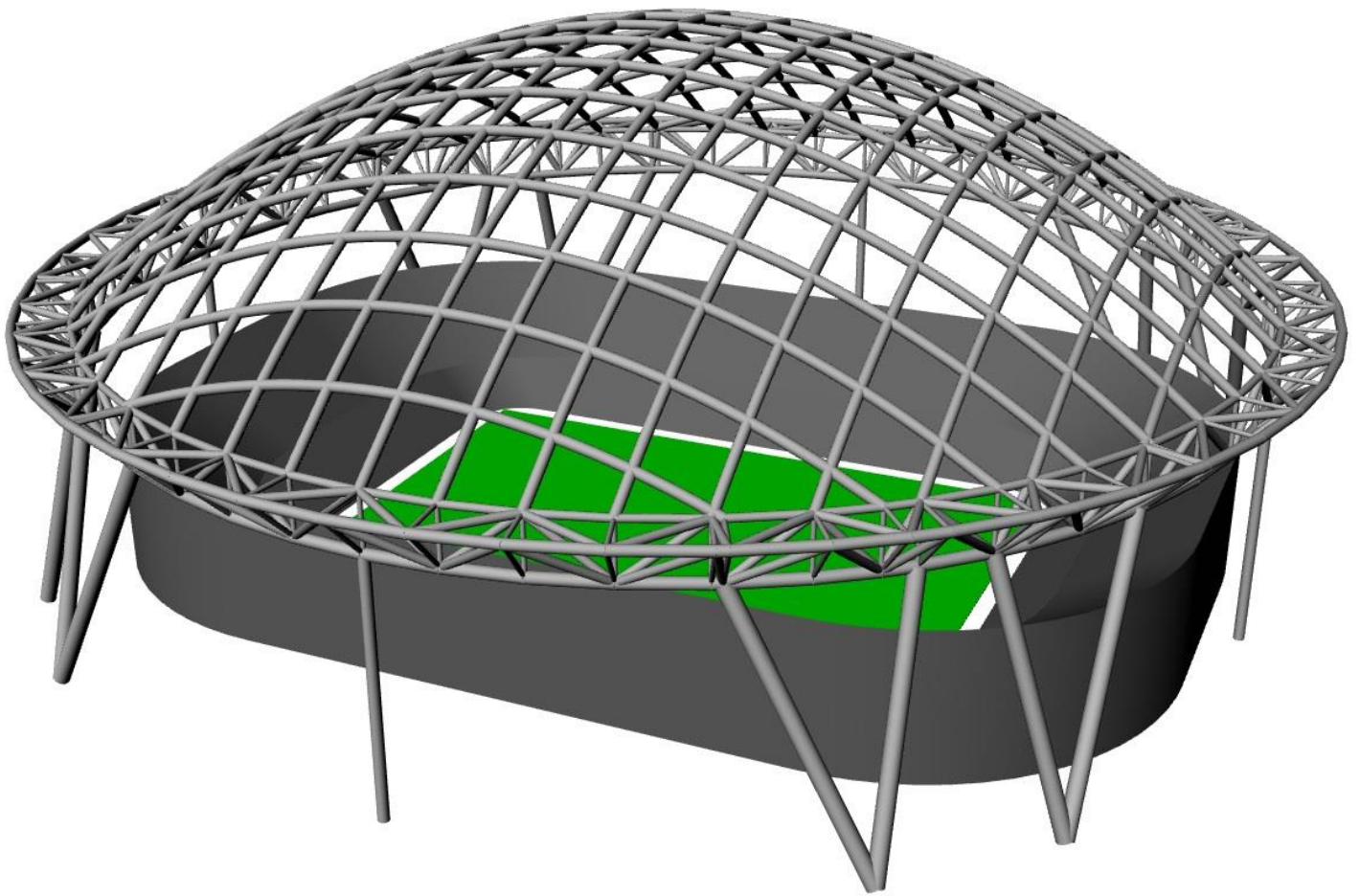




Design and optimisation of
a structurally separated roof structure
for event venues

Lennart Wiltjer

Design and optimisation of a structurally separated roof structure for event venues

by

L. Wiltjer

in partial fulfilment of the requirements for the degree of

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Student

Lennart Wiltjer
Oostmaaslaan 147b2, 3063 AS Rotterdam
Student ID: 1364154
E: lennart_wiltjer@hotmail.com
T: +31 6 522 716 33

Educational institution

Delft University of Technology
Faculty of Civil Engineering and Geosciences
Department of Building Engineering
Stevinweg 1, 2628 CN Delft

Graduation company

IMd Raadgevende Ingenieurs
Piekstraat 77, 3071 EL Rotterdam
E: info@imdbv.nl
T: +31 10 201 2360

Graduation committee

Chairman:
Em. Prof.ir. F.S.K. Bijlaard
Delft University of Technology
E: F.S.K.Bijlaard@tudelft.nl
T: +31 15 278 4581

Prof.ir. R. Nijssse
Delft University of Technology
E: R.Nijssse@tudelft.nl
T: +31 15 278 5488

Dr.ir. R. Abspoel
Delft University of Technology
E: R.Abspoel@tudelft.nl
T: +31 15 278 5358

Ing. R Stark
IMd Raadgevende Ingenieurs
E: R.Stark@imdbv.nl
T: +31 10 201 2360

Preface

This Master's thesis was written as part of the Master curriculum of Building Engineering with a specialisation in Structural Design. Building Engineering is a Master track of Civil Engineering at the Delft University of Technology. The research was carried out in collaboration with the Faculty of Civil Engineering and Geosciences and the Dutch engineering company 'IMd Raadgevende Ingenieurs'.

I would like to acknowledge everyone who was important during the completion of my thesis.

First of all, I would like to express my gratitude to ing. R. Stark from IMd Raadgevende Ingenieurs. He was the one who came up with the basic idea and the first sketch designs of the Event Dome.

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Lastly, on a personal note, I would like to thank my parents, my brother and my friends for supporting me during the process. Without their unconditional support this would not have been possible.



Lennart Wiltjer, Rotterdam, August 2017

Abstract

Nowadays football stadium encounter a lot of problems. Often services are outdated and the capacity is often too large or too small. These capacity problems especially occur in stadium that are built for major tournaments. When the tournament is finished, no appropriate tenants can be found and the stadium often remains empty. Another problem for a football stadium is the infrequent use. Most stadiums are in use only 20-25 times a year. Because these times are spread relatively even throughout the year, there are no large periods in which construction or refurbishment can happen. Besides all this, football stadium are unique structures, which require a large investment. Football clubs are often not capable of making such an investment.

A lot of possible solutions are researched in the past, such as a demountable stadium or a modular, that can easily be enlarged or reduced by adding or removing extra modules.

In this research another possible solution to these problems is drafted. By structurally uncoupling the grandstand structures and the roof structure, a lot of possibilities arise for the stadium. Grandstand structures can easily be enlarged or reduced, services can be refurbished without extensive construction and even movable grandstands can be designed. By moving the grandstand, the stadium can become suitable for other sports or events.

Besides the extra possibilities that arise, uncoupling the two structures also causes the opportunity to design both structures in a structurally efficient way. Bending moments can be prevented as much as possible, which makes the structure much more efficient. A lot of material can be saved, making the total stadium much cheaper.

In this research the roof structure is designed. The starting point in this design was that the structure should be optimal with regards to material usage. Different shapes are designed, which are compared by both structural and functional aspects. A few systems are selected that will be used for the preliminary calculations. In the preliminary calculations each of the different systems is calculated and optimised with Karamba and Galapagos. The total self-weights of the systems are compared and the best scoring system is analyzed further.

This final system is then modeled parametrically. Multiple parameters, like the height, centre-to-centre distances and the different cross-sections, are optimised to determine the optimal design with regards to material usage. After all parameters are optimised, the final design is calculated thoroughly. The results of these calculations are compared to similar structures to check if it really is an optimal design. From the results it follows that similar structures have a larger self-weight. This shows that the design in this research is indeed an appropriate alternative compared to more traditional stadium designs.

Besides the final design, also some design options are researched to see their influence on the structural behaviour of the roof structure. These design options are, for instance, a slightly different shape or a roof with a hole in it. These options can be seen as extras for the design. The determination of the structural consequences of these extras show how much they influence the total price of the system.

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1

Research Setup

In this chapter the problem is introduced, after which a possible solution is stated. This idea of this solution comes from engineering firm IMd Raadgevende Ingenieurs. The main characteristics of this idea are discussed, and advantages and disadvantages are shown. After this the procedure of this research is discussed and the research questions will be stated. Finally, the method of working is discussed briefly.

1.1. Introduction

1.1.1. Problem Description

Nowadays a lot of problems arise in the use of football stadiums. In this section the biggest current problems of football stadiums are discussed.

Capacity Problems

The most visible problem in the stadium is the problem of over- or under-capacity.

Under-capacity is usually not critical for the feasibility of the stadium. In case of under-capacity the stadium is often sold-out, which gives the tenant maximum revenue for a stadium of that size. However, if the capacity of the stadium would be increased, the revenue could increase as well, which is obviously favourable for the tenant.

A much bigger problem for stadiums is over-capacity. This occurs often after large international (football) tournaments. Large, luxurious stadiums are built for these tournaments. If this happens in countries that love sports, and where it's common to visit sport matches, like most of Western Europe, USA and some parts of Asia, new tenants for these stadiums are often easily found. In other countries, like South Africa, new tenants are much harder to find. The FIFA World Cup of 2010 was hosted in South Africa, and for this tournament, large stadiums, with capacities well over 50.000, were built. After the tournament, permanent tenants were impossible to find. No football or other sports club was able to fill out a stadium with a capacity of over 50.000. One of the stadiums, the Cape Town Stadium in Cape Town, was built for this tournament. The government-operated stadium has a capacity of 55.000 seats and has cost more than 530 million US dollar to construct. The annual operating cost in 2011 is estimated at \$6 million after revenues are deducted, and including the annual maintenance cost of \$2 million. After the tournament the only tenant that could be found was the local football club Ajax Cape Town, which plays in the PSL (Premier Soccer League), the highest football league of South Africa. In May 2012, the club had played 10 home games in the PSL with an average attendance number of 4.000, in a stadium with a capacity of 55.000. Since Ajax Cape Town signed a lease contract, where the rent depends on the attendance number, the rent is not enough to outweigh the operating cost. Since the stadium is operated by Cape Town's authorities, the taxpayers of Cape Town pay millions of dollars annually for an empty stadium. [Alm et al., 2012]

Outdated stadiums

Another, less visible, problem with stadiums is that it is outdated. Usually the structure is not at the end of its working life, but the services are outdated, often not meeting the requirements people demand nowadays. The clearest example in the Netherlands is Stadion Feijenoord in Rotterdam, commonly known as 'De Kuip'. It is famous for its atmosphere during matches. The Dutch national football team, however, does not play matches in this stadium anymore, due to the outdated services. The KNVB (the Dutch football association) has chosen not to play any international matches in this stadium. They prefer the Amsterdam ArenA, where sponsors and other business invitees can get the most luxury. The experience of the 'normal' stadium visitors comes in second place, because the KNVB simply makes more money from business related guests. [van der Burg, 2016] In this case, the capacity is not even the biggest problem, even though an increased capacity would increase the revenue of main tenant Feyenoord. Also, the structure is not at the end of its working life. TNO reported in 2004 that the structure of Stadion Feijenoord is still sufficient for another 50 years, if all overdue maintenance will be done. [Bloembergen and van der Walle, 2005]

This example shows that it is important for a stadium to fulfill the demands of current times. Large business areas, where meetings can be held and business can be discussed during the match, create enormous revenues for the stadium.

Refurbishment of a current stadium is a possibility, but since most stadiums are not designed for extensive refurbishment in a later stage, this refurbishment will be costly and time-consuming.

Infrequent use of stadiums

Most sport venues are used about 20-25 times a year. All the other days the stadium is empty, only used for guided tours and maintenance. This makes it hard for a stadium to become financially feasible. Sometimes

sport stadiums are used for other sports or music concerts, but this usually isn't ideal. The layout of a football stadium for instance is not great for most other sports or concerts. The distance between grandstands is often too big or too small, which makes that big international artists preferably choose for venues like Ahoy or the Ziggo Dome, where the experience for the audience is much better. Also the costs of transforming the stadium for other events often do not outweigh the benefits.

1.1.2. Event Dome [Ingenieurs, 2015]

Basic Design

The design philosophy behind the Event Dome is to turn around the way stadiums are designed. Instead of building a permanent stadium and trying to fit every (temporary) function in this stadium, only the roof will be permanent. Underneath this structurally optimised roof, which is stable on its own, all desired functions can be fitted.

If a football stadium is taken as an example, the roof structure is usually supported by the grandstand structure. This makes the grandstand structure highly inefficient because of the large forces and bending moments that come from the roof. By separating the roof structure from the grandstand structure, both structures can have their structurally optimal shape, making the total project much more efficient, and therefore cheaper.

Another advantage that arises if the grandstands and the roof are structurally separated is that it's easy to adjust the grandstand structure to whatever is demanded. For major events, for instance, the grandstand structure can be enlarged to increase the capacity without building a complete new stadium.

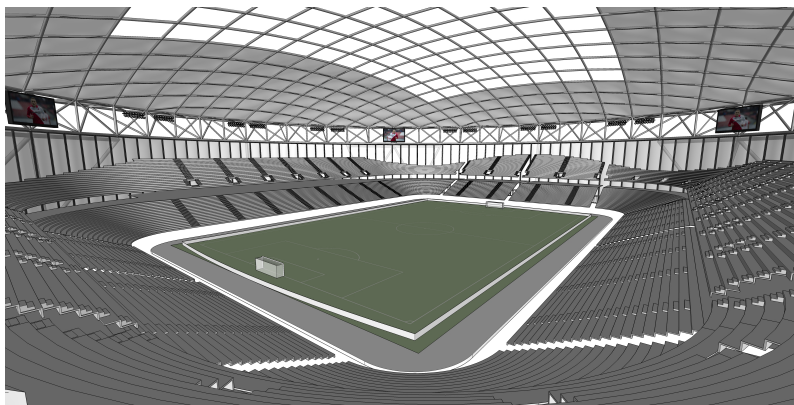


Figure 1.1: Impression of the Event Dome with a grandstand structure

Roof

Because the Event Dome can house multiple functions, the roof should be designed to be either open, closed, movable or a combination of those options. If it will be the roof of a football stadium for instance, the roof needs to have sufficient openings for the grass to live and grow, but also sufficient coverage to keep the spectators dry in case of rain. If the Event Dome will be the roof for a concert venue on the other hand, the roof is preferably fully closed to keep the rain out.

The first idea is to have a structure that will span the entire dome. In this way there is a possibility to add roof cladding to every part where that is desired. If there will never be the need to close the entire roof, the middle part can be free of structural elements by adding some kind of structure around the perimeter of the opening, like a compression or tension ring.

The cladding of the roof is preferably done with a light-weight material to keep the dimensions of the structure as small as possible. One of the possibilities is the usage of ETFE cushions. ETFE is a lightweight plastic that can be translucent. This translucency gives the event dome maximum daylight, but also gives the possibility to light the roof artificially in different colours, depending on the event. [Philips, 2015]

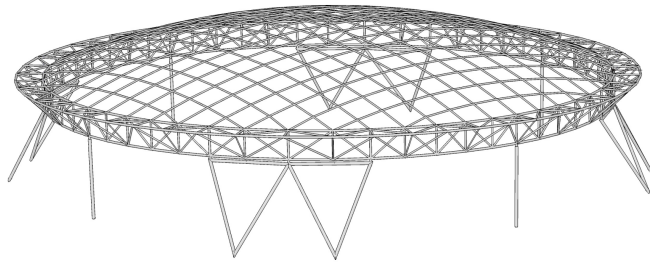


Figure 1.2: Roof of the Event Dome in 3d

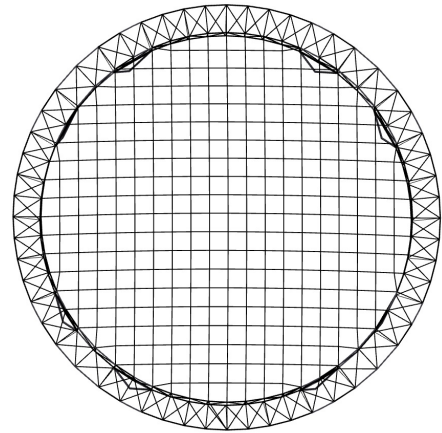


Figure 1.3: Top view of the Event Dome

One of the options is to make a retractable roof. This can be closed in case of bad weather, or if demanded by the event. All other days it can be open to ensure sufficient fresh air and daylight for the grass.

Stadium Possibilities

Because of the structural separation of the roof and the grandstand structure, a lot of possibilities arise for the grandstand structure. One example of those possibilities is a movable grandstand. If, for instance, the grandstand on the long side of the pitch is movable both in and out, a lot of new possibilities arise for hosting other events.

As can be seen in Figure 1.4, with just one movable grandstand there are possibilities to host football, tennis, baseball, extreme sports, indoor and outdoor concerts and many other possibilities.

Another advantage that arises if the grandstands and the roof are structurally separated is that it's easy to adjust the grandstand structure to whatever is demanded. For major events, for instance, the grandstand structure can be enlarged to increase the capacity without building a complete new stadium. In this way scenarios as the one in South Africa, as described in Section 1.1.1 can be prevented.

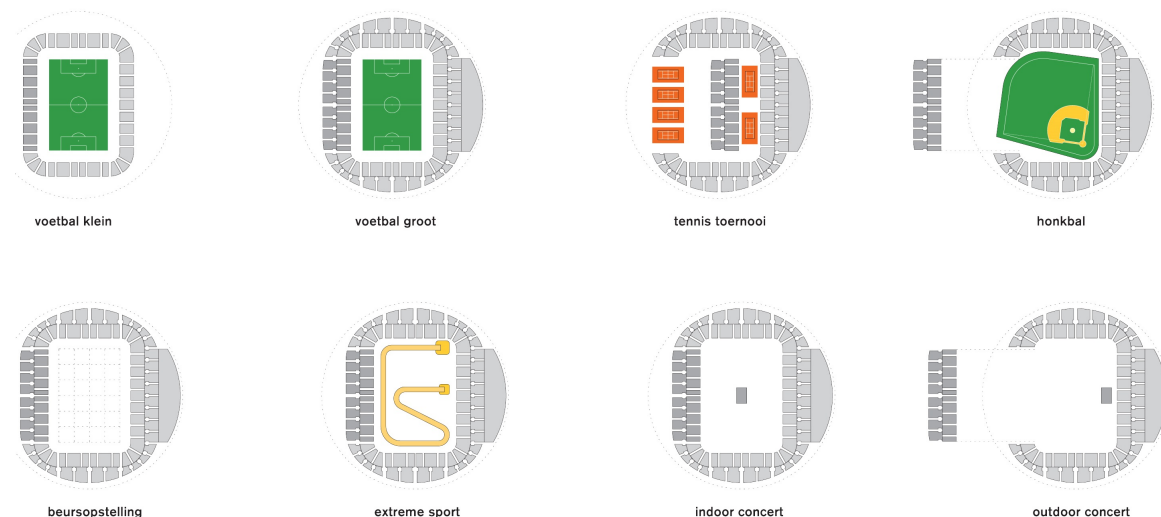


Figure 1.4: Possible stadium configurations

Advantages and Disadvantages of the Event Dome Compared to Other Stadiums

The Event Dome has a lot of advantages compared to other, more traditional, stadiums. Some of these advantages are:

- Relatively low costs
- Stadium can be in use during construction
- Improved multi-functionality
- Grandstand structure can be renovated step by step without drastic measures

Besides advantages there are also disadvantages. Some of these disadvantages are:

- The roof structure has an enormous span
- More conservative people can be skeptic about the new stadium design

Cost Comparison

In order by IMd a cost estimation of a possible design of the Event Dome is drafted by a consultancy company. This cost estimation is based on the original design of the Event Dome, which is described in Section 1.1.4, together with a grandstand structure and all services that are needed for a football stadium. The cost estimation of the Event Dome can be found in Appendix A. The grandstand structure in the cost estimation has a capacity of 28.500 people, but the costs of the Event Dome with a double capacity is also estimated, by doubling the values for the grandstand of the original cost estimation. With these numbers the costs of the Event Dome can be compared to other football stadiums.

In the last column of Table 1.1 the costs per seat for different football stadiums are given. It can be seen that the Event Dome is the cheapest of them all, only coming close to the St. Mary stadium in Southampton.

Table 1.1: Cost comparison (* = Costs from pre-construction estimates)

Stadium	City	Inflation adjusted construction costs (€) [Alm et al., 2012; FC-Groningen, 2003; StadiumDB, 2016]	Capacity	Costs per seat (€)
Event Dome single		50.000.000*	28.500	1.754
Event Dome double		73.000.000*	57.000	1.281
New Kuip	Rotterdam	240.000.000*	68.000	3.529
Red de Kuip	Rotterdam	200.000.000*	70.000	2.857
ArenA	Amsterdam	211.000.000	53.631	3.934
Euroborg	Groningen	54.000.000	22.550	2.395
ESPRIT Arena	Dusseldorf	290.000.000	54.500	5.321
St. Mary	Southampton	60.000.000	32.363	1.854
Emirates	London	568.000.000	60.260	9.426
New Wembley	London	1.080.000.000	90.652	11.914
Etihad Stadium	Manchester	276.000.000	46.708	5.909

1.1.3. Structural System

Probably the most important aspect of the Event Dome is the spanning structure of the roof. Since the circle has a diameter of about 200 meters the design of this structure will have a huge impact on the final costs and feasibility of the design.

To come to an efficient design different structural systems should be examined, like tension-only, compression-only and mixed systems. For each of those systems different types are possible. Examples of these types are

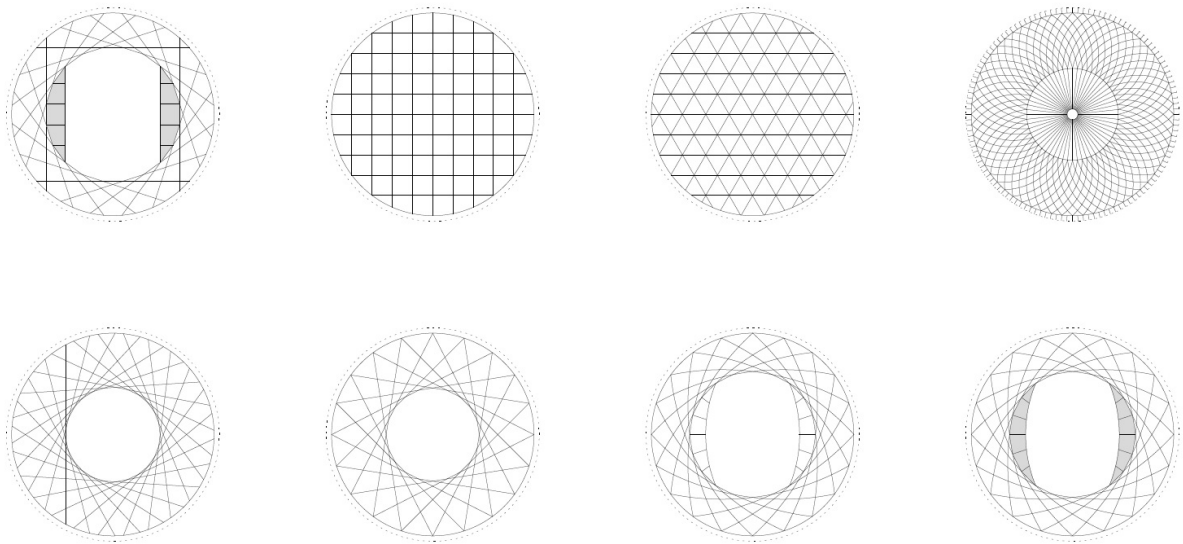


Figure 1.5: Examples of Structural Systems

a system with girders in two directions, a system in three or more directions, a system with radial girders and many others. In Figure 1.5 some examples of structural systems are given. These examples are just for

Obviously the roof needs to be sufficiently strong and stiff. Since the span is large and the structure is relatively slender dynamics are an important aspect as well. For the decision which structural system will be used, it's important to take all these aspects into account.

1.1.4. Original Design by IMd Raadgevende Ingenieurs

The first idea of this Event Dome comes from IMd Raadgevende Ingenieurs, who worked together with architectural firm Cepezed. Their goal was to design a structurally optimal roof structure for a football stadium in the Netherlands. Their solution was to have a kind of dome structure with arches in two directions. This dome structure is lifted upwards to have a minimum clearance height at the edges of the dome. To deal with the large thrust forces that come out of a dome, a tension ring is designed, which is supported by vertical and diagonal columns.

The design can be seen in Figure 1.6 and Figure 1.7.

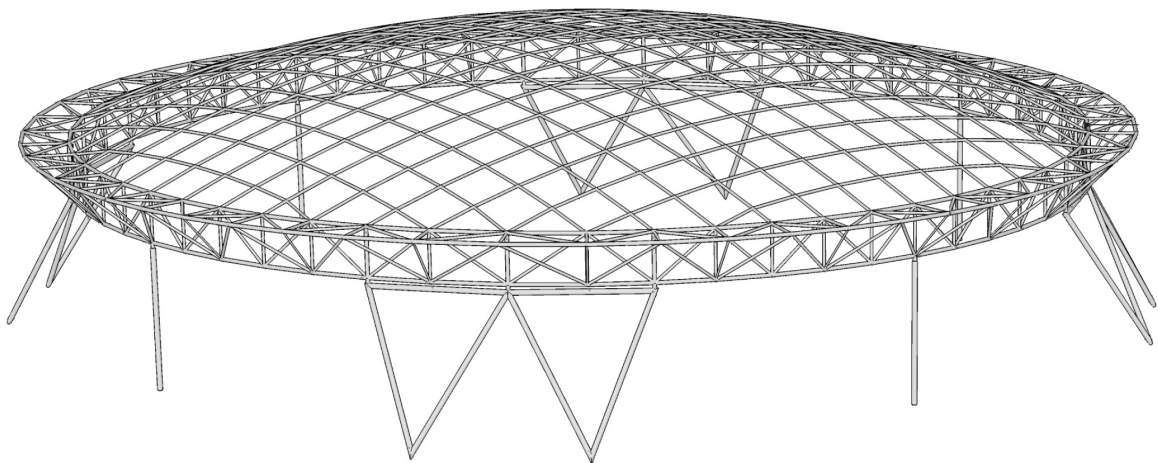


Figure 1.6: Event Dome by IMd Raadgevende Ingenieurs and Cepezed

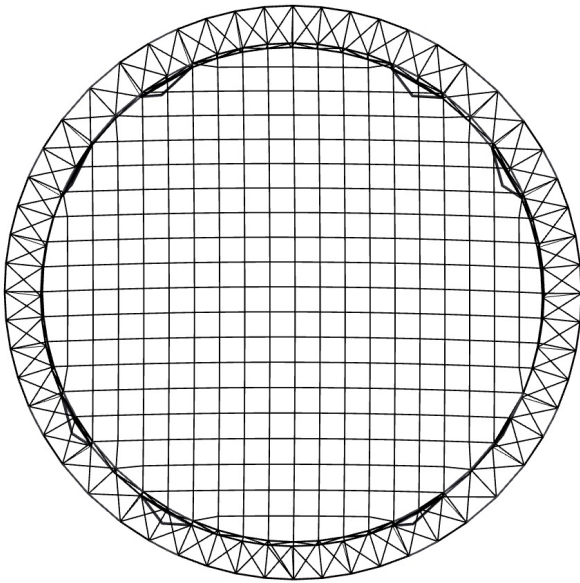


Figure 1.7: Top view of the Event Dome

1.2. Research Design

1.2.1. Research Objective

The main objective of this research is to design the optimal roof structure for an event venue, which is structurally stable on its own. This event venue should be able to host multiple functions, such as football, other sports and concerts.

The result should be a both structural and architectural satisfying design, which also has minimal costs.

Also, a sensitivity analysis should be carried out to identify the effects of certain design changes. Such a sensitivity analysis can be useful if a future client has certain demands for his project. The sensitivity analysis will show easily what effects certain design changes will have on the structure and the total costs. This will make the final design as versatile as possible for all interested parties, both in the current as in the future.

The end product should be one design, with different options that can be added or changed to meet the demands of future clients.

1.2.2. Research Scope

The research will focus mainly on the structural design of the Event Dome. Little attention will be given to the architectural aspects. The grandstand structure is not part of the research. Obviously the dimensions of a grandstand structure are taken into account as a boundary condition.

Multiple possibilities will be examined, such as a tension-only structure, a grid shell or a space truss. All possibilities will be compared with regards to total material usage.

All other aspects of the possibilities, like aesthetics, costs, construction time and versatility will be outside the scope of this research.

In this chapter the project is called 'Event Dome'. This is the name that IMd Raadgevende Ingenieurs used for its preliminary design. Although this name suggests that the final design will be a dome, this has not been decided yet. Until the final shape is decided, the name 'Event Dome' will be used.

1.2.3. Research Methodology

Literature Study

The first part of the research is the literature study. In this part information is gathered that can help during the research. The literature study can be found in Appendix B. The following topics will be covered in the literature study:

- Reference projects
 - (Football) stadiums with a main focus on the roof structure
 - Different large span steel structures like grid shells and cable structures
- Literature about large steel structures
- Requirements for football stadiums

Orientation

In this part the beginning of the research will be made. Different designs will be made, each with a different structural system. A first selection will be made according to functional and structural aspects. The remaining designs will be elaborated a little bit further. Rough calculations will be performed to get some rough dimensions. After these rough calculations a selection is made which design will be developed further.

In this phase a list of possible demands is created for the design. These demands are possible design changes that will be researched in the analysis. The demands can be both structural and architectural. The idea behind this is that different clients will have different demands, and by taking this into account in an early stage these demands can be incorporated in the design.

Analysis

The chosen design from the orientation will be optimised with the use of software. The following software tools will be used for this optimisation:

- Grasshopper: a parametric designing plugin for Rhinoceros 3D, including the following Grasshopper plugins:
 - Galapagos: an optimisation tool
 - Karamba, a FEM tool for Grasshopper for the rough calculations
- A FEM software package

In Grasshopper, a model will be made in which multiple parameters can be changed without the need to change the entire design. Examples of parameters are centre-to-centre distance of the structure, height difference of the structure (height of the top point compared to the height of the lowest point), shape of the structure, and so on.

The first rough calculations will be performed by Karamba. By linking Galapagos and Karamba, the optimal cross-sections can be found for each design. By comparing the total self-weight of the designs, the optimal design can be chosen. A design can be optimal in multiple aspects, such as total weight, aesthetics, costs and construction time. In this research the goal of the optimisation will be to minimize the total weight of the structural members.

The final design will now be remodeled in Grasshopper. This remodeling is necessary because the first designs were modeled to work with Karamba. Parts of the preliminary Grasshopper can be used for the final Grasshopper model. The final design will be calculated with an external, more extensive FEM package. Which FEM software package will be used will be determined later, depending on which design will be elaborated.

By 'playing' with the parameters a sensitivity analysis can be carried out. This sensitivity analysis will show the positive and negative effects of design changes.

After the sensitivity analysis has been carried out, the costs of multiple options for the design will be estimated and compared. In this research these costs are solely based on the self-weight of the structural material. The self-weights will also be compared to the self-weights of other comparable structures, such as sport stadiums or other similar structures.

Conclusions and Recommendations

The last part of the research is to provide conclusions and recommendations following from the research. The results of the analysis will be used to draw conclusions about the good and the bad sides of the designs.

Recommendations will be given about what is needed to further improve and finalize the design.

1.2.4. Research Questions

Based on the current knowledge about the topic the following main- and sub research questions can be formulated.

Main research question

What is the optimal design for a roof structure of an event venue, that has no structural connection with the underlying structures and provides its own stability?

Sub research questions

1. What cost reduction can be achieved for the Event Dome, compared to a more traditional stadium?
2. What options can be changed/added to the design to fulfill the demands of future clients?

1.2.5. Terminology

Optimal Design

In this research the optimal design is defined as the most favourable design, in terms of material usage, estimated construction costs, and overall value. The main focus will be on the material usage, with limited focus on the other aspects.

Flexibility

In this research flexibility means the possibility to house multiple functions in the stadium. Focused on the roof structure, flexibility means that the roof structure is the least obstructive to host any events. One can think of things like a sufficient minimum height for all possible events or a sufficiently open facade to perform construction or move one of the grandstands.

1.3. Method of working

The research will be done mainly in the office of IMd Raadgevende Ingenieurs in Rotterdam and in the faculty of Civil Engineering in Delft. The location will be dependent on the support that is needed.

There will be multiple meetings with the entire committee to discuss progress of the research, including the kick-off meeting and the go/no-go meeting. Besides those meetings, there will also be one-on-one meetings with the different supervisors, dependent on the type of counseling that is needed at that moment. The initiative for counseling will be taken by the student.

The software that will be used for writing the report is L^AT_EX. Lyx is a document processor that combines the L^AT_EX system with a graphical interface. In L^AT_EX the TU Delft layout will be used. The bibliography will be made using JabRef. JabRef is a reference manager which can be used in combination with L^AT_EX.

Calculation software

As explained before, this research will use multiple FEM software packages. The preliminary calculations will be performed with Karamba. Karamba is a plugin for Grasshopper that calculates the structure in a rather short time. The Karamba plugin can also be combined with Galapagos, an optimisation plugin for Grasshopper. If Karamba is combined with Galapagos in the right way, Galapagos can be used to find the combination of cross-sections for which the self-weight is minimal, while still ensuring structural stability.

With this optimisation, a comparison can be made between different structural systems, without extensive and time-consuming labour.

While Karamba is a suitable tool to compare the different structural systems, it is not suitable to perform the final calculations. This is because, first of all, the results of Karamba calculations are not really clear, and to really understand the structural behaviour, Karamba results are just not extensive enough. Besides that, Karamba is not checked by authorities, and Karamba itself explains on their website that, although Karamba is a good design tool, it should not be used for final structural calculations. This is explained a bit further in Section 5.1. Therefore a more extensive FEM package is used for the final calculations.

2

Preliminary Design

To come to an optimal design it is important to consider all options possible for the design. Considering all options will be done in a funnel-way. This means that there will be worked from broad to narrow. First the design criteria will be drafted. After that the possible main internal forces will be described. After that the possible overall shapes of the roof will be looked into. One optimal shape will be chosen to use in the continuation of the preliminary designing stage. Next is the determination of the direction in which the roof will span. Some possibilities will be described and for each of the structural systems, a span direction is chosen. For some systems multiple directions of spanning could be optimal. Each of these directions will therefore be researched. After that different structural systems, based on the reference projects described in Appendix B and other literature [Nixdorf, 2008], will be described with their advantages and disadvantages.

Finally, three or more options will be chosen which will be further elaborated in the next phase of the research.

2.1. Design Criteria

Demands

- The Event Dome should be suitable for at least the functions football stadium and concert venue
- Dimensions of the design should be at least 200 x 150 meter to accommodate the possible functions
- For the function 'football stadium' a minimal clearance height of 30 meter above the pitch is needed according to rules of the UEFA and FIFA [FIFA, 2011]
- There should be no structural elements underneath the roof, besides at the edge
- There should be no structural connection between the dome structure and the underlying structures, like the grandstands
- It should be possible to close (a part of) the roof
- To ensure an optimal structure, a lightweight roof cladding will be used

Starting Points

- The main structural material will be steel
- The structure should be optimal with regards to the use of material
- The structure should be suitable to place anywhere in the Netherlands (other areas can be determined later), so maximum wind and snow loads for any place in the Netherlands have to be used

Boundary Conditions

- The calculations will be done according to CC3 as can be found in the NEN-EN 1990, table NB.20
- The load category for the Event Dome is C5 as can be found in the NEN-EN 1991-1-1, table 6.1

2.2. Internal Forces in Structure

For the determination of the structural system three different types can be distinguished. These types differ in the main internal force that is present in the system. The types that are used here are systems in compression, systems in tension and systems in bending. Each of these systems has its own characteristics with both advantages and disadvantages. These characteristics are briefly described in Table 2.1.

Also combinations of systems can be used, but this will be described later in this chapter.

Table 2.1: Types of internal forces

Main internal force	Structural characteristics	Other characteristics
Compression	- Buckling needs to be taken into account - Structure needs sufficient height difference to prevent bending moments in case of uniform loading	
Tension	- Small member sizes - Low dead load - Upward wind becomes important	- Often interesting aesthetics - Relatively small member sizes lead to large transparency
Bending	- Cross-section becomes enormous for large spans	

2.3. Shape

For the shape of the event dome a lot of possibilities can be examined, such as round, oval, rectangular, rectangular with rounded corners and even polygonal, like pentagonal or hexagonal. They will be described briefly.

Rectangular (with angled corners or rounded corners) & Polygonal

An important characteristic of a rectangular and a polygon is that it has multiple corners. Corners can be seen as discontinuities in the members. Corners are therefore structurally not beneficial because internal forces have to 'travel' around the corners. In these corners large bending moments can occur, due to internal normal forces. If the member continues in another direction, these normal forces create shear forces, which will lead to bending moments. These bending moments lead to a bigger required cross-section at these corners.

By making the corners gradually, or rounding the corners, this bending moment is decreased, but it still occurs.

Oval

An oval does not have any discontinuities. This is an enormous advantage compared to polygons, since the large bending moments that are described before, will not occur.

A disadvantage of ovals is that the shape is not the same in two horizontal directions. This leads to a non-uniform force-distribution and forces out of eccentricity. This obviously leads to different required cross-sections for different parts of the oval.

Round/Circular

Structurally a circle is the most optimal shape for the roof structure, because a circle does not have any discontinuities and is the same in all horizontal directions. All disadvantages stated before are therefore not applicable for a circle.

A functional disadvantage of a circle compared to a rectangle or an oval is that most stadiums and event venues are more or less rectangular. By putting a circular roof above it the roof area will be much larger

than needed. This can be seen in Figure 2.1. The extra area underneath the roof can be used for supporting functions, such as restaurants, offices and dressing rooms.

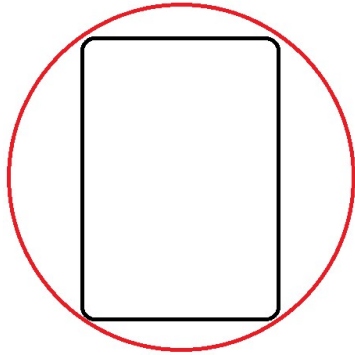


Figure 2.1: Circular roof over rectangular stadium

Conclusion

The goal of this research is to design an optimal roof structure for an event venue. Because of the demand for an optimal structure, the shape of the structure needs to be as optimal as possible as well. Therefore a circular roof will be used for the main design. The option to change the shape slightly towards an oval will also be researched, because although this is structurally not favourable, functionally this could be favourable, because of the reason that is described before. This is shown in Figure 2.2.

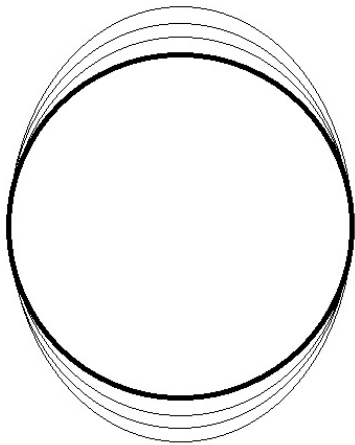


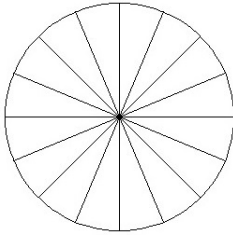
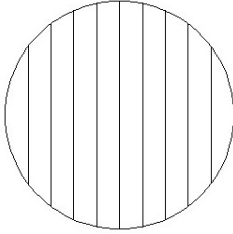
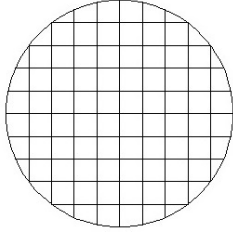
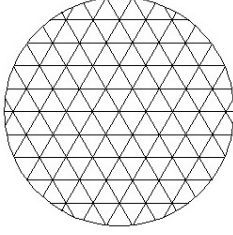
Figure 2.2: Shape of the roof

2.4. Span Directions

For each of the structural systems different span directions are possible. Four span directions are outlined beneath. Combinations of these span directions are also possible, but that probably will not increase the structural efficiency of the design.

