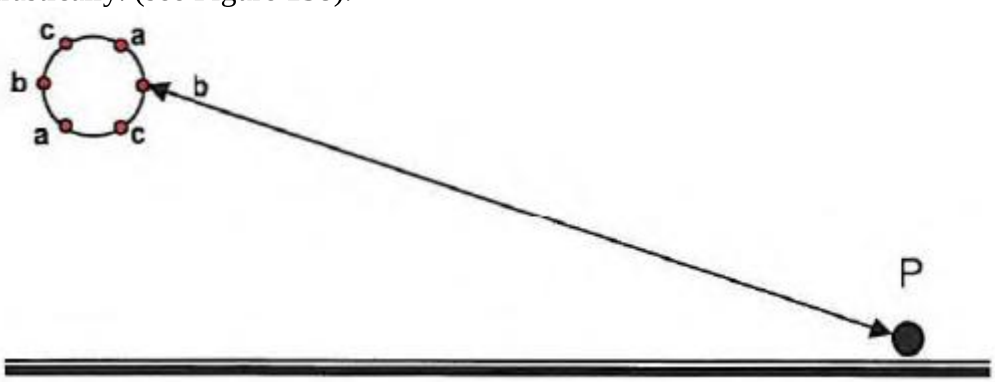


Figure 138: Circular configuration

As the conductors need to be kept in place in the air by the mast and because there are certain distances that must be maintained between the conductors and the mast, the circle will need a minimum diameter to still comply with all these requirements.

To keep the middle conductors in the circle perimeter, necessary provision must be applied in the mast head, so that the middle conductor can be applied further from the mast. However research has shown [37] that this effect is reduced drastically for large mast distances (350 & 400 m). Figure 139 shows that with regards to the 0.4 µT zone width, there is no significant difference between circular and rectangular configurations.

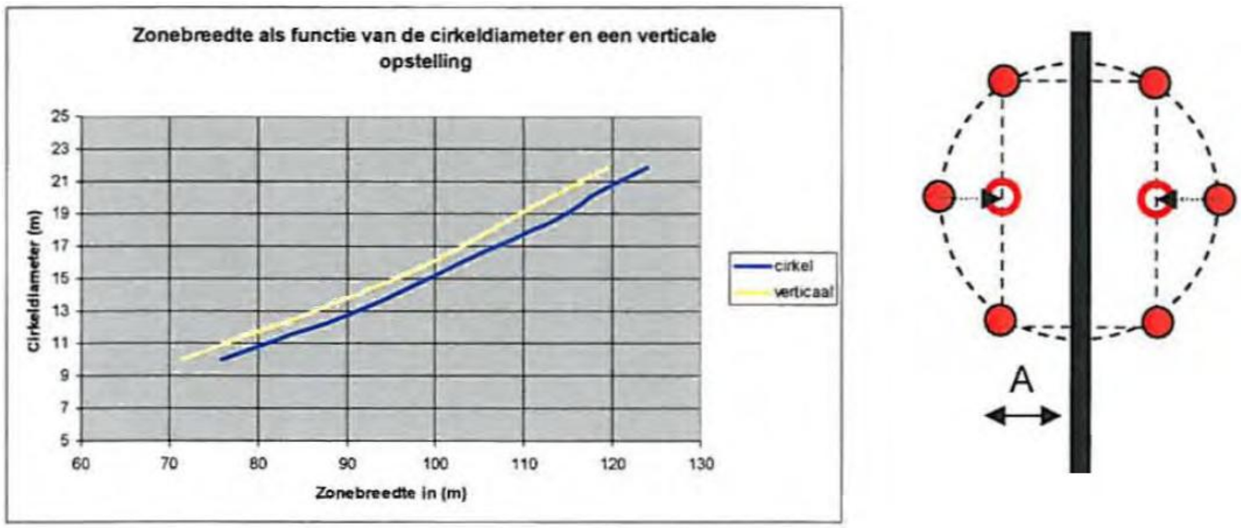


Figure 139: Circular vs. rectangular configuration

Another disadvantage of the two circuits on one mast is that the distance A has to be sufficiently large, so that the mast can be climbed for maintenances. This maintenance has to occur while the other circuit is still active. Thus the minimal circle diameter is mostly decided by the distance A required for safe climbing of the mast.

The mast type with one central mast thus has the disadvantage that due to the presence of the mast body, a minimum distance has to be allowed between the conductors of the circuits. This minimum distance therefore also leads to a minimum value of the 0.4 µT magnetic field zone width.

Bipole mast

A method to reduce the magnetic field zone is to reduce the horizontal distance between the conductors of each circuit. This can be realized by applying two masts instead of one and by applying one circuit to each mast. This type of mast is the so called bipole mast (Figure 140). By reducing the horizontal distance X between the two circuits and by attaching the conductors with braced-V isolators, the magnetic field zone can be reduced. For masts with four circuits the system holds as well.

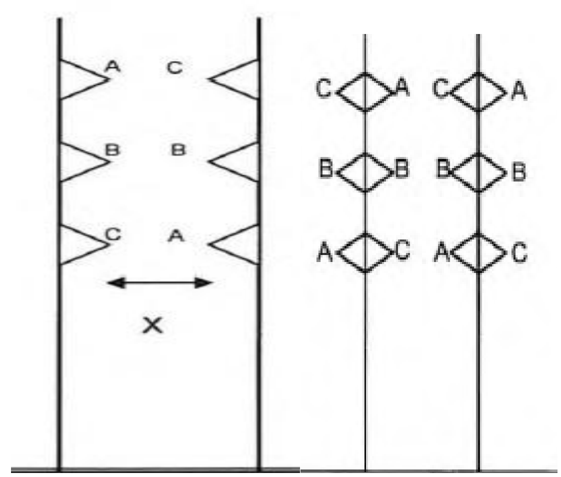


Figure 140: Bipole mast: Two circuits (L) & four circuits (R)

Appendix B: Slenderness

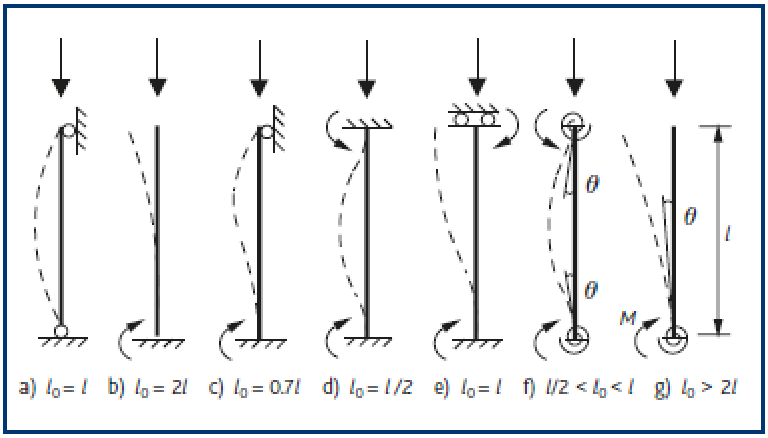


Figure 141: Effective length

$$l_0 = 2 * l = 2 * h = 114 \text{ m}$$

$$i = \sqrt{\frac{I}{A}} = 0.45 \text{ mm} \qquad (13.1)$$

$$\lambda = l_0 * \sqrt{\frac{8}{d}} = 239 \text{ m/m}$$

$$e_i = 1000 * \text{MAX}\left\{1.2 + \frac{l_0}{400}; \frac{d}{300}; 0.02\right\} = 1485 \text{ mm}$$

$$M_{01} = 0 + (N_{ed} * e_i) = 1093 \text{ kNm}$$

$$M_{02} = M_{ed} + (N_{ed} * e_i) = 12340 \text{ kNm}$$

$$\text{r}_m = \frac{M_{01}}{M_{02}} = 0.089 \qquad (13.2)$$

$$n = \frac{N_{ed}}{A_c * f_{cd}}$$

$$\lambda_{\text{lim}} = \frac{20 * A * B * C}{\sqrt{n}} = \frac{20 * 0.7 * 1.1 * (1.7 - r_m)}{\sqrt{n}} = 169 \text{ m/m}$$

The column is slender and 2nd order effects have to be taken into account.

Appendix C: Frequency

$$c = \frac{3.43 * E_{found} * I_{found}}{l_{r,rad}} = 2.89 * 10^{7} \text{ kNm/rad}$$

$$EI = E_{c} * I_{c} = 2.95 * 10^{15} \text{ Nmm}^{2}$$

$$m_{mast} = V * \rho_{c} = 42809 \text{ kg}$$

$$m_{stiffness} = 0.25 * m_{mast} = 11 \text{ ton}$$

$$k = \frac{1}{\frac{h^{3}}{3EI} + \frac{h^{2}}{c}} = 48 \text{ kN/m} \tag{13.3}$$

$$D = 0.02 \text{ (for concrete)}$$

$$f_{e} = \frac{\sqrt{\frac{k}{m_{stiffness}} * (1 - D)}}{2\pi} = 0.332 \text{ Hz}$$

$$\text{Ratio dimensions} = \frac{d_{foot}}{d_{top}} = 4.4 \qquad \text{OK (No vortex shedding)}$$

Appendix D: Wind load

$$\begin{aligned} &\text{Terrain categorie:II} \\ &\nu = 1.50 * 10^{-5}\ \text{m}^2/\text{s} \\ &\rho = 1.25\ \text{kg/m}^3 \\ &b = d_{av} = 1.35\ \text{m} \\ &z_e = h = 57\ \text{m} \\ &A_{ref} = 76.95\ \text{m}^2 \end{aligned} \quad (13.4)$$

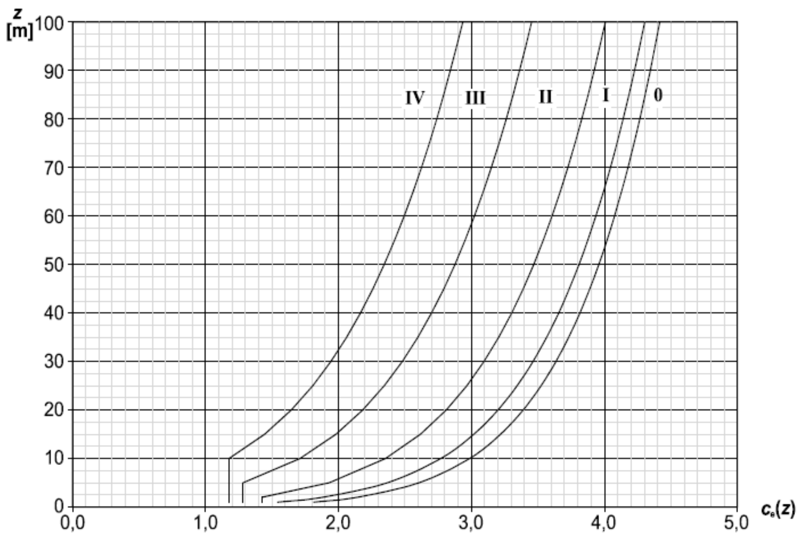


Figure 142: Illustrations of the exposure factor $c_e(z)$ for c_O=1,0, k_I=1,0

$$\begin{aligned} &v_{b,0} = 27\ \text{m/s} \\ &v_b = c_{dir} * c_{season} * v_{b,0} = 27\ \text{m/s} \\ &q_b = 0.5 * \rho * v_b^2 = 0.456\ \text{kN/m}^2 \\ &c_e(z_e) = 3.55 \\ &q_p(z_e) = c_e(z_e) * q_b = 1.62\ \text{kN/m}^2 \end{aligned} \quad (13.5)$$

$$\begin{aligned} &v(z_e) = \sqrt{\frac{2q_p}{\rho}} = 50.87\ \text{m/s} \\ &\text{Re} = \frac{b * v(z_e)}{\nu} = 4.58 * 10^6 \end{aligned} \quad (13.6)$$

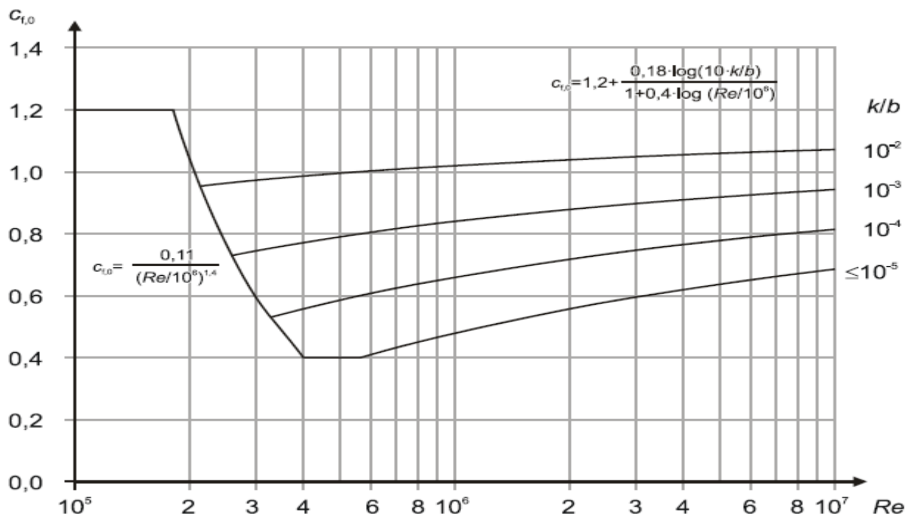


Figure 143: Force coefficient $c_{f,0}$ for circular cylinders without free-end flow and for different equivalent roughness k/b

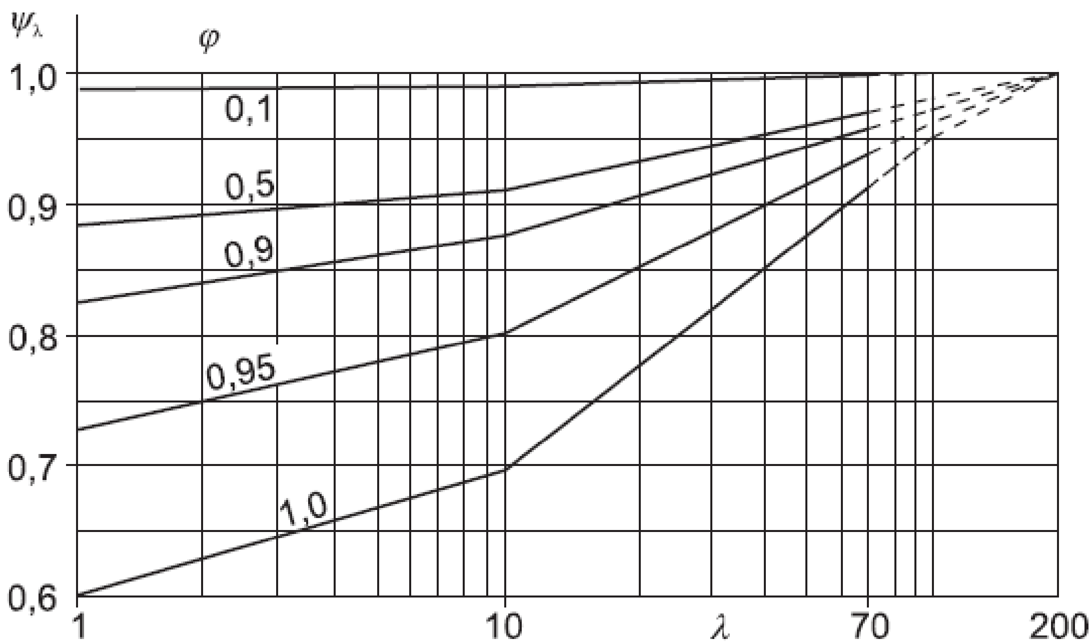


Figure 144: Indicative values of the end-effect factor ψ_λ as a function of solidity ratio ϕ versus slenderness λ

$$
\begin{aligned}
&k = 0.2 \text{ mm} \\
&\text{k/b} = 1.48 * 10^{-4} \\
&c_{f,0} = 0.8 \\
&\psi_\lambda = 0.98 \\
&c_f = c_{f,0} * \psi_\lambda = 0.784
\end{aligned} \tag{13.7}
$$

$$z_0 = 0.05$$
$$k_l = 1.0$$
$$c_0(z) = 1.0$$
$$l_v(z) = \frac{k_l}{c_o(z) * \ln\left(\frac{z_e}{z_0}\right)} = 0.142 \tag{13.8}$$
$$\alpha = 0.67 + 0.05 * \ln\left(z_0\right) = 0.52$$
$$L(z) = 300 * \left(\frac{z_e}{200}\right)^{\alpha} = 156.14 \text{ m}$$

$$\mathrm{B}^2 = \frac{1}{1 + 1.5 * \sqrt{\left(\frac{b}{L(z)}\right)^2 + \left(\frac{b * z_e}{L(z)^2}\right)^2}} = 0.646 \tag{13.9}$$
$$c_s = \frac{1 + 7 l_v(z) * \sqrt{B^2}}{1 + 7 * l_v(z)} = 0.902$$
$$c_r(z) = 0.19 * \left(\frac{z_0}{z_e}\right)^{0.07} * \ln\left(\frac{z_e}{z_0}\right) = 1.337$$
$$v_m(z) = \mathrm{c}_r(z) * c_o(z) * v_b = 36.1 \text{ m/s}$$
$$\mathrm{n}_{1,x} = 0.33$$
$$f_L(z,n) = 0.33$$
$$S_L = \frac{6.8 * f_L(z,n)}{(1 + 10.2 * f_L(z,n))^{1.67}} = 0.19$$
$$G = 5/18$$
$$K_s = \frac{1}{1 + \left(G * 11.5 * b * n_{1,x} / v_m(z)\right)}$$
$$\delta_s = 0.5$$
$$\delta = 0.03 + \frac{c_f * \rho * v_m(z)}{2 * n_{1,x} * \mu_e} + \delta_s = 0.535$$
$$\nu = \mathrm{MAX}\left\{\mathrm{n}_{1,x} * \sqrt{\frac{R^2}{B^2 + R^2}}; 0.08\right\} = 0.28$$

$$k_p = \mathrm{MAX}\left\{\sqrt{2 * \ln\left(\nu * 600\right)} + \frac{0.6}{\sqrt{2 * \ln\left(\nu * 600\right)}}; 3\right\} = 3.39 \tag{13.10}$$
$$R^2 = \frac{\pi^2 * S_L * K_s}{2 * \delta} = 1.697$$

$$c_d = \frac{1+2*k_p*l_v(z)\sqrt{B^2+R^2}}{1+7*l_v(z)*\sqrt{B^2}} = 1.376 \tag{13.11}$$

$$c_s c_d = \mathrm{MAX}\{c_s*c_d;0.85\} = 1.24$$

$$F_w = c_s*c_d*c_f*q_p(z_e)*A_{ref} = 121.09 \text{ kN}$$

$$\mathrm{q}_w = \frac{\frac{1}{3}*F_w*z_e}{0.5*z_e{}^2} = 1.42 \text{ kN/m} \tag{13.12}$$

Appendix E: Preliminary design calculations OSC

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	250	mm

Table 77:: Mast dimensions HSC

Component	Symbol	Value	Unit
Concrete quality	C	45/55	N/mm^2
Compressive strength, characteristic	f_{ck}	45	N/mm^2
Compressive strength, design	f_{cd}	30	N/mm^2
Material density	ρ_c	2405	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$1.75*10^{-3}$	-
Ultimate strain	ε_{cu3}	$3.50*10^{-3}$	-
Young's Modulus, mean	E_{cm}	36000	N/mm^2
Young's Modulus, design	E_{cd}	30000	N/mm^2
Tensile strength, characteristic	f_{ctk}	3.80	N/mm^2
Tensile strength, design	f_{ctd}	253	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	5.7	N/mm^2

Table 78: Material properties HSC

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1100	mm
Area mast average	A_{av}	863938	mm^2
Volume	V	$4.92*10^{10}$	mm^3
Section modulus average	W_{av}	$2.04*10^8$	mm^3
Moment of inertia average	I_{av}	$1.37*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 79: Section properties HSC

Component	SLS		ULS		Unit
Self weight	1184	-	1421	-	kN
Wind	-	1.3	-	1.9	kN/m
Total normal/ shear force	1492	307	1791	469	kN

Table 80: Acting loads SLS & ULS for HSC

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	3860	5790	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	1750	2624	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2061	3092	kNm
Total	13223	16510	kNm

Table 81: Moments in SLS & ULS for HSC

Without prestressing

Stress due to axial load: $\sigma_{Ns;r} = \frac{N_{s;r}}{A_c} = -0.97$ N/mm^2 (13.13)

Stress due to moment: $\sigma_{Ms;r} = \frac{M_{s;r}}{A_c} = 19.66$ N/mm^2 (13.14)

Initial stress at t=0: $\sigma_{c;0} = -3.84$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -20.63$ N/mm$^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 18.68$ N/mm$^2 > f_{ctd}$ NOT OK

With prestressing

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	31776	kN
Governing prestressing force	$P_{min,\infty}$	28598	kN
Required prestressing steel	$A_{p,req}$	22779	mm^2
Number of tendons required	n_{req}	8	-
Number of tendons applied	n_{apl}	8	-
Applied prestressing steel	$A_{p,apl}$	22800	mm^2
Applied prestressing force	$P_{min,0}$	31806	kN
Working prestressing force	$P_{min,\infty}$	28625	kN

Table 82: Prestressing tendons for HSC

Initial stress at t=0: $\sigma_{c;0} = -24.61$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -39.32$ N/mm$^2 > f_{cd}$ NOT OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = -0.02$ N/mm$^2 \leq f_{ctd}$ OK

Losses

Friction losses: $\Delta P_u = 1442$ kN
Shrinkage losses: $\Delta P_{cs} = 1294$ kN
Creep losses: $\Delta P_{cc} = 276$ kN
Relaxation losses: $\Delta\sigma_{pr} = 68$ N/mm^2
Total losses: $\Delta = 11.40\%$

Reinforcement

$A_{s,req} = 56660$ mm^2
$x_u = 1305$ mm

Moment capacity

x_u = 1273 mm	
M_{Rd} = 26352 kN	OK

Deformations

w_k = 0.119 mm	OK
δ_{top} = 1818 mm	NOT OK
δ_{rel} = 114 mm	OK
σ_{tot} = 38.11 N/mm^2	NOT OK
N_{cr} = 3757 kN	OK

Appendix F: Preliminary design calculations HSC

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	250	mm

Table 83: Mast dimensions HSC

Component	Symbol	Value	Unit
Concrete quality	C	90/105	N/mm^2
Compressive strength, characteristic	f_{ck}	90	N/mm^2
Compressive strength, design	f_{cd}	60	N/mm^2
Material density	ρ_c	2410	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$2.30*10^{-3}$	-
Ultimate strain	ε_{cu3}	$2.60*10^{-3}$	-
Young's Modulus, mean	E_{cm}	44000	N/mm^2
Young's Modulus, design	E_{cd}	36667	N/mm^2
Tensile strength, characteristic	f_{ctk}	5.00	N/mm^2
Tensile strength, design	f_{ctd}	3.33	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	7.6	N/mm^2

Table 84: Material properties HSC

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1100	mm
Area mast average	A_{av}	863938	mm^2
Volume	V	$4.92*10^{10}$	mm^3
Section modulus average	W_{av}	$2.04*10^8$	mm^3
Moment of inertia average	I_{av}	$1.37*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 85: Section properties HSC

Component	SLS		ULS		Unit
Self weight	1187	-	1424	-	kN
Wind	-	1.3	-	1.9	kN/m
Total normal/ shear force	1495	316	1794	474	kN

Table 86: Acting loads SLS & ULS for HSC

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	3860	5790	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	1750	2624	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2051	3077	kNm
Total	10997	16496	kNm

Table 87: Moments in SLS & ULS for HSC

Without prestressing

Stress due to axial load: $\sigma_{Ns;r} = \dfrac{N_{s;r}}{A_c} = -0.98$ N/mm^2 (13.15)

Stress due to moment: $\sigma_{Ms;r} = \dfrac{M_{s;r}}{A_c} = 19.65$ N/mm^2 (13.16)

Initial stress at t=0: $\sigma_{c;0} = -3.82$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -20.63$ N/mm$^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 18.67$ N/mm$^2 > f_{ctd}$ NOT OK

With prestressing

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	31776	kN
Governing prestressing force	$P_{min,\infty}$	28598	kN
Required prestressing steel	$A_{p,req}$	22779	mm^2
Number of tendons required	n_{req}	8	-
Number of tendons applied	n_{apl}	8	-
Applied prestressing steel	$A_{p,apl}$	22800	mm^2
Applied prestressing force	$P_{min,0}$	31806	kN
Working prestressing force	$P_{min,\infty}$	28625	kN

Table 88: Prestressing tendons for HSC

Initial stress at t=0: $\sigma_{c;0} = -24.59$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -39.32$ N/mm$^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = -0.02$ N/mm$^2 \leq f_{ctd}$ OK

Losses

Friction losses: $\Delta P_u = 1442$ kN
Shrinkage losses: $\Delta P_{cs} = 1294$ kN
Creep losses: $\Delta P_{cc} = 94$ kN
Relaxation losses: $\Delta\sigma_{pr} = 68$ N/mm^2
Total losses: $\Delta = 12.14\%$

Reinforcement

$$A_{s,req} = 55318 \text{ mm}^2$$
$$x_u = 1247 \text{ mm}$$

Moment capacity

x_u = 1026 mm	
M_{Rd} = 34105 kN	OK

Deformations

w_k = 0.118 mm	OK
δ_{top} = 931 mm	OK
δ_{rel} = 125 mm	OK
σ_{tot} = 38.09 N/mm^2	OK
N_{cr} = 4592 kN	OK

Appendix G: Preliminary design calculations UHSC

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	250	mm

Table 89: Mast dimensions UHSC

Component	Symbol	Value	Unit
Concrete quality	C	170/200	N/mm^2
Compressive strength, characteristic	f_{ck}	170	N/mm^2
Compressive strength, design	f_{cd}	113	N/mm^2
Material density	ρ_c	2500	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$2.35*10^{-3}$	-
Ultimate strain	ε_{cu3}	$3.00*10^{-3}$	-
Young's Modulus, mean	E_{cm}	55000	N/mm^2
Young's Modulus, design	E_{cd}	45833	N/mm^2
Tensile strength, characteristic	f_{ctk}	10.27	N/mm^2
Tensile strength, design	f_{ctd}	6.85	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	41.8	N/mm^2

Table 90: Material properties UHSC

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1100	mm
Area mast average	A_{av}	863938	mm^2
Volume	V	$4.92*10^{10}$	mm^3
Section modulus average	W_{av}	$2.04*10^8$	mm^3
Moment of inertia average	I_{av}	$1.37*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 91: Section properties UHSC

Component	SLS		ULS		Unit
Self weight	1231	-	1477	-	kN
Wind	-	1.4	-	2.1	kN/m
Total normal/ shear force	1539	316	1847	474	kN

Table 92: Acting loads SLS & ULS for UHSC

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	1750	2624	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	3860	5790	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2308	3462	kNm
Total	11254	16881	kNm

Table 93: Moments in SLS & ULS for UHSC

Without prestressing

Stress due to axial load: $\sigma_{Ns;r} = \dfrac{N_{s;r}}{A_c} = -1.01\ \text{N/mm}^2$ (13.17)

Stress due to moment: $\sigma_{Ms;r} = \dfrac{M_{s;r}}{A_c} = 19.73\text{N/mm}^2$ (13.18)

Initial stress at t=0: $\sigma_{c;0} = -3.83\ \text{N/mm}^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -20.73\ \text{N/mm}^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 18.72\ \text{N/mm}^2 > f_{ctd}$ NOT OK

With prestressing

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	32543	kN
Governing prestressing force	$P_{min,\infty}$	29288	kN
Required prestressing steel	$A_{p,req}$	23328	mm^2
Number of tendons required	n_{req}	9	-
Number of tendons applied	n_{apl}	10	-
Applied prestressing steel	$A_{p,apl}$	28500	mm^2
Applied prestressing force	$P_{min,0}$	39578	kN
Working prestressing force	$P_{min,\infty}$	35782	kN

Table 94: Prestressing tendons for UHSC

Initial stress at t=0: $\sigma_{c;0} = -30.19\ \text{N/mm}^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -44.50\ \text{N/mm}^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = -4.24\ \text{N/mm}^2 \leq f_{ctd}$ OK

Losses

Friction losses: $\Delta P_u = 1863$ kN
Shrinkage losses: $\Delta P_{cs} = 2418$ kN
Creep losses: $\Delta P_{cc} = 55$ kN
Relaxation losses: $\Delta\sigma_{pr} = 68\ \text{N/mm}^2$
Total losses: $\Delta = 13.72\%$

Reinforcement

$$A_{s,req} = 60059\ \text{mm}^2$$
$$x_u = 1263\ \text{mm}$$

Moment capacity

x_u = 858 mm

M_{Rd} = 85547 kN OK

Deformations

w_k = 0.076 mm	OK
δ_{top} = 724 mm	OK
δ_{rel} = 97 mm	OK
σ_{tot} = 43.70 N/mm^2	OK
N_{cr} = 5740 kN	OK

Appendix H: Foundation

Component	Symbol	Value	Unit
Inner length	d_1	2.20	m
Outer length	l_2	2.50	m
Total diameter	d	7.20	m
Thickness	t	1.50	m

Table 95: Foundation block dimensions

Component	Symbol	Value	Unit
Pile length	l_p	20	m
Pile diameter	d_p	200	mm
Pile cross section	A_p	0.04	m^2
Grid diameter	d_{grid}	400	mm
Grid cross section	A_{grid}	0.13	m^2
Configuration		Concentric	-
Pile strength	C	45/55	N/mm^2
Young's modulus	E_c	36000	N/mm^2
Bearing capacity	P_p	200	kN
Axial force at foot	N_{ed}	3217	kN
Moment at foot	M_{ed}	12000	kNm

Table 96: Foundation pile properties

Foundation piles

$$n_{\min} = \frac{N_{sd}}{P_p}$$

$$n_{\max} = \frac{0.25 * \pi * d^2}{A_{grid}} \tag{13.19}$$

Piles at the edge: n

Pile spacing: $l_{r,\tan} = \frac{\pi * d}{n-1}$ > 0.400 m

Axial resistance: $N_{Rd} = n * P_p$ > 2263 kN (13.20)