For each structural system different characteristics are important. In Table 2.2 only general characteristic are described. For each of the different structural systems, different span directions can be examined.

Table 2.2: Span directions

	Span direction		Advantages	Disadvantages
1	Radial		+ Uniform cross-sections + Easily possible to add inner compression or tension ring + Few connections	- Difficult connection in middle - Distance between members changes over span
2	One main direction		+ Few connections + Uniform distance between members	- Different spans, therefore different cross-sections - Relatively large cross-sections
3	Two main directions		+ Uniform distance between members + Reduced cross-sections relative to #2 + Relatively easy connections	- Different spans, therefore different cross-sections
4	Three main directions		+ In-plane stability assured because of triangulation + Uniform distance between members + Reduced cross-sections relative to #2 and #3	- Different spans, therefore different cross-sections - Complicated 6-member connections

2.5. Structural Systems

For the roof structure different structural systems can be used. Based on the reference projects and [Nixdorf, 2008] a few systems are lined out beneath with some advantages and disadvantages of each system.

The first selection of the structural systems will be done by comparing the characteristics. Some possibilities can be crossed out without making calculations, just by looking at the characteristics and reference projects with the same system.

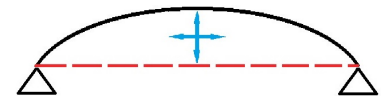
Rough calculations will be made for the systems that remain to make a further selection. After this selection some designs will remain, which will be elaborated further.

In this paragraph no (hinged) connections are shown in the diagrams. The following symbols are used to clarify the mechanics diagrams:

	Spanning in one direction
	Spanning in two directions
	Tension element
	Compression element
	Bending element
	Tension or compression ring
	Tension or compression beam

2.5.1. Dome Structure

The dome system is partially based on the reference project the Singapore National Stadium. The basic idea behind a dome structure is that the roof is made into such a shape that only compression forces will occur in the load case with uniform distributed vertical loads.



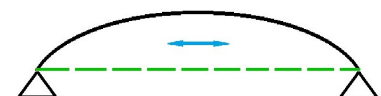
Probably the most important structural consequence of this shape is that it will result in large thrust forces at the supports. These have to be taken up by either the foundation or by tensile members between opposite supports.

A dome structure spans in two directions and can therefore be seen as a rigid structure.

Since there is a minimum height requirement at the perimeter of the circle with a diameter of 200 meters, the dome structure needs a larger ground area than this circle.

2.5.2. Arches on two Supports

The system with arches on two supports works as a dome in one direction, but in the transverse direction, the arches are connected by secondary members.



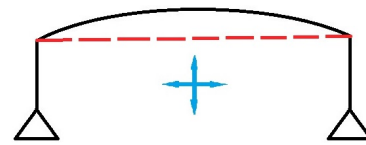
Because of the arch-shape in one direction, this structural system will also have large thrust forces, that have to be taken up by the foundation or tensile members.

The arches system is rigid in one direction.

Just as system 1, the required ground area is larger than the circle of 200 meters, because of the minimum height requirement.

2.5.3. Dome Structure on Columns

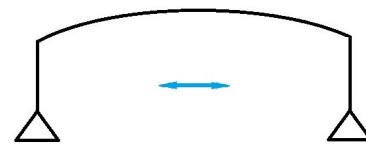
This system is comparable to the dome structure, with the difference that it is raised from the ground and supported by columns. This gives the advantage of requiring a smaller ground area, because the height of the system at the perimeter can be made already sufficient for every function.



The thrust forces that come out of the dome should be taken up at the top of the columns to prevent large bending moments in the columns. This can be solved by adding a tensile ring at the edge of the dome.

2.5.4. Arches on two Supports on Columns

This system is comparable to the Arches on two supports system, with the difference that it is raised from the ground and supported by columns. Again, just as in the previous system, this gives the advantage of requiring a smaller ground area.



The thrust forces from the arches can not be taken by the foundation or a tensile ring. To prevent bending moments in the structure, the columns should be either supported lateral or tensile members have to be used, spanning from one end of the arch to the other. Both solutions will have an impact on the aesthetics of the structure.

2.5.5. Bending Members on Columns

This system consists simply of bending members, supported on columns at both ends of each beam.

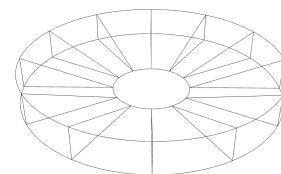
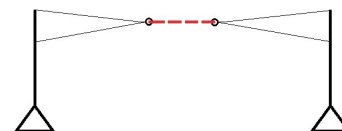
Since the span is quite large, the dimensions of these members will become enormous, even if trusses are used.



2.5.6. Spoked wheel

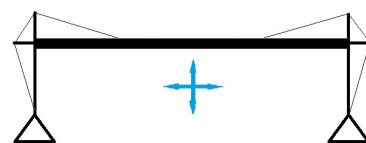
This system's layout is comparable with a bicycle wheel. It consists of three main elements: the inner ring, the outer ring and the spokes that connect both rings. In this system the upper spokes are in tension in case of uniform downward loads. The lower spokes are in tension in case of uniform upward loads.

For the stability of the system it is important that whole system works together. Opposite parts of the circle make equilibrium, giving the system stability in-plane stability. The inner ring in this system works as a tension ring, while the outer ring works as a compression ring.



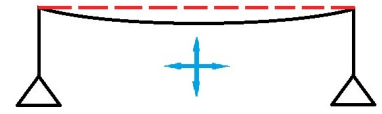
2.5.7. Cable-stayed Bending Members

This system is a variation on system 5. To prevent the dimensions of the bending member from getting too large, the member is supported intermediately by cables that are guided to the column supports.



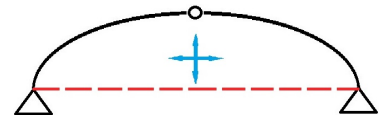
2.5.8. Inverse Dome Structure in Tension on Columns

This system is comparable with the 4th system, with the difference that the dome is inverted. Because of this inversion only tension forces will occur in the system, if loaded with uniform distributed vertical loads. In this system the columns need to be taller than in the other systems, because the minimum height clearance in the centre has to be accomplished.



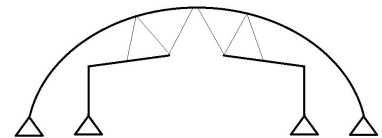
2.5.9. Three-hinged Arches

In this system the main spanning structure are again arches. The difference is that in the middle of each arch a hinge is implemented. This hinge has a great influence on the structural behaviour of the system.



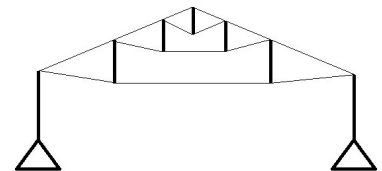
2.5.10. Roof Supported by Separated Structure

In this system the roof structure is supported by a separated structure. This separate can be designed in any way that is preferred.



2.5.11. Tensegrity Structure

A tensegrity structure is a system in which the stability is provided by tensile members. By combining these tensile members with some 'floating' compression members, a stable system can be accomplished.



2.5.12. Summary of Characteristics

Table 2.3: Structural systems

	Type of structure	Reference project	Mechanics diagrams	C/T/B
1	Dome structure	Singapore National Stadium		C
2	Arches on two supports	AT&T Stadium Arlington		C
3	Dome structure on columns	Houston Astrodome		C
4	Arches on two supports on columns	Messe Frankfurt - Halle 3		C
5	Bending members on columns			B
6	Spoked wheel	National Stadium Warsaw, BayArena Leverkusen		T
7	Cable-stayed bending members			T+B
8	Inverse dome structure in tension on columns			T
9	Three-hinged arches	Cargolifter Airship Hangar		C
10	Roof supported by separate structure	Wembley Stadium		
11	Tensegrity structure	Georgia Dome		T

Table 2.4: Summary of structural characteristics

	Structural advantages	Structural disadvantages
1	+ No bending moments in structure + Stability in both directions assured	- Large thrust forces at ground - Bending moments can occur if the height difference is not sufficient
2	+ No bending moments in structure + Stability in one direction assured	- Large thrust forces at ground - Bending moments can occur if the height difference is not sufficient
3	+ No bending moments in structure + Relatively simple foundation	- Large thrust forces at tension ring - Bending moments can occur if the height difference is not sufficient
4	+ No bending moments in structure + Relatively simple foundation	- Columns need to take very large horizontal thrust forces - Bending moments can occur if the height difference is not sufficient
5	+ Relatively simple foundation	- Size of members becomes enormous for large spans
6	+ No bending moments in structure	- Non-continuous structure
7	+ Bending moments reduced + Relatively simple foundation	
8	+ No bending moments in structure + Low dead load + Relatively simple foundation	- Large thrust forces at compression ring - Upward wind force should be taken into account - Stability could be an issue
9	+ Less sensitive for temperature changes and dynamics + Statically determinate structure + Relatively simple foundation	- Large thrust forces - Large deflection at middle support - Bending moments can occur if the height difference is not sufficient
10		- Not efficient because of separated structure
11	+ No bending moments in structure + Low dead load	- Lot of temporary measures needed during construction - Large number of complicated connections - Complex force distribution

Table 2.5: Summary of functional characteristics

	Functional advantages	Functional disadvantages
1	+ Logical structure makes roof cladding easier	- Larger span and ground area required because of lower height at the edge - More obstructive objects around the perimeter of the stadium
2	+ Logical structure makes roof cladding easier	- Larger span and ground area required because of lower height at the edge - More obstructive objects around the perimeter of the stadium
3	+ Logical structure makes roof cladding easier	
4	+ Logical structure makes roof cladding easier	
5	+ Logical structure makes roof cladding easier	- Low transparency due to large member sizes
6		- Difficult cladding in middle part
7		- Obstructive elements on facade
8	+ Logical structure makes roof cladding easier	- Larger column height required because of sag at the centre - Water accumulation in centre could occur
9	+ Logical structure makes roof cladding easier	- Larger span and ground area required because of lower height at the edge
10	+ Easier refurbishment possible	- More obstructive objects around the perimeter of the stadium
11	+ Aesthetically interesting structure	

2.5.13. Selection

As said before the first selection will be based on characteristics. The structural systems that will be crossed out in the first selection are:

- 1 – Structural system 1 will need a large ground area, because of the height requirements. Often the available ground area will not be sufficient. The larger ground area also makes this system more expensive than others. Also, since the span of the dome is larger than the span of the dome on columns, the dimensions of the members will be larger as well. The absence of the tension ring and columns does not weigh up against the extra material in the dome.
- 2 – Structural system 2 will be crossed out as well. This is done mainly for the same reasons as for system 1. The larger ground area will increase the costs and will decrease the flexibility of the design.
- 4 – Structural system 4 will have large thrust forces. These thrust forces have to be taken by the columns, which will lead to large bending moments and the need for clamping the columns, or by a tension elements spanning from one side to the other, which will be obstructive. System 4 does not have any real advantages compared to system 3 and therefore system 4 will be crossed out.
- 6 – System 6 does not have a continuous roof, making it structurally not optimal, since the internal forces have to 'travel' around the middle opening. System 6 is basically a less favourable version of system 8. System 6 will therefore will be crossed out.
- 9 – System 9 needs more material compared to system 2, because of the middle hinge. Also the deformation behaviour will be less favourable, because the structure is less stiff. System 9 will therefore be crossed out.
- 10 – System 10 has a separated main load bearing structure. This makes system 10 not efficient in material usage. System 10 will therefore be crossed out.
- 11 – System 11 is very efficient in material usage, but during construction it is really expensive, because of the large number of temporary measures that have to be taken. System 11 is therefore crossed out.

The remaining systems are the systems 3, 5, 7 and 8. These systems will be disclosed a bit further in the next paragraph.

2.6. Remaining Systems

As said before 4 systems remain after the first selection. Each of these systems will be circular with its spanning direction either in two directions, three directions or radial.

The remaining systems shall be calculated roughly to select the best option from these 4 systems. To do this it is important to understand the structural behaviour of each of the systems. Each of the systems will therefore be explained a bit further, with the focus on their structural behaviour. The internal forces in each of the systems, following from uniformly distributed downward loading, is shown.

2.6.1. System 3: Dome Structure on Columns

This system is basically a dome structure, that has been raised from the ground floor to decrease the ground area and prevent that there will be unusable nooks at the edge of the structure. The dome is then placed on both vertical columns and columns under an angle to support the dome both vertical as lateral.

The principle of a dome structure is a surface that is shaped in such a way that only normal forces occur if loaded with a uniformly distributed vertical load. This shape is called a catenary shape (Dutch: kettinglijn). This is the same curve as if an idealized chain is supported on both ends and loaded with just its own weight.

In Figure 2.3 a schematic diagram of the system can be seen, both in cross-section as in top view.

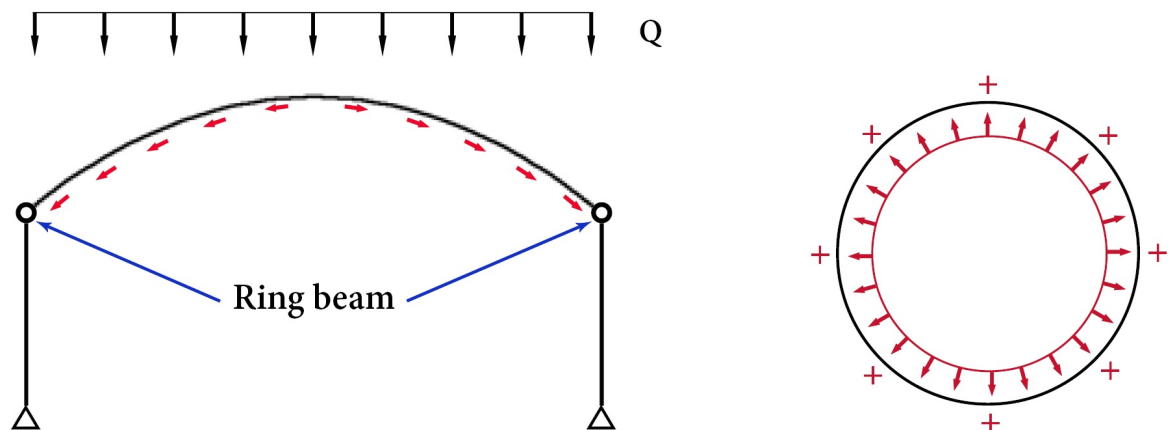


Figure 2.3: Dome structure on columns in cross-section (left) and top view

At the edges of the dome, the structure is still under an angle. Because the dome has only normal forces in an ideal situation, horizontal resultant forces must be taken by the supports of the dome. These horizontal resultant forces are called thrust forces.

In the Dome structure on columns these thrust forces will be taken by a ring beam. In the top view it can be seen that the ring beam will take the normal forces that flow out of the dome. The ring beam will therefore be in tension.

Because the ring beam will take the thrust forces and opposite thrust forces will neutralize each other. Therefore, in this idealized situation where only a uniformly distributed vertical load is present, the columns will only have to take vertical loads.

2.6.2. System 5: Bending Members on Columns

Structurally, system 5 is the most straight-forward. The system basically consists of simply supported beams on two supports, a ring beam that supports the beams on two supports, and columns that support the ring

beam. The connection between the beams on two supports and the ring beam will not actually be simple supports, but this will be discussed in the next chapter.

In Figure 2.4 a schematic diagram of this system can be found with the bending moment diagram of the beam on two supports.

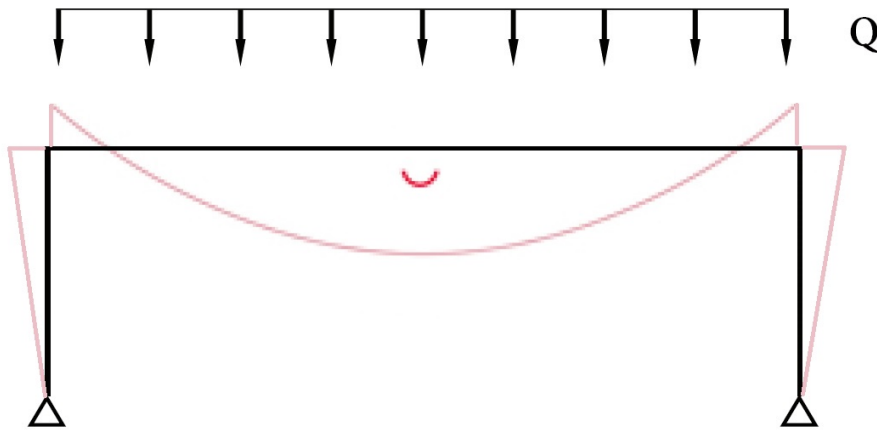


Figure 2.4: Bending members on columns in cross-section

2.6.3. System 7: Cable-stayed Bending Members

This system is a variant on System 5. The basic idea of this system is a bending member, supported on both ends. To decrease the maximum bending moment in the beams, intermediate supports are added. These supports will be steel tensile members, that will be guided to the supports.

The entire structure that is needed for the intermediate supports, so the tensile member and both cantilevering members, will be referred to as supporting structure.

Since the intermediate supports will be under an angle, they will create extra compression forces in the bending members. These compression forces will only occur in the part between the intermediate support and the ring beam supports. This can be seen in the image below. It can also clearly be seen that the bending moment in the beams decreases significantly.

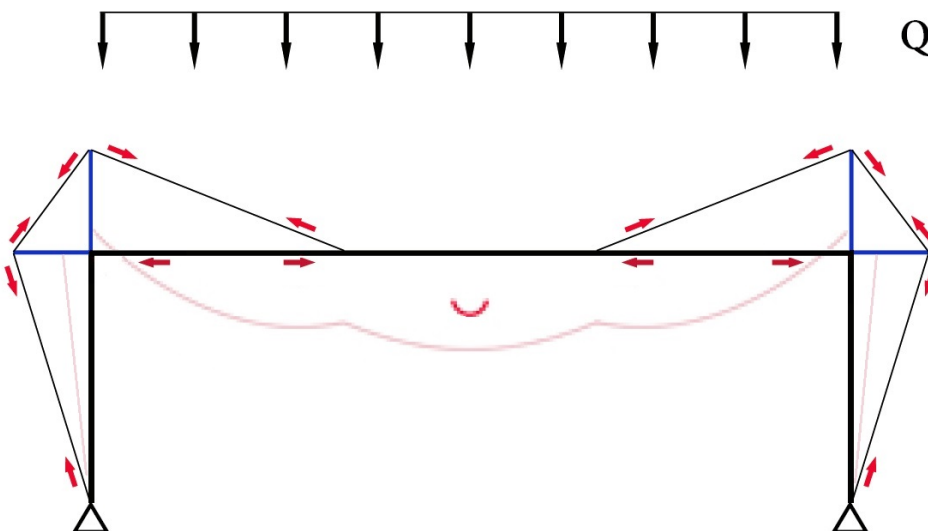


Figure 2.5: Bending members supported by cables in cross-section

The amounts of extra compression forces and reduced bending moments depend strongly on the dimensions of the system. Some of the important dimensions are the location of the intermediate support, the angle

of the supports (and thus the length of the cantilevering supporting structure), the stiffness of the tensile support members. These dimensions should be varied in the research to find the optimal dimensions for the intermediate supports.

For the connection between the intermediate support members and the bending members, an extra ring is created. The bending members inside the inner ring could also be left out, but in this case cladding is not possible inside the inner ring.

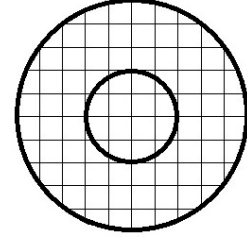


Figure 2.6: Top view

2.6.4. System 8: Inverse Dome Structure on Columns

This main structural idea of this system is comparable to the one of system 3, with the difference that the dome is inverted. Because of this inversion the internal normal force in the dome will be tension, if loaded with a uniformly distributed downward load. This creates compression in the ring beam.

In Figure 2.7 a schematic diagram of the system can be seen, both in cross-section as in top view.

An important aspect of this system is that if the dome material will be tension-only, extra structure is needed for the upward load.

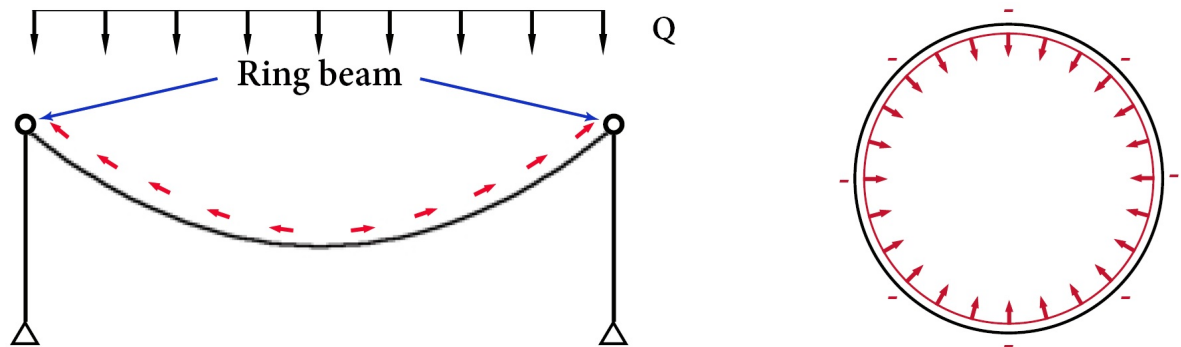


Figure 2.7: Inverse dome structure on columns in cross-section (left) and top view

3

Preliminary Calculations

The remaining systems 3, 5, 7 and 8 will be roughly calculated using Rhino, Grasshopper and Karamba3d to compare their structural behaviour. Each system will have similar starting points to make a reliable comparison. The result of these calculations will be the total value of the required kilograms of steel for each of the systems. The value is calculated by multiplying the steel type by the assumed price for that type of steel.

3.1. Modeling

3.1.1. General Assumptions

Before starting modeling, it is important to set up some starting points of the models. These starting points are briefly given here and explained a bit further in this paragraph.

Summary of General Assumptions

Shape and dimensions

- Shape is a circle with diameter of 200 meters
- The edge of the circle is modeled as a circle beam
- Centre-to-centre distance of linear systems equals 10 meters
- Number of radials in radial system equals 20
- In the radial system there are 4 tangential members spanning from each radial to the next one
- Systems 3 and 8 will be modeled by using a catenary surface
- Members loaded in bending will be modeled as trusses with a centre-to-centre distance of about $L/15$ to $L/20$
- Members loaded in compression will be modeled as CHS (Circular Hollow Sections)
- Trusses are modeled by giving the area of the upper member, lower member and the centre-to-centre distance

Sag/rise of dome structures

- Starting point is a catenary surface with a sag/rise of 20 meters
- Sag/rise is optimised by manual iterations

Columns and supports

- In first calculations the columns will be left out
- Columns will be modeled as point supports that support the ring beam
- Supports will have restricted movement in vertical and tangential direction
- Supports will have free movement in radial direction
- Intermediate supports in system 7 will be modeled as spring supports with the same angle as the tensile supporting members

Loads

4 types of loads can be present:

- Each member will have a dead load, resulting from self-weight of the member
- Uniformly distributed line loads will be put on the main members, that, in total, will approach the uniformly distributed live load of 1 kN/m^2
- If cables are used, pretensioning is applied. The value of the pretensioning will be determined later. This value can also be 0 if pretensioning is not favourable
- Each of the systems will have an asymmetrical load, which is downwards on one half of the system and upwards on the other half
- The asymmetrical load is modeled with point loads which equal a total load of 1 kN/m^2

Load combinations

Each of the systems will have 2 load combinations:

- Load case 0: Self-weight; uniformly distributed live load; pretensioning
- Load case 1: Self-weight; asymmetrical live load; pretensioning

Material

- For all members, except for the cables, steel S355 is used
- For the cables, steel wire ropes are used with an ultimate stress of 1570 N/mm² and a Young's Modulus of 160.000 N/mm²

Connections

- The connections between two trusses will be modeled as fixed connections
- The connections between CHS and trusses will be modeled as hinged connections

Maximum deflection and utilization

- The maximum vertical deflection in any part of the system is 500 mm
- The maximum utilization of each of the elements is a value of 1.0

Calculations

- Calculations are done with Karamba3d, a plugin for Grasshopper
- If no cables are used, only first-order calculations are performed
- If cables are used, second-order calculations are performed

Optimisation

- Optimisation is done with Galapagos, an optimisation component of Grasshopper
- In Galapagos, the evolutionary solver is used
- The optimisation is done by seeking the minimal self-weight with a utilization of all of the elements lower than 1.0

Cost of steel per kg

- The cost of structural steel S355 is taken as 2€/kg
- The cost of cable steel is taken as 8€/kg

Explanation of General Assumptions

Shape and dimensions

As a starting point, a circle with a diameter of 200 meters is used. The edge of the circle is modeled as a circle beam, which is supported by 8 point supports. The spanning structure spans from circle to circle.

Four different spanning directions can be used: radial, one direction, two directions and three directions. The system with three spanning directions will be modeled as a system with two spanning directions, and adding diagonals in one direction in the rectangles that follow from the system with two spanning directions.

The systems that span in either one, two or three directions will have a centre-to-centre distance of 10 meters. The radial system will have a total of 20 radial beams.

The dome structures (system 3 and 8) will be created on a catenary surface.

Karamba3d does not have the option to model a single beam as a truss. To use trusses in calculations, these cross-sections have to be modeled manually. This is done by making number sliders for the centre-to-centre distance, and the areas of both the upper and the lower chord of the truss. By adding formulas for the total area (A), the section modulus (W) and the moment of inertia (I), trusses can still be used. The structural behaviour like the axial stiffness, bending stiffness and the resistance are comparable, so the simple structural behaviour will be comparable as well. Even though the important section properties do match the original truss, it is important to realize that the trusses have to be checked separately for failure.

Sag/rise of dome structures

As said before, the starting point of this system was a dome with a sag/rise of 20 meters. The size of the sag/rise is simply chosen as 1/10 of the span. Obviously, the size of this sag/rise needs to be optimised to prevent unjustified conclusions. Unfortunately, the creation of a catenary surface by Grasshopper is rather time-consuming. Each time the input of the catenary surface, for instance the sag/rise, is changed it takes one to two minutes to recalculate the catenary surface. This makes the optimisation of the sag/rise a process that would take days to come to a somewhat reliable solution. One of the ways to solve this is to decrease the number of control points of the catenary surface. This makes the recalculation of the catenary surface much faster. To decrease the calculation time sufficiently for it to make the optimisation doable the number of control points has to be decreased by a large factor. It can easily be seen from the results of the Karamba3d calculations that the utilization rates of the members increase by over 10%. This makes it clear that to come to the optimal solution, the number of control points has to be kept at the original number. Another way to solve the long duration of the optimisation process is to keep the sag/rise out of the optimisation. To still come to an optimal sag/rise, an optimisation by hand is performed. The value of the sag/rise is first increased in large steps, to see the consequences of this change on the structural behaviour. These iterations will show the rough value of the optimal sag/rise. From this rough value multiple smaller iterations will be done, each with a rough optimisation of the structure, to give a more exact value of the optimal sag/rise.

One side note to this optimisation of the sag/rise has to be made. If the sag/rise of one of the dome structures changes, the length of the columns has to change as well, to satisfy the requirement of the minimum height clearance above the pitch. Since the columns are taken out of this preliminary calculations, the length of the columns is neglected.

Columns and supports

Each of the four systems is elevated from the floor through columns. To simplify the calculations these columns are modeled as point supports on the top point of the column. Each of the systems will have comparable vertical loads, and the horizontal loads will mainly be taken by the ring beam. This means that the columns for each of the systems will have comparable dimensions, and can therefore be taken out of this first rough calculations. Although the inverse dome structure will have longer columns, because of the required clearance height at the centre, this difference can be neglected. It is always good to keep this difference in mind, but it will be negligible.

The point supports (on top of the not-modeled columns) will have restricted movements in the vertical direction and in the direction tangential to the ring. Perpendicular to the ring there won't be a restriction. The

reason for this is that there will be angled columns underneath the ring beam, to assure rotational stability. These angled columns will still be completely underneath the ring beam, and will therefore not give support in the radial direction.

The intermediate supports of system 7 will be modeled as spring supports with the same angle as the tensile supporting members. The spring stiffness will follow from the length and the axial stiffness of the tensile supporting members.

Loads

Naturally the self-weight of the members is put on every member by modeling a gravity load.

The cables will be loaded with a pretensioning load. The optimal value of this load will be determined during the calculations. In case of cables that can not be pretensioned this value will be 0.

For each of the systems an uniformly distributed load of 1 kN/m² is used. In the model this load is put on beams as an uniformly distributed line load, which approaches the uniformly distributed load of 1 kN/m². In the final design the loads will be determined according to the Eurocode. The loads will probably be much larger than the loads that are used in this preliminary design.

To check the systems for their behaviour against asymmetrical loads, asymmetrical point loads are modeled on each of the systems. These loads will be downwards on one half of the system and upwards on the other half of the system. The total value of all the point loads will approach the value of 1 kN/m².

Load combinations

Two load combinations will be used. Both combinations will include the self-weight and the pretensioning. Load Case 0 will also include the uniformly distributed live load. Load Case 1 will also include the asymmetrical loads.

With these load combinations there can be seen which system reacts well to asymmetrical loads.

Material

All of the members will be in steel. Two different steel types will be used: normal Structural steel and Cable steel.

The normal structural steel will be S355. This type of steel has an ultimate stress of 355 N/mm² and a Young's Modulus of 210.000 N/mm².

For the cables steel wire ropes will be used. The type that will be used has an ultimate stress of 1570 N/mm² and a Young's Modulus of 160.000 N/mm². Since this steel type is much more expensive than S355, this difference in costs have to be taken into account to make a reliable comparison.

Connections

The connections between two trusses will be modeled as fixed connections. The reason for this is that both the upper as the lower beam of the truss will be connected. This gives a fixed connection.

The connections between CHS and trusses will be modeled as hinged connections. The reason for this is that it's expensive and time-consuming to create a fixed connection. The connection will always have certain rotational stiffness, but to avoid a complicated model, these connections will be modeled as hinges.

Maximum deflection and utilization

For the maximum deflection the formula $\delta_{max} = span/400$ is used. For a span of 200 meters this means that the maximum vertical deflection in the structure is 500 mm.

The maximum utilization of each of the elements is a value of 1.0.

Optimisation [Davidson and Rutten, 2016]

Each of the systems will be optimised to find the minimal self-weight, while utilization is still below 1.0. This optimisation will be performed with Galapagos, a component of Grasshopper.

Galapagos has two different ways of optimising, the Evolutionary Solver and the Simulated Annealing Solver.

The Evolutionary Solver creates a random group of options. Each of these options is then improved by changing the input sliders and by combining options.

The Simulated Annealing Solver starts with an initial solution. This solution is then changed a little bit every step to improve the initial solution. The settings determine how large each step is, and how many input sliders will be changed in each step.

Because in this calculation, each of the input sliders are greatly dependent on each other, the Evolutionary Solver will be used. This will decrease the chance that the best solution is neglected.

Each optimisation will be performed multiple times to decrease the chance of neglecting the optimal solution.

Because the optimisation in this stage is just a comparison between the different structural systems, the dimensions are not rounded to actual market dimensions. That means that the optimal dimensions of, for instance, a CHS are used, even though these dimensions are not the standard values fabricators use.

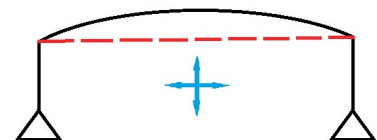
3.1.2. Validation

The Karamba3d plugin will be used to check the structure on utilization, deflection and stability. Especially regarding the stability it is important to check whether the software is reliable in this aspect. To validate the correctness of this stability, simple calculations are performed both by hand as by the use of Karamba3d. The results of these calculations will be compared to validate the calculation methods of Karamba3d.

For the validation of the calculation for stability, a simple beam on two hinged supports is modeled. In the first check the beam is loaded with a point load in the direction of the beam. This will create a uniformly distributed compression force in the beam. In the second check the beam is loaded with a uniformly distributed line load perpendicular to the beam axis.

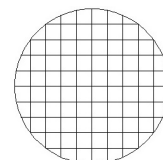
3.1.3. System 3: Dome Structure on Columns

The main structural members of the dome structure on columns are the beams in the dome, the ring beam that carries the thrust forces, and the columns. As said before the columns won't be part of this first calculations.



To create the dome first a catenary curve with a rise of 20 meters is created. After this a catenary surface is created by rotating the curve around the z-axis. A grid of beams is then pulled on to this surface.

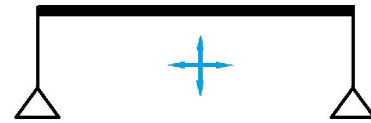
For the compression dome a rectangular grid is used, because with this grid each beam has comparable loads. As said before the centre-to-centre distance is 10 meters.



One of the options was to diagonalize the rectangular grid by adding beams in a third direction. This has been modeled to research the consequences. It turned out that the structural behaviour was not notably increased, while, of course, the self-weight of the structure did increase notably and the connections will become harder to realize. Therefore the system with beams in two directions is used.

3.1.4. System 5: Bending Members on Columns

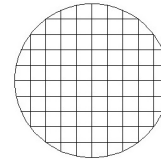
The main structural members of this system are the bending members, the ring beam that carries the forces to the columns, and the columns. As said before the columns won't be part of this first calculations.



The main structural members of this system are the bending members, the ring beam that carries the forces to the columns, and the columns. As said before the columns won't be part of this first calculations.

In this system a grid of beams is modeled on the circular surface that has been created. Each of the bending members spans from one part of the ring to another part, both in x- and y-direction.

For this system a rectangular grid is used, because with this grid each beam has comparable loads. As said before the centre-to-centre distance is 10 meters.



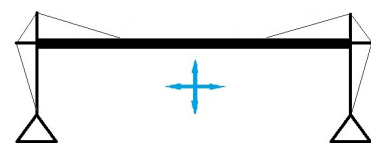
3.1.5. System 5a: Bending Members on Columns with Two Main Bending Members

As a variation on system 5, also the system with two main bending members is researched to see the consequences of using two main members. For this system the model of system 5 is being used. Two of the bending members in one direction, approximately the members on 1/3 and 2/3 of the span, are dimensioned much bigger than all other members. These will be the primary load bearing members. In the opposite direction, secondary members are modeled that transfer the loads to the primary members. Spanning between the secondary members are small tertiary members that give stability to the secondary members.

With this system it can be seen what the influence is of clustering the internal forces on the structural behaviour. It makes sense that if clustering it to two main members has a negative influence on the structural behaviour, the same is true for clustering it to for instance three members.

3.1.6. System 7: Cable-stayed Bending Members on Columns

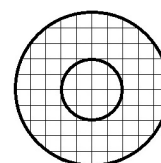
In this system the same bending members as the ones in system 5 are used. The cross-sectional dimensions are obviously not the same, since the intermediate supports reduce the bending moments. The intermediate supports are modeled as spring supports at 1/3 and 2/3 of the span. The spring stiffness is calculated by taking the length of the supporting tensile members in the supporting system and the stiffness of the supporting tensile members. The dimensions of the supporting tensile members and the length of both cantilevering supporting structures will be modeled as number sliders, so they can be incorporated in the optimisation of the structure.



As can be seen in the mechanics diagram before, the intermediate cable supports will lead to extra compression forces in the members, because of the angle of the cables. Because these supports are modeled as spring supports it is important to check whether Karamba3d takes these extra compression forces into account.

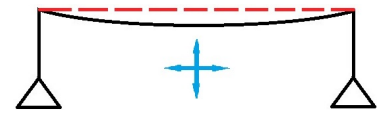
The cantilevering supporting structure is assumed to be CHS323.9x16 for the calculation of the weight of the supporting structure.

For this system a rectangular grid is used, because with this grid each beam has comparable loads. An extra ring is designed, to which the supporting members are connected.

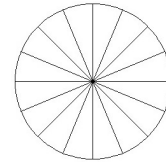


3.1.7. System 8: Tension Dome on Columns

The shape of system 8 is the inverse of the shape of system 5. The model is therefore created in a similar way as the one of system 5.



While researching the rectangular grid for this system, it could be seen that the structural behaviour of the rectangular grid was not favourable. One of the ways to solve this is to diagonalize the grid, but this would lead to a tangle of cables. The other way of solving this unfavourable structural behaviour is by using a radial grid. In this way the main members would create a triangular grid, which leads to a larger stiffness and more favourable structural behaviour.



3.2. Grasshopper Model

The Grasshopper model for the preliminary calculations consists of three parts: the base structure, the spanning structure, and the calculations. Each part will be described briefly in this section, but a more comprehensive description will be given in Appendix C.

3.2.1. Base Structure

As explained in the general assumptions in Section 3.1.1 the base structure is a circle on columns. This circle is modeled with a variable radius, so it can be adjusted in a later phase. In different directions grids are modeled in the circle. For each of the spanning directions, a grid of lines is modeled. These directions are spanning in one, two or three directions and spanning in radial direction. These lines are modeled in the plane of the circle, so they can later be modified to adjust to each of the structural systems. Each of the curves is divided in smaller parts at each connection with another beam. This is done because of the finite element calculations that will be performed later.

3.2.2. Spanning Structure

Each of the different structural systems will be modeled in Grasshopper. The dimensions will be modeled as number sliders, so all dimensions can be adjusted later.

The dome structures (system 3 and 8) both have the shape of a catenary surface. Unfortunately Grasshopper only has the function to create a catenary curve, but not the function to create a catenary surface. To create a catenary surface, first a catenary curve that spans from one side of the base circle to the other side is created. The sag/rise of the catenary curve is again modeled with a number slider, so the sag/rise can be optimised. After that half of the catenary curve is the input for a revolution surface. A revolution surface in Grasshopper means that the curve is rotated around a certain axis to get a revolution surface. In this case the revolution axis is taken as the z-axis, and the revolution angle domain is from 0 to 2π .

After the surface is created, the grid of beams must be modeled on this surface. This can be done in multiple ways. One way Grasshopper has is to "pull" the curves onto the surface. In this case the closes points on this surface are found to build up the new curve. Another way is to project the curves onto the surface. In this case the curves are projected along an axis on the surface. This means that, if the z-axis is taken as the projection axis, only the z-coordinates of the curves change, while by "pulling" the curves, also the x- and y-coordinates can change. Because both the begin end the end point of each spanning beam remains the same, and the curves should remain in the same plane, the projection is the best way to get a set of catenary curves.

The system with simple bending members (system 5) is easily created from the grid of beams that was created before. The geometry of the bending members is exactly the same as the grid of curves in two directions that was created in Section 3.2.1.

The system with bending members with intermediate supports is created by using the grid structure created before, and adding extra members at the position of the intermediate supports with the same direction as

the cables attached to the intermediate supports. In Karamba there is the possibility to give a member the property "spring". These extra members will get this property, with a spring stiffness that is calculated from the design explained in Chapter 2. In Grasshopper the dimensions of all parts of these extra supports are modeled as number sliders. From these number sliders the spring stiffness is calculated automatically.

3.2.3. Calculations

The preliminary calculations are performed with Karamba3d. Karamba is a FEM plugin for Grasshopper. To perform calculations Karamba needs some input. The input that is used for these calculations are elements (beams or shells), supports, loads, cross-sections, materials and joints.

The curves that were created before are transformed to create beams. Each curve is created as a individual beam with all properties of the curve.

For the supports each support has a location and restrictions. The locations of the supports are given by dividing the base ring into 8 equal elements. The 8 points that are created are used as locations for the supports.

For these preliminary calculations a few different load types are used. First of all the gravity loads. Karamba calculates these automatically from the dimensions and the material of each of the elements. The second load type is the pretensioning load. Karamba has a function to model pretensioning. The values of the pretensioning will be part of the optimisation. The third load type is the uniform load. This load will be modeled to act on the grid of beams. Since not all systems have an equal grid the loads on the beams will not be equal as well. Therefore a total downward load is calculated from the area of the base circle, multiplied by the value of the loading. In this case a uniform load of 1 kN/m^2 is used. The total downward load is then divided by the total length of the beams to get the line load per m^1 .

For the cross-sections different cross-sections have to be modeled, one for each beam type. The CHS can easily be chosen from a list of cross-sections that Karamba has included in its software. Unfortunately Karamba does not have an option to model trusses, so to prevent extensive truss modeling for each of the truss elements, trusses are modeled as beams with the properties of a truss. The properties are calculated with a small tool that calculates the cross-sectional properties from the areas of the upper and lower chord, and the centre-to-centre distance.

Karamba has set the standard joint properties to fixed. If hinges are demanded, these joints need to be set manually. Each of the locations where hinges are demanded can be derived from selecting the proper end points of the beams that have joints at one or both ends.

3.3. Results

3.3.1. System 3: Dome Structure on Columns

The first step of optimising the dome structure was to optimise the rise of the catenary surface. As explained before, this is done manually, because of the calculation time of the catenary surface. First large iterations are done to find the rough location of the optimum. This is done by looking at the numbers for the utilization of the members, the self-weight and the deflection. The rise is changed in steps of 2 meters. By looking at the change of the calculation outcome, there can be determined whether the change of the rise is a step in the right direction. If the values of the utilization decrease, it is likely that the rise has changed to a more optimal value. From this first step of optimisation it turned out that the utilization values were decreasing as the value of the rise was increased, while the self-weight was not increasing as much. This continued until the rise reached a value of around 40 meters. After this point it was clear that by increasing the rise further, the structure would become less optimal. Now this has been established, the rise can be further optimised by iterating the rise in smaller steps. For each step an optimisation of the other input values is done, so an optimal structure would be determined for each value of the rise. By comparing these numbers it showed that a rise of 39,0 meters is the optimal value for this structure.

After the rise is optimised, the cross-sections of the members can be optimised. As explained before this is done with the Evolutionary Solver of Galapagos.

The values that are optimised are the diameter and the wall thickness of the CHS, and the area of the upper and lower chord of the ring beam truss.

Table 3.1: Results of optimisation of system 3

	Total weight (kg)	Total weight per m ² (kg)	Total cost per m ² (€)
Roof	562.345	17,9	35,8
Ring	197.920	6,3	12,6
Total	760.265	24,2	48,4

3.3.2. System 5: Bending Members on Columns

In this system only 4 values have to be optimised, namely the area of the upper and lower chord of the ring truss, and the upper and lower chord of the bending members.

Table 3.2: Results of optimisation of system 5

	Total weight (kg)	Total weight per m ² (kg)	Total cost per m ² (€)
Roof	973.894	31,0	62,0
Ring	254.469	8,1	16,2
Total	1.228.363	39,1	78,2

3.3.3. System 5a: Bending Members on Columns with Two Main Bending Members

In this system, multiple values have to be optimised. The system consists of 3 types of trusses, and one type of CHS, the tertiary members.

By setting boundaries to each of the number sliders, it can be prevented that Galapagos will create the same solution as the one for system 5.

Table 3.3: Results of optimisation of system 5a

	Total weight (kg)	Total weight per m ² (kg)	Total cost per m ² (€)
Roof	157.080	5,0	10,0
Ring	1.489.115	47,4	94,8
Total	1.646.195	52,4	104,8

3.3.4. System 7: Cable-stayed Bending Members on Columns

This system has the most values that have to be optimised. These values are: area of both chords of the ring beam truss, area of both chords of the bending trusses, location of the intermediate support (distance from edge), length of the vertical cantilevering support structure and the diameter of the cable.

The length of the horizontal cantilevering support structure is ignored in the optimisation, because in the model this members don't have any participation in the structure, and would therefore be optimised to 0. In reality the length of this member would have influence on the structural behaviour, especially on the supporting structure. This influence is negligible for the determination of the total weight per m², so it can be neglected in the optimisation.

Table 3.4: Results of optimisation of system 7

	Total weight (kg)	Total weight per m ² (kg)	Total cost per m ² (€)
Roof	728.849	23,2	46,4
Ring + extra support structure	405.265	12,9	25,8
Total	1.134.114	36,1	72,2

3.3.5. System 8: Tension Dome on Columns

As said before, this system will have the radial spanning direction. Consequently, this system will have three different cross-sections to optimise, namely the ring beam, the radial cables and the tangential secondary members. Also the pretensioning will be included in the optimisation to see the influence of this pretensioning.

First this model was optimised without asymmetrical loads. This first optimisation already showed something really important. Although the weight of the structure is somewhat lower than the weight of the other structures, the difference is not that big. Because of the price of steel cables is over 4 times higher than the price of normal structural steel, the price of this system is a lot higher than the price of the other systems.

Another aspect that is important in this structure is that this tension-only structure has little to no resistance against any loads besides vertical downward loads. If asymmetrical and upward loads would be incorporated, the weight of this system would even be higher.

These reasons combined gives a good ground to dismiss this system in the research. The values of this system will be shown in the summary in the next paragraph, but these are the values for calculations without asymmetrical loads.

Table 3.5: Results of optimisation of system 8

	Total weight (kg)	Total weight per m ² (kg)	Total cost per m ² (€)
Roof	449.248	14,3	114,4
Ring	113.097	3,6	7,2
Total	562.345	17,9	121,6

3.3.6. Summary of Results

In Table 3.6it can be seen that the compression dome is by far the optimal solution with regards to the total cost. Although the tension dome has a lower dead load per m², the price of cable steel is approximately four times higher than the price of structural steel S355. This makes the total costs of the tension dome 2,5 times higher than the costs of the compression dome. Also, the calculations for the tension dome are performed with only symmetrical downward loads. Because this already gives high costs for this system, there is no need to improve the model by adding asymmetrical and upward loads.

Because of the low costs per m² the system that will be further elaborated is system 3: compression dome. In the next phase of the research a more detailed model of this system will be made and further improvements to the design will be done to come to an optimal design.

Table 3.6: Summary of results

System	Total weight per m ² (kg)	Cost in € per m ²	Notes
3: Compression dome	24,2	48,4	
5: Bending	39,1	78,2	
5a: Bending with two main members	52,4	104,8	
7: Cable-stayed bending	36,1	72,2	
8: Tension dome	17,9	121,6	No asymmetrical loads

3.3.7. Remark

The structures that are designed in this chapter have been designed to resist a uniform load of 1 kN/m². The loads in the final model will be determined according to the Eurocode, and will be significantly larger than 1 kN/m². These higher loads will obviously lead to larger cross-sectional dimensions, and therefore also to a larger self-weight of the structure. The dimensions and the self-weights that are determined in this chapter are for comparison only.

4

Design Input

In Chapter 3 the most optimal system is chosen. This system will be modeled more thoroughly to use it in the final calculations and optimisation. In this chapter the input for the design is considered.

First, the design criteria are drafted. These design criteria are the starting points for the design. From these design criteria, the requirements that are set by the Eurocode can be stated. Finally, the loads on the structure can be determined. For these loads, the Eurocode is used. [NEN-EN 1990, 2011]

For the determination of the wind loads in this chapter, a dome height of 40 meters is used. In the Grasshopper model that will determine the loads in the calculation models, the values of the wind loads are calculated according to the dome height that is modeled as input in the Grasshopper model. In this way, the values of the loads automatically adjust to the dimensions of the structure.

4.1. Design Criteria

Starting Points

- The design consists of a circular roof with the shape of a catenary surface, which is supported by a ring truss on columns
- The radius of the roof is 200 m
- The roof consists of a rectangular grid of beams that follow the catenary shape
- The main structural material will be S355 steel. If demanded other steel types can be used
- The structure should be optimal with regards to the use of material

Boundary Conditions

- The structure should be suitable to place anywhere in the Netherlands
- The structure should be designed according to the Eurocode [NEN-EN 1990, 2011]

4.2. Requirements set by Eurocode

4.2.1. Introduction

The Eurocode is the European standard for structural design. Each structure must comply to the requirements stated in the Eurocode. The Eurocode consists of multiple parts, each for different aspects of the structural design. The Eurocode is a standard for Europe, but a lot of countries have their own national annex. This annex contains information that is specific for the region. In this design the Eurocode with the Dutch national annex is used as the standard.

The parts that are important for this design are described briefly in this paragraph.

- Eurocode 0 - Basis of structural design
This chapter describes the basic requirements for a structure. It also describes the definitions and symbols that are used in the other parts and the requirements with regards to displacements.
- Eurocode 1 - Actions on structures
 - Part 1-1: General actions
This part describes the general actions that act on structures. This includes self-weight and imposed loads.
 - Part 1-3: Snow loads
This part describes the snow loads on a structure
 - Part 1-4: Wind loads
This part describes the wind loads on a structure

4.2.2. Design Class

The Eurocode 0 prescribes that each building is classified in one of the Design classes, according to their design life. For non-monumental structures like the Event Dome a design life of 50 years is taken. From Table 4.1 it follows that the structure is in Design Class 3.

4.2.3. Consequence Class

The Eurocode 0 prescribes that each building is also classified in one of the Consequence classes. The consequence class of a building shows what the consequence of failure of the structure is, regarding loss of human life. According to the Eurocode 0 a building should be in CC3 if more than 500 people are in danger in case of failure. The Event Dome is therefore in CC3.

Table 4.1: Design classes [NEN-EN 1990, 2011]

Design class	Design life	Application
1	5	Temporary structures
2	15	Low-rise industrial buildings and agricultural structures
3	50	Buildings and other regular structures
4	100	Monumental buildings

Table 4.2: Consequence classes [NEN-EN 1990, 2011]

Consequence class	Description
CC3	High consequence regarding loss of human life
CC2	Medium consequence regarding loss of human life
CC1	Low consequence regarding loss of human life

4.2.4. Ultimate Limit State

In the Eurocode 0 two different limit states are distinguished. The first one is the Ultimate Limit State (ULS). The ULS preserves the safety of persons and/or the safety of the structure. In order to fulfill the safety requirements, the following aspects have to be verified [NEN-EN 1990, 2011]:

- Loss of equilibrium of the structure or a part of the structure, that can be seen as a rigid body. This is better known as loss of stability
- Failure by excessive deformations
- Failure caused by fatigue

In order to verify these aspects the Eurocode gives the following design values of actions for CC3, including the γ factors:

Table 4.3: Design values of actions in ULS [NEN-EN 1990, 2011]

Permanent actions		Leading variable action	Accompanying variable actions	
Unfavourable	favourable		Main (if any)	Others
$1.5G_{k,j,\text{sup}}$	$0.9G_{k,j,\text{inf}}$		$1.65\psi_{0,1}Q_{k,1}$	$1.5\psi_{0,i}Q_{k,i} (i>1)$
$1.3G_{k,j,\text{sup}}$	$0.9G_{k,j,\text{inf}}$	$1.65Q_{k,1}$		$1.65\psi_{0,i}Q_{k,i} (i>1)$

The Dutch National Annex gives the ψ factors for buildings. The applicable values for this project are given in Table 4.4.

It can be seen that, since all values for ψ_0 are equal to 0, no combinations of variable actions have to be considered in the ULS

4.2.5. Serviceability Limit State

The second limit state is the Serviceability Limit State (SLS). The SLS preserves the comfort for the users of the structure. These limit states are considering the following topics [NEN-EN 1990, 2011]:

- The functioning of a structure or parts of the structure under regular circumstances
- The comfort of users
- The appearance of the structure

The ULS considers the structural safety of the structure under extreme circumstances. This is the reason why factors have to be applied to the load in the ULS. In the SLS on the other hand the comfort under regular circumstances is considered. For this reason no γ factors have to be used in the SLS. According to the Eurocode 0 the load combinations in the SLS are as shown in Table 4.5.

Table 4.4: ψ factors for buildings [NEN-EN 1990, 2011]

Category	Description	ψ_0	ψ_1	ψ_2
H	Roofs	0	0	0
	Snow loads	0	0.2	0
	Wind loads	0	0.2	0
	Temperature (non-fire)	0	0.5	0

Table 4.5: Design values of actions in SLS[NEN-EN 1990, 2011]

Combination	Permanent actions G_d		Variable actions Q_d	
	Unfavourable	favourable	Main	Others
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	$Q_{k,1}$	$\psi_{0,i} Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{1,1} Q_{k,1}$	$\psi_{2,i} Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{2,1} Q_{k,1}$	$\psi_{2,i} Q_{k,i}$

Vertical deflections

To preserve the comfort for the users of the structure, it is also important to consider maximum deflections and vibrations. According to the Eurocode 0 the following requirements have to be met for the vertical displacements of each of the members.

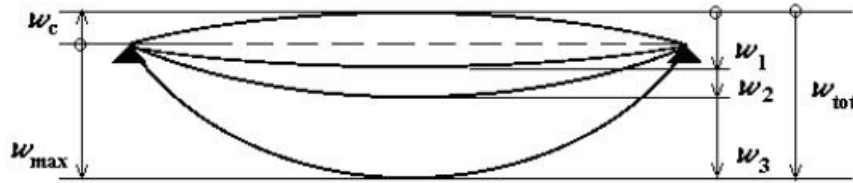


Figure 4.1: Vertical deflections [NEN-EN 1990, 2011]

in which:

w_c	Precamber of the unloaded structure
w_1	Initial part of the deflection under permanent loads
w_2	Long-term part of the deflection under permanent loads
w_3	Additional part of the deflection under variable loads
w_{tot}	The total deflection, which is the sum of w_1 , w_2 and w_3
w_{max}	Remaining deflection, which is the total deflection w_{tot} minus the precamber w_c

The maximum vertical deflections for the SLS are shown in Table 4.6, in which l_{rep} is the length of the span or twice the length of the cantilever. Only applicable values are shown in this table.

The maximum vertical deflection in this design is therefore:

$$w_{max} \quad 1/250 \cdot 200 \quad = \quad 0.8 \text{ m}$$

Horizontal deflections

The Eurocode 0 sets requirements for the maximum horizontal displacement. In this requirement a distinction is made between the horizontal deflection per story and the horizontal deflection for the total building. Since the Event Dome is a building with only one story the maximum deflection per story will be taken as the maximum horizontal deflection for the entire structure. Figure 4.2 shows the horizontal deflections schematically. In this figure the following symbols are used:

H	Height of the building
H_i	Height of a story i
u	Horizontal deflection over the total building height H
u_i	Horizontal deflection of the story height H_i

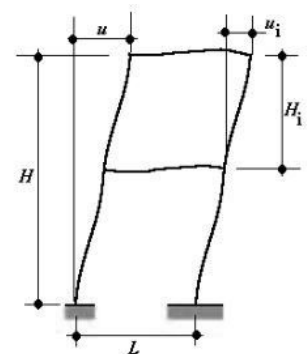


Figure 4.2: Maximum horizontal deflections [NEN-EN 1990, 2011]

Table 4.6: Maximum vertical deflections [NEN-EN 1990, 2011]

	Parts	Maximum	Combination
Other roofs	$w_2 + w_3$	$1/250 \cdot l_{\text{rep}}$	Characteristic

In this case $H = H_i$ and $u = u_i$. The maximum horizontal deflection for the characteristic load combination is $h/300$, in which:

h Lowest height of the building

The maximum horizontal deflection in this design is therefore:

$$u_{\max} \quad 50/300 \quad = \quad 0.17 \text{ m}$$

Vibrations

In order to achieve satisfactory vibration behaviour of buildings and its structural elements the following aspects have to be considered [NEN-EN 1990, 2011]:

- The comfort of users
- The functioning of a structure or parts of the structure

Because in this design the only part considered is the roof, which is not accessible, only the second aspect has to be considered in this case. The only aspect of the vibration that has to be checked is the natural frequencies of the roof, compared to the vibration in the roof induced by the wind. In general, longer spans give lower natural frequencies. Also, light-weight large-span structures will have multiple natural frequencies. Especially in odd-shaped structures like a dome structure, the critical natural frequency is usually not the natural frequency of the first mode. [Nakayama et al., 1998] Since the roof structure has a large span and is also light-weight, it is important to check what the critical natural frequency will be. This natural frequency has to be checked against the wind-induced vibrations to see if vibrations will be a problem in this structure.

4.2.6. Load Combinations

The Eurocode 0 prescribes multiple load combinations for both the ULS and the SLS. These are given in Table 4.3 and Table 4.5. In these load combinations the following load cases are considered:

$G_{k,j}$	Self-weight of the structure
$Q_{w,h}$	Horizontal loads by wind pressure
$Q_{w,v+}$	Upward vertical loads by wind pressure
$Q_{w,v-}$	Downward vertical loads by wind pressure
Q_{snow}	Loads by snow
Q_k	Imposed loads

With these load cases and the load combinations from Table 4.3 and Table 4.5 the following load combinations that can be drafted can be seen in Table 4.7.

Since all values for ψ_0 and ψ_2 are equal to 0, no combinations of variable loads have to be considered, besides the horizontal and vertical wind loads. This is logical because in case of maximum wind load, no snow will stay on the roof and no maintenance will take place. The same holds for the combination of snow and maintenance.

4.3. Actions on Structure

4.3.1. Permanent Actions

The Eurocode 1-1-1 describes permanent actions as the complete dead load of the structure. For the Event Dome this includes the self-weight of the structure, the roof cladding and the fixed installations.

The self-weight of the structure is dependent on the design of the structure and will therefore be determined during the calculations.

Table 4.7: Load combinations [NEN-EN 1990, 2011]

LC	Description	Load combinations with partial factors
1	ULS permanent load	$1.5G_{k,j,\text{sup}}$
2	ULS imposed load	$1.3G_{k,j,\text{sup}} + 1.65Q_k$
3	ULS downward wind	$1.3G_{k,j,\text{sup}} + 1.65(Q_{w,h} + Q_{w,v-})$
4	ULS upward wind	$0.9G_{k,j,\text{inf}} + 1.65(Q_{w,h} + Q_{w,v+})$
5	ULS snow load	$1.3G_{k,j,\text{sup}} + 1.65Q_{\text{snow}}$
6	SLS permanent load characteristic/frequent	$G_{k,j,\text{sup}}$
7	SLS imposed load characteristic	$G_{k,j,\text{sup}} + Q_k$
8	SLS downward wind characteristic	$G_{k,j,\text{sup}} + Q_{w,h} + Q_{w,v-}$
9	SLS upward wind characteristic	$G_{k,j,\text{inf}} + Q_{w,h} + Q_{w,v+}$
10	SLS snow load characteristic	$G_{k,j,\text{sup}} + Q_{\text{snow}}$
11	SLS imposed load frequent	$G_{k,j,\text{sup}} + 0.2Q_k$
12	SLS downward wind load frequent	$G_{k,j,\text{sup}} + 0.2Q_{w,h} + 0.2Q_{w,v-}$
13	SLS upward wind load frequent	$G_{k,j,\text{inf}} + 0.2Q_{w,h} + 0.2Q_{w,v+}$
14	SLS snow load frequent	$G_{k,j,\text{sup}} + 0.2Q_{\text{snow}}$

Roof cladding

Because of the large span of the structure a lightweight roof cladding is used. In this design ETFE cushions are used, because of their light weight, but also because of their possibilities for lighting and translucency. This could create an extra value for the design by increasing the possibilities for different functions.

As said before, ETFE cushions are extremely lightweight. The foil weighs about 1 to 10 N/m², depending on the thickness of the foil. Including all installations and secondary structure required the total weight of the ETFE cushion system can be approximated at 100 N/m² or 0.1 kN/m². [Bessey, 2012]

4.3.2. Variable Actions

The roof of the Event Dome has three different types of variable actions: imposed loads, wind loads and snow loads. The values of each of these loads will be determined according to the Eurocode 1. Accidental actions, as described in [NEN-EN 1991-1-7, 2011], will not be taken into account in this research.

Imposed loads [NEN-EN 1991-1-1, 2011]

Imposed loads should be classified as variable actions. The Eurocode 1-1-1 categorizes roofs according to their accessibility into three categories. This is shown in Table 4.8. The roof in this research is not accessible except for normal maintenance and is therefore of category H.

Table 4.8: Categorization of roofs [NEN-EN 1991-1-1, 2011]

Categories of loaded area	Specific use
H	Roofs not accessible except for normal maintenance and repair
I	Roofs accessible with occupancy according to categories A to G
K	Roofs accessible for special services, such as helicopter landing areas

For roofs of category H the minimum characteristic values Q_k and q_k that should be used are given in Table 4.9.

Wind loads

Wind actions fluctuate with time and act directly as pressures on the external surfaces. They may also act directly on the internal surface of open structures. Pressures act on areas of the surface resulting in forces normal to the surface of the structure. Additionally, when large areas of structures are swept by the wind, friction forces acting tangentially to the surface may be significant.

Table 4.9: Imposed loads on roofs of category H [NEN-EN 1991-1-1, 2011]

Roof slope α	q_k [kN/m ²]	Q_k [kN]
$0 \leq \alpha \leq 15^\circ$	1.0	1.5
$15 \leq \alpha \leq 20^\circ$	$4 - 0.2 \cdot \alpha$	
$\alpha \geq 20^\circ$	0	

In this research tangential friction forces, caused by the wind, are neglected. The wind loads are determined according to the Eurocode 1-1-4 [NEN-EN 1991-1-4, 2011].

Two different types of wind loads on roofs can be considered in the Eurocode. First there is the wind load on a closed building. This wind load is composed by the external wind pressure and the internal wind pressure. These two pressures combined give the total wind load acting on the structure. The other type of wind load is the wind load on canopy roofs. A canopy roof is described in the Eurocode as the roof of a structure that does not have permanent walls. Since this structure must be suitable for both the maximum of both values will be used.

The wind velocity and the velocity pressure are composed of a mean and a fluctuating component. The mean wind velocity v_m should be determined from the basic wind velocity v_b which depends on the wind climate, the height variation of the wind, the terrain roughness and orography. The fluctuating component of the wind is represented by the turbulence intensity. The peak velocity pressure is determined using the turbulence intensity.

Basic wind velocity

The basic wind velocity v_b is calculated from Equation (4.1).

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} \quad (4.1)$$

in which:

c_{dir}	Directional factor (1.0 in the Netherlands)
c_{season}	Season factor (1.0 in the Netherlands)
$v_{b,0}$	Fundamental value of the basic wind velocity, see

The fundamental value of the basic wind velocity is determined using Table 4.10. Since this design should be suitable for every location in the Netherlands, the most critical value for the fundamental value of the basic wind velocity is used.

Table 4.10: Values for $v_{b,0}$ for different wind areas [NEN-EN 1991-1-4, 2011]

Wind area	I	II	III
$v_{b,0}$ [m/s]	29.5	27.0	24.5

These values give a basic wind velocity of:

$$v_b = 1.0 \cdot 1.0 \cdot 29.5 = 29.5 \text{ m/s} \quad (4.2)$$

Mean wind velocity

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b , and should be determined using Equation (4.3). The height z above the terrain is taken as 90 m.

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b \quad (4.3)$$

in which:

$c_r(z)$	Roughness factor
$c_0(z)$	Orography factor, taken as 1.0

The roughness factor $c_r(z)$ is given by Equation (4.4) for $z_{\min} \leq z \leq z_{\max}$.

$$c_r(z) = k_r \cdot \ln \frac{z}{z_0} \quad (4.4)$$

in which:

k_r	Terrain factor depending on the roughness length, calculated using Equation (4.5)
z_0	Roughness length defined in Table 4.11
$z_{\min}$	Minimum height defined in Table 4.11
$z_{\max}$	Maximum height, taken as 200 m

$$k_r = 0.19 \cdot \left(\frac{z_0}{0.05} \right)^{0.07} \quad (4.5)$$

Table 4.11: Terrain categories and terrain parameters [NEN-EN 1991-1-4, 2011]

Terrain category		z_0 [m]	$z_{\min}$ [m]
0	Sea or coastal area	0.005	1
II	Unbuilt area	0.2	4
III	Built area	0.5	7

Again, since this design should be suitable for every location in the Netherlands, the most critical terrain category is used, which is terrain category 0. The mean wind velocity $v_m(z)$ can now be determined using Equation (4.6), Equation (4.7) and Equation (4.8).

$$k_r = 0.19 \cdot \left(\frac{0.005}{0.05} \right)^{0.07} = 0.162 \quad (4.6)$$

$$c_r(90) = 0.162 \cdot \ln \frac{90}{0.005} = 1.585 \quad (4.7)$$

$$v_m(90) = 1.585 \cdot 1.0 \cdot 29.5 = 46.7 \text{ m/s} \quad (4.8)$$

Peak velocity pressure

The peak velocity pressure $q_p(z)$ at height z , which includes mean and short-term velocity fluctuations, should be determined with Equation (4.9).

$$q_p(z) = c_e(z) \cdot q_b \quad (4.9)$$

in which:

$c_e(z)$	Exposure factor given in Figure 4.3
q_b	Basic velocity pressure given in Equation (4.10)

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \quad (4.10)$$

in which:

ρ	Air density, taken as 1.25 kg/m ³ in the Netherlands
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The peak velocity pressure $q_p(z)$ can now be determined with $h = 90\text{m}$, using Equation (4.11), Equation (4.12) and Equation (4.13).

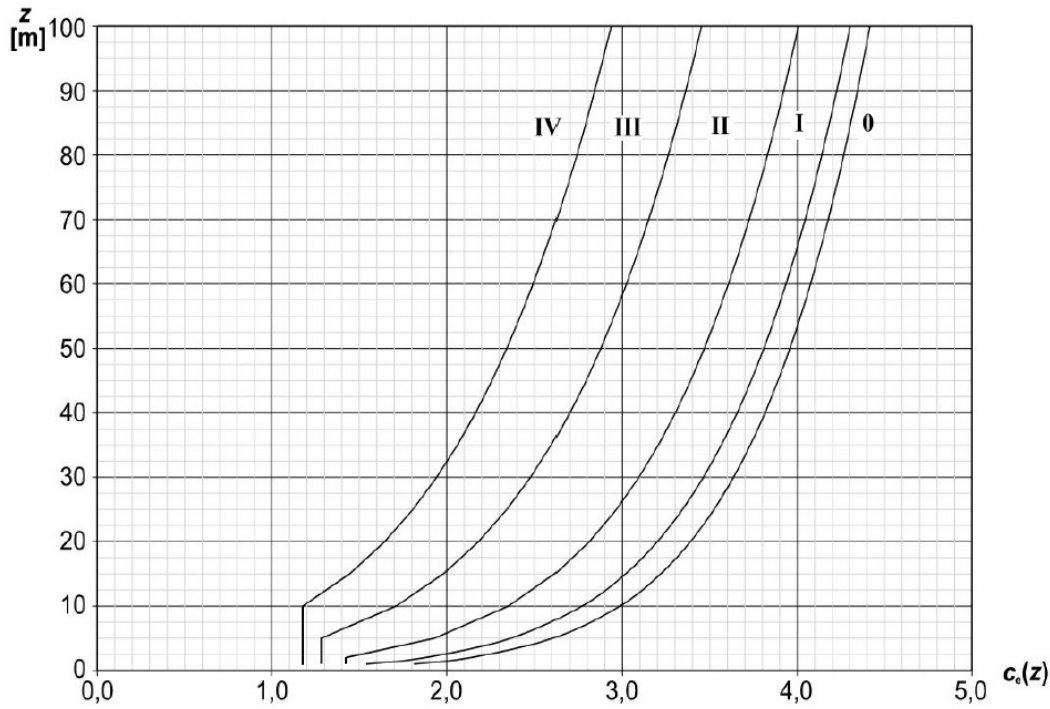


Figure 4.3: Exposure factor c_e for $c_0 = 0$ and turbulence factor $k_I = 0$ [NEN-EN 1991-1-4, 2011]

$$c_e(90) = 4.3 \quad (4.11)$$

$$q_b = \frac{1}{2} \cdot 1.25 \cdot 29.5^2 = 0.544 \text{ kN/m}^2 \quad (4.12)$$

$$q_p(90) = 4.3 \cdot 0.544 = 2.34 \text{ kN/m}^2 \quad (4.13)$$

Pressure and force coefficients

The wind pressure acting on the surfaces w should be obtained from Equation (4.14).

$$w = q_p(z_e) \cdot c_x \quad (4.14)$$

in which:

z_e	Reference height for external pressure
c_x	Pressure or force coefficient

In this research three different pressure or force coefficients are considered:

c_{pe}	External pressure coefficient	Applies to the external surface of a closed building
c_{pi}	Internal pressure coefficient	Applies to the internal surface of a closed building
c_f	Overall force coefficient canopy	Applies to a canopy or surface of an open building

Since the structure in this research can be both open or closed, all three wind forces are considered and the most unfavourable combination is used.

External pressure coefficients

Section 7.2.8 of the [NEN-EN 1991-1-4, 2011] give the external pressure coefficients for vaulted roofs and domes. There are two different types of external pressure coefficients, $c_{pe,10}$ and $c_{pe,1}$. The first one gives the external pressure coefficients for loaded areas of 10 m^2 and the second one gives the external pressure coefficients for loaded areas of 1 m^2 . For vaulted roofs and domes only $c_{pe,10}$ is given. For $c_{pe,1}$ the same value is used.

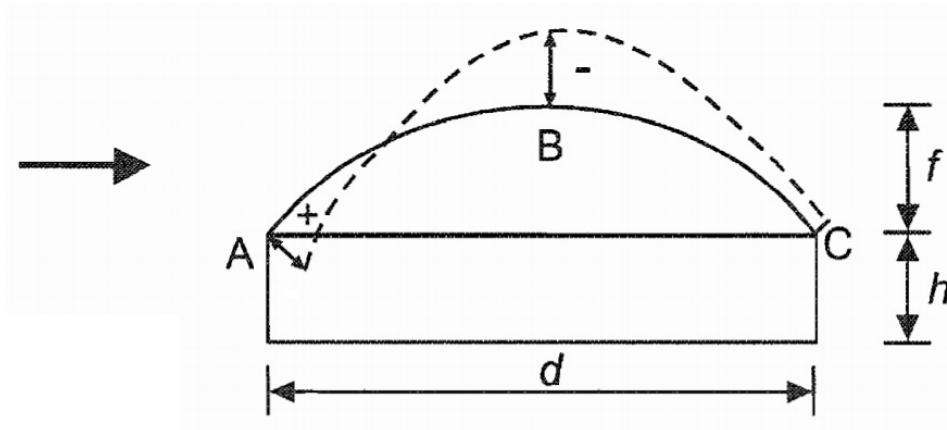


Figure 4.4: Wind pressure domes with circular base [NEN-EN 1991-1-4, 2011]

Figure 4.4 shows that the Eurocode 1-1-4 gives 3 different coefficients for the domed roof. The coefficients for the areas in between can be determined by interpolating between the values in A, B and C. The values of c_{pe} for A, B and C can be determined from Figure 4.5. In this case the f is taken as 40 m, h is taken as 50 m and d is taken as 200 m. This gives $f/d = 0.2$ and $h/d = 0.25$. This will give the following values of c_{pe} for the areas A, B and C:

A	-0.67
B	-0.81
C	-0.25

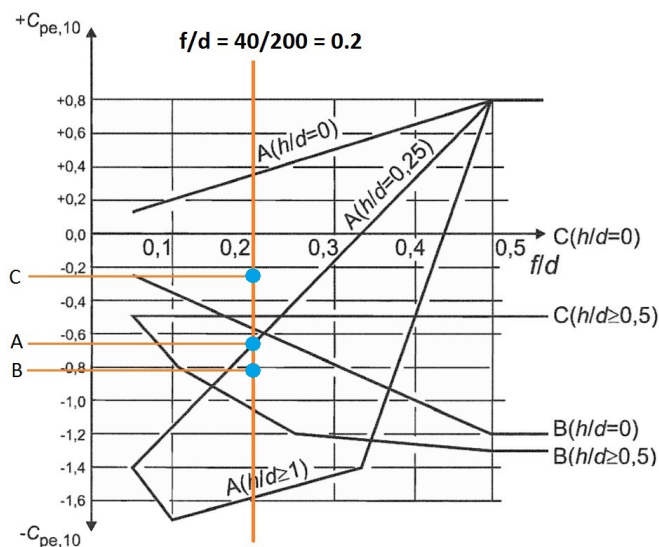


Figure 4.5: External pressure coefficients [NEN-EN 1991-1-4, 2011]

Internal pressure

Besides the external pressure roofs an internal pressure also acts on the surface. The internal pressure coefficient is dependent on the size of the openings in the facade. Since this design must be suitable for all possible functions the worst case is considered for the internal pressure. The maximum value of the internal pressure coefficient is given as:

$$c_{pi} = 0.90 * c_{pe}$$

In this formula c_{pe} is the value for the external pressure coefficient at the location of the openings. Since the openings will only be at the edges, only the values for A and C should be considered. The most unfavourable one is used, which means that c_{pe} is taken as -0.67.

The internal pressure coefficient then becomes:

$$c_{pi} = 0.90 * -0.67 = -0.60$$

The internal pressure acts on the entire roof structure.

Canopy roofs

The Eurocode doesn't describe force coefficients for a dome-shaped canopy. Therefore the values for a duo-pitch roof are used. The shape of the duo-pitch roof can be compared best to the shape of the dome-shaped roof. Research performed in South Africa [Bradley et al., 2016] shows that the wind pressure coefficients for catenary domes, derived from wind tunnel tests, are larger than the coefficients for a duo-pitch roof for any dimensions. For this reason the coefficients for duo-pitch roofs represent a safe representation for the wind actions on a dome-shaped canopy.

The Eurocode gives force coefficients for several parts of a duo-pitch roof. These values reflect the risk of vortices created over edges or corners. Since this design doesn't have these edges or corners, besides the edge of the dome, the overall force coefficients c_f are used for the calculation of the structure.

The Eurocode describes different roof angles, from - 20° to + 30°. For this design only the roof angles above 0° are important, since the negative roof angles describe a situation where the centre is lower than the edges. The roof angle in the dome structure will have a roof angle of 0° in the centre and around +40° at the edges.

The maximum and minimum values for all blockages φ for all positive roof angles are used. The maximum gives the loading in the downward direction and the minimum gives the loading in the upward direction. The maximum and minimum values are:

	c_f
Minimum	-1.3
Maximum	+0.9

Wind actions

It's important that the worst cases of both minimum and maximum load coefficients are used.

For the maximum load coefficient only a value is given in case of a canopy roof. This value is therefore the decisive value.

For the minimum load the determination of the decisive value is a little bit more complex, since all values have to be combined. In case of a closed building, the values of the external and internal pressure have to be combined. The combined load coefficients for this scenario will be:

A	-1.27
B	-1.41
C	-0.85
Weighted average	-1.21

In case of the wind canopy the minimum value is -1.3.

The Eurocode gives for each load situation reference height that has to be used. In case of a closed building the reference height must be taken as the height of the top of the dome, 90 m in this design. In case of a wind canopy the reference height must be taken as the height of the edge of the dome, 50 m in this design. The peak velocity pressure for 50 m is 2.12 and the peak velocity pressure for 90 m is 2.34. Between these values there is a difference of about 10%. If this 10% is included in the comparison, it can be seen that the load coefficients for a closed building are decisive. This conclusion is fortified by the fact that in case of the closed building, the load is more grouped in the middle of the span, which is obviously less favourable for the structural behaviour.

The wind actions on the dome structure therefore become:

A	-1.27 * 2.34	=	-2.97 kN/m ²
B	-1.41 * 2.34	=	-3.30 kN/m ²
C	-0.85 * 2.34	=	-1.99 kN/m ²
Downward	+0.90 * 2.12	=	+1.91 kN/m ²

Snow loads

The snow loads are determined according to the Eurocode 1-1-3 [NEN-EN 1991-1-3, 2011]. Snow loads must be classified as static, variable, fixed actions.

First the characteristic value of snow load on the ground s_k should be determined. According to the Dutch National Annex the characteristic value of snow load on the ground should be taken as 0.7 kN/m^2 for every location in the Netherlands.

Section 5.3.5 of the [NEN-EN 1991-1-3, 2011] describes the snow load on cylindrical roofs. It starts by determining the snow load shape coefficient for this type of roof. The snow load shape coefficient μ_3 is given by the formulas in Table 4.12.

Table 4.12: Snow load shape coefficient [NEN-EN 1991-1-3, 2011]

Roof slope β	μ_3
$\beta > 60^\circ$	0
$\beta \leq 60^\circ$	$0.2 + 10 \cdot h/b$

In which:

- μ_3 Snow load shape coefficient with a maximum of 2.0
- h Height of dome
- b Span of dome

If the roof has a rise of 40 meters and a span of 200 meters, it can be seen that μ_3 equals 2.2. Since the maximum is set as 2.0 by the Eurocode μ_3 is taken as 2.0.

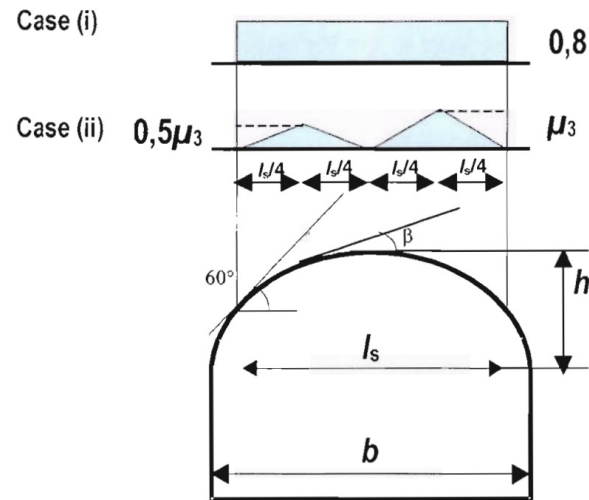


Figure 4.6: Snow load arrangements [NEN-EN 1991-1-3, 2011]

In the Eurocode two types of snow loads are described, the undrifted load arrangement and the drifted load arrangement. These are shown respectively in Figure 4.6 (i) and Figure 4.6 (ii). It can be seen that the loads are only on that part of the roof that has an angle beneath 60° . If the whole roof has an angle beneath 60° l_s is equal to b .

The snow load on the roof is determined with the formula:

$$s = \mu_i \cdot C \cdot C_t \cdot s_k \quad (4.15)$$

in which:

s	Snow load on roof
μ_i	Snow load shape coefficient
C_e	Exposure coefficient (1.0 for the Netherlands)
C_t	Thermal coefficient (1.0 for the Netherlands in most cases)
s_k	characteristic value of snow load on the ground (0.7 kN/m^2 in the Netherlands)

The snow loads on the roof described by Equation (4.16), Equation (4.17) and Equation (4.18), with regards to Figure 4.6:

$$s_i = 0.8 \cdot 1.0 \cdot 1.0 \cdot 0.7 = 0.56 \text{ kN/m}^2 \quad (4.16)$$

$$s_{ii, \text{left}} = 1.0 \cdot 1.0 \cdot 1.0 \cdot 0.7 = 0.70 \text{ kN/m}^2 \quad (4.17)$$

$$s_{ii, \text{right}} = 2.0 \cdot 1.0 \cdot 1.0 \cdot 0.7 = 1.40 \text{ kN/m}^2 \quad (4.18)$$

4.4. Load overview

4.4.1. Load Cases

In this design multiple load cases will be used. A brief overview of all load cases is given in Table 4.13.

Table 4.13: Overview of load cases

Load case	Type	Description
BG2	Permanent	Self-weight steel
BG3	Variable	Imposed loads
BG5	Variable	Upward wind loads
BG6	Variable	Downward wind loads
BG7	Variable	Snow load i
BG8	Variable	Snow load ii
BG10	Permanent	Superimposed loads

4.4.2. Load Combinations

Because multiple load cases can occur simultaneously, load cases are combined to form load combinations. These load combinations follow from the load combinations that are stated in the Eurocode, that are given in Section 4.2.6. The applicable load combinations for this design are given in Table 4.14.

Table 4.14: Load combinations

Load combination	ULS or SLS	Description	Combination with partial factors
LC1	ULS	Self-weight of steel and cladding	$1.5 \cdot (\text{BG2} + \text{BG10})$
LC2	ULS	Imposed loads	$1.3 \cdot (\text{BG2} + \text{BG10}) + 1.65 \cdot \text{BG3}$
LC3	ULS	Downward wind	$1.3 \cdot (\text{BG2} + \text{BG10}) + 1.65 \cdot \text{BG6}$
LC4	ULS	Upward wind	$0.9 \cdot (\text{BG2} + \text{BG10}) + 1.65 \cdot \text{BG5}$
LC5i	ULS	Snow i	$1.3 \cdot (\text{BG2} + \text{BG10}) + 1.65 \cdot \text{BG7}$
LC5ii	ULS	Snow ii	$1.3 \cdot (\text{BG2} + \text{BG10}) + 1.65 \cdot \text{BG8}$
LC6	SLS	Self-weight of steel and cladding	$\text{BG2} + \text{BG10}$
LC7	SLS	Imposed loads	$\text{BG2} + \text{BG10} + \text{BG3}$
LC8	SLS	Downward wind	$\text{BG2} + \text{BG10} + \text{BG6}$
LC9	SLS	Upward wind	$\text{BG2} + \text{BG10} + \text{BG5}$
LC10i	SLS	Snow i	$\text{BG2} + \text{BG10} + \text{BG7}$
LC10ii	SLS	Snow ii	$\text{BG2} + \text{BG10} + \text{BG8}$

Remarks

In the process of determining the actions on the structure some assumptions are done. These assumptions are mainly with regards to the shape of the roof. That is because the Eurocode doesn't describe all actions for a circular dome-shaped roof. The actions that are determined in this chapter are done according to structures that can be compared best to the dome structure, but it is important that one realizes that these loads might not be completely accurate. To come to an accurate description of the actions on the structure, wind-tunnel tests should be performed. These tests can also be done digitally with the use of computational fluid dynamics. These tests are time-consuming and/or costly and therefore will not be within the scope of this research.