Pile eccentricity: $n * e_p$

Moment resistance: M_{Rd} > 11247 kNm

Rotational stiffness

$$\xi_{rigid} = \frac{3.43 * E_c * I_c}{\gamma_c} \tag{13.21}$$

$$\theta = M / \xi \tag{13.22}$$

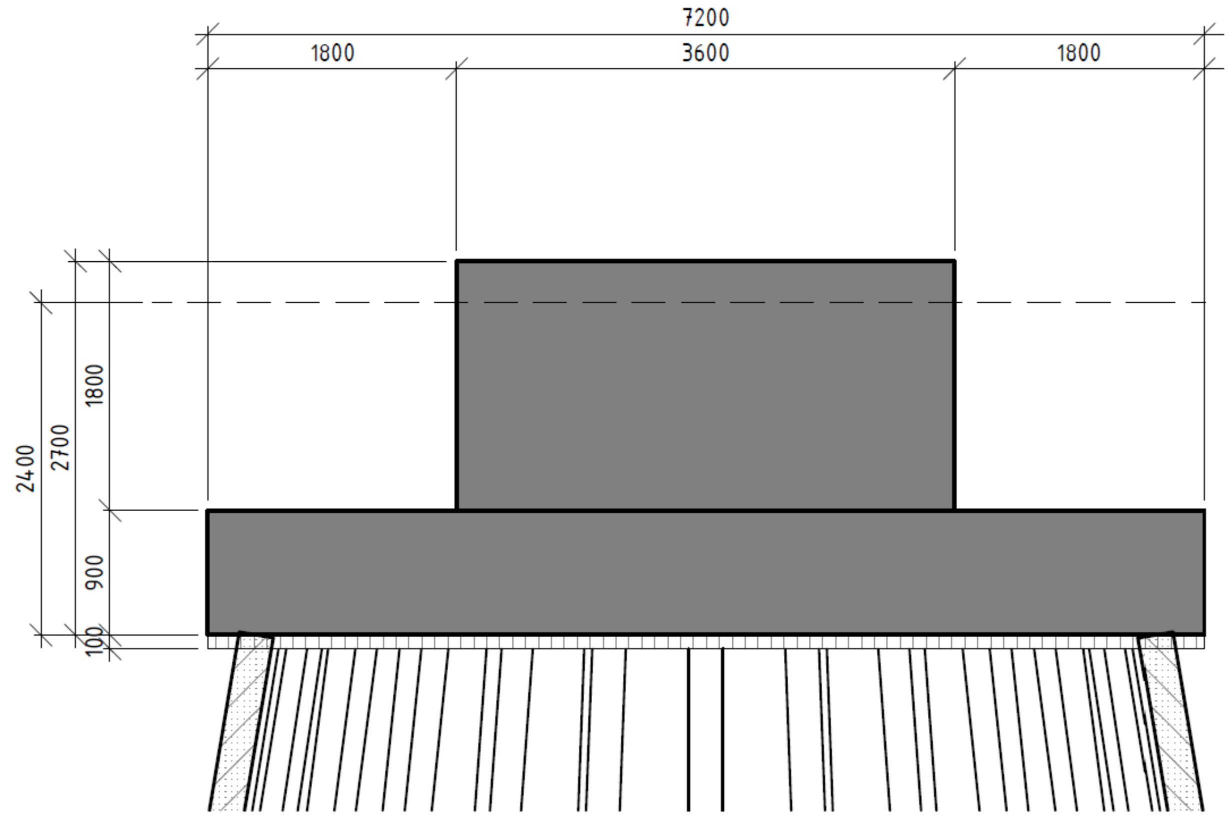
$$\delta = \theta * h \leq 0.001h \tag{13.23}$$

Shear

$$d = 0.9 * t$$
$$k = 1 + \sqrt{\frac{200}{d}}$$
$$c_1 / c_2 = 1.0$$
$$k_c = 0.60$$
$$u_1 = \pi * (d_{mast} + 2d) \quad (13.24)$$
$$e = 0.5 * d_{mast}$$
$$W_1 = e * u_1$$
$$\beta = 1 + k * \frac{M_{Ed}}{V_{Ed}} * \frac{u_1}{W_1}$$
$$v_{Ed} = \frac{\beta * V_{Ed}}{u_1 * d}$$
$$\rho = \frac{A_s}{1 * t}$$
$$\mathrm{v}_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} < 0.12 * k * (100 * \rho * f_{ck})^{1/3} \quad (13.25)$$
$$\sigma_{cp} = \frac{V_{Ed}}{0.25 * \pi * d^2}$$
$$v_{Rd,c} = 0.12 * k * (100 * \rho * f_{ck})^{1/3} + \sigma_{cp} > \mathrm{v}_{Ed}$$

		One mast			Both masts	
Component		**Circular (distributed)**	**Circular (at edge)**	**Square (distributed)**	**Rectangular**	**Units**
Axial force	N_{sd}	2263	2263	2680	9998	kN
Moment	M_{sd}	11247	11247	11247	11247	kNm
Foundation thickness	t	1.5	1.5	2.0	2.5	m
Diameter foundation	d or l*b	7.2	7.2	7.2*5.4	23.9*6.2	m
Minimum number of piles	n_{min}	5	5	13	50	
Maximum number of piles	n_{max}	68	68	69	263	
Number of piles per mast	n	24	21	28	-	-
Total number of piles	n	48	42	56	70	-
Rotational stiffness	ξ	2.89	3.25	3.89	6.25	10^7Nm/rad
Deformation	δ	19.75	18.65	16.74	10.26	mm

Table 97: Summary of foundation calculations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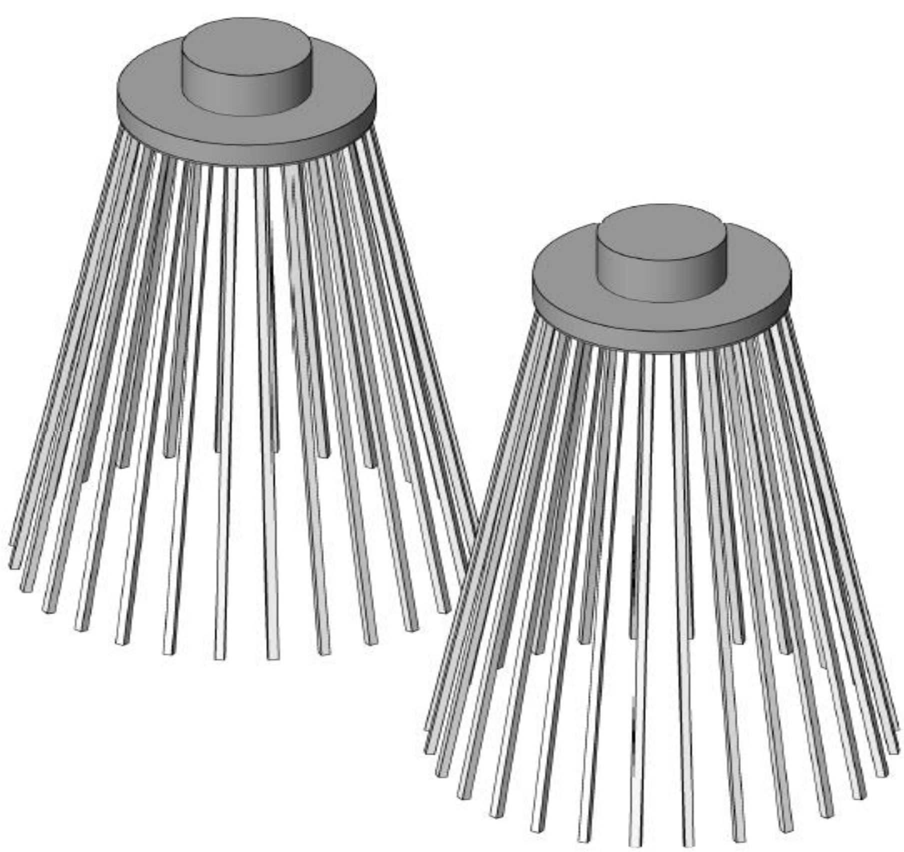
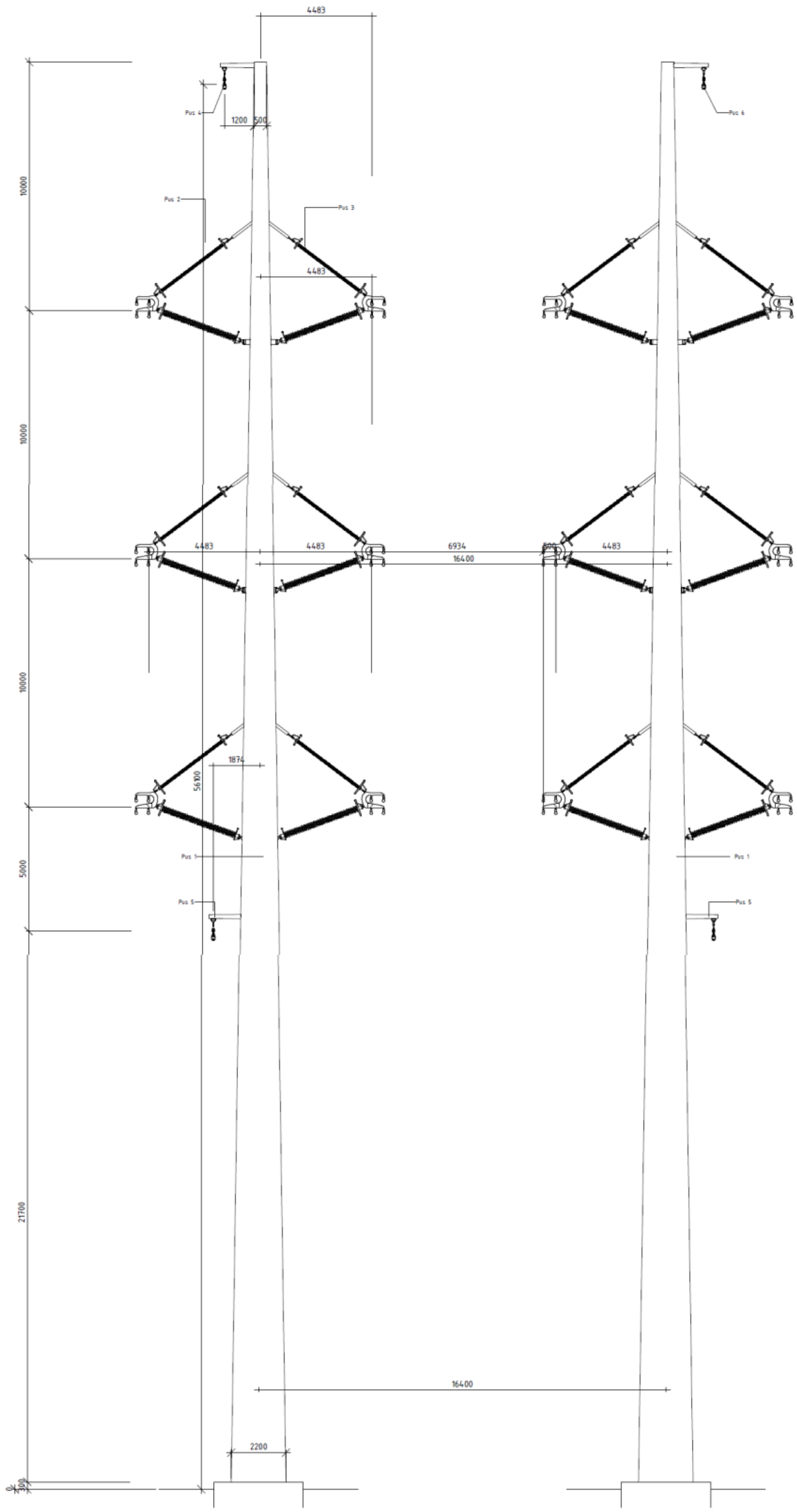
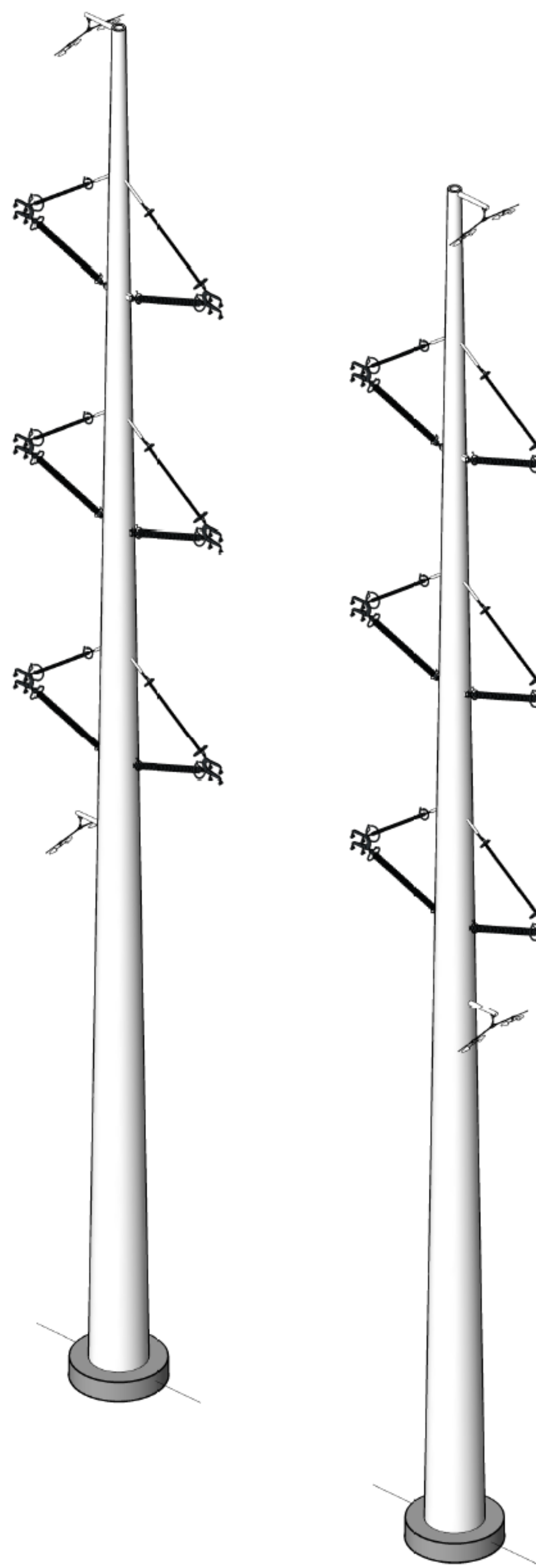
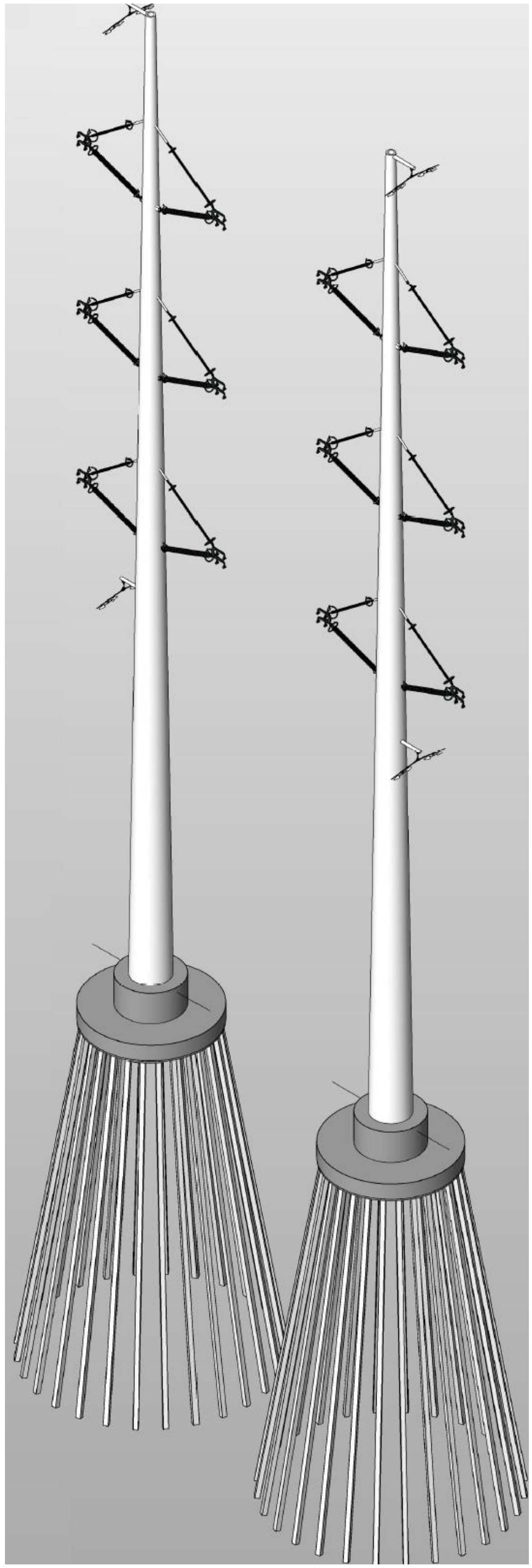