During this research question arose about the process of determining the wind actions. To dispel any doubts prof.ir. A.C.W.M Vrouwenvelder is asked for advice. After reading a summary of this chapter, including links to a research in South Africa [Bradley et al., 2016], which is also given in Appendix B, he advised to continue with the loads determined in this chapter. He pointed out that wind tunnel tests or computational fluid dynamics calculations would be necessary if the structure would become a real project. Therefore this is added to the recommendations at the end of this research.

Another aspect that doesn't come fully clear from the Eurocode is the presence of asymmetrical loads. For wind and snow loads some asymmetry is present, as can be seen in the determination of these loads. But besides this asymmetry that is stated in the Eurocode, a lot of other asymmetrical load arrangements can be present, like a chessboard load arrangement, where elements are alternately loaded and not loaded. Numerous of these combinations can be found, and each combination should be examined to see whether it is a decisive combination. Because the examination of all of these combinations will be a time-consuming process, these combinations will be out of the scope of this research. Only the asymmetry that follows directly from the Eurocode is taken into account in this research.

5

Model

In this chapter the modeling process will be described. A software package, which will be used to calculate the design, is chosen. After that the type of analysis is determined. Next, the characteristics of the design are described and the starting points of the model are drafted. Finally, the modeling procedure will be described briefly. An extensive description of the Grasshopper model will be given in Appendix C.

5.1. Software Choice

5.1.1. FEM package

While Karamba3d functions perfectly for a preliminary design, as performed in Chapter 3, other FEM software packages are more suitable for a final design. One of the reasons is that a lot of FEM packages that are currently on the market are being checked by authorities and have received quality certificates by authorities. These certificates give the users a guarantee that the software is performing the calculations in a correct manner. Karamba doesn't have these certificates, nor do they try to obtain these. This has been made clear by statements on the website of Karamba:

"There is no warranty for the program, to the extent permitted by applicable law. Except when otherwise stated in writing the copyright holders and/or other parties provide the program "as is" without warranty of any kind, either expressed or implied, including, but not limited to, the implied warranties of merchantability and fitness for a particular purpose. The entire risk as to the quality and performance of the program is with you. Should the program prove defective, you assume the cost of all necessary servicing, repair or correction." [karamba3d, 2016]

Because of this reason the final calculations will be performed by another FEM package. A lot of FEM packages are on the market, and they all differ mainly in their calculation possibilities and in the design. The basic calculation possibilities like a linear analysis, non-linear analysis and basic dynamics are present in each of the FEM packages.

Some of the packages also have links from Grasshopper, provided by GeometryGym, but these links are often not ideal. The possibilities are limited to what GeometryGym offers. Another way to link Grasshopper with a FEM package is by the use of Excel-sheets. For the usage of Excel-sheets it is important that the FEM package has a table input. Because linking Grasshopper and the FEM software is important for this research a FEM package is chosen that has a clear and extensive table input.

The price of the FEM package is also something to consider. Full versions of FEM packages usually cost a lot of money. The trial versions are often limited to small structures, making it unfit for use in this research. Therefore a FEM package with a free or cheap student version is preferred.

With all these aspects considered the choice has been made to use SCIA Engineer. SCIA is a global software developer that is originally from Belgium. Their software SCIA Engineer is a widely used FEM package that is used for the design of famous structures like the Markthal in Rotterdam. [SCIA, 2017]

5.1.2. Karamba3d

The aim of this research is to get an optimal design. For this optimisation parameters have to be adjusted to see what the most favourable combination of parameters is. To come to this optimal combination, a large number of calculations have to be performed, for each of the different combinations of parameters. Since the parameters are part of Grasshopper and the calculations will be performed in SCIA, data must be transferred from Grasshopper to SCIA often. This transfer of data can be time-consuming if it has to be done numerous times. To prevent this time-consuming process, rough calculations are performed by Karamba within Grasshopper to get close to the optimal solution. With Karamba one can immediately see what structural consequences changes in parameters have without having to copy the data from one software to the other. When the optimal solution is approximated by using Karamba, the design can be transferred to SCIA to check the calculation and finalize the optimisation.

5.2. Type of Analysis

In SCIA different types of analysis can be performed. The two most important types of analysis are the Static linear calculation and the Static non-linear calculation. The main difference between those two types of analysis is that the non-linear analysis gives the results for which equilibrium conditions are satisfied on a deformed structure. That means that in case of large deflections the non-linear analysis will give different results than the linear analysis.

5.2.1. Linear analysis

The linear analysis is the most basic calculation type within SCIA. It simply calculates the internal forces and deflections for the static system. The checks to determine whether the dimensions of the members are sufficient are performed with the internal forces that follow from the linear analysis. This gives the possibility to easily optimise the cross-sectional dimensions.

The linear analysis also checks the system for stability, but this check works in a simplified way. The properties that are important for stability, such as the buckling length of a member, are calculated through simplified ways. This means that only buckling of single members is considered, and not the stability of the system as a whole. This method is reliable for systems that have clear buckling behaviour, a beam on two supports for instance. If the buckling behaviour of a system is more complicated, the linear analysis could lead to results that are either way too conservative, or not reliable, depending on the input of the system. For a grid of beams in two directions for instance, the members will give each other extra resistance against buckling, because a beam that is supported intermediately in y-direction, also gets (little) buckling resistance in z-direction from the system as a whole.

5.2.2. Non-linear analysis

The non-linear analysis calculates the structure by finding the equilibrium of all input. This means that the deflections will influence the results of the calculations.

The stability of the system as a whole is checked within the non-linear analysis, since the analysis will only give results if the equilibrium conditions are met. In this way the buckling behaviour of the system as a whole is considered. The analysis can only give results if the system is stable, because otherwise the stiffness matrix that is determined in the analysis will be singular and the system will give an error.

5.2.3. Choice of analysis

This check for system stability in the non-linear analysis is useful for the calculation of the final system, but for the comparison of the different parameters it can become quite labour-intensive. While the linear analysis always gives a results, even if the system is unstable for the used cross-sections, the non-linear analysis simply gives a warning that the system is unstable for that load combination. This means that in case of linear analysis, the calculations can be performed, and afterwards the cross-sections can be adjusted to meet the structural requirements. In case of non-linear analysis, the cross-sections should be optimised by hand, making this optimisation a time- and labour-intensive process.

Because of this insensitivity of the non-linear process, the optimisation of the parameters is done with linear analysis. When the optimal values of each of the parameters is determined, the final structure will be determined by the use of non-linear analysis.

5.3. Characteristics of Design

As described in Chapter 3 the final design will be the dome structure in compression. The main elements of this system are the arches, which together make up the dome, the ring beam that is loaded in tension, and two types of columns that support the ring beam. This can all be seen in Figure 5.1.

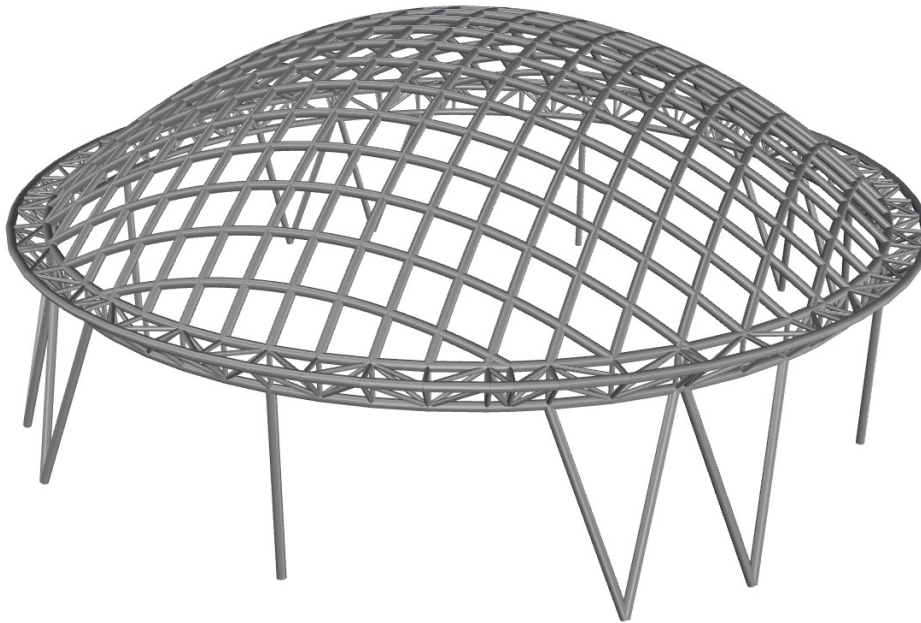


Figure 5.1: 3d-rendering of compression dome

5.3.1. Spanning Structure

The spanning structure consists of a linear grid of beams in two directions which is projected on a dome-shaped surface. Because of this dome-shaped surface each arch has the shape of a catenary. This shape has the advantage that if the arches are loaded with uniformly distributed downward loads, their only internal force will be compression. Since this structure is designed to have normal forces as its main internal force, the spanning members are designed as single members instead of truss members. Truss members behave well under bending moments, but for normal forces, trusses have no advantage over single members.

5.3.2. Ring Beam

A consequence of the dome-shaped roof structure is that large thrust forces will occur at the edge of the dome. These thrust forces are horizontal forces that, in case of a dome structure with uniform downward loads, are directed outwards. To prevent the need for clamped columns and large bending moments in the columns, these forces should be counteracted at the edge of the dome. The easiest way of doing this is by adding a ring structure that can make equilibrium with the thrust forces. The outwards thrust forces will create a tension forces in the ring structure.

The ring structure will also transfer the vertical loads from the roof arches to the columns.

Because this ring beam will have large bending moments in all directions, the ring beam is modeled as a 3-dimensional truss with a triangle as cross-section.

The connection between the spanning beams and the ring truss is somewhat tricky. Because the aim in this research is to get an optimal structure, it makes sense to have both the centre-to-centre distance of the spanning beams and the panel length of the truss uniform. The problem that arises with this configuration is that the centre-to-centre distance is a linear distance, while the panel length is a length that travels around a circle. This means that for some part the spanning beams can end in nodes of the truss, but this can never be the case for the entire structure. This can also be seen in Figure 5.2.

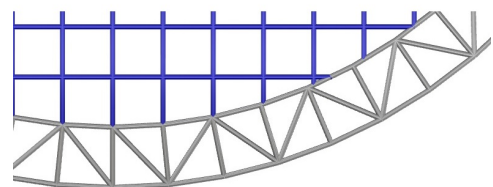


Figure 5.2: Uniform grid and truss

For this problem three different solutions are considered.

- The first solution is to keep the structure as it is. That means that some of the catenary arches will be connected to chords of the truss instead of to nodes of the truss. This can be seen in Figure 5.2. Connections to truss chords are not favourable because of the bending moments that are induced. The main structural advantage of a truss is that each member only has normal forces. If beams are connected to the members in between the nodes bending moments are induced in the truss, which leads to much higher cross-sections. This solution is clearly not the most favourable and will therefore not be used.
- The second solution is to adjust the edge parts of the catenary arches. The end of each arch can be connected to the closest truss nodes to prevent bending moments in the truss. This system can be seen in Figure 5.3. The advantage of this system is that the truss will work as it's supposed to. No bending moments will occur in the truss. The disadvantage is that the catenary arches aren't continuous anymore. Besides that, because of the changing distance between the grid and the ring truss, the edge beams are often asymmetrical. This discontinuity and asymmetry will create extra forces in the edge beams. This can be seen in Figure 5.4, where red colours mean high utilization and blue colours mean low utilization. It can be seen clearly that the edge beams that connect the rectangular grid to the edge truss have the highest utilization.
- The third solution is to adjust the panel length of the truss. That means that the panel length of the truss will not be equal along the truss. This can be seen in Figure 5.5. In the spanning arches there are no discontinuities or asymmetry, so for the arches this is structurally the best solution. For the truss the difference in panel length are structurally not a problem. The difference is that the web beams of the truss will have different forces, because of the different panel lengths.

From this explanation it is clear that the third solution is structurally the best solution. Therefore this solution will be used for the model. To be sure of the assumptions that are done here the second solution is calculated briefly to check the assumptions that are made before. The results can be seen in Figure 5.4.

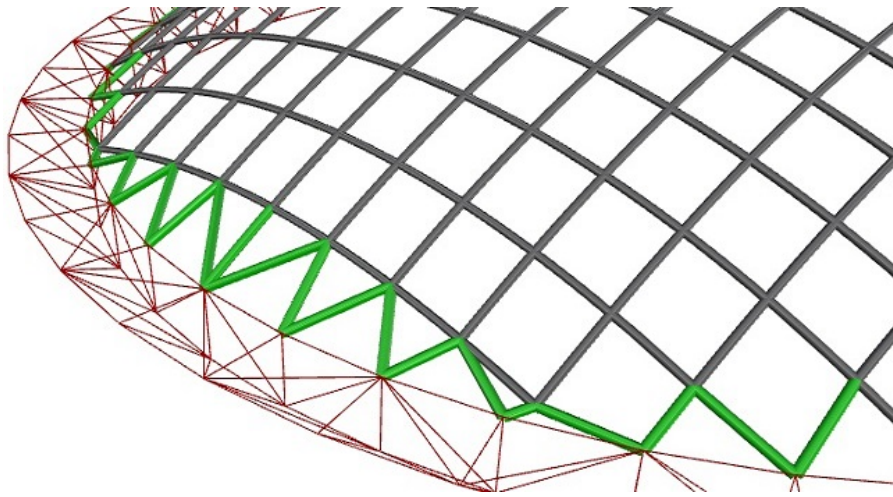


Figure 5.3: Catenary arches adjusted

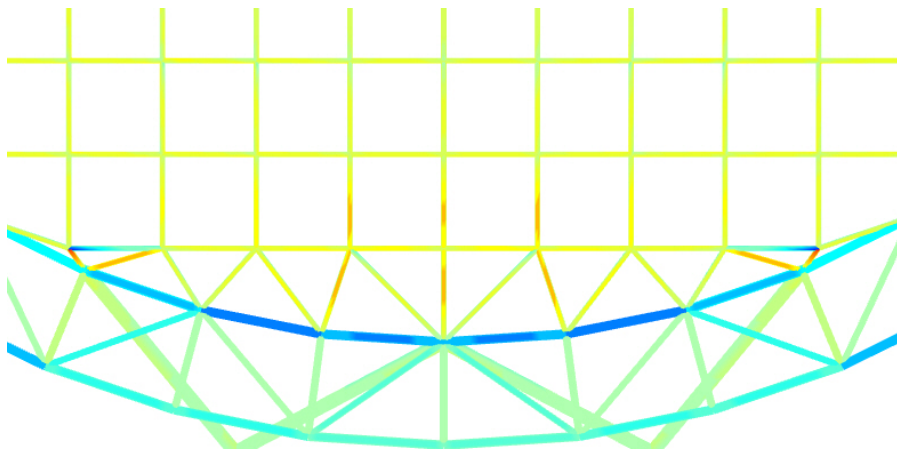


Figure 5.4: Utilization of edge beams

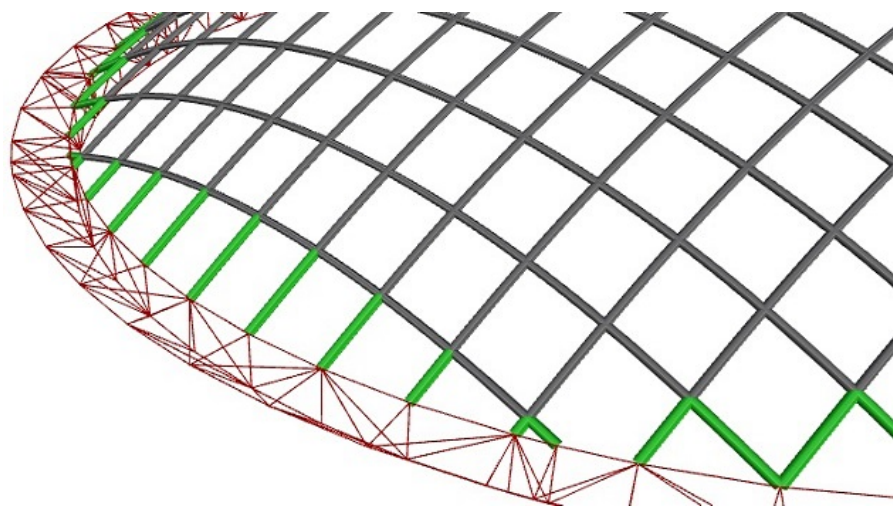


Figure 5.5: Truss panel length adjusted

5.3.3. Columns

The columns will have two main functions. The first one is to transfer the downward loads from the roof structure to the foundation. This can easily be done by compression or tension forces in the columns. The second function of the columns is to stabilize the system in all directions. That means that movement must be prevented in all horizontal directions, but also rotation around all three axis must be fixed.

The easiest way to prevent all degrees of freedom is by adding at least three angled columns, of which their rotation axes don't go through a single point of intersection, to prevent rotation around this pivot point. Because there is a ring beam in this system anyways, it makes sense to add these angled columns in the perimeter of the circle. This automatically makes sure that not all rotation axes go through the same pivot point. This can be seen in Figure 5.6. From this picture it is clear that if another horizontal support in the perimeter of the circle is added, its rotation axis will not go through the pivot point of the other two supports.

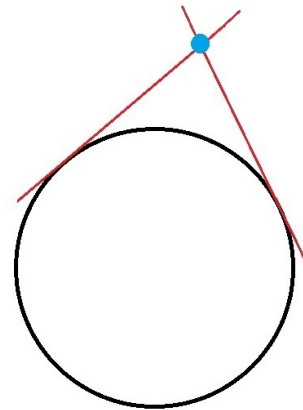


Figure 5.6: Pivot point

The total number of columns is kept low to ensure maximal flexibility for all possible functions. Columns in the perimeter of the structure could be highly obstructive, for instance when large objects have to be placed in the dome.

Since the goal of this research is to find an optimal design with regards to material usage, the structural efficiency from the dome structure should continue in the columns. Because the roof spans in two directions, making it rotationally symmetrical with an order of 4, it makes sense to design the columns in such a way that they are also rotationally symmetrical with an order of 4. Therefore 4 vertical columns and 4 W-shaped columns are used. In Chapter 7 the structural consequences of adding extra columns in the perimeter are researched, although this contradicts with the demand that the flexibility should be as high as possible.

5.3.4. General Assumptions

Goal

The aim of the model is that the model should be able to write Excel sheets with all important information about the model. This information includes the geometry, the supports, the connections and all loads. The information in the Excel sheets should be updated automatically when the Grasshopper model is changed. In this way the SCIA model can be changed accordingly to the changes in the Grasshopper model by simply overwriting the existing information in the SCIA model with the new information from the Excel sheet.

Another function of the Grasshopper model is the usage of the Karamba tool. With this Karamba tool, it can immediately be seen what structural consequences changes in the model will have. With this Karamba tool, it can easily be seen whether the changes in the model will have a positive or a negative effect on the structural behaviour.

Shape and dimensions of roof

- Shape of the roof is a circle with diameter of 200 meters
- Shape of the roof is a catenary surface
- The edge of the roof is modeled as a circular truss beam
- Centre-to-centre distance of linear systems is set to 10 meters
- Members will be modeled as CHS (Circular Hollow Sections)

Rise of catenary surface

- Starting point is a catenary surface with a rise of 40 meters

Columns and supports

- Two types of columns are present in the design: vertical columns and W-shaped columns
- The structure is supported at the bottom of the columns with hinged supports

Loads

- 4 types of loads can be present, which are all given in Section 4.4:
 - Permanent loads:
 - ◊ Each member will have a dead load, resulting from self-weight of the member
 - ◊ The roof arches will also have a superimposed load, following from the weight of the cladding
 - Imposed loads will act on the arches in the negative Z-direction
 - Both upward and downward wind loads will act perpendicular to the catenary surface on the arches
 - Two types of snow loads will act on the arches in the negative Z-direction
- The values of the loads are determined in Chapter 4

Load combinations

- The load combinations are determined in Chapter 4

Material

- For all members steel S355 is used. S355 is used, because it is currently the most common steel type
- If a cross-section type fails on its maximum stress instead of stability, other steel types can be used

Assumed connections

- The connections between two arches are modeled as fixed connections
- All connections in the truss are modeled as hinges
- The connections between an arch and the truss are modeled as hinges

Maximum deflection and utilization

- The maximum vertical deflection in any part of the system is 800 mm, following from Section 4.2.5
- The maximum utilization of each of the elements is 1.0. This utilization consists of three elements, which will be described in Section 6.2.1

5.4. Grasshopper Model

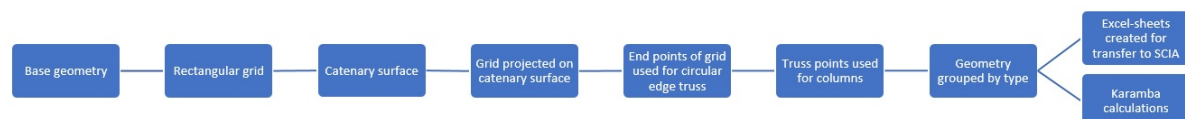


Figure 5.7: Flowchart Grasshopper model

In Figure 5.7 a flowchart of the Grasshopper model can be found. In this flowchart the main components of the model can be seen. The components are described briefly below. A more extensive description of the Grasshopper model can be found in Appendix C.

5.4.1. Geometry

The first part of the model describes the geometry of the dome. As a base for the geometry the model from Chapter 3 is used. In this model the arches are modeled with the use of a catenary surface. Input of the catenary surface is done by adding number sliders for the diameter of the base circle and the rise of the catenary surface. The centre-to-centre distance of the grid of beams is also modeled as a number slider. These values are modeled as number sliders so they can be used in the optimisation of the structure.

By extracting the end points of each arch the base circle of the truss is found. This circle has intermediate points at each of the locations where an arch is connected. This circle is then both scaled to form the outer circle and moved down to form the lower circle. The three circles that are created are the three chords of the ring truss. Between each of the end points of the arches a single truss is modeled. The centre-to-centre distance of the truss is modeled as a number slider, so it can be used in the optimisation of the structure.

The trusses will have a mirrored web configuration compared to their neighbours to form a Warren Truss with verticals. That can be seen in Figure 5.8. A Warren Truss has the advantage that it can deal well with horizontal loads and that the load spreads fairly evenly between the members. A disadvantage of the Warren Truss is that its performance under concentrated loads is not really good. A Warren Truss with verticals has the advantages of a normal Warren Truss, but because of the verticals it deals much better with concentrated loads [Griggs Jr, 2015]. Because the loads from the arches will be transferred to the truss by concentrated loads in the connecting nodes, a Warren Truss with verticals is used in this design.



Figure 5.8: Warren Truss with verticals

The columns are modeled to be spread evenly along the truss. Two different types of columns are used, vertical and W-shaped columns. The W-shaped columns can take the horizontal forces and prevent the structure from rotation around the Z-axis. The structure is rotationally symmetrical around the centre with an order of 4. This means that if the structure is rotated 90° it still looks the same. Because of this symmetry a logical format for the columns would be to follow this symmetry. That means that 4 vertical columns and 4 W-shaped columns are modeled, each at 1/8 of the circle. In this case the columns follow the rotational symmetry from the structure above. The columns are modeled in such a way that they always connect to a node of the truss.

5.4.2. Karamba

As explained before rough calculations will be performed with Karamba to come close to the optimal structure. To really make use of the Karamba software it is important the the Karamba model is as close to the SCIA model as it can get. To achieve this similarity the input for the SCIA model is also used as input for the Karamba model. The geometry is linked directly, while for the loads the most critical load combinations from SCIA are used. For this structure the most critical load combinations are the combinations with downward wind and with upward wind. The structure can be tested in Karamba by looking at the maximum displacements for the SLS load combinations and by looking at the utilization of elements for the ULS load combinations.

5.4.3. Grasshopper Link with SCIA Engineer

As described in Section 5.1 the data has to be transferred from Grasshopper to SCIA. The Grasshopper-plugin GeometryGym does have a feature to do this, but this feature only has limited functions. It is focused solely on the geometry and the section properties, but it disregards loads. In this research the loads should be extracted from the Grasshopper model, since the loads are highly dependent on the geometry. The GeometryGym plugin is therefore not useful in this research. The other way of transferring the data is by the use of Excel-sheets. Grasshopper itself doesn't have a function to write to an Excel-sheet directly, but the plugin Lunchbox does have this feature.

Geometry

The format of the Excel-sheet is based on the table input of SCIA. For the members, for instance, SCIA has a table input which has 10 columns, which can be seen in Figure 5.9. The important columns are briefly described below.

- Name: is used for identification of the members
- Type: type of member, like column, beam, truss beam. This type can be used for checks against the code, because each type has its own requirements
- Start: node at which element starts
- End: node at which element ends
- Cross-section: determines to which cross-section group the element belongs

The two columns that are coloured orange are required. The other columns are optional. For this research the first five columns are used. For the members it is important to have distinctive names to make it more easy to read the results of the calculation. For the cross-sections it is important that each member of the same type has the same cross-section-group, so if the cross-section is changed, it is changed for all members of that type.

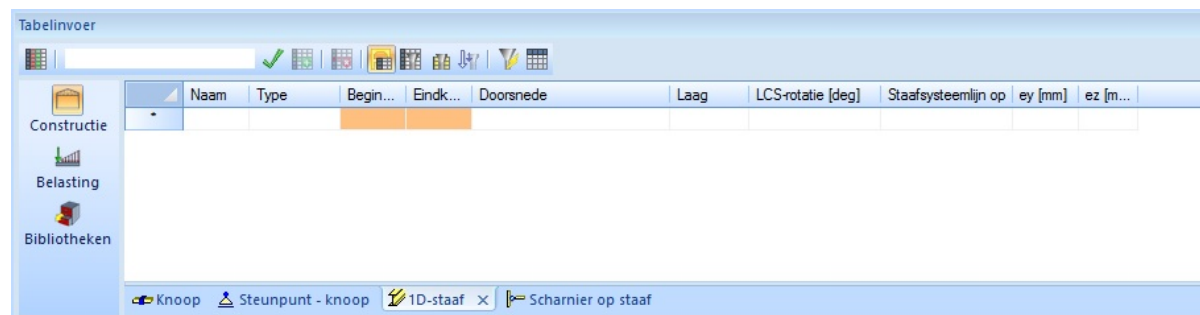


Figure 5.9: SCIA table input members

The next step is to set up Grasshopper in such a way that it writes an Excel-sheet with the correct format. For the members this is done by finding the correct start- and end-nodes for each element. These elements are already grouped so the correct name, type and cross-section can be added to the Excel-sheet. The start- and end-nodes are found by getting the end nodes of each curve, and looking those values up in the total set of nodes. To prevent mistakes as a result of rounding errors, all nodes are rounded to two decimals, equal to 1 cm in the real design. The member names are created by concatenating a small string of text ("X" or "diag") and a series of numbers (1, 2, 3, 4 etc). In this way the members will have names like X105 or diag68, which makes it easy to identify each member.

Loads

The loads are modeled in a similar way. First, the table input for loads in SCIA is considered. The table input for beam loads can be seen in Figure 5.10. The important columns are briefly described below.

- Name: is used for identification of the load
- Type: specifies the type of the load, like a force or self-weight
- Distribution: specifies whether the load is spread uniformly or trapezium-shaped
- Direction: specifies the direction in which the load works. This can be either X, Y or Z
- Angle: specifies the angle of the load compared to the direction. It is specified by the rotation around an axis, like Rx30, which means a rotation of 30° around the X-axis
- Value P1: the value of the load at point 1. In case of uniformly distributed load, value P1 is the value of the load.
- Value P2: this is the value of the load in point 2 in case of trapezium-shaped load

- Member: member on which the load acts
- Load case: load case which the load is part of
- System: specifies which coordinate system is used for the determination of the load. Can be either GCS (global coordinate system) or LCS (local coordinate system)
- Position x1: location of point 1 along the local X-axis of the member
- Position x2: location of point 2 along the local X-axis of the member

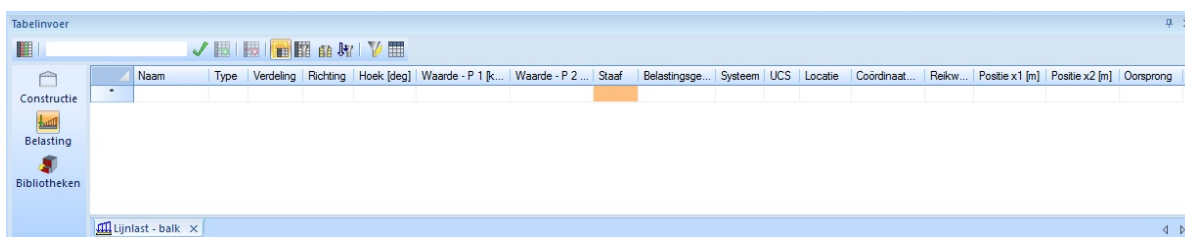


Figure 5.10: SCIA table input beam loads

In Grasshopper a few different types of loads are present: imposed loads, snow loads 1 and 2 and both upward and downward wind load. All loads besides the wind loads act in a global Z-direction. The wind loads, however, act perpendicular to the surface. For the table input this means that the wind loads should use a LCS, while the other loads should use the GCS. The wind loads should then be rotated to act perpendicular to the surface in the direction perpendicular to the members X-axis. A rotation is therefore added, which is calculated by evaluating the catenary surface at the middle of each element.

Since the load is distributed uniformly along the entire surface, each beam will have a triangular load distribution. This can be seen in Figure 5.11. The value of the beam load Pm in the middle of each beam is specified as the centre-to-centre distance of the beams, multiplied by the distributed load per area unit (kN/m^2). The load is 0 at both ends of each beam.

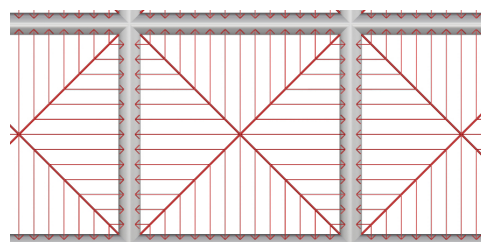


Figure 5.11: Load distribution

Since SCIA doesn't have the option to specify a triangular load distribution like the one that is shown in Figure 5.11, the load is divided in two parts. The first part will have a P1-value of 0, a P2-value of Pm , a P1-location of 0 and a P2-location of 0.5. The second part will have a P1-value of Pm , a P2-value of 0, a P1-location of 0.5 and a P2-location of 1.

The value of some of the loads is dependent on the dimensions of the design. This holds for the imposed loads and the wind loads. Both types of loads can change in value, direction or area for different dimensions of the structure. These loads are therefore modeled to be dependent on the dimensions of the structure.

5.4.4. Cross-sections

For this design 10 different cross-sections are used. The locations of these cross-sections can be seen in Figure 5.12. In Table 5.1 the corresponding SCIA cross-sections and a brief explanation can be found.

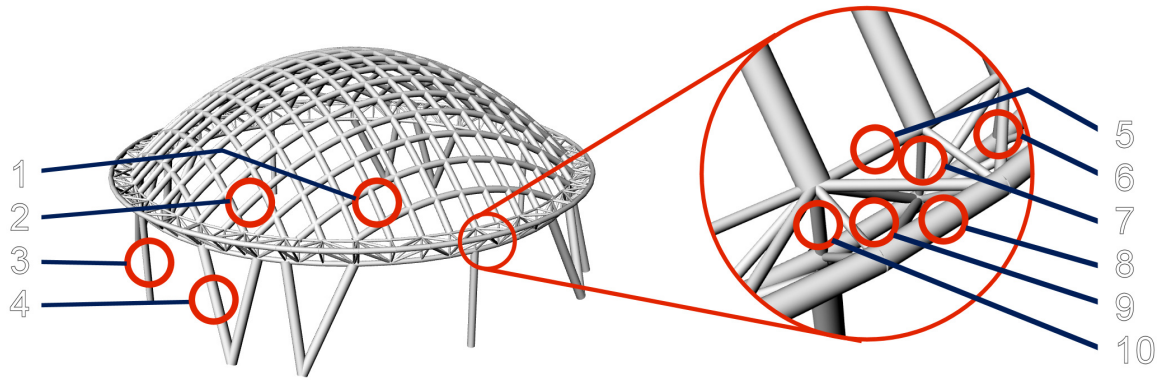


Figure 5.12: Cross-sections

Table 5.1: Overview of cross-sections

#	SCIA cross-section	Description
1	CS7	Arches in X-direction
2	CS8	Arches in Y-direction
3	CS3	Vertical columns
4	CS2	W-shaped columns
5	CS4	Inner, upper truss chords
6	CS6	Truss diagonals
7	CS5	Truss verticals
8	CS10	Outer, upper truss chords
9	CS9	Lower truss chords
10	CS11	Truss verticals above columns

5.4.5. Parameters for optimisation

In Grasshopper some input of the design is modeled as number sliders. Some of these number sliders are used for the input, like the radius of the circle. Others, like the centre-to-centre distance of the arches or the rise of the catenary, are modeled as number sliders so they can be used in the optimisation. These number sliders represent important input for which a logical number is chosen as a starting point. The aim of the optimisation is to find a combination of parameters, for which the material use will be minimal.

The starting values that are shown below are determined with an optimisation in Karamba. These calculations are not exact, since some of the input is approximated to keep the model simple. The optimised parameters therefore are not exact either, but they should give a good approximation of the optimal value. This is obviously checked in SCIA by calculating model with the starting value, as well as some models with parameters around the starting value.

In Figure 5.13 all parameters for optimisation can be seen. The parameters that will be used for the optimisation are:

Symbol	Function	Starting value
s	Rise of the catenary surface	40 m
a	Centre-to-centre distance of the arches	10 m
b	Centre-to-centre distance of the ring truss	8 m
r	Shape of ring truss	
CS	All cross-sections	-

Because of the high number of possible combination of the parameters, the optimisation is done separately. Each parameter is optimised separately, while using this optimal value as input for the optimisation of the next parameter. Since the parameters can influence each other, each optimised value is looped back to the other optimisations to make sure that the combination of the optimal values actually is the optimal solution.

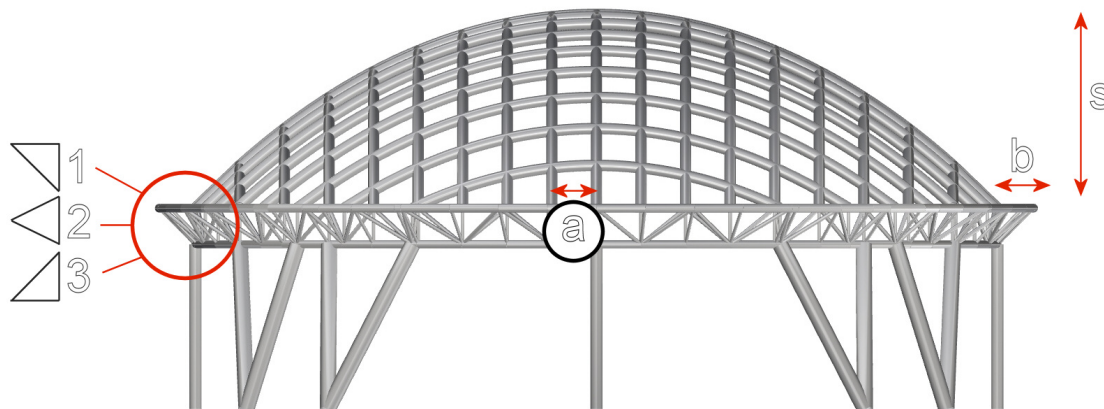


Figure 5.13: Optimisation parameters

The optimisations are done from biggest influence to smallest influence. In this way the chance that looping back leads to different solutions is minimised. First, the centre-to-centre distance of the arches is optimised. Consequently, the centre-to-centre distance of the ring truss is optimised. After that, the rise of the catenary surface is optimised.

Also, the shape of the ring truss is checked. Three different shapes are examined. All three shapes have an upper and a lower inner chord. The location of the outer chord is different for each shape. As a starting point the outer chord is at the same height as the upper inner chord. For the second and the third shape, the location of the outer chord is at the intermediate height of both upper and lower inner chords, and at the height of the lower inner chord.

5.4.6. Optimisation process

General

The optimisation is performed in multiple steps. In the first step the SCIA model is created for three different values of the parameters that is being optimised. The results are checked to see if some logic can be seen in the results of these three calculations. With this logic the next optimisation step can be determined. For instance, if value 1 is much higher than value 2 and 3, the optimal results is most likely close to values 2 and 3. The next optimisation step is to create new SCIA models for some values close to the optimal solution of the three calculations that are performed in the previous step. This last step is repeated until a satisfying optimal solution has been found.

After the option with the lowest self-weight has been found, the total self-weight is divided into individual self-weights for each of the cross-sections. In this way it is easy to see which cross-sections become smaller, and which ones becomes larger when some parameter is changed. With these individual self-weights the results of the calculations can be explained and verified. If the changes in self-weight of some of the cross-sections can not be explained, it is important to check whether the model is created correctly, to prevent errors slipping into the model, which can lead to false conclusions.

If all results can be explained by structural logic, the next parameter can be optimised. As explained before, each optimised parameter is looped back into the other parameters to check whether the results are still correct with other combinations of parameters.

SCIA

The optimisation process in SCIA makes use of the auto-design function of SCIA. Auto-design is a function that can optimise structures by optimising different parameters for a structure. In this case the parameter that will be optimised is the cross-sectional area of the steel. This means that auto-design finds the combination

of cross-sections that lead to the lowest total self-weight, while still keeping its structural safety and stability. After each Auto-design step the model is calculated again to determine the structural behaviour of the model with the new cross-sections. Auto-design is a process that should be repeated several times to come to reliable results. The input for auto-design are the combinations of cross-sections, each with its critical load combination. Each cross-section will be optimised with regards to its critical load combination.

To determine which load combination(s) should be used, a test calculation is performed. From this calculation it follows clearly that the critical aspect of this model are the stresses in the members, since no node comes even close to the maximum vertical displacement of 800 mm in any of the SLS load combinations. Therefore the internal stresses of each of the members is determined for each of the load combinations. With these results it can easily be seen that load combinations 3 and 4 are the critical load combinations. Each of these load combinations is critical for different cross-sections.

These results are used as input for the auto-design. When the input is set up, the optimisation routine can be started. The optimisation routine for a certain number of steps. The number of steps is determined by looking at the changes in the results. If the results remain roughly the same during some auto-design step, it can be assumed that more optimisation steps are not needed. This is checked by performing some extra auto-design steps. When a sufficient result is found, the total self-weight and cross-sectional dimensions are extracted from SCIA and used in the comparison of each of the models.

Columns

Since the number of columns is kept at a minimal to prevent obstructive elements around the dome, each of the columns will have large internal forces. Especially the W-shaped columns will have very large internal forces. In combination with their large buckling lengths, the cross-sectional dimensions of the columns will be enormous, especially if non-composite columns are used. Because the auto-design process only works with cross-sections from lists, all columns are kept out of this auto-design optimisation process. Each of the calculations will use the same dimensions for the columns.

Another aspect that arises with the design of the columns is the aspect of structural safety. Since the number of columns is kept low, the removal of one column could lead to structural failure. Removal of one of the columns is important, because they could be susceptible to, for instance, impact loads. Impact loads are loads that follow from impact of cars or trucks. Obviously, such an impact shouldn't lead to failure of the structure. The columns should therefore be resistance against these types of loads. To create a sufficient resistance, the columns should have a higher resistance than the forces that follow from the SCIA calculations. This higher resistance leads to even higher dimensions than the dimensions calculated in SCIA. If these dimensions turn out to be too large, a new design for the columns will be made, based on the internal forces.

6

Results

In this chapter the results of the optimisation are discussed. As always, with computational calculations, it is important that the results can be explained in a logical way to prevent wrong results. With the enormous amount of data that is used for calculations of this type, a mistake is easily made. Even a small error, like a wrongfully placed hinge, can have big impact on the results of the calculation. For this reason, in each step of the optimisation, the results are explained, with rough percentages.

The extended results can be found in Appendix D. In this appendix the total weights of each of the cross-sections can be found for each of the calculations. In these tables one can see which cross-sections have increased or decreased in size for any optimisation step, as well as its portion in the total weight of the system.

In Section 6.1 the results of each optimisation are given as the total reaction for load case 1 (self-weight of steel) in kN. This total reaction shows the total weight of the structural steel that is used for that design.

6.1. Optimisation

6.1.1. Centre-to-centre distance of arches

The first step in the optimisation of the system is the optimisation of the centre-to-centre (c/c) distance of the arches. The starting point of this c/c is 10 meters. This means that the diameter of 200 meters is divided into 20 spans. This means that the system will have 19 arches in both directions. Since the system should be optimal, it is logical to keep the division of the diameter in rounded numbers. The diameter should be divided in, for instance, 18, 19 or 20 spans, giving a c/c distance of 11.1, 10.5 or 10 meters respectively. From calculations it becomes clear that an odd number of spans has a negative effect on the structural behaviour of the system. This can be explained by the fact that in case of an odd number of spans, there is no arch exactly in the middle, where a vertical column is designed. Because there is no column at the end of the middle arches, which have the biggest span, the arches stiffness of the ring beam is less compared to the situation where there is a column at the end of the arch. Because of this decreased stiffness, the arches need bigger cross-sections to maintain the stability of the roof system. Because of this reason, only even number of spans are considered.

As stated before the starting point is a c/c distance of 10 meters, which correspond to a division into 20 spans. All even numbers of spans between 16 and 24 are considered to get a clear view of the structural behaviour in comparison to the different c/c distances.

The optimal value of the c/c distance of the arches is 11.11 meters, corresponding to a number of spans of 18. The results of the optimisation process can be seen in Figure 6.1

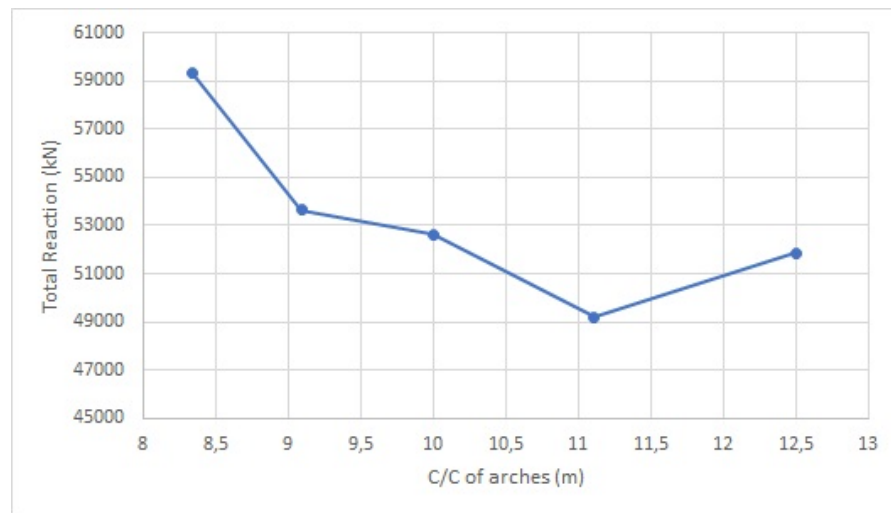


Figure 6.1: Optimisation of the c/c distance of the arches

Validation of results

From the results it can be seen that the difference between the result of the calculations for 18 and 24 spans is about 10.000 kN in total, which corresponds to a difference of about 21%. To explain where this difference is coming from, the individual cross-sections have to be compared.

It becomes clear that the biggest difference comes from the weight of the arches. Even though the cross-section of the arches can be smaller if the c/c distance is smaller, the total length of the arches increases. From the results it becomes clear that this optimisation was mainly about finding the optimal ratio between total length of the arches and the cross-sectional dimensions of the arches. The total length of the arches for 24 is about 34% larger than the total length of the arches for 18. This difference is larger than the difference of the total weight. This can be explained by the fact that the dimensions of the arches can be decreased.

The same explanation holds for the other calculations that are performed. The more arches there are, the more length they have in total, but the smaller the cross-sectional dimensions can get. The optimum of this ratio corresponds to the optimal c/c distance of 11.11 meters.

Varying c/c distance

Besides the option to have to c/c distance uniform, there is also the possibility to have the c/c distance vary. This can be seen in Figure 6.2. This varying c/c distance could be a favourable option because the middle arches have a much longer span then the arches at the edge. By making the c/c distance at the edges larger, the internal forces will be spread more evenly among the arches. With this varying c/c distance, something important arises. Because the c/c distance is larger at the edges, the buckling length of the arches in the other direction becomes larger as well. Since the arches will be in compression in case of downward loading, this increased buckling length can influence the required cross-sectional dimensions of the arches significantly.

To research the option of varying c/c distance, a few calculations are performed with models with varying c/c distance. Multiple options, with a different number of arches, 15, 17 and 19, are calculated and the results are compared to the results of the calculations for the uniform c/c distance. From these calculations it becomes clear that varying the c/c distance is not favourable for the total self-weight of the structure. This can be explained by the fact that the buckling length of the arches is increased. The required cross-sectional dimensions of the arches subsequently increase significantly, making the option of varying c/c distances not favourable.

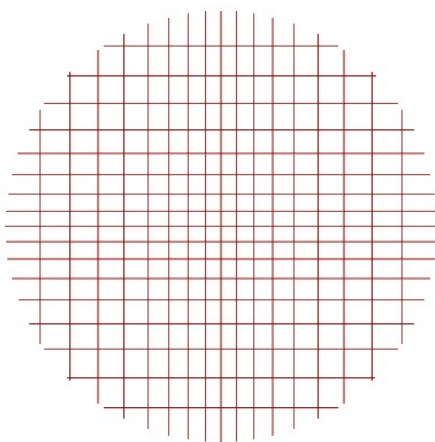


Figure 6.2: Varying c/c distance

Conclusion

The optimal c/c distance of the arches is a uniform c/c distance of 11.11 meters, corresponding to 18 spans, or 17 arches in each direction.

6.1.2. Internal centre-to-centre distance of ring truss

The second step in the optimisation is the optimisation of the c/c distance of the ring truss. This c/c distance is the distance between the upper, inner chord and both other chords. The starting point of this parameter is 8 meters.

The ring truss has two main functions. The first one is that it have to make equilibrium with the thrust forces coming from the dome. These thrust forces are horizontal forces that are pointed outwards in case of downward loading, putting the ring beam in tension. The second function of the ring beam is that it has to transfer the vertical (gravity) loads to the columns. Since the distance between two columns can be as much as 47 meters, the ring beam must be capable of spanning this distance. Both functions will have great influence on the structural behaviour of the ring beam, and will therefore both be used in the validation of the calculation results.

The results can be seen in Figure 6.3.

Validation of results

From the results it follows clearly that the optimal c/c distance of the ring truss is 9 meters. Two aspects play a major role in the structural behaviour of this truss beam. The first aspect follows clearly from the way trusses

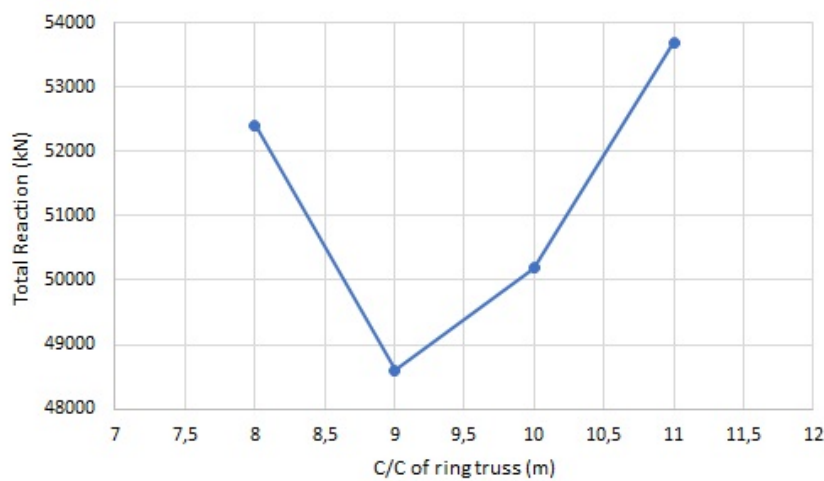


Figure 6.3: Optimisation of the centre-to-centre distance of the ring truss

work. The larger the c/c distance, the larger the leverage arm. If the leverage arm is increased, the normal forces in the chords is decreased. The second aspect is partially specific for this design, partially general for all trusses. This aspect implies that when the c/c distance is increased, the length of some of the members is increased. In general, this is true for the verticals and diagonals of every truss. But in this case particularly it is also true for the outer chords. If the c/c distance is increased, the radius of the outer circle is increased as well. Because the truss is still divided in an equal amount of segments, the length of each segment will increase. For members in tension, the increased length doesn't make a difference, but for members in compression, the buckling length increased quadratic with the increase in member length.

Both aspects can be seen easily in the results of the calculations. When the c/c distance of the truss gets too small, the chords must get bigger too deal with the increasing normal forces. When the c/c distance gets too big, the outer and the bottom chord have to have great cross-sectional dimensions to deal with the problem of buckling. This last aspect doesn't arise clearly for the upper, inner chord, because, in contrast to the other two chords, this chord is never the only compression member in the truss. The other two chords will be the only compression chord for either an upward or an inward bending moment in the truss.

Also, increasing the c/c distance of the truss increases the lengths of the verticals and diagonals. Because each vertical or diagonal can be both in tension and in compression, the buckling length of all these chords is an important aspect of the design. It can be seen clearly that increasing the c/c distance of the truss increases the required cross-sectional dimensions of the verticals and diagonals.

All aspects combined, there are both advantages as disadvantages in increasing the c/c distance. From the results of the SCIA calculations, the optimal value can easily be found.

Conclusion

The optimal c/c distance of the ring truss is 9 meters.

6.1.3. Rise of the catenary surface

The third step in the optimisation of the system is the optimisation of the rise of the catenary surface. As stated before the starting value for the rise of this catenary surface is 40 meters. This starting value is not really reliable, because the wind loads also depend on the rise and shape of the catenary surface. In the Grasshopper model, the loads are independent of the rise of the catenary surface. This optimisation is therefore carried out in small steps. Another important note that has to be made is that the values for the rise in this optimisation aren't the actual values. These values are the input numbers in Grasshopper, but because of the way Grasshopper deals with catenary curves, the actual value is a little bit different. For practical reasons, the Grasshopper input values are shown in this section, but in Section 6.1.3 the real value of the rise is given as well.

First, the structure with a rise of 40 meters is calculated with SCIA and the cross-sectional dimensions are optimised for this system. The total weight of the steel is then extracted from SCIA by looking at the sum of the reaction forces for the dead load only. Subsequently, the same is done for the structure with a rise of 44 and 48 meters. By looking at the weight of the steel for each of these systems, it can be seen that, for this domain of the rise, the weight of the system is lower for both 44 and 48 meters, as it is for 40 meters. Therefore, an intermediate value of 46 meters is used for the calculations. Because the weight of the structure for 46 meters is lower than the weight of the structure for both 44 and 48 meters, the structure is calculated for values of 45 and 47 meters as well.

The same steps are then repeated with different values of the rise, until a clear minimal weight is found. The results of this optimisation can be found in Figure 6.4.

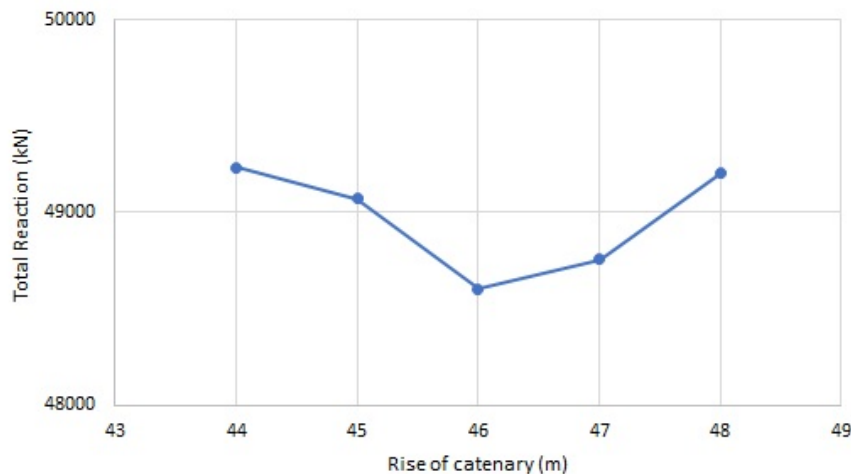


Figure 6.4: Optimisation of Rise of catenary surface

Validation of results

For this optimisation two main aspects are important. The first aspect is the structural behaviour of the arches. If the height of the dome is increased, the member length of the arches increases. But if the height of the dome is decreased, bending moments occur in the arches, which is unfavourable for the required cross-sectional dimensions. So a higher dome will increase the total member length of the arches, but will decrease the cross-sectional dimensions of the arches. The second aspect that is important in this optimisation are the thrust forces that come out of the arches into the ring beam. The thrust force is a normal force in the arch, and therefore works in the same direction as the edge of the dome. This means that by increasing the dome height, the direction of the thrust force becomes more vertically. Subsequently, the horizontal thrust force decreases, while the vertical thrust force increases. This influences the structural behaviour of the ring truss.

Both aspects are visible in the results of the optimisation. The calculations show that the cross-sectional dimensions of the arches decrease for an increased dome height, but the length increases. The second aspect is most clearly visible in the outer truss chords. Its cross-sectional dimensions increase for a lower dome height, because of the increased horizontal thrust force, that has to be taken partly by the outer truss chords.

Again, in this optimisation, the optimal ratio of these aspects has to be found.

Conclusion

The optimal rise of the dome is 46 meters. As explained this 46 meters is the input value for the Grasshopper model. The actual height of the dome in this case is 48.2 meters.

6.1.4. Shape of ring truss

Besides the c/c distance, the shape of the ring truss also influences the structural behaviour. To deal sufficiently with bending moments in both horizontal and vertical directions, a 3d-truss is used. The two inner chords are placed above each other to transfer the vertical loads to the columns, without inducing extra horizontal forces in the truss. For the horizontal bending moments, induced by the horizontal thrust forces from the dome, an extra chord is needed. This chord is placed on the outer side of the other two chords. The vertical location of this chord is determined by performing calculations for three different locations of the outer chord: at the top, at the middle, or at the bottom of the truss. Each of these options is calculated and optimised to compare the required cross-sectional dimensions. The results of this optimisation can be found in Figure 6.6. In this figure, option 1 is the truss with the outer chord at the top location, option 2 is the one with the outer chord in the middle location, and option 3 is the one with the outer chord at the bottom location. In Figure 6.5 the three different truss shapes can be seen.

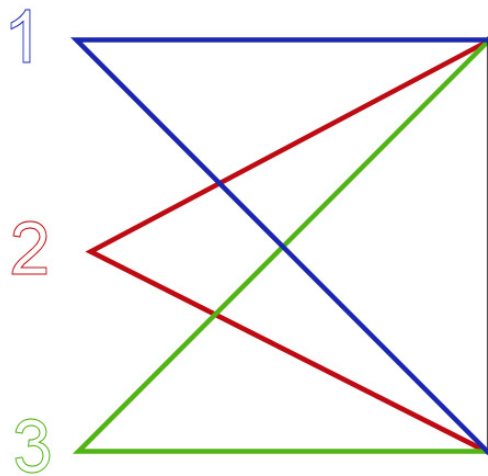


Figure 6.5: Different shapes of ring truss

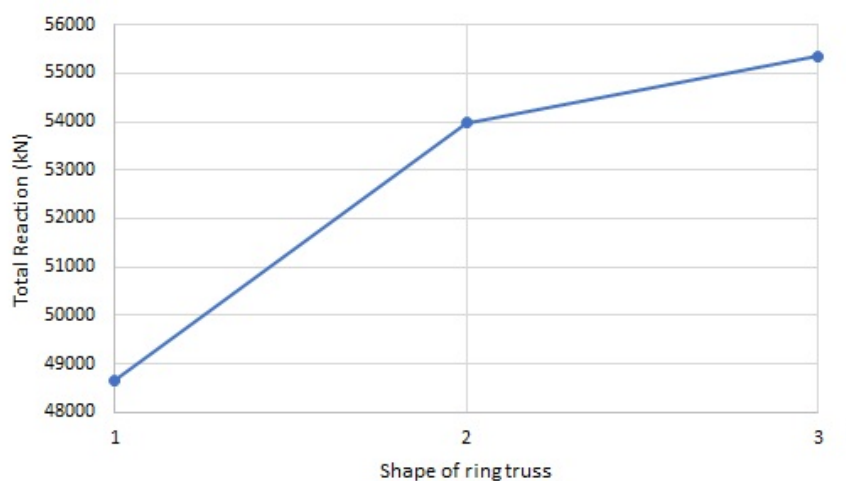


Figure 6.6: Shape of ring truss

Validation of results

From the optimisation it follows clearly the option 1 is structurally the favourable option. This can be explained by the fact that both the vertical load and the horizontal loads are induced to the ring truss through the upper inner chord. Because the vertical loads are transferred to the columns by the two inner chords, the upper chord and the outer chord must take up the horizontal loads. Therefore it is favourable to have the outer chord at the same height as the upper inner chord. In this way the horizontal loads are taken up by the two chords that, together, form a horizontal truss.

Conclusion

The most favourable shape of the ring truss is the truss where the outer chord is at the same height as the inner chord. This can be seen in Figure 6.7

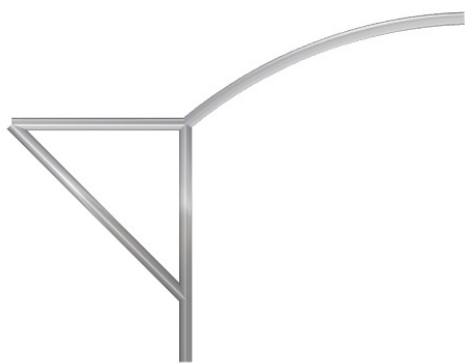


Figure 6.7: Cross-section of optimal ring truss

6.2. Non-linear Calculations

Now that the final values of each of the optimisation parameters is defined, the final calculations can be done. As explained before the final calculations will be done with non-linear analysis.

As can be seen in the extended results in Appendix D, the ratio between diameter and wall thickness is not always the most logical one. That is because SCIA has looked for the cross-section with the smallest self-weight, while still fulfilling its structural job. Therefore the optimised cross-sections will be adjusted for better diameter-thickness ratios. For this ratio it is important that the wall thickness is not too small, to prevent local failure. For the connections it is also important that the wall thicknesses of two connected elements have a good ratio to prevent extensive welding or failure of the connections, but this latter will be discussed later.

6.2.1. Requirements

The non-linear analysis is performed multiple times to find the minimal required cross-sectional dimensions of each of the cross-sections, while still complying to the structural requirements that are set. These structural requirements are:

- The maximum displacement in the model for all SLS load combinations is lower than the maximum displacement that is set in Chapter 4.
- The utilization of each of the elements for all SLS and ULS load combinations is below 1.0. SCIA calculates three types of utilization checks:
 - Stability check: this check calculates the ratio between the destabilizing forces and the stability resistance. A value lower than 1.0 means that that individual member is stable against (lateral) buckling

- Section check: this check calculates the ratio between the internal forces and the resistance of the member. A value lower than 1.0 means that that individual member is sufficiently resistant against the internal forces that occur
- Unity check: this check determines the largest values of the stability and unity checks. A unity check lower than 1.0 means that the member is sufficiently strong and stiff
- The structural stability for all SLS and ULS load combinations is assured; no buckling will occur. The local buckling of the members is already checked in the stability checks of the individual members. The stability of the system as a whole will be checked by calculating the critical load factors. If the critical load factors are equal to or larger than 1.0 for each load combination, the system is stable.

6.2.2. Wall thickness

For the ratio between diameter and wall thickness the book *Hollow Section in Structural Applications* [Wardenier et al., 2002] is used. In this book, formulas are given to calculate the strength of joints between hollow sections. These formulas are only valid in a certain range of the diameter-thickness-ratio. For CHS members this range of validity is given in Equation (6.1). This formula holds for all CHS members and this ratio will be used as a boundary condition in the design of the cross-sections.

$$d_i / t_i \leq 50 \quad (6.1)$$

6.2.3. Results

The results of the non-linear calculations are described briefly in this paragraph. In Appendix E an extended Engineering Report can be found. In this report both the input and the output of the SCIA calculations are shown.

Cross-sectional dimensions

The minimal cross-sectional dimensions are given in Table 6.1. With these cross-sectional dimensions each of the members has a unity check below 1.0. The dimensions for the columns are the dimensions that are used for the SCIA calculations. The actual design of the columns will be given in Section 6.4.

Table 6.1: Final cross-sections

Cross-section	Description	Diameter (mm)	Wall thickness (mm)
CS2	W-shaped columns	1010	56
CS3	Vertical columns	880	19
CS4	Inner, upper truss chords	600	17
CS5	Truss verticals	360	8
CS6	Truss diagonals	630	15
CS7	Arches in X-direction	800	16
CS8	Arches in Y-direction	800	16
CS9	Lower truss chords	910	20
CS10	Outer, upper truss chords	800	19
CS11	Truss verticals above columns	830	20

Displacements

The maximum displacements in this model are determined with SCIA. Only SLS load combinations are taken into account for the consideration of the maximum displacement.

The maximum vertical displacement in the model is found for Load Combination 7. The maximum displacement is 534,1 mm, while the maximum is 800 mm. The structure therefore complies to the requirement set by the Eurocode.

Stability

A non-linear stability calculation is performed in SCIA. In this calculation critical load coefficients are determined. The structure is stable if the critical load coefficient is equal to or larger than 1.0 for each Load Combination. The smallest critical load coefficient in this design is 1.0. The structure is therefore sufficiently resistant against buckling.

Columns

From the overview of the cross-sections it can be seen that the cross-sectional dimensions of the vertical columns are not extremely large. Members with these dimensions are still standardized members. The cross-section of the W-shaped columns, however, is fairly large. The wall thickness that is calculated by SCIA particularly is large. This large wall thickness does not only create problems in the fabrication process, but the connections that will be made between these angled columns and the other members of the structure will be hard to make. On-site welding is basically impossible with members with this wall thickness.

As explained in Section 5.4.6 the columns also need some extra resistance to ensure structural safety in case of accidental loads, such as impact loads. This extra resistance increases the dimensions of the angled columns even further.

Because of this large wall thickness and the problems that arise with this the cross-section of the W-shaped columns that is calculated with SCIA will not be used. Instead of this cross-section a new column design will be made for these columns in Section 6.4, based on its internal forces.

6.3. Joint efficiencies

The efficiency of a joint is a somewhat complicated calculation, because it is dependent on the type of joint, the material strength (which in this case is taken all equal), the ratio between diameter and wall thickness, and the angle in which the two members connect. The general, simplified rule that is given in chapter 8 of [Wardenier et al., 2002] is that a joint is 100% efficient if it complies with Equation (6.2). This rule is not complete, since it neglects other aspects, such as the chord stress effect Q_f . For simply supported members, however, the influence of the other factors is small. Equation (6.2) is therefore sufficient for calculations in this stage of the design. This formula also only holds for a joint between two members of the same steel type. If different steel types are used, a correction factor is introduced. Since only S355 steel is used in this design, this correction factor is not needed for the calculations of the joint efficiencies.

$$\frac{t_0}{t_1} \geq \frac{\sin \theta}{C_k} \quad (6.2)$$

in which:

t_0	Wall thickness of chord
t_1	Wall thickness of brace
θ	Angle in which two members connect
C_k	Joint efficiency, following from figures 8.17, 8.18 and 8.19 from [Wardenier et al., 2002]

Equation (6.2) can be rewritten to a formula that describes the joint efficiency, where 1.0 means a 100% efficiency:

$$Efficiency = \frac{t_0}{t_1} \cdot \frac{C_k}{\sin \theta} \quad (6.3)$$

In this design the following cross-sections are present:

These cross-sections connect with each other with different connections, T, Y, K and X-joints. In Table 6.3 an overview of the joints present in this design are shown. The joints between the angled columns and other members are neglected in this phase, because these joints will be special joints, because of the large internal forces that will occur in these joints. Also, the final cross-section of the W-shaped columns will be different than the one that is used in the SCIA calculations.

Table 6.2: Cross-sections

Name	Description
CS2	W-shaped columns
CS3	Vertical columns
CS4	Inner upper truss chords
CS5	Truss verticals
CS6	Truss diagonals
CS7	Arches in x-direction
CS8	Arches in y-direction
CS9	Bottom truss chords
CS10	Outer upper truss chords
CS11	Truss verticals above columns

Table 6.3: Joints in design

Joint no.	Chord	Brace	Type of joint
1	CS4, CS9, CS10	CS5, CS11	T
2	CS4, CS9, CS10	CS6	K
3	CS4	CS7, CS8	T, Y
4	CS7	CS8	X
5	CS9	CS3	T

The efficiency of each of the joints is checked with Equation (6.2). If the joint efficiency is too low, wall thickness should be adjusted to ensure structural safety. The efficiency of each of the joints is calculated with Equation (6.3). An overview of the efficiencies is given in Table 6.4.

Table 6.4: Joint efficiency

Joint no.	Chord	Brace	Type of joint	Maximum angle	C_k	Joint efficiency
1.1	CS4	CS5	T	90	0.28	0.60
1.2	CS9	CS5	T	90	0.2	0.50
1.3	CS10	CS5	T	90	0.2	0.48
1.4	CS4	CS11	T	90	0.35	0.30
1.5	CS9	CS11	T	90	0.29	0.29
1.6	CS10	CS11	T	90	0.29	0.28
2.1	CS4	CS6	K	70	0.49	0.59
2.2	CS9	CS6	K	70	0.36	0.51
2.3	CS10	CS6	K	70	0.37	0.50
3.1	CS4	CS7	T, Y	90	0.29	0.31
3.2	CS4	CS8	T, Y	90	0.29	0.31
4	CS7	CS8	X	90	0.2	0.2
5	CS9	CS3	T	90	0.29	0.31

Table 6.4 shows clearly that the joint efficiencies are often far below 1.0. This means that in the design of the joints a much larger joint strength is required than the internal forces that occur in the joints. These joint efficiencies are therefore important to consider in the design of the joints.

Another aspect that has to be taken into account for the design of the joints is that some joints are modeled as hinges. In real-life, a connection between two members is never a full hinge. But to make sure the joint will work as a hinge without failing, the hinged joints must have sufficient rotational capacity. The required rotation can be extracted from SCIA, by looking at the maximum rotation for each joint. The hinged joints must be able to rotate this maximum rotation without breaking.

6.4. Columns

6.4.1. Introduction

For the design of the W-shaped columns the internal forces of the columns are used. These internal forces can easily be extracted from the SCIA model. The internal forces can be found in Table 6.5. It can be seen that the shear forces are small compared to the normal forces and the bending moments. To simplify the calculations, the shear forces are neglected in the design of the columns.

Besides the internal forces, another aspect is very important in the design of the columns, the buckling length. The angled columns have a buckling length of about 45 meters. This large buckling length in combination with the large internal forces that can occur, make the required cross-sectional dimensions of the columns enormous. These large dimensions are not favourable because of multiple aspects. Problems arise in the manufacturing process, during transport and during construction.

To prevent these large dimensions, multiple options arise.

The first option is to decrease the buckling length by adding intermediate lateral supports. There should be lateral supports in two directions to prevent buckling in both directions. An intermediate connection between the angled columns can be made, but this supports the columns only in one direction. In the other direction there are no structural elements that can support the columns laterally.

The second option is to decrease the buckling length by fixing one or both ends of the columns. By clamping the column, the buckling length can be decreased by 30% for one fixed end, or 50% for two fixed ends.

The third option is to have a composite column. A composite column can be a column with multiple materials, like a concrete-filled steel hollow section, or it can be multiple columns that act as one column. If multiple columns are used, they should be connected at one or more locations to ensure collaboration between the individual columns. The advantage of multiple columns is that material is moved away from the gravitational centre of the composite column, thus creating a larger leverage arm, thus creating an increased moment of inertia, making the column more resistant to buckling. The disadvantage of using multiple columns is that if one of the columns fails, the composite column is likely to fail as well. Since columns are the most important part of a structure, this is not favourable. Concrete filled sections therefore have the preference above multiple columns.

Since there are no structural elements that can support the columns laterally, the option of supporting the columns intermediately to decrease the buckling length is not possible. Creating fixed connections, as stated in the second solution, can be labour-intensive and expensive. Because these aspects are difficult to determine in this research, the option of creating fixed connections is not used in this research. Since this could be a good solution, this will be added to the recommendations.

Therefore the columns will be designed as concrete filled CHS in this research. An advantage that comes together with the use of concrete filled CHS is that these sections behave relatively well in case of fire or accidents. Because of the large dimensions of the columns, attention should be given to the pouring of the concrete. This can not be done in one go, because of the shrinkage of the concrete, local failure of the steel and temperature differences. These aspects will not be taken into account in this research and will therefore be added to the recommendations.

6.4.2. Design of columns

As explained before, the columns will be designed as concrete filled CHS. For the calculations of these columns the Design Guide of Tata Steel is used [Wang, 2014]. In this design guide a simplified design method is given. For this research this simplified design method is used. Assumed dimensions and materials for which the simplified design method is applicable are used.

The calculation of the composite column can be found in Appendix F. In these calculations it can be seen that the utilization of the columns is about 0.57. This means that the columns will have some overcapacity. Since the calculations neglect the shear forces and the bending moment around the member axis, this overcapacity is required. Because of this overcapacity, the columns will have extra resistance against accidental loads, such as impact loads. This overcapacity might be a little bit too much, but that also follows from the restrictions of the simplified design method.

Table 6.5: Internal forces diagonal columns (in kN and kNm)

CS13	N-	N+	Vy	Vz	Mx	My	Mz
LC1	-4338.66		44.56	230.32	296.32	1743.51	1403.15
LC2	-4007.33		43.64	198.82	231.62	1511.28	1443.28
LC3	-9272.87		149.19	268.67	773.31	3776.43	3141.39
LC4		9008.45	90.15	164.28	313.63	2515.09	2000.89
LC5i	-5732.00		71.09	207.95	369.20	2024.60	1914.12
LC5ii	-7823.77		104.80	227.19	422.67	2434.63	2371.18
LC6	-2894.19		24.75	150.26	185.91	1106.94	957.20
LC7	-3045.50		29.10	150.79	174.21	1124.24	1089.88
LC8	-6217.40		75.58	172.30	465.37	2274.56	1939.05
LC9		5036.31	36.93	152.93	133.13	1302.26	724.01
LC10i	-4088.85		42.75	154.43	250.36	1417.86	1379.48
LC10ii	-5327.47		57.31	162.13	277.63	1588.04	1577.51
MAX	-9272.87	9008.45	149.19	268.67	773.31	3776.43	3141.39

The final column will be a CHS with a diameter of 1200 mm and a wall thickness of 20 mm. The hollow section is filled with concrete C20/25 with 0,5% of its area reinforcement. This equals to about 20 rebars with a diameter of 20 mm in each column. A cross-section of the composite column can be seen in Figure 6.8.

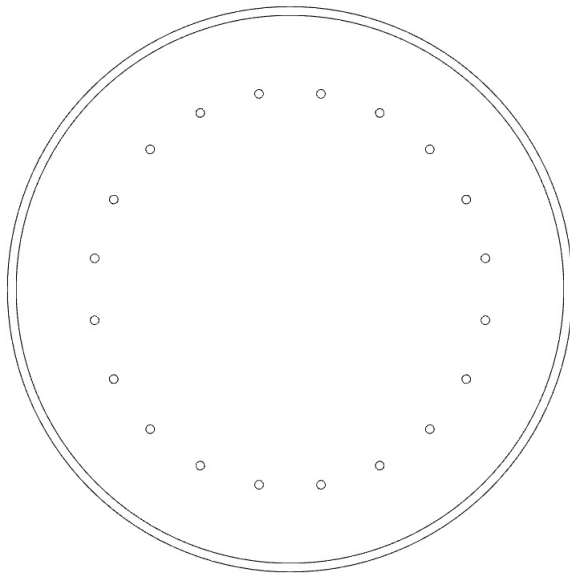


Figure 6.8: Cross-section of composite column

6.5. Conclusion

6.5.1. Self-weight

The goal of this research is to design a structure that is optimal with regards to material usage. The material usage can be stated as the total weight of the structural material. In this design only steel is used, so the total weight of the structural steel is taken as a measure to check whether this structure is favourable in comparison to other similar structures. The comparison with other structures is a somewhat difficult comparison, since no other structure is exactly the same as this design. Some factors that influence the required self-weight of a structure are the structural system, the shape and the span. Only if all these factors are similar in another design, the self-weight can be compared one-on-one. Therefore it is important to have a further look at the comparisons between this design and other existing structures. Often differences can be explained by looking at the span, for instance.

In this paragraph the self-weight of the structure is described. For this self-weight a distinction is made between the self-weight of the total structure, the self-weight of the roof including the ring and the self-weight of the spanning arches only. This distinction is made so comparisons with other structures can be made more easily. Each of these self-weights is easily read from the results of the SCIA calculations, by looking at the self-weight of the different cross-sections. The weight of the total structure, including the columns, is not given, because this is not important for the comparison. No other structures have columns of this length. Besides that, composite columns are used, which makes the self-weight of the total structure including the columns not important.

Roof only

The weight of the spanning arches is 1.915.660 kg. With a total roof area of 31.415 m², this gives a self-weight of 60,98 kg/m².

Roof including ring

The total weight of the spanning arches and the ring truss is 3.464.840 kg. With a total roof area of 31.415 m², this gives a self-weight of 110.3 kg/m².

Comparison with preliminary calculations

In Chapter 3 preliminary calculations are performed. The self-weight of the roof only is calculated to be 17,9 kg/m². This value is about 3.5 times smaller than the self-weight that is calculated in this chapter (61,0 kg/m²). This difference can easily be explained by the fact that in the preliminary calculations different loads are used. The loads that are used in the preliminary calculations are self-weight of the structure and a uniform loading of 1 kN/m². The loads that are used in the final calculations are the loads that are described in Chapter 4. The wind loads, which turn out to be decisive, are approximately three times higher. This explains the large difference in self-weight.

Comparison with other structures

The comparison between the self-weight of this design and the self-weight of other structures is an important one to show the strength of this design. The problem with this comparison is that the self-weight of a structure is often not public. Some structures of which the structural self-weight is made public can be compared to the Event Dome, but since the shape and the structural behaviour often differs from the Event Dome, these comparisons wouldn't be of any use. In this paragraph comparisons are made with two reference projects, that are also described in Appendix B.

Houston Astrodome The Houston Astrodome is a dome with a diameter of 220 meters. The total self-weight of the structural elements of the roof excluding the tension ring is about 69 kg/m². Including the tension ring the self-weight of the roof structure of the Astrodome is 81 kg/m². If the ring beam is excluded, the Event Dome has a lower self-weight than the Astrodome, but if the ring beam is included, the self-weight of the Event Dome is much larger than the self-weight of the Astrodome. This can be explained by the fact that the Event Dome has a minimal number of columns to ensure maximal flexibility. The Astrodome has much more columns in its perimeter, which decreases the column-to-column span of the ring beam. Because of this smaller span, the required cross-sectional dimensions of the ring beam are much smaller, thus reducing the self-weight of the ring beam. Another factor that reduces the self-weight of the ring beam of the Astrodome is the fact that the Astrodome also has shear walls in its perimeter. These shear walls take up a significant part of the internal forces of the ring beam, thus reducing the required self-weight of the ring. These shear walls are made of concrete, and are therefore not included in the self-weight that is stated in this paragraph.

Messe Frankfurt - Halle 3 Halle 3 consists of five adjacent groups of steel arches that span 165 meters in one direction. The self-weight of the structural steel is about 210 kg/m². Although Halle 3 has a different structural system than the Event Dome, it is still a relatively good comparison, because both structures use arches as the main spanning elements. Even though the span is somewhat smaller (165 meters and 200 meters), Halle 3 is about two times heavier than the Event Dome. This comparison shows that the Event Dome is in fact a relatively optimal design.

7

Sensitivity analysis

In Chapter 6 the optimal design for the Event Dome has been given. But sometimes other demands can arise for the structure. One can think of an oval roof or a roof with a hole in it. These two options will be researched briefly in this chapter and the structural consequences on the material usage will be considered.

7.1. Extra columns in perimeter

In the final design the number of columns is intentionally kept low to ensure maximal flexibility. Since the ring beam transfers all loads from the roof to the columns, this small number of columns lead to large spans in the ring beam. For the horizontal thrust forces coming from the dome this is not really an issue, since the thrust forces make equilibrium inside the ring beam. For the vertical loads, however, this large span leads to large cross-sectional dimensions in the ring beam. In this paragraph extra columns are added to research the structural consequences of adding more columns.

The largest span that occurs within the ring beam is the span between a vertical column and its nearest set of W-shaped columns. This span is about 45 meters. By adding a vertical column in the middle of this span, this span can be reduced to 22,5 meters. By reducing the span with a factor 2, the bending moment in the beam is reduced by a factor $2^2 = 4$.

Although this extra column might be good for the structural behaviour, the flexibility is also influenced. The maximum width of elements that can pass through the perimeter of the structure is reduced by a factor 2. This could become a problem for several functions, such as a movable grandstand for instance. The decision whether or not to add the extra columns can be made for each project individually.

7.1.1. Results

For these calculations the same procedure as the one in Chapter 6 is used. After the extra columns are added manually, the cross-sections are optimised with Auto-design, using linear analysis. When each of the members is optimised, the structure is calculated with a non-linear analysis to check the system for stability. The results of this non-linear optimisation are checked manually to see if the system complies to the requirements stated in Section 6.2.1. The results of this analysis can be seen in Table 7.1, together with the results of the analysis of the final design that is performed in Chapter 6. The D in the table stands for diameter, and the t for wall thickness.

Table 7.1: Results analysis for system with extra columns

Cross-section	System with extra columns			System without extra columns			Weight difference (kg)
	D (mm)	t (mm)	Weight (kg)	D (mm)	t (mm)	Weight (kg)	
CS2	1010	45	763.149	1010	56	938.871	-175.722
CS3	790	18	168.572	880	19	66.150	+102.422
CS4	580	17	148.204	600	17	153.469	-5.265
CS5	360	8	132.581	360	8	132.581	0
CS6	580	13	531.468	630	15	665.146	-133.678
CS7	800	16	957.830	800	16	957.830	0
CS8	800	16	957.830	800	16	957.830	0
CS9	800	19	229.777	910	20	275.627	-45.850
CS10	800	19	250.457	800	19	250.457	0
CS11	830	20	71.898	830	20	71.898	0
TOTAL			4.211.768			4.469.861	-258.093

7.1.2. Validation of results

From the results it follow that adding 8 extra columns in the perimeter, the total weight of the structure is reduced by 258.093 kg, which is about 6%. It can be seen in Table 7.1 that the self-weight of 5 cross-sections is changed. The changes in the weight of the columns can easily be explained. The weight of the vertical columns increases, because instead of 4 vertical columns, there will be 8. Even though the cross-sectional dimensions decrease, the total weight of these vertical columns increases. The self-weight W-shaped columns, however, decreases, because there are more vertical columns present in the design. This higher number of columns lead to a smaller internal load in the W-shaped columns, thus decreasing their self-weight.

Furthermore, it can be seen that the self-weight of the lower truss chord decreases. This can be explained by the fact that this is the chord that takes up half of the vertical bending moment in the truss. Since the span of

the truss is decreased, the bending moment is also decreased. The same holds for the upper, inner chord of the truss, but since this chord 'shares' the vertical bending moment with the upper, outer chord, the decrease in self-weight of the upper, inner chord is small.

The final difference that is clearly visible is the decrease in the self-weight of the diagonals. This is also explained by the fact that the span of the truss is decreased. The diagonals of the truss are important in transferring the shear forces, that follow from the bending moment. Since the bending moment is decreased, these shear forces also decrease.