Appendix I: Design drawings







Appendix J: Transport with whole segments

OSC		t_{mast}:	420	n_{tendon}:	10	**Number of segments: 12**								**Units**
h	0	2	5	8	11	15	18	22	27	32	40	49	57	m
d_{bot}	2,20	2.14	2.14	2.05	1.96	1.87	1.75	1.66	1.54	1.39	1.25	1.01	0.74	m
d_{top}	2,20	2.14	2.05	1.96	1.87	1.75	1.66	1.54	1.39	1.25	1.01	0.74	0.50	m
d_{av}	2,20	2.14	2.10	2.01	1.92	1.81	1.71	1.60	1.47	1.32	1.13	0.87	0.62	m
G	0,00	4.54	16.58	15.70	14.81	18.37	12.75	15.62	17.31	14.85	18.64	13.44	5.26	ton
Total G	167.68	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	
σ	0,00	0,03	0.08	0.08	0.09	0.16	0.10	0.20	0.35	0.40	1.28	2.23	2.21	N/mm²

Table 98: Segment lengths for OSC with whole segments

HSC		t_{mast}:	145	n_{tendon}:	8	**Number of segments: 6**		**Units**
h	0	7	15	24	35	47	57	m
d_{bot}	2.20	2.20	1.99	1.75	1.48	1.16	0.80	m
d_{top}	2.20	1.99	1.75	1.48	1.16	0.80	0.50	m
d_{av}	2.20	2.10	1.87	1.62	1.32	0.98	0.65	m
G	0.00	15.55	15.73	15.10	14.72	11.37	5.74	ton
Total G	78.22	PASS	PASS	PASS	PASS	PASS	PASS	
σ	0.00	0.32	0.48	0.72	1.37	2.38	2.84	N/mm²

Table 99: Segment lengths for HSC with whole segments

UHSC	t_{mast}:	75	n_{tendon}:	8	**Number of segments: 4**	**Units**
h	0	13	29	44	57	m
d_{bot}	2.20	2.20	1.81	1.34	0.89	m
d_{top}	2.20	1.81	1.34	0.89	0.50	m
d_{av}	2.20	2.01	1.57	1.11	0.69	m
G	0.00	14.79	14.12	9.16	4.74	ton
Total G	42.81	PASS	PASS	PASS	PASS	
σ	0.00	1.13	2.24	2.89	3.77	N/mm²

Table 100: Segment lengths for UHSC with whole segments

Appendix K: Transport with half segments

OSC	**Half segments: 4**			t_{mast}:	420	n_{tendon}:	10	**Whole segments: 8**				**Units**
h	0	5	11	15	18	22	26	32	40	49	57	m
d_{bot}	2.20	2.20	2.05	1.87	1.77	1.66	1.54	1.42	1.25	1.01	0.74	m
d_{top}	2.20	2.05	1.87	1.77	1.66	1.54	1.42	1.25	1.01	0.74	0.50	m
d_{av}	2.20	2.13	1.96	1.82	1.72	1.60	1.48	1.34	1.13	0.87	0.62	m
G	0.00	14.06	15.25	16.16	14.96	15.62	14.04	18.11	18.64	13.44	5.26	ton
Total G	131.48	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	PASS	
σ	0.00	0.53	0.82	0.13	0.14	0.20	0.22	0.57	1.28	2.23	2.21	N/mm^2

Table 101: Segment length for OSC with half segments

HSC	**Half segments: 2**		t_{mast}:	145	**Whole segments: 4**		**Units**
h	0	12	21	32	47	57	m
d_{bot}	2,20	2,20	1.84	1.57	1.25	0.81	m
d_{top}	2,20	1,83	1.57	1.25	0.81	0.50	m
d_{av}	2,20	2,02	1.71	1.41	1.03	0.66	m
G	0.00	12.82	16.02	15.84	14.60	6.12	ton
Total G	59.28	PASS	PASS	PASS	PASS	PASS	
σ	0.00	3.07	0.68	1.27	3.25	3.09	N/mm^2

Table 102: Segment length for HSC with half segments

HSC	**Half segments: 2**		t_{mast}:	75	**Whole segments: 3**	**Units**
h	0	13	26	44	57	m
d_{bot}	2.20	2.20	1.81	1.42	0.89	m
d_{top}	2.20	1.81	1.42	0.89	0.50	m
d_{av}	2.20	2.01	1.62	1.16	0.69	m
G	0.00	7.39	11.82	11.46	4.74	ton
Total G	35.42	PASS	PASS	PASS	PASS	
σ	0.00	3.53	1.43	3.99	3.77	N/mm^2

Table 103: Segment length for UHSC with half segments

Appendix L: Final design calculations OSC

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.30	m
Diameter at top	d_{top}	0.70	m
Wall thickness	t	420	mm

Table 104:: Mast dimensions HSC

Component	Symbol	Value	Unit
Concrete quality	C	45/55	N/mm^2
Compressive strength, characteristic	f_{ck}	45	N/mm^2
Compressive strength, design	f_{cd}	30	N/mm^2
Material density	ρ_c	2405	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$1.75*10^{-3}$	-
Ultimate strain	ε_{cu3}	$3.50*10^{-3}$	-
Young's Modulus, mean	E_{cm}	36000	N/mm^2
Young's Modulus, design	E_{cd}	30000	N/mm^2
Tensile strength, characteristic	f_{ctk}	3.80	N/mm^2
Tensile strength, design	f_{ctd}	253	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	5.7	N/mm^2

Table 105: Material properties HSC

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1150	mm
Area mast average	A_{av}	1425026	mm^2
Volume	V	$8.12*10^{10}$	mm^3
Section modulus average	W_{av}	$3.19*10^8$	mm^3
Moment of inertia average	I_{av}	$2.39*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 106: Section properties HSC

Component	SLS		ULS		Unit
Self weight	1954	-	2344	-	kN
Wind	-	1.4	-	2.1	kN/m
Total normal/ shear force	2262	315	2714	473	kN

Table 107: Acting loads SLS & ULS for HSC

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	3860	5790	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	1750	2624	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2287	3430	kNm
Total	11233	16849	kNm

Table 108: Moments in SLS & ULS for HSC

Without prestressing

Stress due to axial load: $\sigma_{Ns;r} = \frac{N_{s;r}}{A_c} = -0.96$ N/mm^2 (13.26)

Stress due to moment: $\sigma_{Ms;r} = \frac{M_{s;r}}{A_c} = 16.35$ N/mm^2 (13.27)

Initial stress at t=0: $\sigma_{c;0} = -3.39$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -17.31$ N/mm$^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 15.38$ N/mm$^2 > f_{ctd}$ NOT OK

With prestressing

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	37682	kN
Governing prestressing force	$P_{min,\infty}$	33914	kN
Required prestressing steel	$A_{p,req}$	27012	mm^2
Number of tendons required	n_{req}	10	-
Number of tendons applied	n_{apl}	10	-
Applied prestressing steel	$A_{p,apl}$	28500	mm^2
Applied prestressing force	$P_{min,0}$	39758	kN
Working prestressing force	$P_{min,\infty}$	35782	kN

Table 109: Prestressing tendons for HSC

Initial stress at t=0: $\sigma_{c;0} = -19.10$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -29.92$ N/mm$^2 > f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = -0.75$ N/mm$^2 \leq f_{ctd}$ OK

Losses

Friction losses: $\Delta P_u = 1803$ kN
Shrinkage losses: $\Delta P_{cs} = 1355$ kN
Creep losses: $\Delta P_{cc} = 317$ kN
Relaxation losses: $\Delta\sigma_{pr} = 68$ N/mm^2
Total losses: $\Delta = 11.42\%$

Reinforcement

$A_{s,req} = 38300$ mm^2
$x_u = 1262$ mm

Moment capacity

x_u = 1458 mm

M_{Rd} = 23082 kN OK

Deformations

w_k = 0.139 mm OK

δ_{top} = 1070 mm OK

δ_{rel} = 76 mm OK

σ_{tot} = 28.17 N/mm^2 OK

N_{cr} = 6539 kN OK

Appendix M: Final design calculations HSC

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	145	mm

Table 110: Mast dimensions HSC

Component	Symbol	Value	Unit
Concrete quality	C	90/105	N/mm^2
Compressive strength, characteristic	f_{ck}	90	N/mm^2
Compressive strength, design	f_{cd}	60	N/mm^2
Material density	ρ_c	2410	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$2.30*10^{-3}$	-
Ultimate strain	ε_{cu3}	$2.60*10^{-3}$	-
Young's Modulus, mean	E_{cm}	44000	N/mm^2
Young's Modulus, design	E_{cd}	36667	N/mm^2
Tensile strength, characteristic	f_{ctk}	5.00	N/mm^2
Tensile strength, design	f_{ctd}	3.33	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	7.6	N/mm^2

Table 111: Material properties HSC

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1100	mm
Area mast average	A_{av}	548915	mm^2
Volume	V	$3.13*10^{10}$	mm^3
Section modulus average	W_{av}	$1.50*10^{8}$	mm^3
Moment of inertia average	I_{av}	$1.01*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 112: Section properties HSC

Component	SLS		ULS		Unit
Self weight	754	-	905	-	kN
Wind	-	1.3	-	1.9	kN/m
Total normal/ shear force	1062	307	1275	460	kN

Table 113: Acting loads SLS & ULS for HSC

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	3860	5790	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	1750	2624	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2043	3065	kNm
Total	10989	16483	kNm

Table 114: Moments in SLS & ULS for HSC

Without prestressing

Stress due to axial load: $\sigma_{Ns;r} = \dfrac{N_{s;r}}{A_c} = -1.13$ N/mm^2 (13.28)

Stress due to moment: $\sigma_{Ms;r} = \dfrac{M_{s;r}}{A_c} = 27.83$ N/mm^2 (13.29)

Initial stress at t=0: $\sigma_{c;0} = -5.33$ N/mm$^2 \le 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -28.97$ N/mm$^2 \le f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 26.70$ N/mm$^2 > f_{ctd}$ NOT OK

With prestressing

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	27771	kN
Governing prestressing force	$P_{min,\infty}$	24994	kN
Required prestressing steel	$A_{p,req}$	19907	mm^2
Number of tendons required	n_{req}	7	-
Number of tendons applied	n_{apl}	8	-
Applied prestressing steel	$A_{p,apl}$	22800	mm^2
Applied prestressing force	$P_{min,0}$	31806	kN
Working prestressing force	$P_{min,\infty}$	28625	kN

Table 115: Prestressing tendons for HSC

Initial stress at t=0: $\sigma_{c;0} = -39.31$ N/mm$^2 \le 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -59.55$ N/mm$^2 \le f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = -3.88$ N/mm$^2 \le f_{ctd}$ OK

Losses

Friction losses: $\Delta P_u = 1442$ kN
Shrinkage losses: $\Delta P_{cs} = 1323$ kN
Creep losses: $\Delta P_{cc} = 109$ kN
Relaxation losses: $\Delta\sigma_{pr} = 68$ N/mm^2
Total losses: $\Delta = 10.80\%$

Reinforcement

$$A_{s,req} = 64906 \text{ mm}^2$$
$$x_u = 1399 \text{ mm}$$

Moment capacity

x_u = 1500 mm
M_{Rd} = 29597 kN OK

Deformations

w_k = 0.105 mm OK
δ_{top} = 1265 mm OK
δ_{rel} = 170 mm OK
σ_{tot} = 59.45 N/mm^2 OK
N_{cr} = 3377 kN OK

Appendix N: Final design calculations UHSC

Component	Symbol	Value	Unit
Height	h	57	m
Diameter at foot	d_{bot}	2.20	m
Diameter at top	d_{top}	0.50	m
Wall thickness	t	75	mm

Table 116: Mast dimensions UHSC

Component	Symbol	Value	Unit
Concrete quality	C	170/200	N/mm^2
Compressive strength, characteristic	f_{ck}	170	N/mm^2
Compressive strength, design	f_{cd}	113	N/mm^2
Material density	ρ_c	2500	kg/m^3
Material factor	γ_c	1.5	-
Maximum strain in pure compression	ε_{cu}	$2.35*10^{-3}$	-
Ultimate strain	ε_{cu3}	$3.00*10^{-3}$	-
Young's Modulus, mean	E_{cm}	55000	N/mm^2
Young's Modulus, design	E_{cd}	45833	N/mm^2
Tensile strength, characteristic	f_{ctk}	10.27	N/mm^2
Tensile strength, design	f_{ctd}	6.85	N/mm^2
Tensile strength, flexural	$f_{ctk,fl}$	41.8	N/mm^2

Table 117: Material properties UHSC

Component	Symbol	Value	Unit
Neutral line section	z_{sec}	1100	mm
Area mast average	A_{av}	300415	mm^2
Volume	V	$1.71*10^{10}$	mm^3
Section modulus average	W_{av}	$9.08*10^{8}$	mm^3
Moment of inertia average	I_{av}	$6.13*10^{11}$	mm^4
Slenderness mast	λ	239	m/m

Table 118: Section properties UHSC

Component	SLS		ULS		Unit
Self weight	428	-	513	-	kN
Wind	-	1.4	-	2.1	kN/m
Total normal/ shear force	736	316	884	474	kN

Table 119: Acting loads SLS & ULS for UHSC

Moments	SLS	ULS	Units
Retour current conductor	107	161	kNm
Conductors 1st level	1750	2624	kNm
Conductors 2nd level	2807	4211	kNm
Conductors 3rd level	3860	5790	kNm
Lighting wire	422	633	kNm
Wind (at foot)	2301	3451	kNm
Total	11247	16870	kNm

Table 120: Moments in SLS & ULS for UHSC

Without prestressing

Stress due to axial load: $\sigma_{Ns;r} = \dfrac{N_{s;r}}{A_c} = -1.47$ N/mm^2 (13.30)

Stress due to moment: $\sigma_{Ms;r} = \dfrac{M_{s;r}}{A_c} = 46.89$ N/mm^2 (13.31)

Initial stress at t=0: $\sigma_{c;0} = -8.72$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -48.37$ N/mm$^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = 45.42$ N/mm$^2 > f_{ctd}$ NOT OK

With prestressing

Component	Symbol	Value	Unit
Governing prestressing force	$P_{min,0}$	25869	kN
Governing prestressing force	$P_{min,\infty}$	23282	kN
Required prestressing steel	$A_{p,req}$	18544	mm^2
Number of tendons required	n_{req}	7	-
Number of tendons applied	n_{apl}	8	-
Applied prestressing steel	$A_{p,apl}$	28500	mm^2
Applied prestressing force	$P_{min,0}$	39578	kN
Working prestressing force	$P_{min,\infty}$	35782	kN

Table 121: Prestressing tendons for UHSC

Initial stress at t=0: $\sigma_{c;0} = -73.32$ N/mm$^2 \leq 0.6f_{cd}$ OK

Compressive stress at t = ∞: $\sigma_{c;\infty} = -106.61$ N/mm$^2 \leq f_{cd}$ OK

Tensile stress at t = ∞: $\sigma_{t;\infty} = -10.67$ N/mm$^2 \leq f_{ctd}$ OK

Losses

Friction losses: $\Delta P_u = 1442$ kN
Shrinkage losses: $\Delta P_{cs} = 1985$ kN
Creep losses: $\Delta P_{cc} = 71$ kN
Relaxation losses: $\Delta\sigma_{pr} = 68$ N/mm^2
Total losses: $\Delta = 11.53\%$

Reinforcement

$A_{s,req} = 72198$ mm^2
$x_u = 1527$ mm

Moment capacity

x_u = 1494 mm

M_{Rd} = 26681 kN OK

Deformations

w_k = 0.074 mm OK

δ_{top} = 1623 mm OK

δ_{rel} = 218 mm OK

σ_{tot} = 108.71 N/mm^2 OK

N_{cr} = 2559 kN OK

Appendix O: Segment-segment connections for UHSC

Connection number: 1 (h = 44 m)

$$
\begin{aligned}
&n = 1\\
&a_1 = 50 \text{ mm } \geq 50 \text{ mm}\\
&\mathrm{a}_L = 100 \text{ mm } \geq 0.5L\\
&\mathrm{a}_e = 20 \text{ mm}\\
&L = a_1 + a_L + a_e = 170 \text{ mm} \qquad \text{OK}\\
&b_1 = 50 \text{ mm}\\
&\mathrm{b}_W = 100 \text{ mm } \geq 0.5L\\
&\mathrm{b}_e = 50 \text{ mm}\\
&W = b_1 + b_W + b_e = 200 \text{ mm} \qquad \text{OK}\\
&h = 400 \text{ mm}
\end{aligned}
\tag{13.32}
$$

$$
\begin{aligned}
&a = 200\\
&l_{ov} = 400\\
&l_{ov}/h = 1 \qquad \text{Deep beam}\\
&z = 0.2 * l + 0.4 * h = 240 \text{ mm} < 0.8l
\end{aligned}
\tag{13.33}
$$

$$
\begin{aligned}
&F_v = 3976 \text{ kN}\\
&\mathrm{F}_h = 0 \text{ kN}\\
&M_d = F_v * a = 795 \text{ kNm}\\
&A_{s,Fv} = \frac{M_{sd}}{f_y * z} = 7616 \text{ mm}^2\\
&A_{s,Fh} = 0 \text{ mm}^2\\
&\phi = 25 \text{ mm}\\
&n = \frac{A_s}{\phi} = 16
\end{aligned}
\tag{13.34}
$$

Now that the amount of reinforcement necessary to introduce the prestressing into the wall is known, the shear resistance of the joint can be calculated.

$$V = 54 \text{ kN}$$
$$n_{bolt} = 2 \text{ bolts}$$
$$F_{v,Ed} = \frac{V}{n} = 27 \text{ kN}$$
$$a_{bolt,l} = 20 \text{ mm} \qquad \text{OK} \qquad (13.35)$$
$$a_{bolt,r} = L - a_{bolt,l} = 150 \text{ mm}$$
$$b_{bolt,l} = 20 \text{ mm}$$
$$b_{bolt,m} = W - b_{bolt,l} - b_{bolt,r} = 160 \text{ mm} \qquad \text{OK}$$
$$b_{bolt,r} = 20 \text{ mm}$$

Bolt quality: 5.6

$$f_{yb} = 300 \text{ N/mm}^2$$
$$f_{ub} = 500 \text{ N/mm}^2$$
$$\alpha_v = 0.6 \qquad (13.36)$$
$$\gamma_m = 1.25$$
$$M = 16 \text{ mm}$$
$$F_{v,Rd} = \frac{\alpha_v * f_{ub} * A_{b,s}}{\gamma_m} = 35 \text{ kN} \qquad \text{OK}$$

Finally it is checked if the moment and compression force can be transferred from the top segment, to the segment below it.

$$f_{mk} = 45 \text{ N/mm}^2$$
$$f_{md} = 30 \text{ N/mm}^2$$
$$f_{mm} = 27 \text{ N/mm}^2 \qquad (13.37)$$
$$v_o = 20 \text{ mm}$$
$$v = v_o + 20 = 40 \text{ mm}$$

$$d = 0.89 \text{ m}$$
$$\text{x}_u = 460 \text{ mm}$$
$$\text{bx}_u = 98167 \text{ mm}$$
$$\text{k}_1 = 0.9$$
$$k_2 = 0.98 \qquad (13.38)$$
$$k_3 = 0.12$$
$$k_4 = 38.18$$
$$k_5 = 0.5$$

$$f'_v = k_1 * k_2 * f_{md} = 26.41 \text{ N/mm}^2$$
$$A_{bv} = \pi * d * t = 209164 \text{ mm}^2$$
$$N'_{uv} = A_{bv} * f'_v = 5524 \text{ kN}$$
$$M_{Ed} = 2060 \text{ kNm}$$
$$N_c = 0.5 * bx_u * f'_v = 1277 \text{ kN}$$
$$e_c = 301.5 \text{ mm} \qquad (13.39)$$
$$M_{Rd,\text{As}} = M_{Ed} * N_c * e_c = 1675 \text{ kNm}$$
$$d_s = d - x_u - c = 418 \text{ mm}$$
$$d_s' = x_u - c = 430 \text{ mm}$$
$$A_s = \frac{M_{Rd,As}}{f_y(d_s + d_s')} = 4543 \text{ mm}^2$$
$$M'_{uv} = N_c * e_c + f_y * A_s * (d_s + d_s') = 2070 \text{ kNm} \qquad \text{OK}$$

Connection number: 2 (h = 29 m)

$$n = 1$$
$$a_1 = 50 \text{ mm } \geq 50 \text{ mm}$$
$$\text{a}_L = 100 \text{ mm } \geq 0.5L$$
$$\text{a}_e = 20 \text{ mm}$$
$$L = a_1 + a_L + a_e = 170 \text{ mm} \qquad \text{OK}$$
$$b_1 = 50 \text{ mm} \qquad (13.40)$$
$$\text{b}_W = 100 \text{ mm } \geq 0.5L$$
$$\text{b}_e = 50 \text{ mm}$$
$$W = b_1 + b_W + b_e = 200 \text{ mm} \qquad \text{OK}$$
$$h = 400 \text{ mm}$$

$$a = 200$$
$$l_{ov} = 400 \qquad (13.41)$$
$$l_{ov}/h = 1 \qquad \text{Deep beam}$$
$$z = 0.2 * l + 0.4 * h = 240 \text{ mm} < 0.8l$$

$$\begin{aligned}
F_v &= 3976 \text{ kN} \\
\mathrm{F}_h &= 0 \text{ kN} \\
M_d &= F_v * a = 795 \text{ kNm} \\
A_{s,Fv} &= \frac{M_{sd}}{f_y * z} = 7616 \text{ mm}^2 \\
A_{s,Fh} &= 0 \text{ mm}^2 \\
\phi &= 25 \text{ mm} \\
n &= \frac{A_s}{\phi} = 16
\end{aligned} \tag{13.42}$$

Now that the amount of reinforcement necessary to introduce the prestressing into the wall is known, the shear resistance of the joint can be calculated.

$$\begin{aligned}
V &= 51 \text{ kN} \\
n_{bolt} &= 2 \text{ bolts} \\
F_{v,Ed} &= \frac{V}{n} = 26 \text{ kN} \\
a_{bolt,l} &= 20 \text{ mm} && \text{OK} \\
a_{bolt,r} &= L - a_{bolt,l} = 150 \text{ mm} \\
b_{bolt,l} &= 20 \text{ mm} \\
b_{bolt,m} &= W - b_{bolt,l} - b_{bolt,r} = 160 \text{ mm} && \text{OK} \\
b_{bolt,r} &= 20 \text{ mm}
\end{aligned} \tag{13.43}$$

$$\begin{aligned}
&\text{Bolt quality: 5.6} \\
f_{yb} &= 300 \text{ N/mm}^2 \\
f_{ub} &= 500 \text{ N/mm}^2 \\
\alpha_v &= 0.6 \\
\gamma_m &= 1.25 \\
M &= 16 \text{ mm} \\
F_{v,Rd} &= \frac{\alpha_v * f_{ub} * A_{b,s}}{\gamma_m} = 35 \text{ kN} && \text{OK}
\end{aligned} \tag{13.44}$$

Finally it is checked if the moment and compression force can be transferred from the top segment, to the segment below it.

$$f_{mk} = 45 \text{ N/mm}^2$$
$$f_{md} = 30 \text{ N/mm}^2$$
$$f_{mm} = 27 \text{ N/mm}^2 \quad (13.45)$$
$$v_o = 20 \text{ mm}$$
$$v = v_o + 20 = 40 \text{ mm}$$
$$d = 1.34 \text{ m}$$
$$\text{x}_u = 732 \text{ mm}$$
$$\text{bx}_u = 157834 \text{ mm}$$
$$\text{k}_1 = 0.9$$
$$k_2 = 0.98 \quad (13.46)$$
$$k_3 = 0.12$$
$$k_4 = 38.18$$
$$k_5 = 0.5$$
$$f'_v = k_1 * k_2 * f_{md} = 26.41 \text{ N/mm}^2$$
$$A_{bv} = \pi * d * t = 314573 \text{ mm}^2$$
$$N'_{uv} = A_{bv} * f'_v = 8308 \text{ kN}$$
$$M_{Ed} = 2060 \text{ kNm}$$
$$N_c = 0.5 * bx_u * f'_v = 2084 \text{ kN}$$
$$e_c = 489.1 \text{ mm} \quad (13.47)$$
$$M_{Rd,\text{As}} = M_{Ed} * N_c * e_c = 1041 \text{ kNm}$$
$$d_s = d - x_u - c = 585 \text{ mm}$$
$$d_s' = x_u - c = 710 \text{ mm}$$
$$A_s = \frac{M_{Rd,As}}{f_y(d_s + d_s')} = 1847 \text{ mm}^2$$
$$M'_{uv} = N_c * e_c + f_y * A_s * (d_s + d_s') = 2060 \text{ kNm} \qquad \text{OK}$$

Connection number: 3 (h = 13 m)

$$n = 3$$
$$a_1 = 50 \text{ mm } \geq 50 \text{ mm}$$
$$\mathrm{a}_L = 100 \text{ mm } \geq 0.5L$$
$$\mathrm{a}_e = 20 \text{ mm}$$
$$L = a_1 + a_L + a_e = 170 \text{ mm} \qquad \text{OK} \tag{13.48}$$
$$b_1 = 50 \text{ mm}$$
$$\mathrm{b}_W = 100 \text{ mm } \geq 0.5L$$
$$\mathrm{b}_e = 50 \text{ mm}$$
$$W = b_1 + b_W + b_e = 200 \text{ mm} \qquad \text{OK}$$
$$h = 400 \text{ mm}$$

$$a = 200$$
$$l_{ov} = 400$$
$$l_{ov}/h = 1 \qquad \text{Deep beam} \tag{13.49}$$
$$z = 0.2 * l + 0.4 * h = 240 \text{ mm} < 0.8l$$

$$F_v = 3976 \text{ kN}$$
$$\mathrm{F}_h = 0 \text{ kN}$$
$$M_d = F_v * a = 795 \text{ kNm}$$
$$A_{s,Fv} = \frac{M_{sd}}{f_y * z} = 7616 \text{ mm}^2 \tag{13.50}$$
$$A_{s,Fh} = 0 \text{ mm}^2$$
$$\phi = 25 \text{ mm}$$
$$n = \frac{A_s}{\phi} = 16$$

Now that the amount of reinforcement necessary to introduce the prestressing into the wall is known, the shear resistance of the joint can be calculated.

$$\begin{aligned}
V &= 149 \text{ kN} \\
n_{bolt} &= 4 \text{ bolts} \\
F_{v,Ed} &= \frac{V}{n} = 37 \text{ kN} \\
a_{bolt,l} &= 20 \text{ mm} && \text{OK} \\
a_{bolt,r} &= L - a_{bolt,l} = 150 \text{ mm} \\
b_{bolt,l} &= 20 \text{ mm} \\
b_{bolt,m} &= W - b_{bolt,l} - b_{bolt,r} = 160 \text{ mm} && \text{OK} \\
b_{bolt,r} &= 20 \text{ mm}
\end{aligned} \tag{13.51}$$

$$\begin{aligned}
&\text{Bolt quality: 5.6} \\
f_{yb} &= 300 \text{ N/mm}^2 \\
f_{ub} &= 500 \text{ N/mm}^2 \\
\alpha_v &= 0.6 \\
\gamma_m &= 1.25 \\
M &= 18 \text{ mm} \\
F_{v,Rd} &= \frac{\alpha_v * f_{ub} * A_{b,s}}{\gamma_m} = 46 \text{ kN} && \text{OK}
\end{aligned} \tag{13.52}$$

Finally it is checked if the moment and compression force can be transferred from the top segment, to the segment below it.

$$\begin{aligned}
f_{mk} &= 45 \text{ N/mm}^2 \\
f_{md} &= 30 \text{ N/mm}^2 \\
f_{mm} &= 27 \text{ N/mm}^2 \\
v_o &= 20 \text{ mm} \\
v &= v_o + 20 = 40 \text{ mm}
\end{aligned} \tag{13.53}$$

$$\begin{aligned}
d &= 1.81 \text{ m} \\
\text{x}_u &= 980 \text{ mm} \\
\text{bx}_u &= 178063 \text{ mm} \\
\text{k}_1 &= 0.9 \\
k_2 &= 0.98 \\
k_3 &= 0.12 \\
k_4 &= 38.18 \\
k_5 &= 0.5
\end{aligned} \tag{13.54}$$

$$f'_v = k_1 * k_2 * f_{md} = 26.41 \text{ N/mm}^2$$

$$A_{bv} = \pi * d * t = 427009 \text{ mm}^2$$

$$N'_{uv} = A_{bv} * f'_v = 11278 \text{ kN}$$

$$M_{Ed} = 2060 \text{ kNm}$$

$$N_c = 0.5 * bx_u * f'_v = 2849 \text{ kN}$$

$$e_c = 656.6 \text{ mm} \tag{13.55}$$

$$M_{Rd,\text{As}} = M_{Ed} * N_c * e_c = 189 \text{ kNm}$$

$$d_s = d - x_u - c = 812 \text{ mm}$$

$$d_s' = x_u - c = 960 \text{ mm}$$

$$A_s = \frac{M_{Rd,As}}{f_y(d_s + d_s')} = 245 \text{ mm}^2$$

$$M'_{uv} = N_c * e_c + f_y * A_s * (d_s + d_s') = 2060 \text{ kNm} \qquad \text{OK}$$

Appendix P: Isolator connections for UHSC

Connection number: $1_{tension}$ (h = 49.63 m)

$$n = 1_t$$

$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$b_w = 50 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 45 + 60 = 105 \text{ mm} \qquad \text{OK}$$

$$\text{k=1+}\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 1000 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0.23$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} = 0.47 \text{ N/mm}^2 > 0.2 f_{ck}$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$

$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 7 \text{ kN}$$

$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 20 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 3.84 \text{ N/mm}^2 \qquad (13.56)$$

$$D = 60 \text{ mm}$$

$$\text{F}_p = \pi * D * d * v_{Rd,c} = 76 \text{ kN} \qquad \text{OK}$$

Connection number: $1_{compression}$ (h = 46.13 m)

$$n = 1_c$$

$$V_{Ed} = F_{c,H} = 91 \text{ kN}$$

$$b_w = 90 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 45 + 75 = 120 \text{ mm} \qquad \text{OK} \qquad (13.57)$$

$$d^* = 2b = 180 \text{ mm}$$

$$\text{u}_0 = \pi * d^* = 565 \text{ mm}$$

$$v_{Ed} = \frac{\beta * V_{Ed}}{u_0 * d} = 1.33 \text{ Nmm}^2$$

$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$\text{k=1+}\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 0 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} = 0.55 \text{ N/mm}^2 > 0.2 f_{ck}$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$

$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 7 \text{ kN}$$

$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 0 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 1.37 \text{ N/mm}^2 \qquad \text{OK} \qquad (13.58)$$

Connection number: $2_{tension}$ (h = 39.63 m)

$$n = 2_t$$

$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$b_w = 50 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 55 + 20 = 75 \text{ mm} \qquad \text{OK}$$

$$\text{k}=1+\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 0 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0.00$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} > 0.2 f_{ck} = 34.00 \text{ N/mm}^2$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$

$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 24 \text{ kN}$$

$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 19 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 6.39 \text{ N/mm}^2 \qquad (13.59)$$

$$D = 50 \text{ mm}$$

$$\text{F}_p = \pi * D * d * v_{Rd,c} = 75 \text{ kN} \qquad \text{OK}$$

Connection number: $2_{compression}$ (h = 36.13 m)

$$n = 2_c$$

$$V_{Ed} = F_{c,H} = 88 \text{ kN}$$

$$b_w = 50 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 55 + 0 = 55 \text{ mm} \qquad \text{OK} \qquad (13.60)$$

$$d^* = 2b = 100 \text{ mm}$$

$$\text{u}_0 = \pi * d^* = 314 \text{ mm}$$

$$v_{Ed} = \frac{\beta * V_{Ed}}{u_0 * d} = 5.07 \text{ Nmm}^2$$

$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$\text{k=1+}\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 0 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} = 33.30 \text{ N/mm}^2 < 0.2 f_{ck}$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}^{\ 0.5} = 1.29 \text{ N/mm}^2$$