7.1.3. Conclusion

From the calculations it follows clearly that the addition of 8 extra columns in the perimeter of the structure is favourable in terms of material usage. In terms of flexibility, however, this addition of extra columns is not favourable. More columns in the perimeter mean more obstructive elements. The maximum free width in the perimeter is decreased by a factor 2. This doesn't have to be a problem for every possible function, but for some it might become a problem. One can think of a movable grandstand, for instance. The maximum width of the movable grandstand is decreased by a factor 2 as well.

The addition of the extra columns could therefore be a good option, but the future user of the structure should think it through to make sure these extra columns won't become a problem in a later stage.

7.2. Hole in roof

The first option that is considered is the design with a hole in the roof. This hole can be demanded for several reasons. One can think of the demand to light fireworks in the stadium or even create the possibility to let a helicopter land inside the stadium. For this reason the structure is calculated with a hole in the roof with several different diameters. At the edge of the hole a inner ring beam is modeled to connect the ends of the arches.

The structure will again be calculated with SCIA, and the structural behaviour will be explained to verify the results.

The brief results can be found in Figure 7.1, while the extended results can be found in Appendix D.

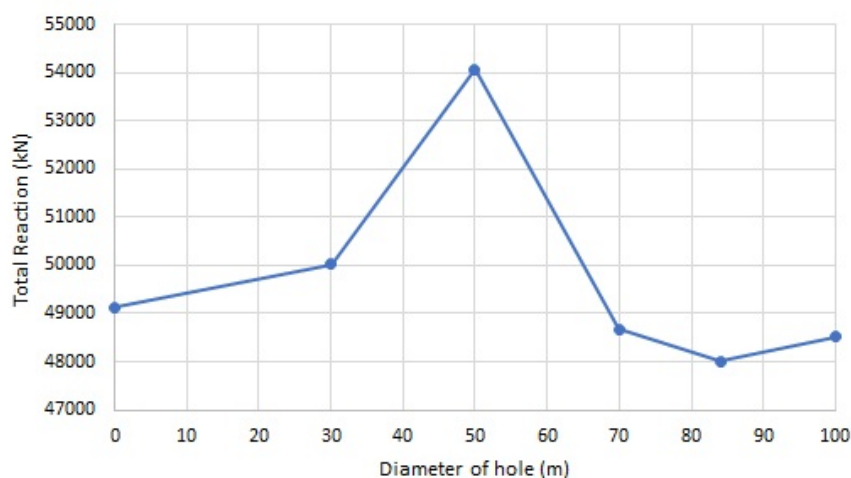
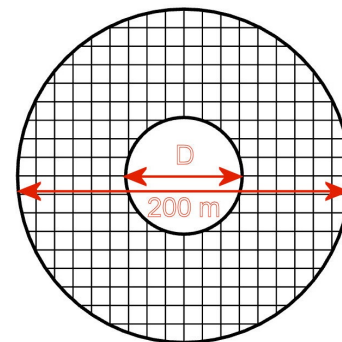


Figure 7.1: Results of the calculations with hole in roof

7.2.1. Validation of results

In the results it can clearly be seen that the total self-weight of the structure increases if a small hole is added to the roof. This can be explained by the fact that the dome is not continuous anymore. This means that the structural behaviour of the dome changes as well, adding larger bending moments to the arches. This increases the required cross-sectional dimensions of the arches. For a small hole with diameter 30, the difference is relatively small, since only a few arches are not continuous anymore. In this case the dome still acts as a dome, just with a discontinuity in it. If the size hole is increased to a diameter of 50 meters, this behaviour can be seen more clearly. The dimensions of the arches increase enormously, due to the changed structural behaviour. The arches now have to take large bending moments, which increases the required cross-sectional dimensions.

If the diameter of the hole is increased even further, however, the total self-weight of the structure decreases. This is explained by the fact that the total loads decrease as well. Obviously, loads can not work on a hole. For the smaller holes, this decrease in loads is small compared to the change in the structural behaviour. If the diameter of the hole is increased, the reduced loads become larger than the change in structural behaviour. Despite the fact that the dome does not work as a perfect dome structure anymore, the total self-weight decreases.

If the diameter of the hole is increased even further, to 100 meters, the roof arches start acting as cantilevering members. The roof structure does not work as a continuous structure anymore. Because of the cantilevering behaviour, large bending moments occur. The required cross-sectional dimensions of the arches increase because of these large bending moments, thus increasing the total self-weight of the structure.

7.3. Hinged connections in one direction

The optimal solution with regards to minimal self-weight is a design where the arches in both direction have moment-resisting connections with each other. It can, however, be more practical to have hinges in one direction. Hinged connections are easier to be made on the construction site and could therefore be favourable in comparison to the system with moment-resisting connections in both directions. Moment-resisting connections are often made with welds, while most hinges are bolted connections. Bolted connections are erected more easily, especially in this design, where most connections are at high altitude.

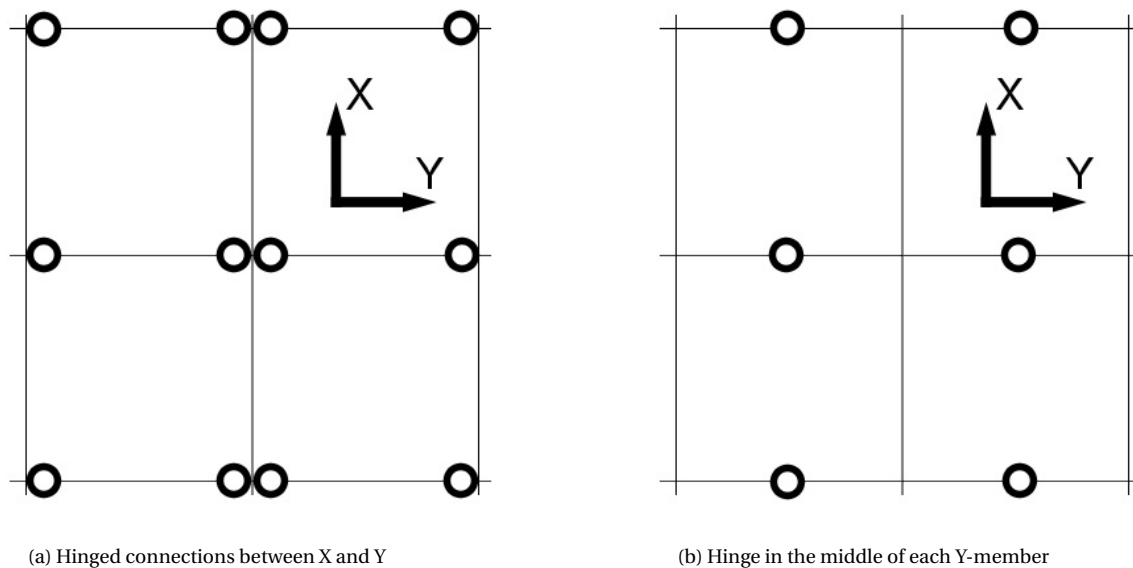
On the other hand, hinges can largely influence the structural behaviour of the system. With moment-resisting connections, the arches in both directions work together as a continuous system. In this way, the shape of the dome is used completely, which means that no bending moments occur if the structure is loaded uniformly. If hinged connections in Y-direction are used, the structure doesn't work as a complete dome anymore. Even though the arches in X-direction will still have no bending moments in an ideal situation, the arches in Y-direction will have behave as simply supported members, that are supported by the arches in X-direction. This change in structural behaviour will increase the required cross-sectional dimensions of the members. In this paragraph the system with hinged connections in one direction is researched. The results will be compared to the results of the system with moment-resisting connections.

Two different locations for the hinges will be considered: hinged connections between Y and X members (a), and a system where X and Y members are connected with a fixed connection, but where each Y-member has a hinge in the middle of its span (b). Both options can be seen in Figure 7.2. The circles in this image represent hinges.

7.3.1. Option A

The starting point for the calculation of option A is the design that is made in Chapter 6. When the calculations are performed with the cross-sectional dimensions that are stated in Chapter 6 the system becomes unstable. Linear calculations can be performed, but the results of these calculations show that the system is highly unstable. This can also be seen by performing non-linear calculations. These non-linear calculations can not be finished, because SCIA gives the warning that the stiffness matrix is singular. This means that no equilibrium can be found for a deformed structure.

From the results of the linear calculations it can be seen that the beams in X- and Y-direction both fail. This can be explained by the fact that the arches in X-direction have to take all the load instead of about half of it.



(a) Hinged connections between X and Y

(b) Hinge in the middle of each Y-member

Figure 7.2: Hinges on the arches

Because of this less-favourable structural behaviour the deformations become much larger, inducing extra internal forces in both the X- and Y-members.

To create a stable system, the cross-sectional dimensions of both types of members should be increased. By iteration the smallest cross-sectional dimensions for which the system is stable can be found. This iteration shows that the self-weight of the arches becomes about 6 times larger if hinged connections are used ($1.9 \cdot 10^6$ kg against $12 \cdot 10^6$ kg). Because of this larger self-weight of the arches, also the dimensions of the ring truss and the columns should be increased. The total self-weight of the system with hinged connections between X- and Y-members is about 3 times larger than the self-weight of the system with moment-resisting connections.

Validation of results

At first the increase in self-weight seems to be extremely high. This raises questions whether the results are correct. To make sure the results and the conclusions that are drawn, are correct, a closer look is taken to the extended results of the calculations. For this validation the results of a uniform downward loading is used. This is done to prevent that changes in structural behaviour due to asymmetricality, for instance, influence the validation of the results.

The first thing that will be looked into are the internal forces of the arches. A distinction is made between the internal forces of the X-arches and the internal forces of the Y-arches. The hinges are all modeled in the Y-arches, which means that the X-arches are still continuous in this option.

For the arches in X-direction, the addition of hinges does not significantly change the normal forces in the arches. The bending moments in the X-arches, however, increase significantly. This introduction of large bending moments in the arches show that the arches will not work as perfect arches anymore. The induced bending moments have a large influence on the required cross-sectional area of the arches.

For the arches in Y-direction, it can be seen that the bending moments in the Y-arches do not change significantly, but the normal forces are about 30% larger if hinges are added. This can be explained by the fact that the structure does not work as perfect dome anymore. The addition of hinges prevent the arches in both direction to act together fully. This creates bending moments in the X-arches, and increased normal forces in the Y-arches.

The second thing that will be looked into are the critical load coefficients. The critical load coefficients are determined for the structures with equal cross-sections to make a good comparison. It can be seen clearly that the critical load coefficients for all load combinations decrease. This means that the structure is much more susceptible for buckling.

Both the increased internal forces, and the increased risk of buckling increase the required cross-sectional areas of the arches. Increasing the cross-sections also means increasing the load from the self-weight. This increased load increases the required cross-sectional area again. Since the forces that flow from the arches to the rest of the structure increase, the required cross-sectional dimensions of all elements increases. This combination of factors explains the large increase in total self-weight.

Conclusion

Even though the use of hinged connections do have some advantages over moment-resisting connections, such as easier constructability, this option is not favourable, because the material usage is 3 times larger. When costs are considered, the advantage of easier connections doesn't weigh up against the higher material usage.

7.3.2. Option B

In option B hinges are added in the middle of each Y-member. This could be favourable compared to the system that is used in option A, because the hinges are located where the bending moments in the Y-members are small. The disadvantage that the structure doesn't work as a complete dome anymore, which is described in Section 7.3.1, still occurs in this system.

Calculations show that if the cross-sectional dimensions of the system described in Chapter 6 are used, the new system is again highly unstable. The same procedure as for option A is used. By iteration the smallest self-weight is found, for which the system is stable. In this case, the Y-members can be smaller than the Y-members for option A, because the hinges are located where the bending moments are the smallest. This makes the total self-weight of the arches somewhat smaller than the ones for option A, about 10%, but still much larger than the self-weight of the system with moment-resisting connections.

Validation of results

Again, both the internal forces and the critical load coefficients are considered. For this system it can be seen that the internal forces of most X- and Y-arches do not increase significantly. The arches close to the ring, however, do have much bigger internal forces if hinges are added. This can be explained by looking at the critical load coefficients. The critical load coefficients show that the structure is much more susceptible to buckling if hinges are added. This decreased buckling resistance creates large internal forces at the edges of the dome, following from instability of the total structure.

The instability that follows from the addition of hinges increase the required cross-sectional area of the arches, because much more material is needed to make the structure as a whole stable. Even though most members will be stable, even if hinges are used, the system as a whole becomes unstable. This explains the increase in self-weight of the structure if hinges are used.

7.3.3. Conclusion

The total self-weight of each of the systems is given in Table 7.2. The self-weight of the final design that is performed in Chapter 6 is also given to make a good comparison between the different systems.

Although hinges could be favourable for the construction, the self-weight of the structure increases enormously if hinges are used. This increase in self-weight doesn't only increase the cost of the material, but it also increases the complexity of the construction significantly. Also the complexity of the connections increases, due to this increase in size of the members.

If all aspects are considered, the addition of hinged connections are not favourable. The calculations that are made in this section show that the material usage increases significantly, easily weighing out the possible advantages of the use of hinged connections. If costs are considered, it is highly unlikely that the system with hinged connections costs less than the system with moment-resisting connections.

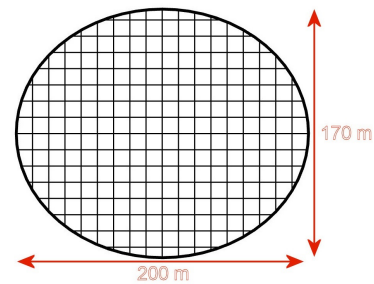
Table 7.2: Results analysis with hinges in Y-direction

	System without hinges	Hinged option A	Hinged option B
Total weight (kg)	4.469.861	14.284.707	13.071.140
Difference with system without hinges (kg)		+ 9.814.846	+ 8.601.279

7.4. Oval roof shape

Since the goal of this research was to design an optimal roof structure, a circular roof shape is used in this research. This circular shape is the most optimal shape, since it has no discontinuities. Often, however, the functions below the roof demand other shapes than a circle. A football stadium, for instance, has an oval shape. To explore the structural consequences of this oval shape, a roof with an oval shape is calculated and compared to the circular roof. The radii of the oval that are used are 100 and 85 meters, corresponding to the approximate size of football stadium De Kuip in Rotterdam.

A top view image of this oval roof can be seen to the right.



7.4.1. Results

The procedure for the calculations is the same procedure that is used in Chapter 6. First the cross-sections are optimised with Auto-design, using the results from linear calculations. After this non-linear analysis is performed to check the system for stability. If all requirements stated in Section 6.2.1 the results can be compared to the results of the analysis on the circular system.

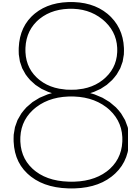
The results of the calculations can be found in Table 7.3. From the results it can be seen that the total weight of the oval system is lower than the total weight of the circular system. The area of the oval roof, however, is 15% smaller than the area of the circular roof. The weight per m^2 of the oval roof therefore is higher than the weight of the circular roof. These calculations show that a circular roof is indeed the favourable roof shape in comparison to the oval shape.

Table 7.3: Results analysis with hinges in Y-direction

	Circular system	Oval system
Total weight (kg)	4.469.861	4.062.370
Weight per m^2 (kg/m^2)	142,3	152,1
Difference per m^2 (kg/m^2)		+ 9,8 (7%)

7.4.2. Conclusion

The weight per m^2 of the oval roof is higher than the weight of the circular roof. But sometimes, not the entire circular area is needed. If, for instance, a football stadium is considered, parts of the circular roof will not be in use. These parts will only cover an area outside of the stadium. In this case, it could be favourable to consider an oval roof shape, because the total weight can be smaller. The decision whether to use an optimal circular roof shape, or a slightly less optimal oval roof shape can be made for each future project individually.



Construction

The design of the Event dome has a large roof that is built up from a large number of members, connected with a large number of connections. The design of the connections and the construction of this design are therefore important aspects to ensure the success of the design.

The main focus of this research is to design an optimal structure. But since the construction does have a large influence on the costs of the structure, some attention will be given to the construction of the Event Dome. The main friction points of the construction of a structure with such dimensions are discussed in this chapter. Also some issues that can arise during the construction of the Event Dome will be discussed, and possible solutions will be given.

The construction plan that is given in this chapter might not be the optimal plan. This plan is just given here to show an example of a possible construction plan.

8.1. Starting points

In this research some assumptions are made for the construction. These assumptions are taken as boundary conditions to which the construction planning must comply.

In general, it can be said that welds are preferably done in the shop. In the shop the workers do not have to deal with rain, wind, changing temperatures. Some connections, however, have to be made on the site, because it is impossible to transport the entire structure from the shop to the site. For the welds that will be made on the site, the welds preferably are made on the ground instead of high in the air. This makes it safer for the construction workers and the conditions can be controlled more easily.

During multiple steps of the construction process, the maximum dimensions of elements are limited. These limitations follow from the limitations for fabrication and transport, but also from the maximum capacity of the cranes that are used. In this chapter no exact numbers of these maximum dimensions are given, but procedures are given that can be used for a range of maximum dimensions.

8.2. Construction process

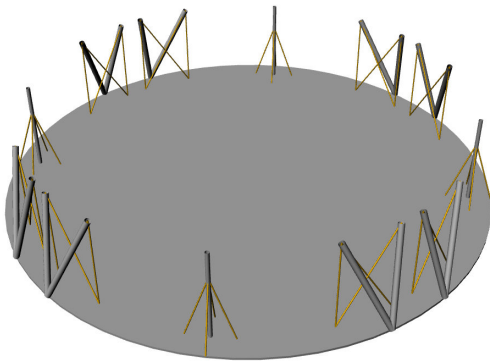
In this paragraph the basic construction planning is given. In the next sections this planning will be elaborated further, where each part will be described more extensively. Figure 8.1 shows the different phases of the construction, where yellow elements are the temporary support structures.

In the first phase the foundation will be constructed. Since the foundation is outside the scope of this research, no further attention will be given to the foundation.

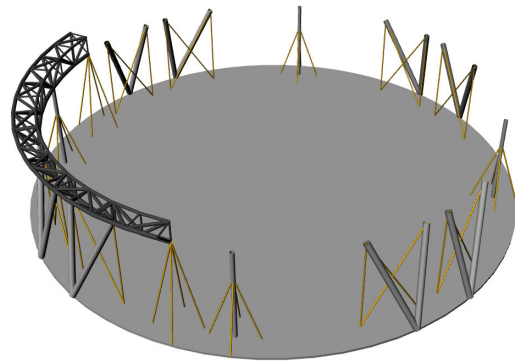
After the foundation is finished, the columns will be constructed. Both the W-shaped frames and the vertical columns are constructed. During construction, the columns need temporary structures, because they are not stable on their own.

The third phase is the construction of the ring. The ring beam will be fabricated on the construction site. Because of the maximum crane capacity, the ring beam will be fabricated on the ground in multiple segments. By fabricating segments instead of lifting each separate element the number of welds that have to be made at a height of 50 meters is reduced. This reduces the risk and the complexity of the welding process. The length of each segment is dependent on the capacity of the cranes that will be used. After the segment is fabricated on the ground, the entire segment is lifted to the required height and is connected to the columns. Since the structure is not stable on its own until the entire ring beam is constructed, temporary support structures are needed to support the ring segments during construction. After the final ring segment is put in place, the structure becomes stable on its own and the temporary support structures can be removed.

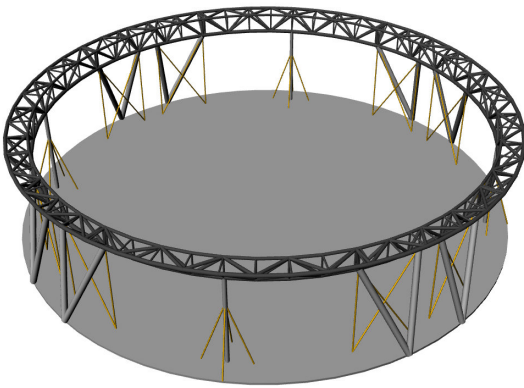
The fourth phase of the construction is the construction of the roof. Because of the limited crane capacity the roof is divided into segments. The size of these segments is dependent on the capacity of the cranes that will be used. The roof will be constructed in one direction. That means that first the most left arch is constructed. After that the second most left arch is constructed, and so on. The second arch is connected with the first arch. In this way a finished arch is stable on its own, because on both ends it is supported by the ring truss, and laterally it is supported by the arch members in the other direction. During the construction of an arch, that arch is not yet stable, since it only has a support at one of its ends. Therefore temporary measurements are needed. These temporary measurements can be some kind of scaffolding tower. That tower can be moved to the next location, and the same structure can therefore be used for the entire roof structure.



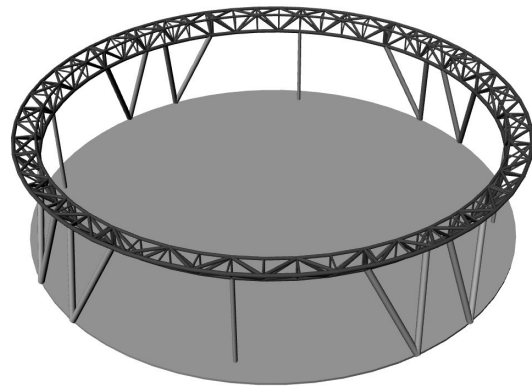
(a) Second phase: construction of columns with temporary supports



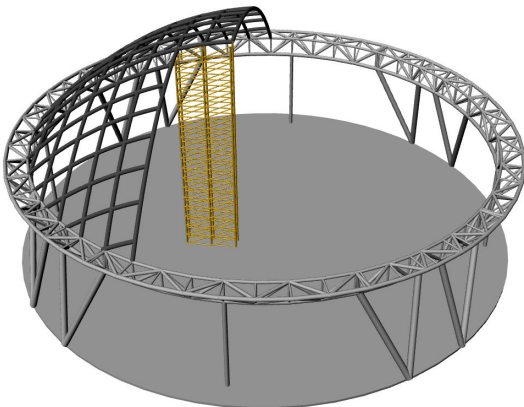
(b) Third phase: Construction of ring with temporary supports



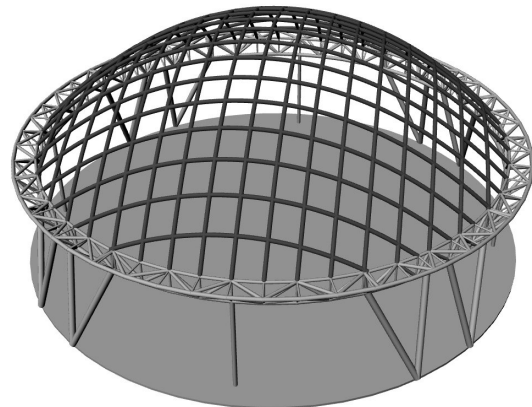
(c) Third phase: Ring is completed



(d) Third phase: Temporary supports are removed



(e) Fourth phase: Construction of roof arches with temporary supports



(f) Construction finished

Figure 8.1: Construction phases of the Event Dome

8.2.1. Second phase: columns

After the foundation is finished, the columns will be constructed.

The vertical columns are simple CHS steel sections. These sections have a length of about 41 meters. With special transport these sections can be transported to the construction site in one piece. On site a crane will lift them to stand vertically. Since the supports of these columns are hinged, temporary structures are needed to support the columns laterally.

The W-shaped columns are composite columns that are made up of a CHS steel section, filled with concrete. Because of the large dimensions of these columns, the concrete can not be poured in one go. This could lead to local failure of the CHS, because the concrete is not hardened yet, and the pressure on the CHS would become enormous due to the large length of the column. Besides this local failure of the CHS, pouring all concrete in one go could also lead to problems in the concrete itself. Because of its large dimensions, the hardening of the concrete would create heat, and stresses would occur in the concrete. This could lead to cracking of the concrete. Therefore extra attention should be given to the filling of the CHS. Extra measurements could be necessary to prevent problems during construction.

8.2.2. Third phase: ring

After all columns are constructed, the next step is the construction of the ring beam. As explained in Section 8.1 the connections between the different elements of the truss are preferably made on the ground. The truss is therefore divided into segments. The size of these segments is limited by the capacity of the cranes that are used. Larger segments are preferred over smaller segments, because less connections have to be made at a high altitude.

Because the system is not stable as a whole until the entire ring is finished, temporary support structures are needed to support the truss vertically and laterally.

First the segments are connected to the temporary support structures. After the segment is supported by the temporary structures, the connections with the other elements can be made. The segments are welded to the different columns. After this is done, the next segment is hoisted into place. Again, first the structure is supported by temporary structures. After that the connections between the previous segment are made. Lastly the connections with columns are made.

This routine continues until the entire ring is complete. Once the ring is completed and all connections are made, the temporary support structures can be removed, because the system is now stable on its own.

8.2.3. Fourth phase: arches

After the columns and the ring truss are constructed, the roof structure can be constructed. Because the number of connections that have to be made at high altitude preferably is minimized, the roof is divided into segments of maximal dimensions. The dimensions of these segments are dependent on the capacity of the cranes that are used. In Figure 8.2 a possible division into segments can be seen. The green edges in the image are the location where temporary support structures are needed during the construction of the roof.

Figure 8.2 also shows the order in which the segments will be constructed. The first segment that will be constructed is segment 1. This segment is hoisted into place. When the segment is in place it can be connected to the ring truss. The segment is not stable yet when all connections to the ring beam are finished, because it is not supported at the two free edges. It can be, however, that the segment is stable, but detailed calculations should be performed to check whether the segment is stable yet. If the segment is not stable yet, temporary support structures should be added to one of the free edges of the segment. The most logical one is the short edge of the segment. This short edge is shown in green in Figure 8.2. When the first segment is connected to the ring beam and the temporary support structure, the next segment can be constructed.

Segment 2 is hoisted into place and connected to the ring beam and segment 1. When all connections are made that part of the roof is stable on its own, because it can be seen as an arch on two supports, that is supported laterally by the elements in the other direction. The temporary support structure can therefore be removed.

The next step is the construction of segment 3. The same procedure is used as the one for segments 1 and 2. The temporary support structure that is needed for segment 3 is the same support structure that is used for segment 1, eventually with some extra altitude added. For this support structure, some kind of scaffolding structure can be used for which it is easy to add or remove some extra altitude.

The same procedure is repeated until the entire roof structure is constructed. After the entire roof is constructed, the structure can be finished by adding installations and roof cladding.

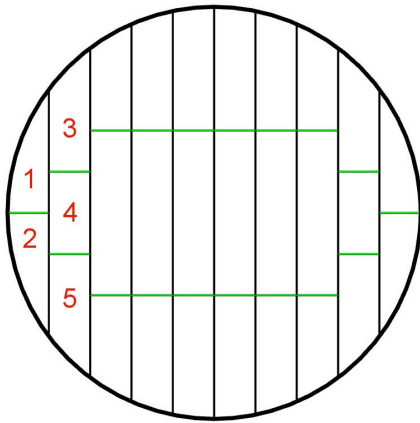


Figure 8.2: Possible segment division

8.3. Conclusion

In this chapter a possible construction process for this structure is described. Obviously, multiple other construction options can be thought of, and the solution that is given in this chapter is not necessarily the best option. Therefore additional research is recommended to draft a waterproof construction plan.

It can be seen that a lot of difficulties arise during the construction of a structure with such large dimensions. Therefore a thorough, detailed construction plan should be drafted before the start of the construction. If possible problems are examined beforehand, solutions can be found to prevent delays and budget overruns.

Because of the importance of the construction in such a design, a recommendation is added that the construction plan is researched further.

9

Structural Robustness

For a design like the Event Dome, where the consequences are high in case of failure of the structure, it is important to check the structure for robustness. The procedure that is used to check this structural robustness is described in the Eurocode [NEN-EN 1991-1-7, 2011], paragraph 3.3: accidental design situations.

The Eurocode describes the robustness of a structure as follows: "*Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.*" [NEN-EN 1991-1-7, 2011]

The structural robustness is checked by removing one member, and recalculating the structure. From the results of this calculation, one can determine whether the structure is sufficiently safe. A structure is sufficiently safe if removing one member does not lead to a collapse. The structure doesn't have to comply to all structural requirements stated in Section 6.2.1, like maximum displacement, but it is important that the failure of one member does not lead to the failure of the complete structure.

9.1. Process

In this chapter some critical members are removed one at the time. The new structure is then calculated and the system is checked to see if it is sufficiently robust. The members that will be removed are the critical members of each cross-section. That means that the members with the highest utilization rate are the ones that are removed. Sometimes an extra calculation is performed, where another member than the most critical one, is removed. This extra calculation will be performed if multiple locations can be found, for which the removal of a member would have a unique structural behaviour. For the arches, for instance, the removal of an edge member will have different consequences than the removal of one of the middle members. Both calculations will therefore be performed.

In this chapter the assumption is made that the structure is sufficiently robust if the critical load coefficient for each load combination is equal to or larger than 1.0. The utilization of the elements is also checked for all SLS load combinations. If the utilization of one of the elements for a SLS load combination is above 1.0, this element is checked to see if the ultimate stress will be reached. If the ultimate stress will be reached, this element will fail. To check if the structure is still safe with this element failing, this element is removed as well, and the structure is recalculated. This process continues until none of the elements will fail.

Only if the critical load coefficient for each load combination is equal to or larger than 1.0, and if no extra elements will fail, the structure will be said to be sufficiently robust.

The engineering reports where the critical load coefficients and the utilization rates for the SLS load combinations are included, are given in Appendix F.

9.2. Arches in X- and Y-direction

For the arches, the X- and Y-members are considered to be of the same cross-section. Since both elements have the same cross-section, and the structure is identical in X- and Y-direction, the critical member of all X- and Y-members is sufficient to check the structure. For the arches, the location of the removed member will have large influence on the structural behaviour of the new structure. Therefore multiple calculations will be performed. The first check that will be done is the one where the most critical member is removed (1). Besides this most critical member, also one of the middle members will be removed (2). The third removed member will be one of the edge members, from the arch with the largest span (3). The three locations can be seen in Figure 9.1.

9.2.1. Most critical member

The first member that will be removed, because it has the largest utilization rate, is member X13.

From the results of the calculations it follows that the critical load coefficient for the new structure is 1.0 for load combination 3. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member X13 is removed.

9.2.2. Middle member

The second member that will be removed is member X133.

The lowest critical load coefficient is 1.0 for load combination 3. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member X133 is removed.

9.2.3. Edge member

The third member that will be removed is member X125.

The lowest critical load coefficient is 1.0 for load combination 3. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member X125 is removed.

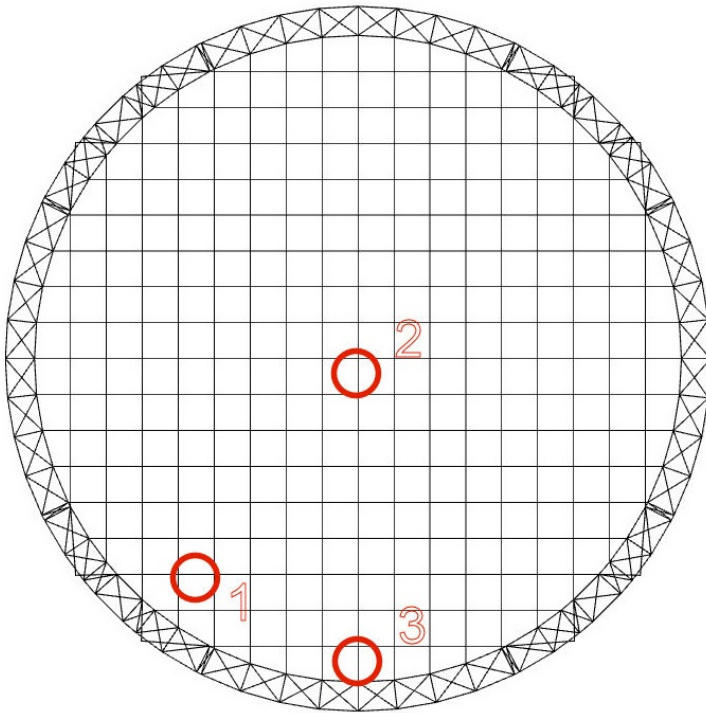


Figure 9.1: Location of removed members X and Y

9.2.4. Conclusion

Three different arch members are removed. The removal of each of these members has a different influence on the structure. Each of the three calculations show that the structure is sufficiently robust, even if one of the arch members is removed, because the lowest critical load combination is 1.0, and each of the elements has a maximum utilization rate of 1.0 or lower, for each SLS load combination. Regarding the X- and Y-arches, therefore, the structure is sufficiently robust.

9.3. Truss members

The truss consists of three chords, the upper inner chord, the upper outer chord and the lower chord. For each of these chords the most critical one is removed to check the robustness of the truss.

Besides the chords, the truss also consists of verticals and diagonals. For both these cross-sections, the most critical one is removed as well.

The locations of all truss members that are removed are around the W-shaped columns. For each of the truss cross-sections the most critical elements are in that location.

9.3.1. Upper, inner chord

The member that will be removed is member Chordi38.

The lowest critical load coefficient is 1.0 for load combinations 3 and 5i. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member Chordi38 is removed.

9.3.2. Upper, outer chord

The member that will be removed is member Chordo7.

During the calculations SCIA showed the error that the stiffness matrix is singular. This means that the structure will buckle under the giving loads. This shows that the structure will collapse. Adjustment of the structure, or additional measurements are needed to ensure robustness of the structure. Because calculations can not be performed, no engineering report will be given in Appendix G for the removal of this cross-section.

9.3.3. Lower chord

The member that will be removed is member Chord15.

The lowest critical load coefficient is 1.0 for load combinations 3 and 5i. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member Chord15 is removed.

9.3.4. Verticals

The member that will be removed is member Vert48.

The lowest critical load coefficient is 1.0 for load combination 3. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member Vert48 is removed.

9.3.5. Diagonals

The member that will be removed is member Diag12.

The lowest critical load coefficient is 1.0 for load combinations 3 and 5i. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member Diag12 is removed.

9.3.6. Conclusion

Calculations have shown that the removal of the critical outer chord lead to instability. This means that the structure will collapse if the critical outer chord fails. This is an issue that must be solved before construction of this design, because the structure is not sufficiently robust at this stage. The adjustment of the structure, or additional measurements that should be taken are outside the scope of this research, and will therefore be added to the recommendations.

9.4. Columns

Two types of columns are present in this design: vertical columns, and W-shaped columns. The W-shaped columns consist of 4 columns. These 4 columns can be divided in two groups: the outer columns, and the inner columns. With the W-shaped columns divided into two groups, the total number of different columns becomes 3: vertical columns (1), outer angled columns (2), and inner angled columns (3). For each of these 3 types of columns, the most critical one is removed to check the robustness of the structure. The locations of these elements can be seen in Figure 9.2.

For the W-shaped columns, the assumed cross-section of the angled column is the steel-only cross-section that was calculated in Chapter 6. This cross-section is not as strong and stiff as the one designed in Section 6.4. The use of the steel-only cross-section is therefore a safe assumption.

9.4.1. Vertical columns

The member that will be removed is member Cv3.

The lowest critical load coefficient is 1.0 for load combination 3. Some of the lower truss chords have a utilization above 1.0 (1.04) for load combination 8. These members are checked to see if they reach their ultimate strength, because then these members will fail. Since the stresses in these members are 369 N/mm^2 , while the ultimate strength of S355 is about 470 N/mm^2 , these members will yield, but will not fail. That means that the structure will not collapse, even if member Cv3 is removed.

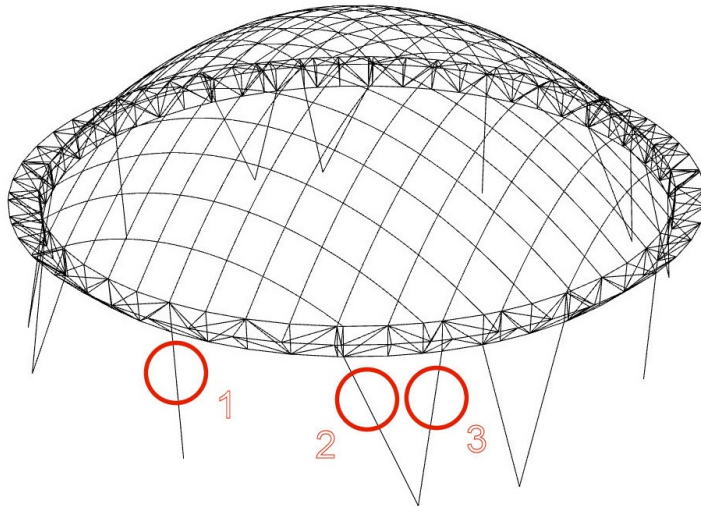


Figure 9.2: Location of removed columns

9.4.2. Outer angled columns

The member that will be removed is member Cw15.

The lowest critical load coefficient is 1.0 for load combinations 2, 3, 5i and 5ii. The maximum utilization of the vertical columns becomes 1.31. The stresses in this member are again compared to the ultimate strength of S355. The maximum stress in the vertical columns is 465 N/mm^2 , which is close to the ultimate strength of 470 N/mm^2 . With the use of the composite column that is determined in Section 6.4, the maximum stress is probably reduced. Therefore additional research is needed to check the robustness of the structure if one of the angled columns is removed.

9.4.3. Inner angled columns

The member that will be removed is member Cw3.

The lowest critical load coefficient is 1.0 for load combinations 2, 3 and 5ii. No members have a utilization rate larger than 1.0 for a SLS load combination. That means that the structure will not collapse, even if member Cw3 is removed.

9.4.4. Conclusion

With the steel-only cross-sections that are used in this paragraph, the structure is not completely robust. With the use of composite columns, however, the structural robustness could very well be ensured. To ensure this structural robustness, additional research is needed. This additional research is outside the scope of this research, and will therefore be added to the recommendations.

9.5. Overall Conclusion

For most cross-sections the structural robustness is ensured. Even if the most critical member of that cross-section is removed, the structure as a whole will not fail. Only for the upper, outer chords, and the W-shaped columns, this robustness is not completely ensured. Additional research is needed to check whether the structure is completely robust. Since this extra research is outside the scope of this research, this will be added to the recommendations.

10

Conclusions and Recommendations

In this chapter, conclusions are drawn from the research that was conducted to answer the main research question 'what is the optimal design for a roof structure of an event venue, that has no structural connection with the underlying structures and provides its own stability?' Recommendations are given to improve the concept and design of the Event Dome.

10.1. Conclusions

10.1.1. Optimal structure

Structural system The optimal structural system with regards to material usage is a dome structure that is supported by a ring truss. Because the main internal forces in a dome structure are normal forces, the required cross-sectional dimensions of the members is relatively small, thus reducing the total self-weight of the structure. In comparison with a system that has bending as main internal force, a weight saving of about 30-35% can be accomplished. In comparison with a tension dome, the total self-weight increases about 35%, but since cable steel is much more expensive than regular structural steel, a cost saving of about 60% can be accomplished, if the price of cable steel is taken as 4 times the price of regular structural steel.

Span direction The most favourable span direction is a rectangular grid which spans in two directions. Spanning in three directions adds a lot of weight, without reducing the internal forces in the other spanning members significantly. A third spanning direction also increases the complexity of the joints, because instead of 4 members, 6 or even 8 members will join in each joint. The advantage, however, is that the joints can be designed as hinges in the plane of the roof, but this does not outweigh the disadvantages. The joints will still be complicated because in the direction perpendicular of the plane, the joints should still be fixed, to ensure that the structure still works as a dome.

Addition of a third spanning direction thus increases the total self-weight of the structure with about 20%.

Height of dome The height of the dome, or the rise, influences the total self-weight of the structure. The optimal value of the rise is a function of multiple aspects, where an increase in rise is favourable for some aspects, but unfavourable for other aspects. The most important aspects that influence the total self-weight are:

- Bending moments in arches - The lower the rise of the dome, the more bending moments will occur in the arches
- Total length of arches - The lower the rise of the dome, the lower the total length of the arches, thus the lower the total weight of the arches. The total length of the arches increases with about 0.5% per meter of increase in rise
- Thrust forces - The lower the rise of the dome, the larger the horizontal thrust forces become
- Wind load on roof - The lower the rise of the dome, the lower the total wind load on the dome. The total wind load changes with about 0.5% per meter of change in rise

In this design the optimal value is found for this structure with its own boundary conditions. If other boundary conditions are used, however, the optimal height of the dome could be different. In this design the optimal value of the rise of the dome is 48 meters. Decreasing the rise 1 meter increases the total self-weight about 1%. Increasing the rise 1 meter increases the total self-weight about 0.4%.