$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 33 \text{ kN}$$

$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 26 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 6.29 \text{ N/mm}^2 \qquad \text{OK} \qquad (13.61)$$

Connection number: $3_{tension}$ (h = 29.63 m)

$$n = 3_t$$

$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$b_w = 50 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 65 + 5 = 70 \text{ mm} \qquad \text{OK}$$

$$\text{k}=1+\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 0 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0.00$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} > 0.2 f_{ck} = 34.00 \text{ N/mm}^2$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$

$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 22 \text{ kN}$$

$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 18 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 6.39 \text{ N/mm}^2 \qquad (13.62)$$

$$D = 50 \text{ mm}$$

$$\text{F}_p = \pi * D * d * v_{Rd,c} = 70 \text{ kN} \qquad \text{OK}$$

Connection number: $3_{compression}$ (h = 26.13 m)

$$n = 3_c$$

$$V_{Ed} = F_{c,H} = 82 \text{ kN}$$

$$b_w = 50 \text{ mm} \qquad \text{OK}$$

$$d = t + x = 65 + 0 = 65 \text{ mm} \qquad \text{OK} \qquad (13.63)$$

$$d^* = 2b = 100 \text{ mm}$$

$$\text{u}_0 = \pi * d^* = 314 \text{ mm}$$

$$v_{Ed} = \frac{\beta * V_{Ed}}{u_0 * d} = 4.03 \text{ Nmm}^2$$

$$C_{Rd,c} = 0.18/\gamma_c = 0.12$$

$$f_{ck} = 170 \text{ N/mm}^2$$

$$\text{k=1+}\sqrt{\frac{200}{d}} \geq 2$$

$$A_{sl} = 0 \text{ mm}^2$$

$$\rho_{sl} = \frac{A_{sl}}{b_w * d} = 0$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \frac{N_{sd} + P_{m0}}{A_c} > 0.2 f_{ck} = 34.00 \text{ N/mm}^2$$

$$v_{\min} = 0.035 * k^{1.5} * f_{ck}{}^{0.5} = 1.29 \text{ N/mm}^2$$

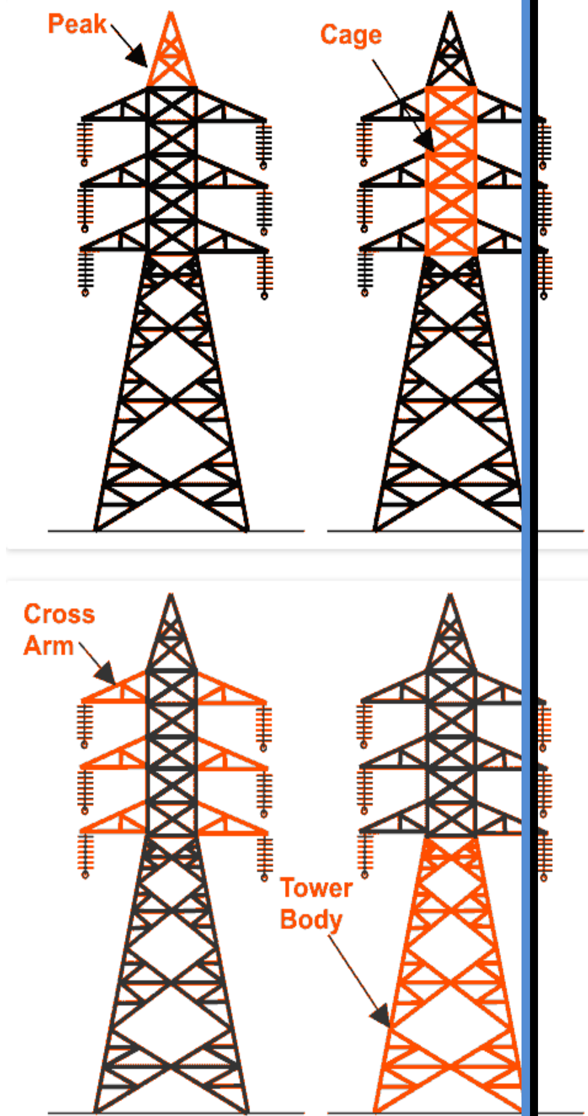
$$V_{Rd,c} = (v_{\min} + k_1 * \sigma_{cp}) * b_w * d = 34 \text{ kN}$$

$$\text{V}_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_{sl} * f_{ck})^{1/3} + k_1 * \sigma_{cp}) * b_w * d = 27 \text{ kN}$$

$$v_{Rd,c} = \frac{V_{Rd,c}}{b_w * d} = 6.39 \text{ N/mm}^2 \qquad \text{OK} \qquad (13.64)$$

Appendix Q: Steel truss dimensions[20]

Height mast	h	60	m
Truss thickness	t	12	mm
Peak			
Width peak	bpeak	3,50	m
Height peak	hpeak	4,00	m
Area peak	Apeak	2,10	m^2
Cage			
Width cage	bcage	3,50	m
Height cage	hcage	28,00	m
Area cage	Acage	39,20	m^2
Cross arms			
Width middle cross arms	bcross,mid	5,00	m
Height middle cross arms	hcross,mid	4,00	m
Area cross arms	Across	66,64	m^2
Tower body			
Width tower body, top	bbody,top	3,50	m
Width tower body, bot	bbody,bot	6,00	m
Height tower body	hbody	29,00	m
Area tower body	Abody	82,65	m^2
Total area mast	Aav	190,59	m^2
Volume mast	Vav	3,05	m^3
Foundation			
Foundation thickness	t	1,50	m
Foudation dimensions	l or b	4,00	m
Foundation area per leg	Aleg	16,00	m^2
Foundation volume per leg	Vleg	24,00	m^3
Number of piles per leg	n	15	



[20] Note that in practice for the steel truss mast, every individual slat is calculated. However as this falls outside of the scope of this thesis, a general estimation is made about the amount of steel in the mast. This estimation is based on previous projects.

Appendix R: Steel tube dimensions

Height mast	h	57	m
Wall thickness	t	17	mm
Wall thickness refinement	n	35%	
Wall thickness top	t_{top}	11,05	mm
Wall thickness average	tav	15,04	mm
Diameter foot	dbot	2,20	m
Diameter top	dtop	0,50	m
Diameter average	dav	1,35	m
Height lightning wire	hlw	57,00	m
Height 1st conductors 380 kV	h380,1	47,00	m
Height 2nd conductors 380 kV	h380,2	37,00	m
Height 3rd conductors 380 kV	h380,3	27,00	m
Height passive loops	hpl	22,00	m
Section properties			
Neutral line	zbot	1100	mm
Area mast at top	Atop	25796	mm2
Area mast at foot	Abot	116588	mm2
Area mast average	Aav	71192	mm2
Area mast average with refinement	Aav,ref	63062	mm2
Volume	V	4,06E+09	mm3
Volume with refinement	V,ref	3,59+09	mm3
Foundation			
Foundation thickness	t	1,40	m
Foundation diameter	d	7,00	m
Foundation area	A	38,48	m2
Foundation volume	V	53,88	m3
Number of piles	n	25	

Appendix S: Energy unit rates

Energy unit rate	**Embodied energy**	**Embodied emission**
Material	[MJ/kg]	[kg CO_2/kg]
Concrete precast C45/55	2.10	0.159
Concrete precast C90/105	2.60	0.196
Concrete precast C170/200	5.39	0.329
Foundation block C30/37	1.80	0.120
Reinforcing steel B500B	10.88	0.720
Prestressing steel Y1860S	36.00	1.250
Galvanized steel S235	21.63	0.686
Structural steel S355	12.34	1.350
Coating	97,00	3.130

Table 122: Energy unit rates for embodied energy & emission

Appendix T: Embodied energy

Embodied energy	**Steel truss**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Galvanized steel S235	21.63	MJ/kg	21.828	472129	MJ
Foundation block C30/37	1080	MJ/kg	424.800	764640	MJ
Foundation reinforcement B500B	10.88	MJ/kg	7.488	81469	MJ
Total energy				**1.318.239**	MJ

Table 123: Embodied energy steel truss

Embodied energy	**Steel tube**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Construction steel S355	12,34	MJ/kg	73.011	900664	MJ
Coating	97,00	MJ/kg	738	71543	MJ
Foundation block C30/37	1,80	MJ/kg	420.616	757109	MJ
Foundation reinforcement B500B	10,88	MJ/kg	8.405	91447	MJ
Total energy				**1.820.762**	MJ

Table 124: Embodied energy steel tube

Embodied energy	**OSC**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C45/55	2,10	MJ/kg	365.573	767703	MJ
Reinforcing steel B500B	10,88	MJ/kg	22.524	245057	MJ
Prestressing steel Y1860S	36,00	MJ/kg	10.724	386055	MJ
Foundation block C30/37	1,80	MJ/kg	488.625	879525	MJ
Foundation reinforcement B500B	10,88	MJ/kg	10.615	115495	MJ
Total energy				**2.393.835**	MJ

Table 125: Embodied energy OSC

Embodied energy	**HSC**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C90/105	2,60	MJ/kg	135.244	351635	MJ
Reinforcing steel B500B	10,88	MJ/kg	4.377	47624	MJ
Prestressing steel Y1860S	36,00	MJ/kg	8.579	308844	MJ
Foundation block C30/37	1,80	MJ/kg	471.661	848989	MJ
Foundation reinforcement B500B	10,88	MJ/kg	10.064	109496	MJ
Total energy				**1.666.588**	MJ

Table 126: Embodied energy HSC

Embodied energy	UHSC				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C170/200	5,39	MJ/kg	74.894	403677	MJ
Reinforcing steel B500B	10,88	MJ/kg	2.337	25423	MJ
Prestressing steel Y1860S	36,00	MJ/kg	8.579	308844	MJ
Foundation block C30/37	1,80	MJ/kg	455.148	819267	MJ
Foundation reinforcement B500B	10,88	MJ/kg	9.527	103657	MJ
Total energy				**1.660.868**	MJ

Table 127: Embodied energy UHSC

Appendix U: Embodied emissions

Embodied emissions	Steel truss				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Galvanized steel S235	1,350	kg CO_2/kg	21.828	29467	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	424.800	50976	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	7.488	5391	kg CO_2
Total energy				**85.835**	kg CO_2

Table 128: Embodied emissions steel truss

Embodied emissions	Steel tube				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Construction steel S355	0,686	kg CO_2/kg	73.011	50056	kg CO_2
Coating	3,130	kg CO_2/kg	738	2309	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	420.616	50474	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	8.405	6052	kg CO_2
Total energy				**108.890**	kg CO_2

Table 129: Embodied emissions steel tube

Embodied emissions	OSC				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C45/55	0,159	kg CO_2/kg	365.573	58126	kg CO_2
Reinforcing steel B500B	0,720	kg CO_2/kg	22.524	16217	kg CO_2
Prestressing steel Y1860S	1,250	kg CO_2/kg	10.724	13405	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	488.625	58635	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	10.615	7643	kg CO_2
Total energy				**154.026**	kg CO_2

Table 130: Embodied emissions OSC

Embodied emissions	HSC				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C90/105	0,196	kg CO_2/kg	135.244	26508	kg CO_2
Reinforcing steel B500B	0,720	kg CO_2/kg	4.377	3152	kg CO_2
Prestressing steel Y1860S	1,250	kg CO_2/kg	8.579	10724	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	471.661	56599	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	10.064	7246	kg CO_2
Total energy				**104.229**	kg CO_2

Table 131: Embodied emissions HSC

Embodied emissions	**UHSC**				
Material	Energy unit rate	Unit	Quantity	Total	Unit
Concrete segments C170/200	0,329	kg CO_2/kg	74.894	24640	kg CO_2
Reinforcing steel B500B	0,720	kg CO_2/kg	2.337	1682	kg CO_2
Prestressing steel Y1860S	1,250	kg CO_2/kg	8.579	10724	kg CO_2
Foundation block C30/37	0,120	kg CO_2/kg	455.148	54618	kg CO_2
Foundation reinforcement B500B	0,720	kg CO_2/kg	9.527	6860	kg CO_2
Total energy				**98.524**	kg CO_2

Table 132: Embodied emissions UHSC

Appendix V: Price list[21]

Activity	Price	Unit
Material		
Concrete segments C45/55	€ 200	/m^3
Concrete segments C90/105	€ 365	/m^3
Concrete segments C170/200	€ 1110	/m^3
Steel moulds	€ 250	/m^2
Steel tube rolling process	€ 40	/m^2
Steel section galvanizing process	€ 80	/m^2
Structural steel S355 for tubes	€ 2.00	/kg
Structural steel S255 for sections	€ 2.50	/kg
Coating steel tubes	€ 50	/m^2
Foundation block(s) C30/37	€ 80	/m^3
Foundation piles C45/55	€ 35	/p
Reinforcing steel B500B[22]	€ 1	/kg
Prestressing steel Y1860S[23]	€ 4	/kg
Gravel layer	€ 5	/m^3
Equipment		
Pile driver	€ 2500	/day
Truck (small)	€ 300	/unit
Truck (large)	€ 500	/unit
Main crane	€ 2500	/day
Support	€ 1500	/day
Prestressing	€ 5000	/layer
Labor		
Labor costs	€ 40	/mh
Labor man-hours concreting segments	€ 4	/m^3
Labor man-hours steel segments	€ 0.1	/kg
Labor man-hours coating	€ 1	/ m^2

[21] The presented prices are not fixed and may change from manufacturer to manufacturer.
[22] Includes costs for cutting, placement, etc.
[23] Includes costs for ducts, stressing, anchorage, etc.