Ring beam The ring beam is important in the structure since it equilibrates the thrust forces following from the dome structure. The combination of the low number of columns, and the large dimensions of the total structure, makes the weight of the ring beam relatively large. The total self-weight of the ring beam is equal to about 80% of the weight of the dome arches. The design of the ring is therefore an important aspect in designing an optimal structure.

A future client should be aware that some design decisions, like the low number of columns, have a large influence on the structural behaviour, and therefore the total self-weight of the structure. A recommendation is therefore added.

Columns The columns that support the Event Dome are a large part of the total self-weight of the structure (about 20% of the structural steel). This can be explained by the fact that each column has a very large cross-sectional area. This is partly because of the low number of columns that are used to ensure maximum flexibility, and partly because of the large buckling length of the columns.

A future client should be aware of the fact that the required dimensions of these columns follow from decisions that are made. The structural engineer should inform him about the possible changes in design that decrease the weight of the columns. A recommendation about this is therefore added.

10.1.2. Sensitivity analysis

Number of columns The number of columns is intentionally kept low to ensure maximum flexibility of the Event Dome. This small number of columns does create large spans for the ring beam, thus increasing the required cross-sectional dimensions of the ring beam. The addition of extra columns decreases the total self-weight of the structure, but it also decreases the flexibility of the design, because there will be more obstructive elements.

The addition of 8 extra vertical columns, in the middle of the largest spans decreases the total self-weight with about 260.000 kg, which is about 6% of the total.

A future client should be aware that the low number of columns is structurally highly unfavourable. The client should be aware that the demand to have flexibility does have great consequences on the total weight and costs of the structure. A recommendation is therefore added.

Shape The circular shape of the roof is the most favourable shape with regards to material usage per m^2 . If, however, the functions underneath the Event Dome require a more oval shape, this can be done without increasing the total self-weight of the structure enormously. Calculations show that the total self-weight of the structure decreases if an elliptical shape with radii 100 and 85 meters is used with about 9%. The weight per m^2 , however, does increase for this system with about 7%.

If the parts of the circle that are outside of the elliptical shape will never be used, an elliptical roof shape is favourable with regards to material usage.

Roof connections The connections between the arches in both direction should be designed as fixed connections. If hinged connections are used, the self-weight of the roof increases by a factor 3, because the structure does not work as a complete dome anymore. Hinged connections could be favourable for the construction, since hinged connections are constructed more easily, but the enormous increase in self-weight makes the usage of hinged connections not favourable in comparison to the usage of fixed connections.

10.1.3. Comparison to other projects

Similar structures The Event Dome has a smaller self-weight than comparable structures that have equal or similar spans. This shows that the Event Dome really has an optimal design. Because of its structural logic, all loads are transferred to the foundation in a simple and direct way.

Costs of football stadiums Since the Event Dome is structurally separated from underlying structures, such as grandstands, both the Event Dome and the underlying structures can be designed in a structurally optimal way. This reduces the self-weight of both structures, thus reducing the total costs of the project. This makes the Event Dome a cheap alternative to a more traditional football stadium.

10.2. Recommendations

10.2.1. Actions on Structure

Wind loads Since the Eurocode is not fully applicable to a dome-shaped structure, extra tests should be performed to determine the exact wind loads on the structure. In this research assumptions are made for the wind loads on the structure, supported by other researches performed on similar structures. These other researches showed that the assumptions that are made in this research are safe representations for the wind loads on this structure. The determination of the wind loads is checked and agreed to by Prof.ir. A.C.W.M. Vrouwenvelder from the Delft University of Technology. For more realistic wind loads on this structure, wind tunnel tests or computational fluid dynamics calculations should be performed.

Asymmetrical loads In this research asymmetrical loads following from the Eurocode are used. There are, however, other asymmetrical loading patterns that can be thought of, like a chessboard-pattern loading. In this loading pattern elements are alternately loaded and not loaded. Numerous of these combinations can be found, and each combination should be examined to see whether it is a decisive combination. To further complete the loading on this structure, research must be performed to see which loading pattern is governing.

Accidental loads In this research only normal load cases are considered. For structures with a large consequence in case of failure, it is important to consider all possible load cases that can be present in the lifetime of the structure. Accidental loads, such as impact loads or explosion loads, should be taken into account to ensure the safety of this structure.

10.2.2. Roofing

EFTE cushions In this research EFTE cushions are used as cladding, because they are extremely lightweight. If EFTE cushions will be used, it is important that further research is performed to investigate the behaviour of these EFTE cushions. At this moment not too much information is available about, for instance, the aging of the cushions. Also, additional research must be performed to design the installations that are needed for these cushions.

EFTE cushions and daylight If a grass pitch will be present underneath the Event Dome, it is important that research will be performed to see how grass grows if only lighted through EFTE cushions. Grass needs a lot of light to sufficiently grow, or recover from a match. Because EFTE cushions filter the sunlight, it must be checked that this will not become a problem for the pitch.

10.2.3. Structural behaviour

Flexibility against structural behaviour A future client should be highly aware that the demand for flexibility does have large consequences on the structural behaviour of the design. As explained before the addition of extra columns decreases the total self-weight significantly. The low number of columns makes the span of the ring beam large. By adding extra columns in the middle of this span, the span is reduced and the required cross-sectional area of the ring elements is decreased as well.

The future client should be warned about the structural consequences of the design choices that are made.

Dynamics In this research little to no attention is given to the dynamics of the structure. Since the roof of the Event dome has a large span, one can imagine that dynamics or vibrations could become critical for the structure. Further research to investigate the dynamic behaviour of the Event Dome is needed, to conclude decisively that the structure is sufficiently strong and stiff.

Fatigue of steel Some of the connections in this design will have large internal forces. These internal forces will occur numerous of times. These joints are therefore susceptible to fatigue. Research should be performed to check the fatigue behaviour of the steel connections.

Structural robustness As was shown in the calculations in Chapter 9, not every member can be removed without the structure collapsing. Extra attention is needed to ensure structural robustness on these aspects. A possible solution could be to increase the cross-sectional dimensions of some elements, but extra research is needed to fully check the structural robustness of this structure, since the structure has a high risk category.

Joints A number of different cross-sections is used, each with its own diameter and wall thickness. Connections between two cross-sections with different dimensions could be hard. The joint efficiencies that are calculated in Chapter 6 are far below 1.0. This means that the joint should be stronger than the internal forces that will occur. Some joints will have large internal forces, which makes the design of the joints even more important. Therefore extra attention should be given to the design of the different joints.

Hinged joints Some of the joints that are present in this design are modeled as hinged joints. These hinged joints should have sufficient rotational capacity to prevent brittle failure of a joint. This rotational capacity should be taken into account in the design of the joints that are modeled as hinges.

Design of angled columns In this design composite columns are used for the angled columns. These composite columns are designed with the simplified design method from the Eurocode. For the final design of the columns it might be profitable to use a more extended design method, because this might decrease the cross-sectional dimensions of the columns. The accidental loads against which the angled columns should be resistant, should be determined, to accurately determine the required resistance of the angled columns.

Also, other options, like fixed column connections, should be researched to see whether they are favourable, because they decrease the buckling length, but increase the complexity of the supports and connections.

High-strength steel In this research only S355 steel is used. From the calculations it follows that the critical factor of each cross-section is the stability. Because the resistance for stability is not increased if steel of a higher strength is used, S355 is used for each cross-sections. It can, however, be favourable to use a higher-strength steel for some cross-sections. Especially for the structural safety this might be favourable. Therefore extra research must be performed to see if high-strength steel for some members could be favourable.

10.2.4. Construction

Little attention is given to the construction in this research. An example of a construction plan is given, but this construction plan is not elaborated extensively and serves more as a example how problems during construction can be solved.

The large number of connections in this design can make the construction time-consuming and expensive. Especially fixed connections and connections at a large altitude could be complicated.

Therefore a clever construction plan should be designed to decrease the risk and the complexity of the construction. Also, the joints should be designed in such a way that they can be constructed relatively easy at high altitude.

Part I

Lists

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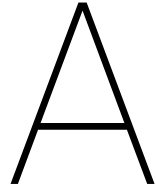
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Part II

Appendices



Cost Estimations

In this appendix the cost estimations of the Event Dome are shown. The first estimation has been done before the start of this thesis and was performed by the company Bremen Bouwadviseurs in Rotterdam. This cost estimation is done for a stadium with a capacity of 28.500 seats.

The second cost estimation is an extrapolation of the first cost estimation, where the costs for the roof have stayed the same, but all costs for grandstands, services and installations are doubled.

elementomschrijving	toelichting	hoeveelheid kosten/eenheid		totaal kosten
1 Grond				
A aankoop grond	nvt, behoort tot investeringskosten	40000		-
D sloopwerk	nvt	40000		-
totaal Grond				-
2 Bouwkundige werken				
A Onderbouw		40000		
A11.1 Peilen en maten	bouwrijp maken, afschuiven toplaag	40000	1	40.000,00
A11.3 grondverzet	grondwerk (gesloten grondbalans)	40000	5	200.000,00
	diversen grondwerk	1	60000	60.000,00
A16.1 funderingsconstructies	fundering/poeren kapconstructie	1	36000	36.000,00
	fundering tribune	15500	40	620.000,00
A17.1 paalfunderingen	palen onder kap (450*450*20m)	1440	35	50.400,00
	palen onder tribune (450*450*20m)	3200	35	112.000,00
totaal Onderbouw		27,96 /m2		1.118.400,00
B Skelet		40000		
B23.1 dragende vloeren	1e ring (staal + prefab beton)	8000	350	2.800.000,00
	2e ring (staal + prefab beton)	7500	350	2.625.000,00
	verplaatsbare tribune	1	150000	150.000,00
B23.2 secundaire vloeren	stoelen 1e ring	15000	50	750.000,00
	stoelen 2e ring	13500	50	675.000,00
B27.1 dragende daken	staalconstructie dak	38000	100	3.800.000,00
B95.1 diversen ruwbouw	trappenhuizen incl. bordesvloeren	6	50000	300.000,00
totaal Skelet		277,5 /m2		11.100.000,00
C Dak				
C27.1 dakafbouwconstructie	EFE overkapping	38000	250	9.500.000,00
totaal Dak				9.500.000,00
D Gevel				
D21.1 buitenwandafbouw	gevel rondom; tentdoek	22000	100	2.200.000,00
totaal Gevel				2.200.000,00
totaal Bouwkundige werken				23.918.400,00
3 Installaties				
A Werktuigbouwkundige installaties				
A59 Complete installaties	HWA-installatie stadion	1	100000	100.000,00
	W-installaties (sanitaire units; compleet)	12	10000	120.000,00
totaal Werktuigbouwkundige installaties				220.000,00
B Elektrotechnische installaties				
B69 Complete installaties	E-installaties stadion	15500	25	387.500,00
	E-installaties stadionverlichting	1	1000000	1.000.000,00
totaal Elektrotechnische installaties				1.387.500,00
C Transportinstallaties				
C66.1 Lift en transport	liftinstallatie stadion	nvt		
totaal Installaties				1.607.500,00
5 Terrein		40000		
A90 terrein	bestrating maaiveld	30000	35	1.050.000,00
	kunstgrasveld	8000	50	400.000,00
	verplaatsbare doelen	2	2500	5.000,00
	verplaatsbare dug-out's	2	4000	8.000,00
totaal Terrein				1.463.000,00
X Backbone	bouwkosten backbone: kantoren/ kleedruimtes/ horeca/ skyboxen, deels casco (skyboxen) - deels gebruiksklaar	8000	1000	8.000.000,00
totaal Backbone				8.000.000,00
6 Staartkosten				
A Algemene uitvoeringskosten		34988900	0,1	3.498.890,00
totaal Algemene uitvoeringskosten				3.498.890,00
B Opslagen	algemene kosten over 1D, 2, 4, 5 en 6A	28880290	0,07	2.021.620,30
	winst & risico over 1D t/m 6A + alg kosten	30901910	0,02	618.038,21
totaal Opslagen				2.639.658,51
C Bijkomende kosten	coördinatie installaties	1607500	0,03	48.225,00
totaal Bijkomende kosten				48.225,00
totaal Staartkosten				6.186.773,51
TOTALE BOUWKOSTEN EXCL. BTW		1444,76 /zitplaats		41.175.673,51
8 Bijkomende kosten				
	honoraria adviesdiensten	41175674	0,1	4.117.567,35
	legeskosten	41175674	0,0175	720.574,29
	financieringskosten	41175674	0	-
	onvoorzien	41175674	0,1	4.117.567,35
totaal Bijkomende kosten				8.955.708,99
TOTALE INVESTERINGSKOSTEN EXCL. BTW		1759,00 /zitplaats		50.131.382,49
Gemiddelde kosten		1759,00 /zitplaats		
		1253,28 /m2		

	elementomschrijving	toelichting	hoeveelheid	kosten/eenheid	totaal kosten
1	Grond				
A	aankoop grond	nvt, behoort tot investeringskosten	40000		-
D	sloopwerk	nvt	40000		-
	totaal Grond				-
2	Bouwkundige werken				
A	Onderbouw		40000		
A11.1	Peilen en maten	bouwrijp maken, afschuiven toplaag	40000	1	40.000,00
A11.3	grondverzet	grondwerk (gesloten grondbalans)	40000	5	200.000,00
		diversen grondwerk	1	60000	60.000,00
A16.1	funderingsconstructies	fundering/poeren kapconstructie	1	36000	36.000,00
		fundering tribune	31000	40	1.240.000,00
A17.1	paalfunderingen	palen onder kap (450*450*20m)	1440	35	50.400,00
		palen onder tribune (450*450*20m)	6400	35	224.000,00
	totaal Onderbouw		46,26 /m2		1.850.400,00
B	Skelet		40000		
B23.1	dragende vloeren	1e ring (staal + prefab beton)	16000	350	5.600.000,00
		2e ring (staal + prefab beton)	15000	350	5.250.000,00
		verplaatsbare tribune	1	150000	150.000,00
B23.2	secundaire vloeren	stoelen 1e ring	30000	50	1.500.000,00
		stoelen 2e ring	27000	50	1.350.000,00
B27.1	dragende daken	staalconstructie dak	38000	100	3.800.000,00
B95.1	diversen ruwbouw	trappenhuizen incl. bordesvloeren	12	50000	600.000,00
	totaal Skelet		456,25 /m2		18.250.000,00
C	Dak				
C27.1	dakafbouwconstructie	EFTE overkapping	38000	250	9.500.000,00
	totaal Dak				9.500.000,00
D	Gevel				
D21.1	buitenwandafbouw	gevel rondom; tentdoek	22000	100	2.200.000,00
	totaal Gevel				2.200.000,00
	totaal Bouwkundige werken				31.800.400,00
3	Installaties				
A	Werktuigbouwkundige installaties				
A59	Complete installaties	HWA-installatie stadion	2	100000	200.000,00
		W-installaties (sanitaire units; compleet)	24	10000	240.000,00
	totaal Werktuigbouwkundige installaties				440.000,00
B	Elektrotechnische installaties				
B69	Complete installaties	E-installaties stadion	31000	25	775.000,00
		E-installaties stadionverlichting	1	1000000	1.000.000,00
	totaal Elektrotechnische installaties				1.775.000,00
C	Transportinstallaties				
C66.1	Lift en transport	liftinstallatie stadion	nvt		
	totaal Installaties				2.215.000,00
5	Terrein		40000		
A90	terrein	bestrating maaiveld	30000	35	1.050.000,00
		kunstgrasveld	8000	50	400.000,00
		verplaatsbare doelen	2	2500	5.000,00
		verplaatsbare dug-out's	2	4000	8.000,00
	totaal Terrein				1.463.000,00
X	Backbone	bouwkosten backbone: kantoren/ kleedruimtes/ horeca/ skyboxen, deels casco (skyboxen) - deels gebruiksklaar	16000	1000	16.000.000,00
	totaal Backbone				16.000.000,00
6	Staartkosten				
A	Algemene uitvoeringskosten		51478400	0,1	5.147.840,00
	totaal Algemene uitvoeringskosten				5.147.840,00
B	Opslagen	algemene kosten over 1D, 2, 4, 5 en 6A	38411240	0,07	2.688.786,80
		winst & risico over 1D t/m 6A + alg kosten	41100027	0,02	822.000,54
	totaal Opslagen				3.510.787,34
C	Bijkomende kosten	coördinatie installaties	2215000	0,03	66.450,00
	totaal Bijkomende kosten				66.450,00
	totaal Staartkosten				8.725.077,34
TOTALE BOUWKOSTEN EXCL. BTW			2112,40 /zitplaats		60.203.477,34
8	Bijkomende kosten				
		honoraria adviesdiensten	60203477	0,1	6.020.347,73
		legeskosten	60203477	0,0175	1.053.560,85
		financieringskosten	60203477	0	-
		onvoorzien	60203477	0,1	6.020.347,73
	totaal Bijkomende kosten				13.094.256,32
TOTALE INVESTERINGSKOSTEN EXCL. BTW			2571,85 /zitplaats		73.297.733,66

Gemiddelde kosten	1285,93 /zitplaats
	1832,44 /m2

B

Literature Study

B.1. Literature

B.1.1. Wind loads on catenary structures [Bradley et al., 2016]

Some options that will be considered are catenary structures. The determination of the wind loads on catenary structures can be tough, since this isn't covered by the Eurocode completely. To make a well-educated guess for the wind loads on catenary structures, literature has been found. In South Africa a research is conducted that has compared the wind tunnel test results for catenary structures with the results for wind tunnel tests for a duo-pitch structure. The results of this research show that the wind loads on a catenary structure are lower than the wind loads on a duo-pitch structure. This means that the wind loads of duo-pitch structures can be used for the design of a catenary structure. Because the wind loads of the duo-pitch structure are actually higher, the use of these wind loads is a safe approach.

B.1.2. Requirements football stadiums [FIFA, 2011]

One of the functions that the Event Dome should be able to include is the function of a football stadium. The world football association (FIFA) has set up requirements that each international football stadium must meet. For this design the most important requirement is that at least 30 meters of free height is required above the pitch.

B.2. Reference Projects

B.2.1. Cargolifter Airship Hangar [Barnes and Dickson, 2000]

The Cargolifter Airship Hangar is an airship hangar, finished in the year 2000. The hangar is 360 m long, 220 m wide and 106 m high, which made it the largest freestanding hall in the world. The structure spans in the 220 m direction, using a three-hinge system with steel truss arches. The structure is clad with two layers of PVC fabrics to enclose the hall.

Each pair of truss arches was built at the same time while supporting the free sides until the two arches are connected.

After insolvency of the Cargolifter company in 2002 the hangar was transformed to a tropical theme park. For this transformation parts of the PVC fabrics are replaced by translucent EFTE cushions. For these EFTE cushions a cable net structure is designed to make the required spans in the EFTE possible.

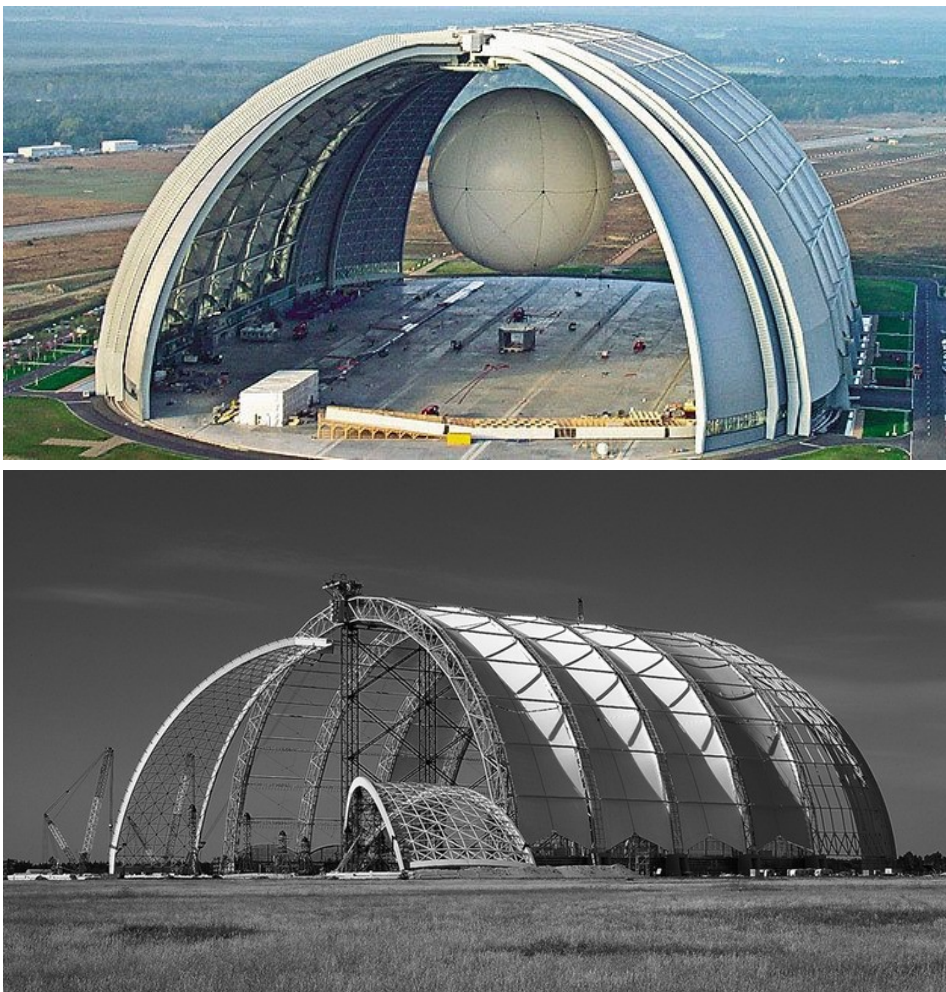


Figure B.1: Cargolifter Airship Hangar

B.2.2. Messe Frankfurt – Halle 3 [Barnes and Dickson, 2000]

Hall 3 of the Messe Frankfurt is a trade fair hall. The suspended roof spans 165 m and consists of five adjacent groups of steel arches spanning in one direction. These arches are connected by a net of tendons and supported on each side by six A-shaped trestles to transfer the thrust forces to the foundation.

About 4200 tons of structural steel is used for the structure of this hall. With an area of about 20.000 m² this gives a material usage of 210 kg/m².

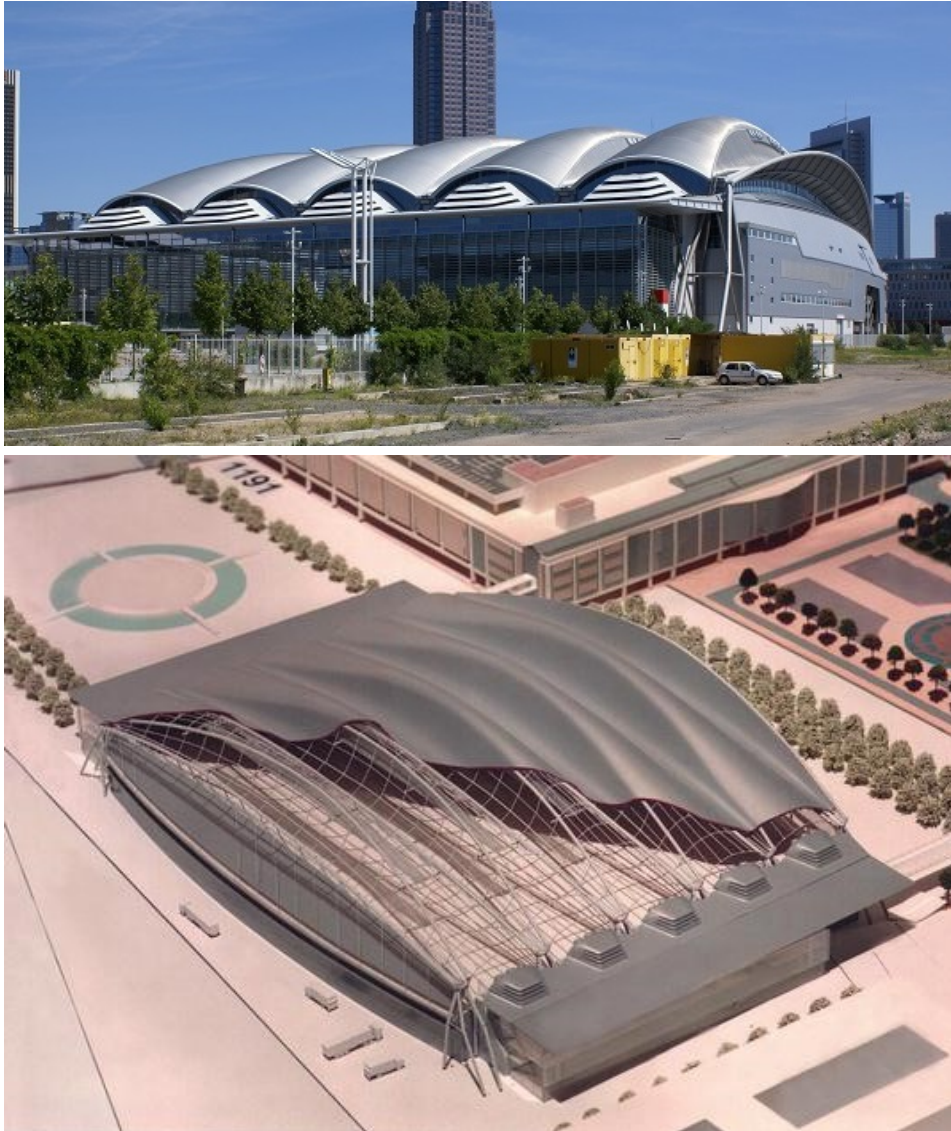


Figure B.2: Messe Frankfurt - Halle 3

B.2.3. Singapore National Stadium [StadiumDB, 2016]

The Singapore National Stadium is a multi-purpose stadium, which is mostly used for football matches. With a retractable bottom tier the stadium has multiple seating arrangements, to host football, rugby, cricket, athletics and concerts. The stadium also has a retractable roof which can open or close in approximately 25 minutes. The roof is made out of ETFE. At night this roof can function as a giant projector screen.

The main members in the dome-shaped roof structure are 5 3d-trusses. These trusses span in the lateral direction of the stadium with a maximum span of 310 meter. These main members are the only structural members in the open part of the roof. In the closed part secondary 3d-trusses span in multiple directions between the main members.

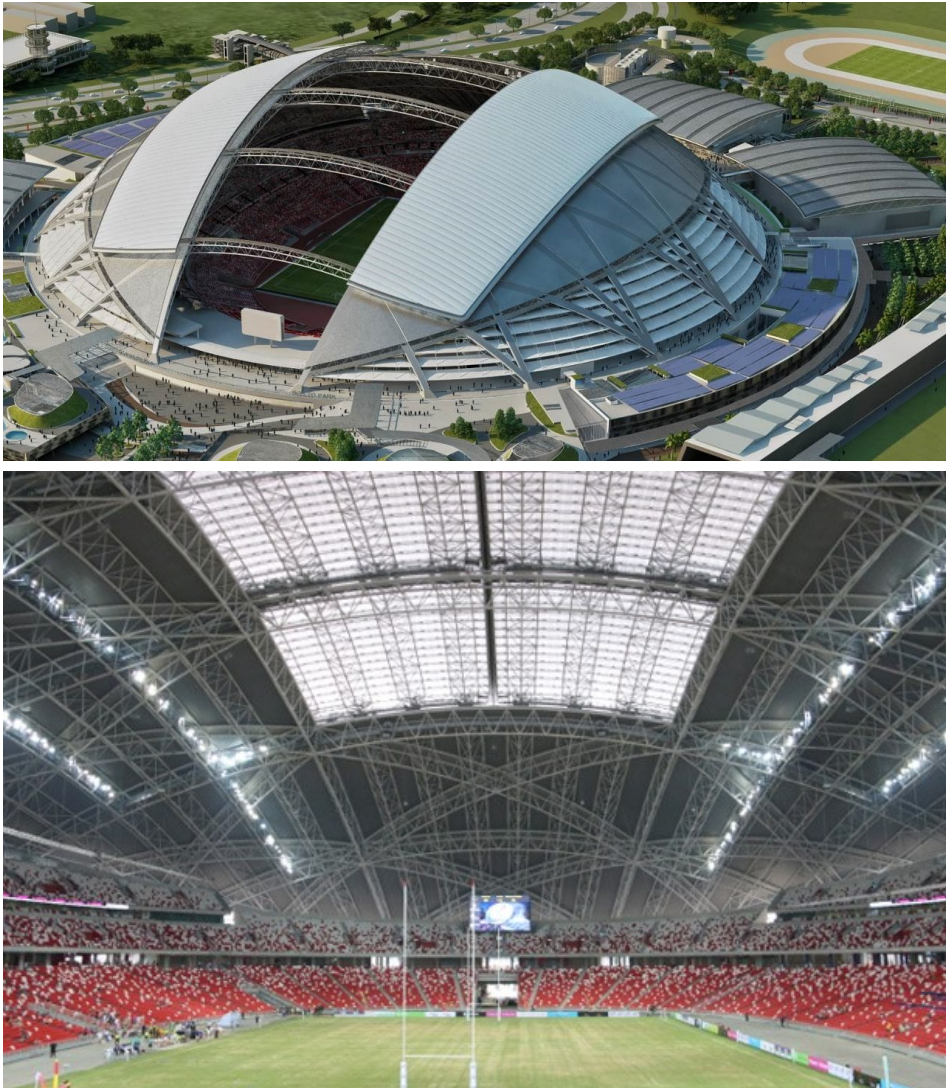


Figure B.3: Singapore National Stadium

B.2.4. Houston Astrodome [Bass, 1965]

The Houston Astrodome is a multi-purpose domed sports stadium. It is opened in 1965 and was the world's first domed sports stadium. The span of the dome is 196 meters. The roof of the stadium was completely closed with over 4000 skylights to let sunlight in the stadium. The skylights smeared the sun into a blinding wall of light. This gave a lot of problems for the sports events that were hosted in the stadium. To block the sun they painted the skylights, but this made the grass die from lack of sunlight. As a solution artificial grass was installed.

The structure works as a dome that is raised above the ground. At the edge of the dome there is an enormous tension ring to provide stability to the dome. This tension ring is supported by columns. The connection between the column and the tension ring is rigid in the tangential direction, but the columns are free to move radially. The entire dome is divided into 12 triangles by using radial members. These triangles are subdivided into smaller triangles by using members in 3 directions, including the tangential direction. The diameter of the dome is about 220 meters. The total weight of the roof structure, including the tension ring is 81 kg/m². Excluding the tension ring the self-weight of the roof structure is about 69 kg/m².



Figure B.4: Houston Astrodome

B.2.5. National Stadium Warsaw [StadiumDB, 2016]

The National Stadium Warsaw is the national football stadium of Poland. The stadium has a unique retractable roof.

The structure of the roof is built up from mostly tension rods and cables. The fixed part of the roof is supported by cables that are anchored to the ground by tension rods.

The retractable part of the roof consists of cables that are spanning from the centre spire to the edge of the fixed roof. These cables function as rails for the foldable roof. The foldable part of the roof is made out of fibreglass covered with Teflon. If the roof is opened the fibreglass roof is folded into the centre spire.

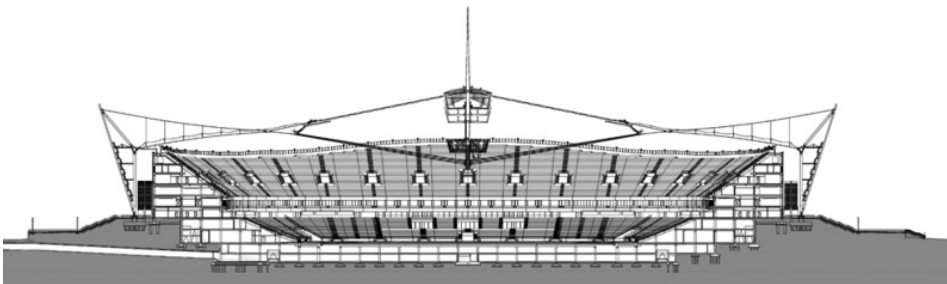
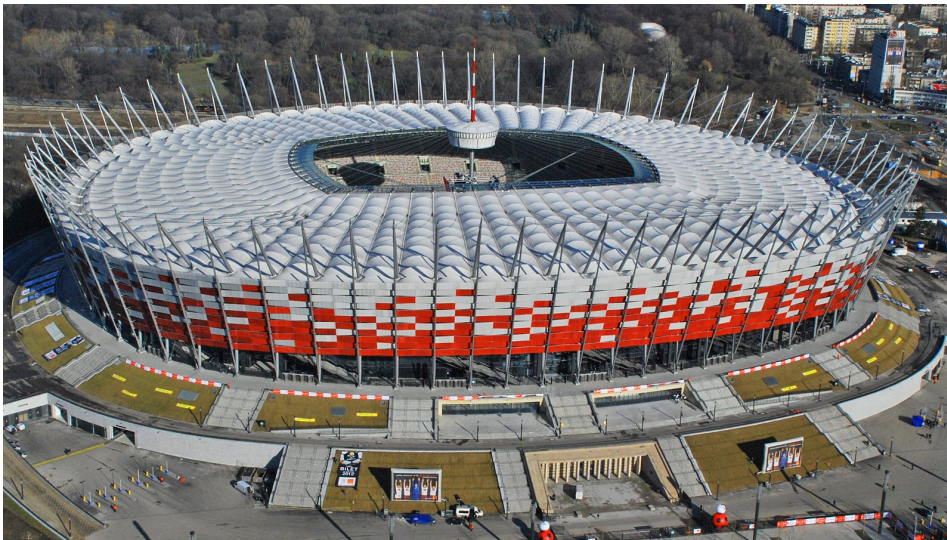


Figure B.5: National Stadium Warsaw

B.2.6. BayArena Leverkusen [StadiumDB, 2016]

The BayArena in Leverkusen, Germany is the stadium of football club Bayer 04 Leverkusen. In 2008 the stadium has had an extensive redevelopment. A second tier was built above the existing stands and an impressive roof was constructed. The roof structure consists of an outer truss ring beam, an inner ring beam and Y-shaped cables that span between the inner ring beam and both chords of the outer ring beam. The outer ring beam is vertically supported by 8 V-shaped columns.

The roof cladding is supported by the bottom part of the Y-shaped cables that span between the two ring beams.

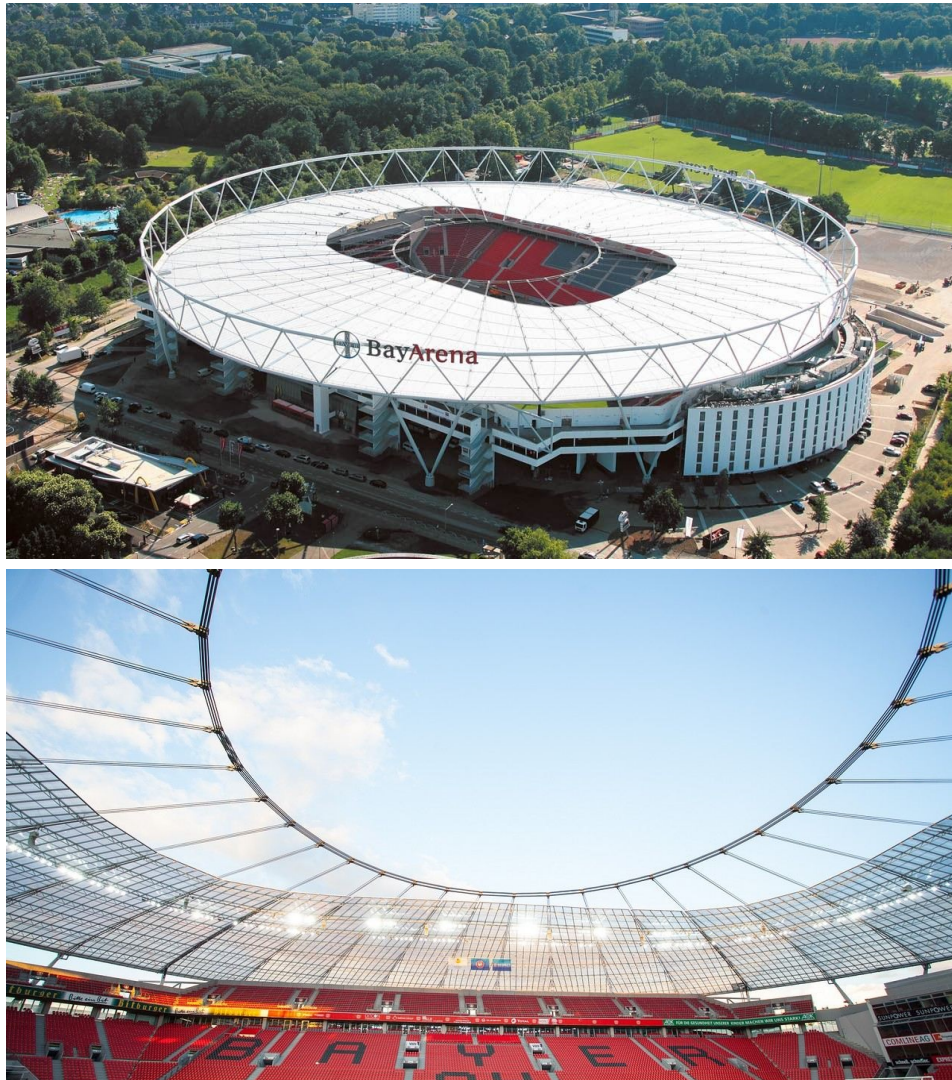


Figure B.6: BayArena Leverkusen

B.2.7. New Wembley Stadium [StadiumDB, 2016]

The Wembley Stadium is a football stadium in London that opened in 2007. With 90.000 seats it's the second largest stadium in Europe. The stadium has a retractable roof.

The biggest eye catcher is the Wembley Arch. This 134 meter high arch supports a big part of the roof by cables, and some other parts by being one support for cable-truss-beams.

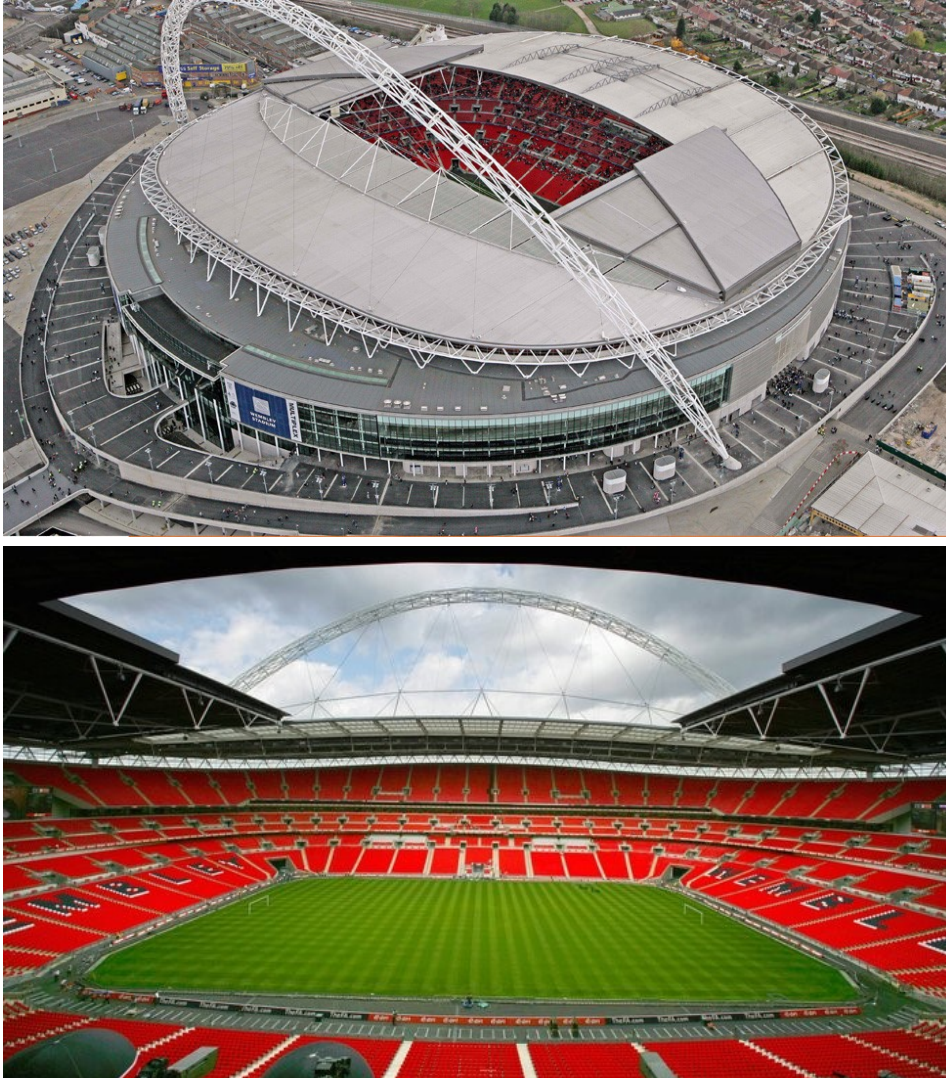


Figure B.7: New Wembley Stadium

B.2.8. AT&T Stadium Arlington [StadiumDB, 2016]

The AT&T Stadium, former known as the Cowboys Stadium is an American Football Stadium with a retractable roof in Arlington, USA. The facility is also used for other functions, such as concerts, basketball games, soccer games and motocross.