Labor man-hours foundation	€ 4	/m³
Labor man-hours reinforcement	€ 0.4	/kg
Labor man-hours assembly OSC	€ 60	/m³
Labor man-hours assembly HSC	€ 80	/m³
Labor man-hours assembly UHSC	€ 160	/m³
Labor man-hours assembly steel tubes S355	€ 0.2	/kg
Labor man-hours assembly steel sections S235	€ 0.4	/kg
Labor man-hours prestressing	€ 2	/kg
Maintenance		
Conductors (maintenance, replacement)	€ 100.000	-
Cleaning steel tubes	€ 5	/m²
Cleaning steel sections	€ 25	/m²
Repainting steel tubes	€ 25	/m²
Platforms etc.	€ 216	/day
Prestressing 1x/10 years	€ 50	/tendon

Table 133: Price list

Appendix W: Costs for OSC

Material/Activity	Cost unit rate	Unit	Quantity	Total
Direct Costs				
Material costs				
Concrete segments C45/55	€ 200	/m³	152,01	€ 30.401
Steel moulds[24]	€ 250	/m²	336,37	€ 84.093
Foundation block(s) C30/37	€ 80	/m³	136,09	€ 10.888
Foundation piles C45/55[25]	€ 35	/p	50,00	€ 1.750
Reinforcing steel B500B[26]	€ 1	/kg	33138,91	€ 24.854
Prestressing steel Y1860S[27]	€ 4	/kg	10723,75	€ 42.895
Total material costs				*€ 194.880*

Building costs				
Building site preparation	€ 5	/m³	1640	€ 8.200
Work floor	€ 25	/m²	200	€ 5.000
Foundation activities	€ 350	/m³	53,01	€ 18.555
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 300	/unit	11	€ 3.300
Main crane	€ 2.500	/day	6	€ 15.000
Support crane	€ 1.500	/day	6	€ 9.000
Connections (% of material+prestressing)	5%	perc.	€ 73.296	€ 40.313
Prestressing per layers	€ 5.000	/layer	10	€ 50.000
Landscape restoration	€ 5	/m³	1640	€ 8.200
Total building costs				*€ 165.068*

Labor costs	with 1mh=€40			
Labor man-hours concreting segments	€ 4	/m³	152,01	€ 608
Labor man-hours foundation	€ 4	/m³	186,09	€ 744
Labor man-hours reinforcement	€ 0,4	/kg	33138,91	€ 13.256
Labor man-hours assembly	€ 60	/m³	152,01	€ 9.120

[24] Outer and inner mould
[25] Including reinforcement
[26] Including cutting, placing, etc.
[27] Including ducts, stressing and anchorage

Labor man-hours prestressing	€ 2	/kg	10723,75	€ 21.448
Total labor costs				€ 45.176

Primary direct costs				€ 405.124
Direct costs to be detailed	10,00%			€ 40.512
Direct costs				€ 445.636

Indirect costs				
Maintenance costs (100 years)				
Conductors 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 4x/100 years	€ 5	/m²	537,21	€ 3.126
Surface treatment 4x/100 years	€ 20	/m²	537,21	€ 12.504
Platforms etc. 4x/100 years	€ 216	/day	5	€ 1.257
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Connections (% of material) 10x/100 years	10%	perc.	40313	€ 23.457
Prestressing 10x/100 years	€ 50	/tendon	20	€ 2.909
Total maintenance costs				€ 251.667

Primary indirect costs				
Investment costs	5,00%	-	€ 445.636	€ 22.282
Design costs	5,00%	-	€ 445.636	€ 22.282
General construction costs	1,00%	-	€ 445.636	€ 4.456
Execution costs	5,00%	-	€ 494.656	€ 24.733
Overhead costs	5,00%	-	€ 519.389	€ 25.969
Insurance costs	1,50%	-	€ 545.359	€ 8.180
Profit	3,00%	-	€ 553.539	€ 16.606
Risk	3,00%	-	€ 553.539	€ 16.606
Primary indirect costs				€ 141.115
Anticipated costs				€ 659.972

Additional indirect costs				
Risk	10,00%	-	€ 659.972	€ 65.997

R&D	1,00%	-	€ 659.972	€ 6.600
Engineering costs	2,00%	-	€ 659.972	€ 13.199
Fees & license permits	1,70%	-	€ 659.972	€ 11.220
Additional indirect costs				€ 97.016

Total costs per mast location		***€ 756.988***
Total costs per km		***€ 1.892.469***

Table 134: Costs for OSC

Appendix X: Costs for HSC

Material/Activity	Cost unit rate	Unit	Quantity	Total
Direct Costs				
Material costs				
Concrete segments C90/105	€ 365	/m^3	56,12	€ 20.483
Steel moulds[10]	€ 250	/m^2	264,71	€ 66.178
Foundation block(s) C30/37	€ 80	/m^3	129,03	€ 10.322
Foundation piles C45/55[11]	€ 35	/p	50,00	€ 1.750
Reinforcing steel B500B[12]	€ 1	/kg	43997,48	€ 32.998
Prestressing steel Y1860S[13]	€ 4	/kg	8579,00	€ 34.316
Total material costs				*€ 166.047*

Building costs				
Building site preparation	€ 5	/m^3	1640	€ 8.200
Work floor	25	/m^2	200	€ 5.000
Foundation activities	€ 350	/m^3	53,01	€ 18.555
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 300	/unit	10	€ 3.000
Main crane	€ 2.500	/day	4	€ 10.000
Support crane	€ 1.500	/day	4	€ 6.000
Connections (% of material+prestressing)	5%	perc.	€ 54.799	€ 13.700
Prestressing per layers	€ 5.000	/layer	8	€ 40.000
Landscape restoration	€ 5	/m^3	1640	€ 8.200
Total building costs				*€ 120.155*

Labor costs				
Labor man-hours concreting segments	€ 4.5	/m^3	56,12	€ 253
Labor man-hours foundation	€ 4	/m^3	179,03	€ 716
Labor man-hours reinforcement	€ 0,4	/kg	43997,48	€ 17.599
Labor man-hours assembly	€ 80	/m^3	56,12	€ 4.489
Labor man-hours prestressing	€ 2	/kg	8579,00	€ 17.158
Total labor costs				€ 40.215

Primary direct costs		€ 326.417
Direct costs to be detailed	10,00%	€ 32.642
Direct costs		€ 359.059

Indirect costs

Maintenance costs (100 years)				
Conductors 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 3x/100 years	€ 5	/m^2	519,31	€ 1.770
Surface treatment 3x/100 years	€ 15	/m^2	519,31	€ 5.311
Platforms etc. 3x/100 years	€ 216	/day	5	€ 736
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Connections (% of material) 10x/100 years	10%	perc.	13700	€ 7.972
Prestressing 10x/100 years	€ 50	/tendon	16	€ 2.328
Total maintenance costs				€ 150.383

Primary indirect costs				
Investment costs	5,00%	-	€ 359.059	€ 17.953
Design costs	5,00%	-	€ 359.059	€ 17.953
General construction costs	1,00%	-	€ 359.059	€ 3.591
Execution costs	5,00%	-	€ 398.555	€ 19.928
Overhead costs	5,00%	-	€ 418.483	€ 20.924
Insurance costs	1,50%	-	€ 439.407	€ 6.591
Profit	3,00%	-	€ 445.998	€ 13.380
Risk	3,00%	-	€ 445.998	€ 13.380
Primary indirect costs				€ 113.699
Anticipated costs				€ 520.843

Additional indirect costs				
Risk	10,00%	-	€ 520.843	€ 52.084
R&D	1,00%	-	€ 520.843	€ 5.208
Engineering costs	2,00%	-	€ 520.843	€ 10.417
Fees & license permits	1,70%	-	€ 520.843	€ 8.854

Additional indirect costs		€ 76.564

Total costs per mast location		***€ 597.406***
Total costs per km		***€ 1.493.516***

Table 135: Costs for HSC

Appendix Y: Costs for UHSC masts

Material/Activity	Cost unit rate	Unit	Quantity	Total
Direct Costs				
Material costs				
Concrete segments C170/200	€ 110	/m^3	29,96	€ 33.253
Steel moulds[10]	€ 250	/m^2	253,40	€ 63.351
Foundation block(s) C30/37	€ 80	/m^3	122,15	€ 9.772
Foundation piles C45/55[11]	€ 35	/p	50,00	€ 1.750
Reinforcing steel B500B[12]	€ 1	/kg	9527,32	€ 7.145
Prestressing steel Y1860S[13]	€ 4	/kg	8579,00	€ 34.316
Total material costs				*€ 149.587*

Building costs				
Building site preparation	€ 5	/m^3	1640	€ 8.200
Work floor	€ 25	/m^2	200	€ 5.000
Foundation activities	€ 350	/m^3	53,01	€ 18.555
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 300	/unit	8	€ 2.400
Main crane	€ 2.500	/day	3	€ 7.500
Support crane	€ 1.500	/day	3	€ 4.500
Connections (% of material+prestressing)	20%	perc.	€ 67.569	€ 40.541
Prestressing per layers	€ 5.000	/layer	6	€ 30.000
Landscape restoration	€ 5	/m^3	1640	€ 8.200
Total building costs				*€ 132.396*

Labor costs				
Labor man-hours concreting segments	€ 5	/m^3	29,96	€ 150
Labor man-hours foundation	€ 4	/m^3	172,15	€ 689
Labor man-hours reinforcement	€ 0,4	/kg	9527,32	€ 3.811
Labor man-hours assembly	€ 160	/m^3	29,96	€ 4.793
Labor man-hours prestressing	€ 2	/kg	8579,00	€ 17.158
Total labor costs				€ 26.600

Primary direct costs		€ 308.583
Direct costs to be detailed	10,00%	€ 30.858
Direct costs		€ 339.442

Indirect costs

Maintenance costs (100 years)				
Conductors 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 2x/100 years	€ 5	/m²	519,31	€ 922
Surface treatment 2x/100 years	€ 5	/m²	519,31	€ 922
Platforms etc. 2x/100 years	€ 216	/day	5	€ 384
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Connections (% of material) 10x/100 years	10%	perc.	40541	€ 11.795
Prestressing 10x/100 years	€ 50	/tendon	16	€ 2.328
Total maintenance costs				€ 141.754

Primary indirect costs				
Investment costs	5,00%	-	€ 339.442	€ 16.972
Design costs	5,00%	-	€ 339.442	€ 16.972
General construction costs	1,00%	-	€ 339.442	€ 3.394
Execution costs	5,00%	-	€ 376.780	€ 18.839
Overhead costs	5,00%	-	€ 395.619	€ 19.781
Insurance costs	1,50%	-	€ 415.400	€ 6.231
Profit	3,00%	-	€ 421.631	€ 12.649
Risk	3,00%	-	€ 421.631	€ 12.649
Primary indirect costs				€ 107.487
Anticipated costs				€ 493.247

Additional indirect costs				
Risk	10,00%	-	€ 576.052	€ 49.325
R&D	1,00%	-	€ 576.052	€ 4.932
Engineering costs	2,00%	-	€ 576.052	€ 9.865
Fees & license permits	1,70%	-	€ 576.052	€ 8.385

Additional indirect costs		€ 72.507

Total costs per mast location		***€ 565.754***
Total costs per km		***€ 1.414.385***

Table 136: Costs for UHSC

Appendix Z: Material costs for steel truss mast

Material/Activity	Cost unit rate	Unit	Quantity	Total
Direct Costs				
Material costs				
Steel sections S235	€ 2,50	/kg	23786	€ 59.465
Galvanizing process	€ 80	/m²	190,59	€ 15.247
Foundation block(s) C30/37	€ 80	/m³	96,00	€ 7.680
Foundation piles C45/55	€ 35	/p	60,00	€ 2.100
Reinforcing steel B500B	€ 1	/kg	7488,00	€ 7.488
Total material costs				*€ 91.981*

Building costs				
Building site preparation	€ 5	/m³	1640	€ 8.200
Work floor	€ 25	/m²	200	€ 5.000
Foundation activities	€ 350	/m³	63,62	€ 22.266
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 300	/unit	3	€ 900
Main crane	€ 2.500	/day	2	€ 5.000
Support crane	€ 1.500	/day	2	€ 3.000
Attachments (% of material)	20%	perc.	€ 59.465	€ 11.893
Landscape restoration	€ 5	/m³	1640	€ 8.200
Total building costs				*€ 71.959*

Labor costs				
Labor man-hours steel sections S235	€ 0,2	/kg	23786,02	€ 5.709
Labor man-hours foundation	€ 4	/m³	156,00	€ 624
Labor man-hours reinforcement	€ 0,4	/kg	7488,00	€ 2.995
Labor man-hours assembly	€ 0,4	/kg	23786,02	€ 9.514
Total labor costs				€ 18.842

Primary direct costs				€ 182.782
Direct costs to be detailed	10,00%			€ 18.278
Direct costs				€ 201.060

Indirect costs				
Maintenance costs (100 years)				
Conductors 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 10x/100 years	€ 25	/m²	190,59	€ 22.716
Platforms etc. 10x/100 years	€ 216	/day	15	€ 15.446
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Attachments (% of material) 10x/100 years	20%	perc.	59465	€ 34.602
Total maintenance costs				€ 102.731

Primary indirect costs				
Investment costs	5,00%	-	€ 201.060	€ 10.053
Design costs	5,00%	-	€ 201.060	€ 10.053
General construction costs	1,00%	-	€ 201.060	€ 2.011
Execution costs	5,00%	-	€ 223.177	€ 11.159
Overhead costs	5,00%	-	€ 234.335	€ 11.717
Insurance costs	1,50%	-	€ 246.052	€ 3.691
Profit	3,00%	-	€ 249.743	€ 7.492
Risk	3,00%	-	€ 249.743	€ 7.492
Primary indirect costs				€ 63.668
Anticipated costs				€ 367.459

Additional indirect costs				
Risk	10,00%	-	€ 367.459	€ 36.746
R&D	1,00%	-	€ 367.459	€ 3.675
Engineering costs	2,00%	-	€ 367.459	€ 7.349
Fees & license permits	1,70%	-	€ 367.459	€ 6.247
Additional indirect costs				€ 54.016

Total costs per mast location		***€ 421.475***
Total costs per km		***€ 1.053.688***

Appendix AA: Material costs for steel tube masts

Material/Activity	Cost unit rate	Unit	Quantity	Total
Direct Costs				
Material costs				
Steel segments S355	€ 2,00	/kg	56075	€ 112.149
Rolling process	€ 40	/m²	483,49	€ 19.340
Coating	€ 50	/m²	483,49	€ 24.175
Foundation block(s) C30/37	€ 80	/m³	107,76	€ 8.621
Foundation piles C45/55	€ 35	/p	50,00	€ 1.750
Reinforcing steel B500B	€ 1	/kg	8405,02	€ 8.405
Total material costs				*€ 174.439*

Building costs				
Building site preparation	€ 5	/m³	1640	€ 8.200
Work floor	€ 25	/m²	200	€ 5.000
Foundation activities	€ 350	/m³	53,01	€ 18.555
Pile driver	€ 2.500	/day	3	€ 7.500
Transport (trucks)	€ 500	/unit	4	€ 2.000
Main crane	€ 2.500	/day	1	€ 2.500
Support crane	€ 1.500	/day	1	€ 1.500
Attachments (% of material)	20%	perc.	€ 112.149	€ 22.430
Landscape restoration	€ 5	/m³	1640	€ 8.200
Total building costs				*€ 75.885*

Labor costs				
Labor man-hours steel segments S355	€ 0,1	/kg	56074,51	€ 5.607
Labor man-hours coating	€ 1	/m²	483,49	€ 483
Labor man-hours foundation	€ 4	/m³	157,76	€ 631
Labor man-hours reinforcement	€ 0,4	/kg	8405,02	€ 3.362
Labor man-hours assembly	€ 0,2	/kg	56074,51	€ 11.215
Total labor costs				€ 21.299

Primary direct costs				€ 271.622
Direct costs to be detailed	10,00%			€ 27.162
Direct costs				€ 298.785

Indirect costs				
Maintenance costs (100 years)				
Conductors 1x/100 years	€ 100.000	-	1	€ 29.094
Cleaning 5x/100 years	€ 5	/m²	483,49	€ 4.502
Recoating 5x/100 years	€ 25	/m²	483,49	€ 22.508
Platforms etc. 5x/100 years	€ 216	/day	5	€ 2.011
Incidental corrective maintenance 1x/year	€ 30	-	1	€ 873
Attachments (% of material) 10x/100 years	10%	perc.	22430	€ 13.052
Total maintenance costs				€ 72.040

Primary indirect costs				
Investment costs	5,00%	-	€ 298.785	€ 14.939
Design costs	5,00%	-	€ 298.785	€ 14.939
General construction costs	1,00%	-	€ 298.785	€ 2.988
Execution costs	5,00%	-	€ 331.651	€ 16.583
Overhead costs	5,00%	-	€ 348.234	€ 17.412
Insurance costs	1,50%	-	€ 365.645	€ 5.485
Profit	3,00%	-	€ 371.130	€ 11.134
Risk	3,00%	-	€ 371.130	€ 11.134
Primary indirect costs				€ 94.613
Anticipated costs				€ 465.438

Additional indirect costs				
Risk	10,00%	-	€ 465.438	€ 46.544
R&D	1,00%	-	€ 465.438	€ 4.654
Engineering costs	2,00%	-	€ 465.438	€ 9.309
Fees & license permits	1,70%	-	€ 465.438	€ 7.912
Additional indirect costs				€ 68.419

Total costs per mast location		***€ 533.857***
Total costs per km		***€ 1.334.642***

Table 137: Material costs for steel tube masts

Appendix AB: Weight of criteria for MCDA

Criteria	Technical feasibility	Magnetic field influence	Execution feasibility	Nuisance	Sustainability	Unforeseen maintenance	Landscape integration	Marketability	Costs	Result	Correction	Weight
Technical feasibility	X	0	0	2	1	0	0	0	0	3	4.5	5%
Magnetic field influence	2	X	1	2	2	2	2	2	0	13	14.5	17%
Execution feasibility	2	1	X	2	1	1	0	1	0	8	9.5	11%
Nuisance	0	0	0	X	0	0	0	0	0	0	1.5	2%
Sustainability	1	0	1	2	X	0	1	1	0	6	7.5	9%
Unforeseen maintenance	2	0	1	2	2	X	2	2	0	11	12.5	15%
Landscape integration	2	0	2	2	1	0	X	1	0	8	9.5	11%
Marketability	2	0	1	2	1	0	1	X	0	7	8.5	10%
Costs	2	2	2	2	2	2	2	2	X	16	17.5	20%
									Total:	72	85.5	100%

Table 138: Weight of criteria for MCDA

Table 138 can be filled as follows. First a criteria in the far left column is chosen. This criteria is then compared with any criteria in the upper most row. If the left criteria is thought to be superior, then a value of two is applied. If it is thought to be of less importance, then a zero is applied. If the compared criteria seem to be equally important, than a value of one is applied.
E.g. When comparing the 3rd row, magnetic field influence, with the 5th column, nuisance, it can be concluded that magnetic field influences are of far greater importance that nuisance during construction. So a two is filled in the box located in the 3rd row and the 5th column.

Appendix AC: Weight of criteria for VM

Criteria	Technical feasibility	Magnetic field influence	Execution feasibility	Nuisance	Sustainability	Unforeseen maintenance	Landscape integration	Marketability	Result	Correction	Weight
Technical feasibility	X	0	0	2	1	0	0	0	3	4.5	7%
Magnetic field influence	2	X	1	2	2	2	2	2	13	14.5	21%
Execution feasibility	2	1	X	2	1	1	0	1	8	9.5	14%
Nuisance	0	0	0	X	0	0	0	0	0	1.5	2%
Sustainability	1	0	1	2	X	0	1	1	6	7.5	11%
Unforeseen maintenance	2	0	1	2	2	X	2	2	11	12.5	18%
Landscape integration	2	0	2	2	1	0	X	1	8	9.5	14%
Marketability	2	0	1	2	1	0	1	X	7	8.5	13%
								Total:	72	85.5	100%

Table 139: Weight of criteria for VM

Appendix AD: Weight & score comparison for MCDA

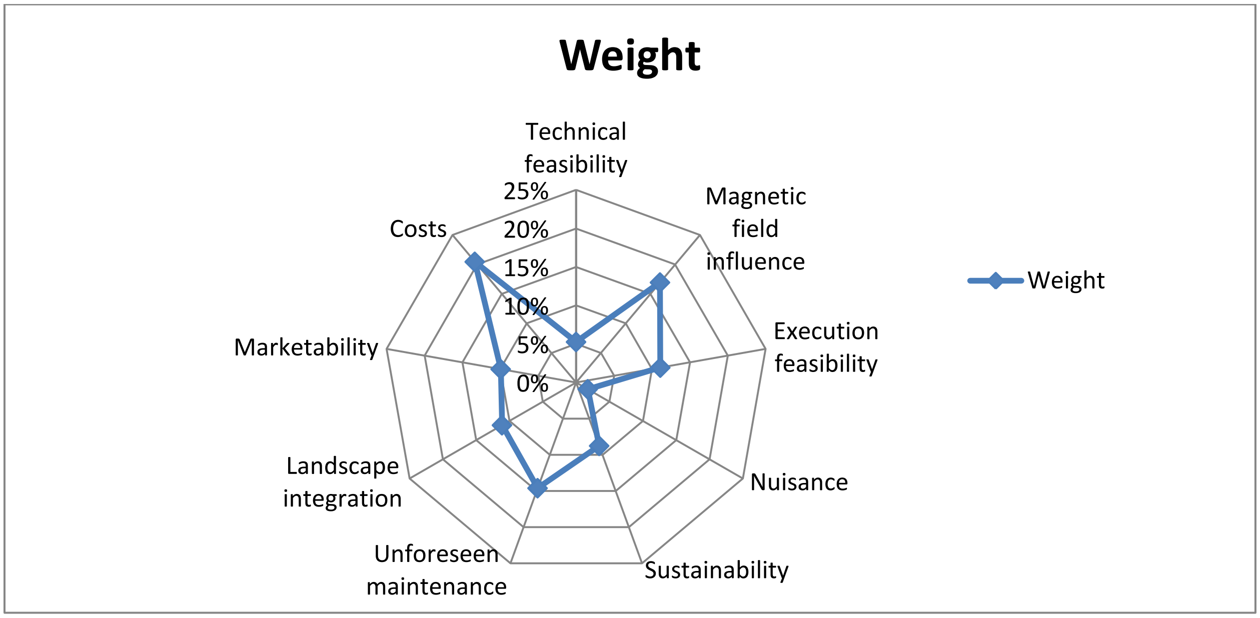


Figure 145: Weight of criteria for MCDA

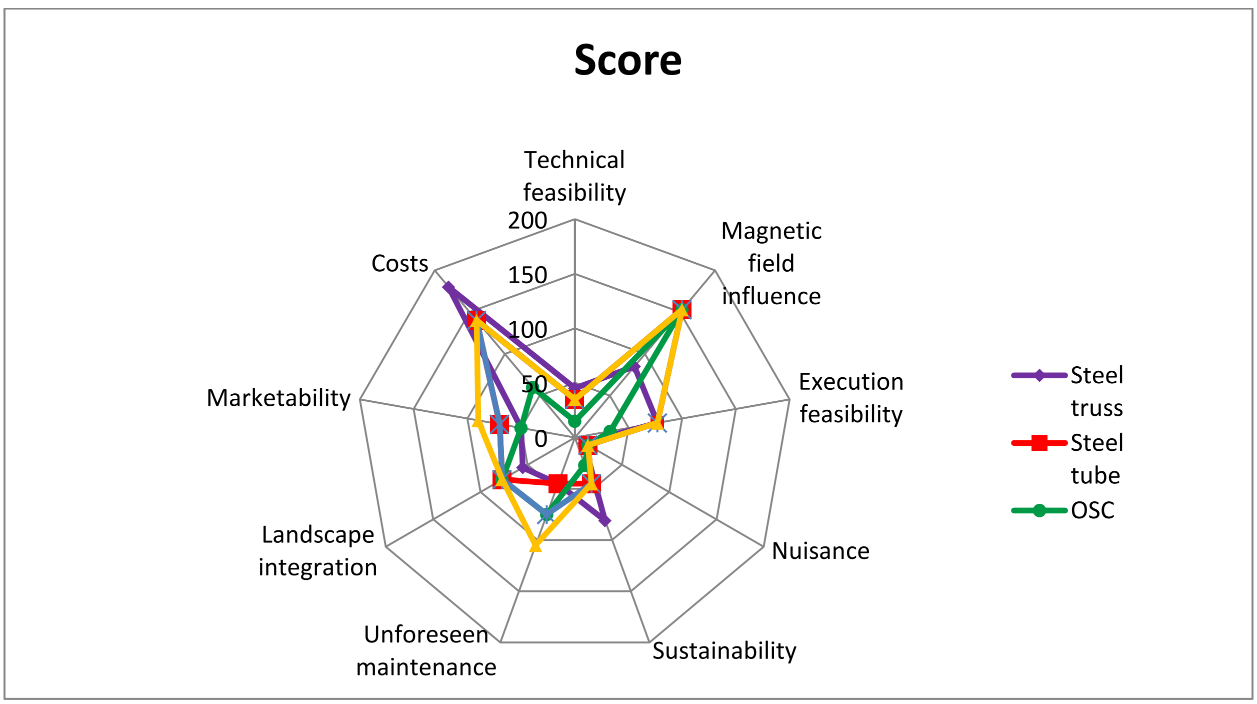


Figure 146: MCDA score

Appendix AE: Weight & score comparison for VM

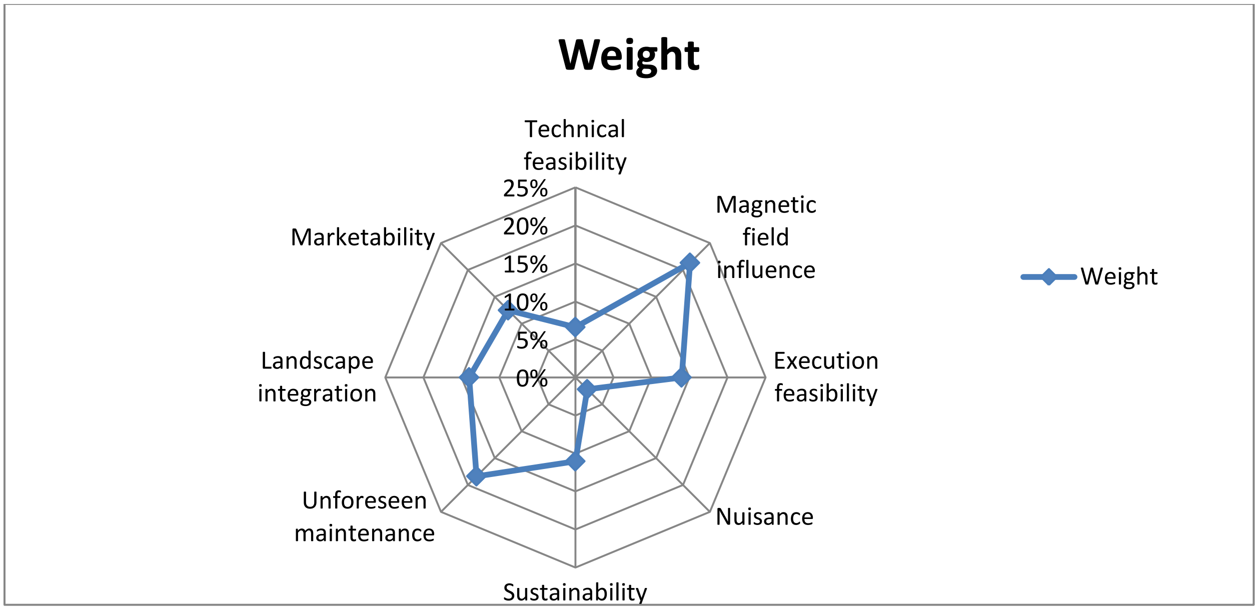


Figure 147: Weight of criteria for VM

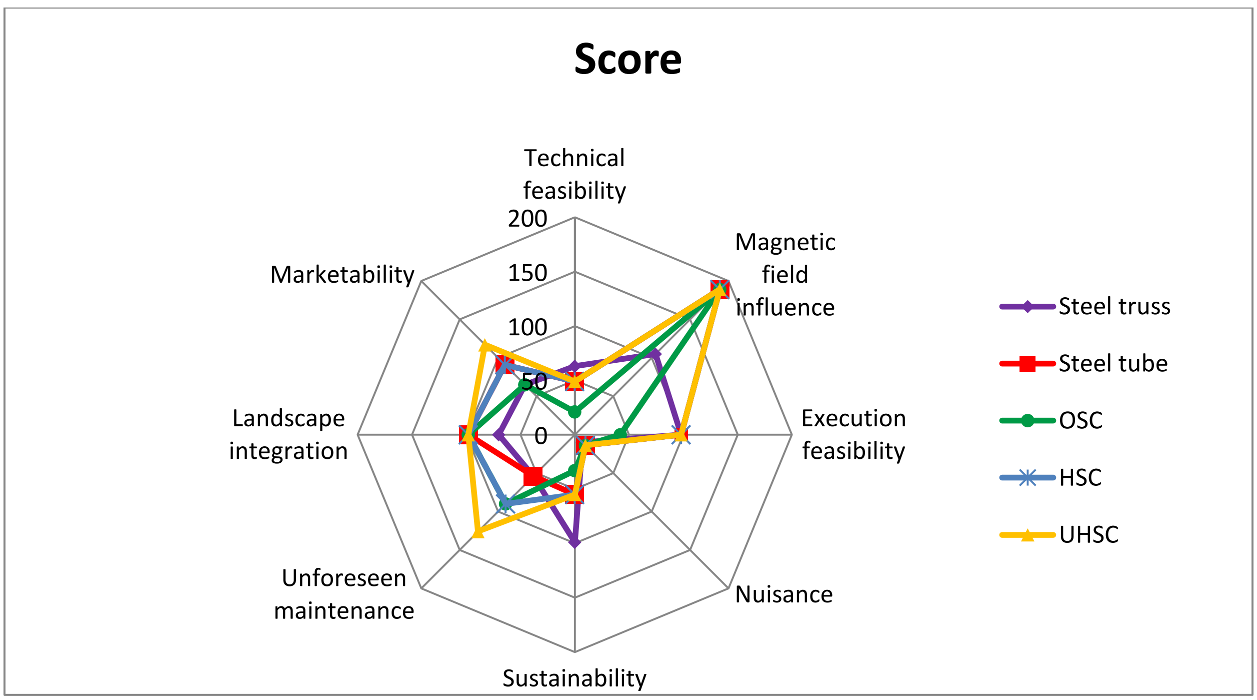


Figure 148: VM score

Appendix AF: Summary

Component	Values					Units
Material	**Concrete**			**Steel**		
Dimension mast	**OSC**	**HSC**	**UHSC**	**Tube**	**Truss**	
Height	57,00	57,00	57,00	57,00	60,00	m
Diameter bottom	2,30	2,20	2,20	2,20	6,00	m
Diameter top	0,70	0,50	0,50	0,50	3,50	m
Thickness	420	145	75	17	12	mm

Segments	OSC	HSC	UHSC	Tube	Truss	Units
Number of segments	12	6	4	2	-	-
Shortest segment	3	7	13	24	-	m
Longest segment	9	12	16	33	-	m
Lightest segment	5	6	5	18	-	m
Heaviest segment	19	16	15	13	-	kg

Foundation	OSC	HSC	UHSC	Tube	Truss	Units
Foundation block thickness	1,5	1,5	1,5	1,4	1,5	m
Foundation block diameter	7,2	7,2	7,2	7,0	7,2	m
Number of piles	21	21	21	21	50	-
Pile dimensions	20*0,40*0,40	20*0,40*0,40	20*0,40*0,40	20*0,40*0,40	20*0,40*0,40	m

	Material	OSC	HSC	UHSC	Tube	Truss	Units
Mast	m^3 of concrete / kg steel	152	56	30	56075	23786	m^3\kg
	kg of reinforcing steel / m^2 coating	33139	43997	9527	483	-	kg\ m^2
	kg of prestressing steel	10724	8579	8579	-	-	kg
Foundation	m^3 of concrete	136	129	122	108	96	m^3
	kg of reinforcing steel	33138	43997	9527	8405	7488	kg
Piles	m^3 of concrete	134	134	134	134	161	m^3
	kg of reinforcing steel	102	102	102	102	123	kg
Entire	m^3 of concrete	422	319	286	242	257	
	kg of steel	77105	96678	27738	64584	31400	
	m^2 of coating	-	-	-	483	-	

Sustainability	OSC	HSC	UHSC	Tube	Truss	Units
Embodied energy	2.393.835	1.666.588	1.660.868	1.774.106	1.318.239	MJ/kg
Embodied emissions	154.026	104.229	98.524	105.780	85.835	kg CO_2/kg

Direct costs	OSC	HSC	UHSC	Tube	Truss	Units
Material costs	194880	166047	149587	174159	91981	€
Building costs	165068	120155	132396	75885	71959	€
Labor costs	45176	40215	26600	21267	18842	€
Total direct costs	445636	359059	339442	298442	201060	€

Indirect costs	OSC	HSC	UHSC	Tube	Truss	Units
Maintenance costs	73221	48084	46317	72040	102731	€
Anticipated costs	659972	520843	493247	464986	367459	€
Total indirect costs	238131	190263	179995	162857	117684	€

Total costs per mast location	**757000**	**597000**	**566000**	**533000**	**421000**	€

Quality	**552**	**684**	**746**	**628**	**656**	-

Value Quality/Costs	**0,73**	**1,14**	**1,32**	**1,21**	**1,34**	-