The stadium is one of the largest dome structures in the world. On both short sides of the stadium the dome continues to the ground with two space trusses on each side. On both long sides the dome structure stops and is supported on columns and underlying facilities.

The main structural elements in the dome are two 91 meter tall arches that span the length of the stadium dome, almost 374 meters. On both sides the arches are anchored in the ground.



Figure B.8: AT&T Stadium Arlington

B.2.9. Georgia Dome [StadiumDB, 2016]

The Georgia Dome is a domed American Football stadium in Atlanta in the United States. The stadium is also used for other sports and concerts. The dome structure doesn't work as a regular dome structure, where most elements are loaded in compression. In the Georgia Dome the main structural system is a tensegrity system, which means that the continuous elements are loaded in tension, supported by some compression members. In this system no members will experience a bending moment.

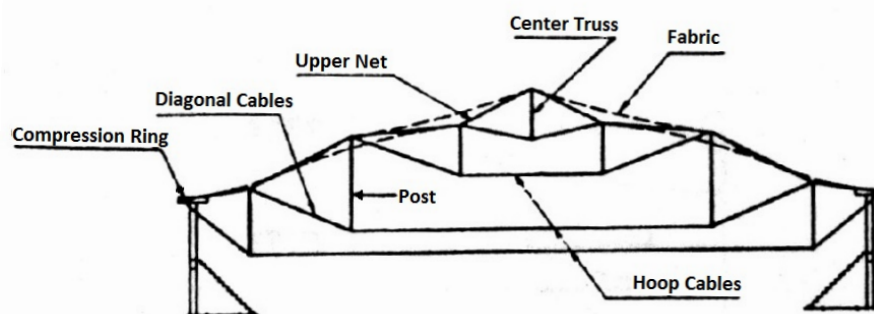
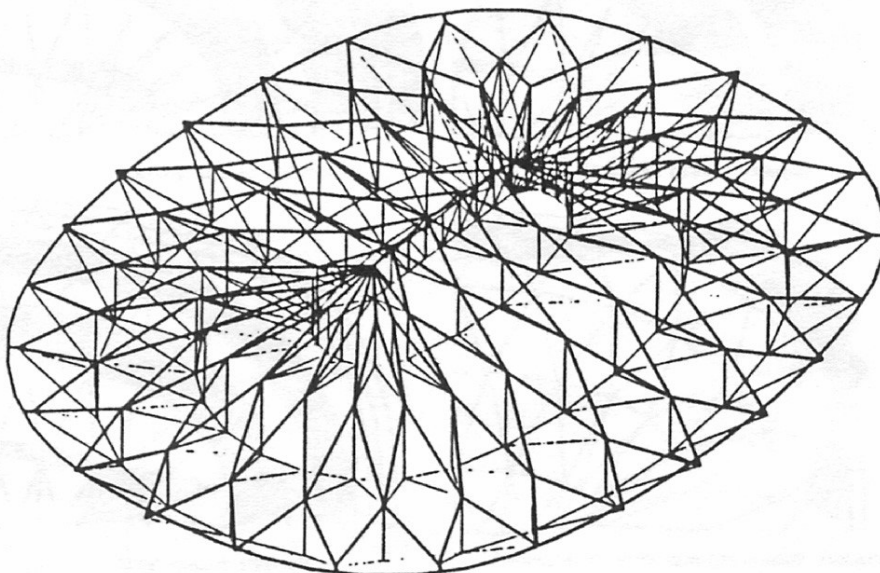
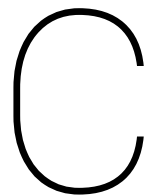


Figure B.9: Georgia Dome



Grasshopper model

In this appendix a description of the Grasshopper model is given. The model for the final calculations is used, because this is an extended version of the Grasshopper model for the preliminary calculations. In the preliminary calculations multiple options are calculated. The procedure for each of the options is similar to the one that is described in this appendix, so only the Grasshopper model for the final design is shown in this appendix.

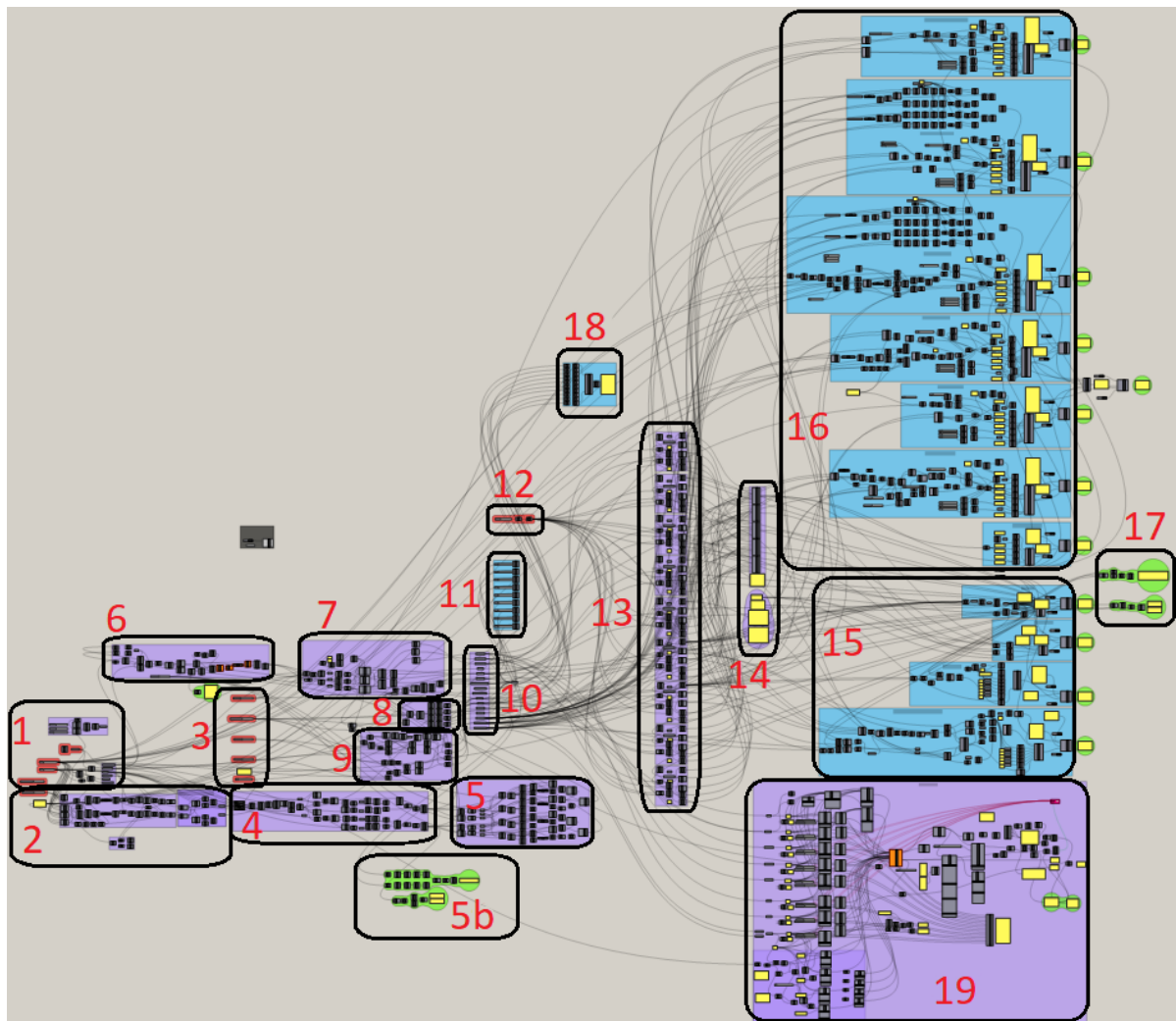


Figure C.1: Overview of Grasshopper model

Figure C.1 shows an overview of the total Grasshopper model. In the Grasshopper model 4 component colours are used. The component colour is dependent on the function of that component. The component colours that are used are:

Red Red groups show the input of the model. This input is mostly modeled with number sliders, so the input can easily be adjusted in the optimisation of the design.

Purple Purple groups are computing components. These computing groups create the model from all input through several computations.

Green The green groups are checking groups. Each model has its boundary conditions. The wind load that is used in this model, for instance, is only applicable for a certain range of diameter and dome height. The green groups show checks whether the model is within its boundary limits and whether the model is correct. This correctness holds, for instance, that all members connect to each other and no points are too close to each other.

Blue The blue groups are output groups. These groups represent the data or geometry that can be outputted to SCIA or Rhino.

The Grasshopper model is divided into 19 groups of components. Each of these groups has a separate function, which will be described in this appendix. Each of the groups will be described briefly, where attention will be given to the points of attention of each group. Since some parts of the model are quite straight-forward, there is no point in describing each component individually.

Group 1

In group 1 the base geometry is described. For this design that means that the shape of the roof is modeled, together with its height and diameter. Some important aspects, such as the middle point of the circle, are created here for further use in the model.

Group 2

In group 2 a rectangular grid of beams is created on the flat circle, with the centre-to-centre distance of the beams as input. This grid of beams is then trimmed with the circle that was created in group 1. Next the spanning beams are shattered at the intersections to create individual beams. This is necessary for the Karamba and SCIA calculations.

Group 3

Group 3 shows some basic input data for the model. The input that is given in group 3 is for instance the rise of the dome, the centre-to-centre distance of the ring truss and the number and angle of the columns.

Group 4

In group 4 the geometry of the roof is created. First a catenary curve is created, which is rotated around the middle point of the circle. This rotation creates a catenary surface. The next step is to combine the catenary surface and the rectangular grid of beams to create a grid of beams that follow the shape of the catenary surface. This can be done by either 'pulling' or 'projecting'. Pulling means that each beam is pulled on the closest points of the surface, while projecting means that the beams are moved in a specified direction until it reaches the catenary surface. Since the centre-to-centre distance in this design is preferably kept equal, projecting is used in this design to create the arches.

Group 5

Because of the small imperfections that come along with creating the catenary surface and projecting the curves on this surface, the edge beams are remodeled to intersect with the circle at the specified height. This imperfection does not always occur, but dependent on the input that is used, the maximum imperfection that is found in this research was about 20 mm. This imperfection will not change the structural behaviour significantly, but it does influence the Karamba and SCIA calculations, because the beams don't intersect, and thus are not connected.

Group 5b

Group 5b checks the imperfections that are described in group 5. The size of this imperfection is checked, and the remodeled members are checked to see if they intersect with the circle at the specified height. Also the angle of the edge beams are checked to see the maximum angle of the roof.

Group 6

In group 6 the end points of the spanning arches are used to find the input nodes of the ring truss. In this group these points are also checked to check the minimal distance between two points. If this distance is too low, the points are combined. The members that used these points are remodeled with the combined point as new end point.

Group 7

In group 7 the ring truss is modeled. The nodes that are specified in group 6 are used as input, together with the centre-to-centre distance from group 3. The output of this group are all members of the truss, grouped in chord, verticals and diagonals.

Group 8

In group 8 the vertical columns are modeled. The number of vertical columns is specified in group 3. The columns are spread evenly among the perimeter of the roof. The columns are modeled to always connect to a node of the ring truss.

Group 9

In group 9 the angled columns are designed. The number of columns, and the angle of the columns are specified in group 3. Again, the columns are spread evenly among the perimeter of the roof and will always connect to a node of the ring truss.

Group 10

In group 10 all members are grouped according to their function.

Group 11

Group 11 creates pipes in Rhino that can be used for renderings of the model. Each of the member groups has an individual number slider for the diameter of the pipe. This group is not used for the calculations, but for graphic use only.

Group 12

Group 12 is an input group that specifies the tolerance that is used in the calculations. This tolerance is important since some computations in Grasshopper lead to non-exact results. The calculations software will see two points as different, even if there is a difference of 0.01 mm. Therefore a tolerance is introduced to make sure that there won't be any problems with points or members not coinciding. In this research a tolerance of 1 cm is used.

Group 13

In group 13 each of the members is prepared for Grasshopper. Instead of defining members as geometry, each member is now described with its end points, type of member, member name and cross-section. The end points that are used are found by getting the end points of each curve, and looking those values up in the total set of nodes. In this way all members use the same set of nodes. Members will therefore automatically be connected if they have the same begin or end node.

Group 14

Group 14 groups some important aspects of the geometry, such as member name, end points, type of member and cross-section. This grouping is needed so that all members can be exported to SCIA at once.

Group 15

Group 15 is the output group to export the geometry to SCIA. In this group all nodes, members, connections, and support points are exported to Excel-sheets. These Excel-sheets can be copied to SCIA directly, because the layout of SCIA is used in the modeling of these Excel-sheets.

If, for instance, nodes are considered, SCIA has a table input of 4 columns: node name, X-coordinate, Y-coordinate and Z-coordinate. The Excel-sheet that is created in Grasshopper, therefore also has 4 columns, with the same lay-out as the one in SCIA. The required information for these Excel-sheets is taken from group 13 and 14, and is then re-organized to fit the SCIA table input.

Group 16

Group 16 is the output group to export the loads to SCIA. In this group, the loads are calculated automatically from the geometry of the structure. If the height changes, for instance, the loads are changed accordingly. Since the wind loads act perpendicular to the surface of the roof, the wind loads are rotated. This rotation is also calculated automatically from the geometry of the structure. All export from the different load types are combined into one Excel-sheet, so all loads can be exported to SCIA in one go.

Group 17

Group 17 is a checking group. This group checks the minimal distance between two points. This check is different from the one that is performed in group 6, because now the node coordinates for SCIA are used. The other check that is performed in this group is the check whether the structure falls inside the boundary conditions that are stated by the Eurocode or otherwise. The boundary conditions that are stated by the Eurocode are the ranges for which the determination of the loads is correct. If the geometry of the structure is not within the boundary conditions, the loads should be determined with other formulas than the one used in this Grasshopper model.

Group 18

Group 18 is an output group that gives the total length of each of the cross-sections. In this way the self-weight of each of the total structure can be divided into the self-weight of each cross-section. With this, the weight percentage of, for instance, the ring truss, can be determined. This group is not used for SCIA or Karamba calculations, but for manual calculations only.

Group 19

Group 19 is the group where the Karamba module is. Because this group is a bit more extensive than the other groups, this group will also be described more extensively. In Figure C.2 an overview of the Karamba module can be found. In this module new groups are specified, which in this appendix will be named with group 19.x, where x is the number of the group in Figure C.2.

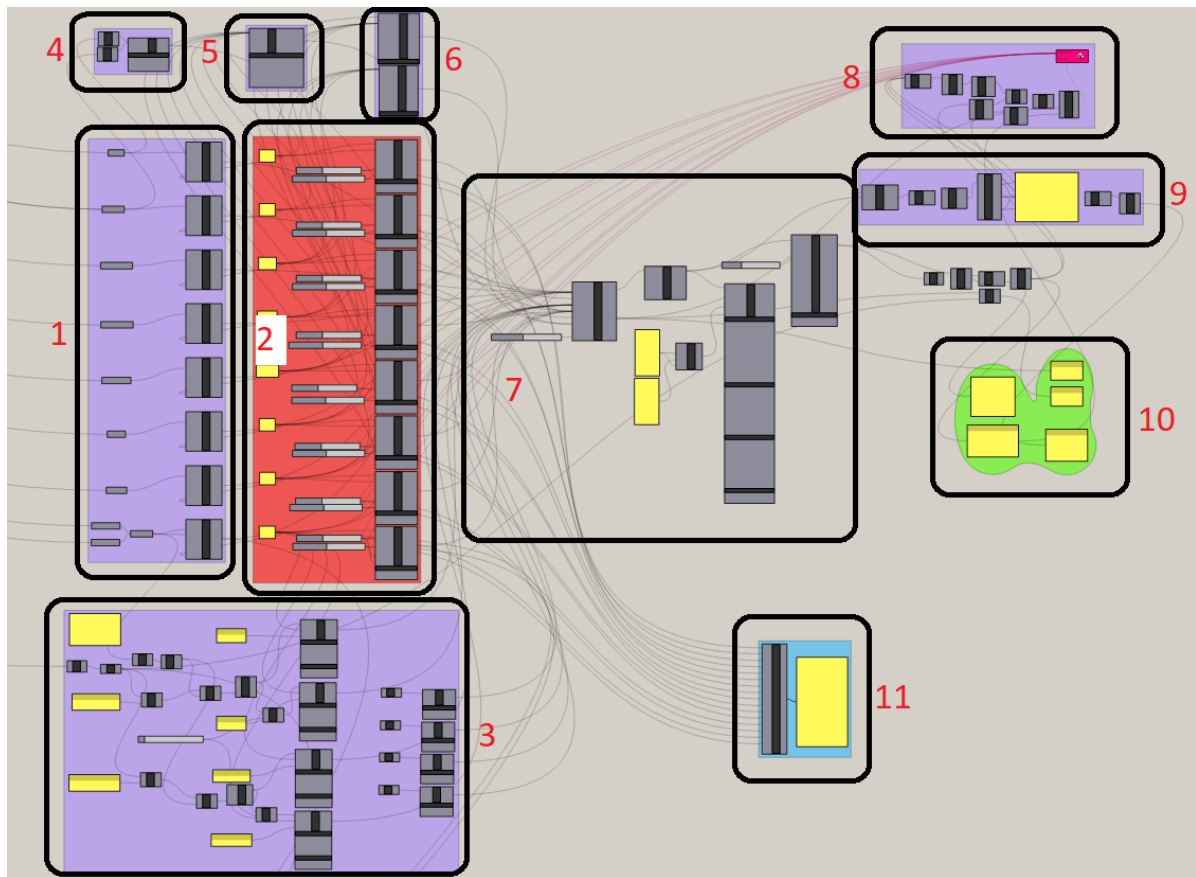


Figure C.2: Overview of Karamba module

Group 19.1

Group 19.1 transforms the line into Karamba elements. Each line type is transformed into a separate element group. These Karamba elements are the geometry of the Karamba model.

Group 19.2

Group 19.2 shows the cross-sections of each of the member groups. The dimensions of the cross-sections are modeled as number sliders. In this way the dimensions can easily be adjusted, and can also be used in a Galapagos optimisation.

Group 19.3

Group 19.3 represents the load input for Karamba calculations. Different load cases are used, which together form different load combinations. For the values of these loads approximated values are used.

Group 19.4

Group 19.4 shows the support points. The support conditions can be adjusted here as well. The support conditions tell whether the support is hinged or fixed in x-, y- and/or z-direction.

Group 19.5

Group 19.5 represents the material selection.

Group 19.6

Group 19.6 represents the connections. This module has two member types as input. The connection between these two member types can be hinged or fixed in x-, y- and/or z-direction. For every connection different conditions can be specified.

Group 19.7

Group 19.7 represents the calculations module of Karamba. First all input members (group 19.1-19.6) are assembled. Then a first-order or second-order calculations can be performed. The results of this calculation can then be found by some Karamba modules, such as utilization, displacements or model view.

Group 19.8

Group 19.8 is the Galapagos optimisation module. This optimisation module optimises the dimensions of the cross-sections, to minimize a certain value. The value that has to be minimized in this design is the total weight of the structure, whilst still fulfilling its structural job. The requirements that are used to check whether the structure is still sufficiently strong and stiff are the utilization and the maximum displacement. The utilization must be lower than 1.0, and the maximum displacement must be lower than 800 mm. If both these conditions are met, the total weight can be minimized.

Group 19.9

This group shows the maximum displacements for the different load combinations. The maximum displacements for the SLS load combinations are used in group 19.8 to check the condition that the maximum displacement must be lower than 800 mm.

Group 19.10

This group shows the results of the calculations. The results that are shown here are the total self-weight, the self-weight per m², the maximum displacement, the maximum utilization and the member for which the utilization is the highest.

Group 19.11

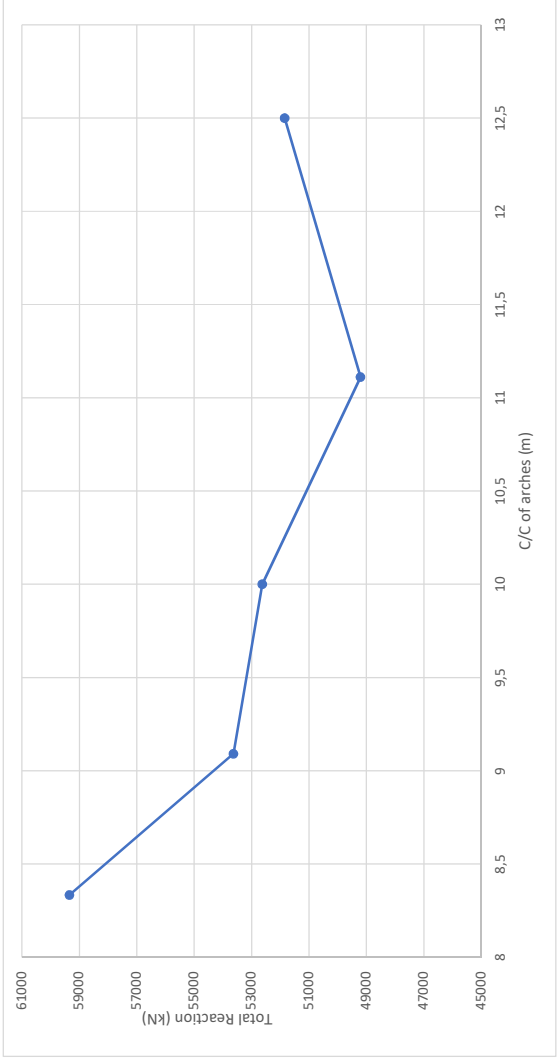
Group 19.11 shows all cross-sectional dimensions in a single list. The dimensions that are shown are the diameter and wall thickness of each member type. This group is not used for the Karamba calculations, but it is used for manual calculations only.

D

Results Optimisation

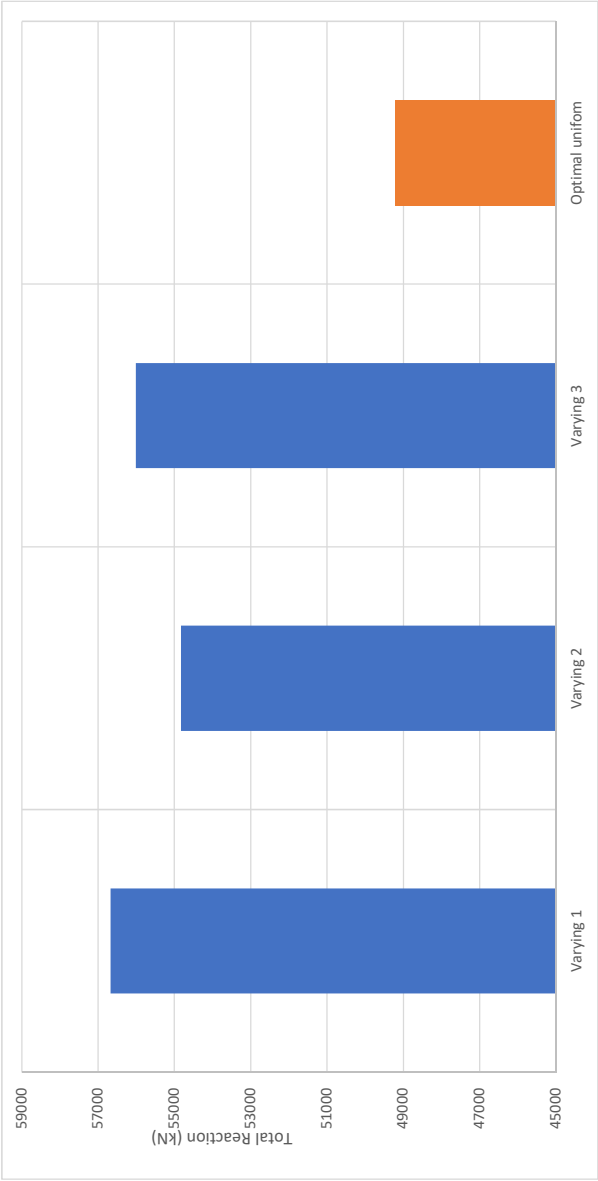
Centre-to-centre distance of arches

C/C	nr. of arches	12,5					11,11111111					10					9,090909091					8,333333333											
		16					18					20					22					24											
		L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)				
CS2	W-columns	705	2134	22,2	1,06E+11	713	2134	22,2	1,07E+11	708	2134	22,2	1,06E+11	719	2134	22,2	1,08E+11	698	2134	22,2	1,05E+11												
CS3	V-columns	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10												
CS4	chord inner	628	508	6,4	6,5E+09	628	660	6,4	8,41E+09	628	660	6,4	8,41E+09	628	610	6,4	7,78E+09	628	762	6,4	9,7E+09												
CS5	truss vert	1.664	457	6,4	1,55E+10	1.909	457	6,4	1,78E+10	1.909	457	6,4	1,78E+10	2.401	457	6,4	2,24E+10	2.401	457	6,4	2,24E+10												
CS6	truss diag	2.763	711	6,4	3,99E+10	2.924	610	6,4	3,62E+10	2.909	559	6,4	3,31E+10	3.273	508	6,4	3,39E+10	3.289	559	6,4	3,74E+10												
CS7	X	2.772	2083	9,5	1,73E+11	3.126	1829	9,5	1,72E+11	3.480	1930	9,5	2,01E+11	3834	1676	10,3	2,09E+11	4188	1727	10,3	2,35E+11												
CS8	Y	2.772	2083	9,5	1,73E+11	3.126	1981	9,5	1,86E+11	3.480	1981	9,5	2,07E+11	3834	1626	10,3	2,03E+11	4188	1778	9,5	2,23E+11												
CS9	chord bottom	628	2032	9,5	3,83E+10	628	1778	10,3	3,63E+10	628	1778	10,3	3,63E+10	628	1930	9,5	3,64E+10	628	1727	10,3	3,53E+10												
CS10	chord outer	684	1981	20,6	8,86E+10	685	2083	9,5	4,28E+10	685	1778	11,1	4,27E+10	685	1880	11,1	4,52E+10	685	2083	15,9	7,18E+10												
CS11	vert above col	180	2083	14,3	1,7E+10	180	2134	14,3	1,74E+10	180	2134	12,7	1,54E+10	180	2032	14,3	1,65E+10	180	2083	14,3	1,7E+10												
		Total volume (mm3) 6,83E+11					Total volume (mm3) 6,48E+11					Total volume (mm3) 6,93E+11					Total volume (mm3) 7,07E+11					Total volume (mm3) 7,82E+11											
		Total weight (kg) 5358318					Total weight (kg) 5087148					Total weight (kg) 5440599					Total weight (kg) 5550514					Total weight (kg) 6138914											
		Total reaction (kN) 52565,1					Total reaction (kN) 49904,93					Total reaction (kN) 53372,28					Total reaction (kN) 54450,54					Total reaction (kN) 60222,75											
		Reaction SCIA (kN) 51846,88					Reaction SCIA (kN) 49204,06					Reaction SCIA (kN) 52629,56					Reaction SCIA (kN) 53631,49					Reaction SCIA (kN) 59342,81											



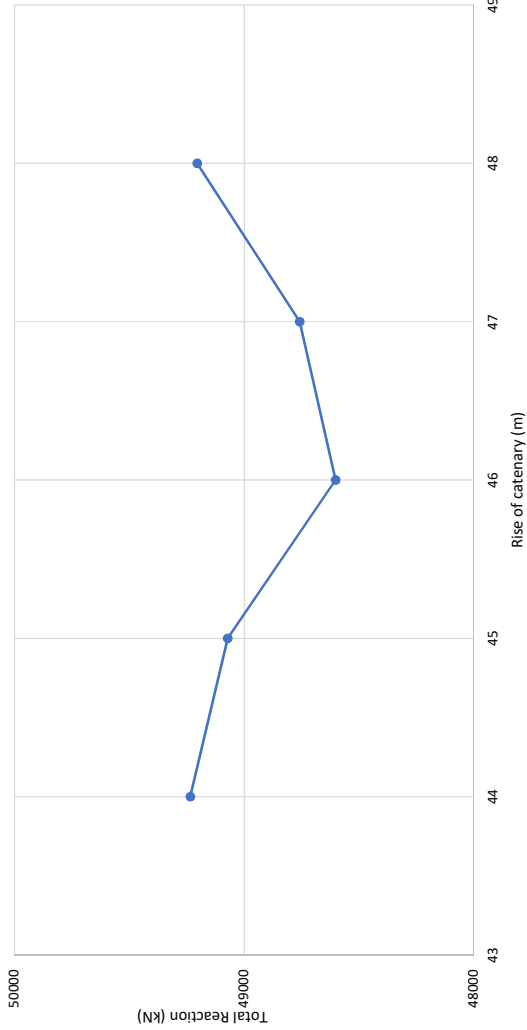
Varying centre-to-centre distance arches

C/C nr. of arches	Varying 1 15				Varying 2 17				Varying 3 19				Optimal for uniform c/c 18					
	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)		
CS2	699	2134	22,2	1,05E+11	697	2134	22,2	1,05E+11	701	2134	22,2	1,05E+11	713	2134	22,2	1,07E+11		
CS3	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10		
CS4	628	610	6,4	7,78E+09	628	660	6,4	8,41E+09	628	508	6,4	6,5E+09	628	660	6,4	8,41E+09		
CS5	1418	610	7,9	2,17E+10	1909	457	6,4	1,78E+10	1909	508	6,4	1,97E+10	1.909	457	6,4	1,78E+10		
CS6	2548	711	7,9	4,55E+10	2929	660	6,4	3,92E+10	2879	610	6,4	3,57E+10	2.924	610	6,4	3,62E+10		
CS7	2879	2083	11,1	2,1E+11	3295	1981	9,5	1,96E+11	3660	1880	9,5	2,06E+11	3.126	1829	9,5	1,72E+11		
CS8	2879	1930	11,1	1,95E+11	3295	1930	9,5	1,91E+11	3660	1829	9,5	2,01E+11	3.126	1981	9,5	1,86E+11		
CS9	628	1524	14,3	4,34E+10	628	1829	11,9	4,32E+10	628	1829	11,9	4,32E+10	628	1778	10,3	3,63E+10		
CS10	684	1524	22,2	7,38E+10	684	1930	17,5	7,32E+10	685	1524	22,2	7,39E+10	685	2083	9,5	4,28E+10		
CS11	180	2134	17,5	2,13E+10	180	2134	20,6	2,51E+10	180	2083	19,1	2,27E+10	180	2134	14,3	1,74E+10		
Total volume (mm3)				7,48E+11	Total volume (mm3)				7,23E+11	Total volume (mm3)				Total volume (mm3)				6,48E+11
Total weight (kg)				5874442	Total weight (kg)				5675184	Total weight (kg)				Total weight (kg)				5087148
Total reaction (kN)				57628,28	Total reaction (kN)				55673,56	Total reaction (kN)				Total reaction (kN)				49904,93
Reaction SCIA (kN)				56674,13	Reaction SCIA (kN)				54824,47	Reaction SCIA (kN)				Reaction SCIA (kN)				49204,06



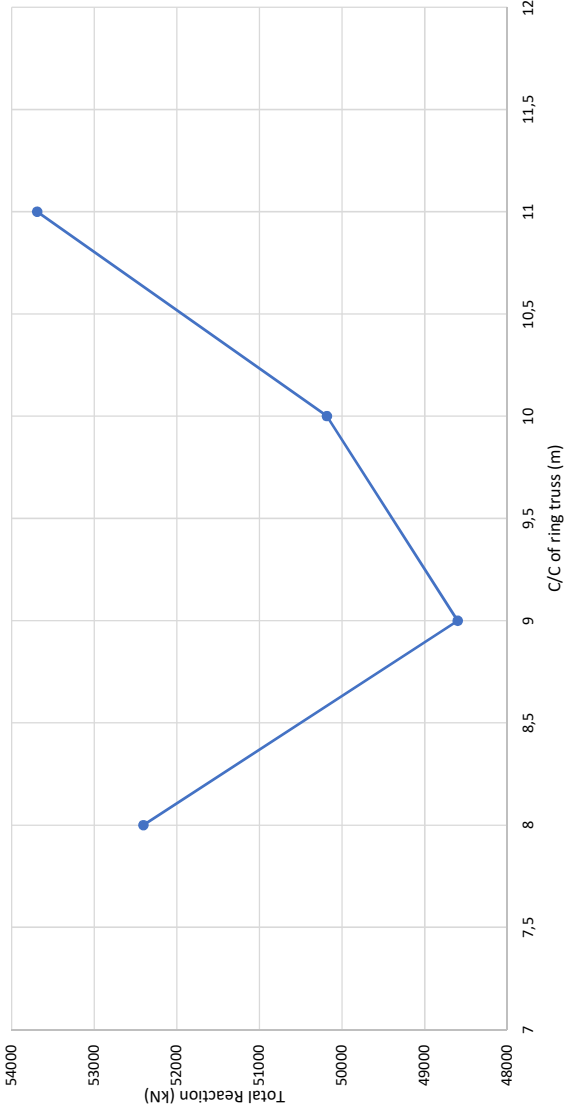
Rise of catenary surface

Rise	44				45				46				47				48			
	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)
CS2 W-columns	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11
	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10
	628	508	10,3	1,05E+10	628	508	10,3	1,05E+10	628	508	10,3	1,05E+10	628	711	6,4	9,06E+09	628	660	6,4	8,41E+09
	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10
	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10
CS5 truss vert	3071	1880	9,5	1,73E+11	3084	1880	9,5	1,74E+11	3098	1829	9,5	1,7E+11	3112	1829	9,5	1,71E+11	3126	1829	9,5	1,72E+11
CS7 X	3071	1930	9,5	1,78E+11	3084	1930	9,5	1,79E+11	3098	1930	9,5	1,79E+11	3112	1778	10,3	1,8E+11	3126	1981	9,5	1,86E+11
CS8 Y	628	1880	9,5	3,54E+10	628	1880	9,5	3,54E+10	628	1829	9,5	3,45E+10	628	1880	9,5	3,54E+10	628	1778	10,3	3,63E+10
CS9 chord bottom	685	2134	10,3	4,75E+10	685	2134	9,5	4,38E+10	685	2032	9,5	4,17E+10	685	2083	9,5	4,28E+10	685	2083	9,5	4,28E+10
CS10 chord outer	180	2032	15,9	1,84E+10	180	2032	15,9	1,84E+10	180	2032	15,9	1,84E+10	180	2032	15,9	1,84E+10	180	2134	14,3	1,74E+10
CS11 vert above col	Total volume (mm3)				Total volume (mm3)				Total volume (mm3)				Total volume (mm3)				Total volume (mm3)			
	Total weight (kg)				Total weight (kg)				Total weight (kg)				Total weight (kg)				Total weight (kg)			
	Total reaction (kN)				Total reaction (kN)				Total reaction (kN)				Total reaction (kN)				Total reaction (kN)			
	Reaction SCIA (kN)				Reaction SCIA (kN)				Reaction SCIA (kN)				Reaction SCIA (kN)				Reaction SCIA (kN)			



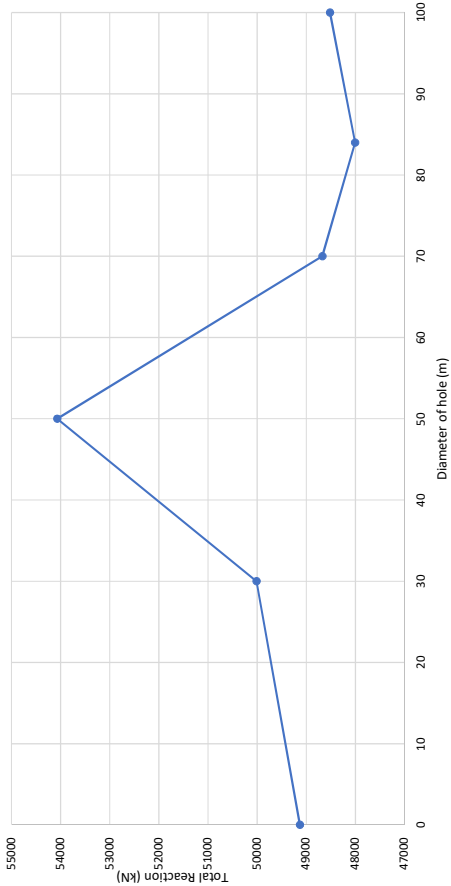
Centre-to-centre distance of ring truss

C/C	8				9				10				11			
	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)
CS2 W-columns	728	2134	22,2	1,09E+11	713	2134	22,2	1,07E+11	698	2134	22,2	1,05E+11	683	2134	22,2	1,03E+11
CS3 V-columns	168	2134	22,2	2,53E+10	164	2134	22,2	2,47E+10	160	2134	22,2	2,41E+10	156	2134	22,2	2,35E+10
CS4 chord inner	628	457	6,4	5,85E+09	628	508	10,3	1,05E+10	628	711	6,4	9,06E+09	628	508	6,4	6,5E+09
CS5 truss vert	1697	457	6,4	1,58E+10	1909	457	6,4	1,78E+10	2122	457	6,4	1,98E+10	2334	457	6,4	2,17E+10
CS6 truss diag	2753	610	6,4	3,41E+10	2924	610	6,4	3,62E+10	3103	660	6,4	4,16E+10	3287	711	6,4	4,74E+10
CS7 X	3098	1829	9,5	1,7E+11	3098	1829	9,5	1,7E+11	3098	1829	9,5	1,70E+11	3098	1829	9,5	1,7E+11
CS8 Y	3098	1880	9,5	1,75E+11	3098	1930	9,5	1,79E+11	3098	1930	9,5	1,79E+11	3098	1880	9,5	1,75E+11
CS9 chord bottom	628	2083	9,5	3,92E+10	628	1829	9,5	3,45E+10	628	1930	9,5	3,64E+10	628	2083	10,3	4,25E+10
CS10 chord outer	678	2134	22,2	1,02E+11	685	2032	9,5	4,17E+10	691	2134	11,9	5,54E+10	697	2134	20,6	9,72E+10
CS11 vert above col	160	2032	14,3	1,47E+10	180	2032	15,9	1,84E+10	200	2032	15,9	2,05E+10	220	2134	14,3	2,12E+10
Total volume (mm3)				6,91E+11	Total volume (mm3)				Total volume (mm3)				Total volume (mm3)			
Total weight (kg)				5425138	Total weight (kg)				Total weight (kg)				Total weight (kg)			
Total reaction (kN)				53220,61	Total reaction (kN)				Total reaction (kN)				Total reaction (kN)			
Reaction SCIA (kN)				52406,8	Reaction SCIA (kN)				Reaction SCIA (kN)				Reaction SCIA (kN)			



Roof with hole

	Without hole				With hole 30				With hole 50				With hole 70				With hole 84				With hole 100			
	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)	L (m)	D (mm)	t (mm)	V (mm3)
CS2 W-columns	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11	713	2134	22,2	1,07E+11
CS3 V-columns	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10	164	2134	22,2	2,47E+10
CS4 chord inner	628	711	6,4	9,06E+09	628	711	6,4	9,06E+09	628	559	7,9	8,84E+09	628	457	7,9	7,25E+09	628	457	6,4	5,85E+09	628	457	6,4	5,85E+09
CS5 truss vert	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10	1909	457	6,4	1,78E+10
CS6 truss diag	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	610	6,4	3,62E+10	2924	559	6,4	3,32E+10
CS7 X	3098	1880	9,5	1,75E+11	3027	1981	9,5	1,8E+11	2911	2032	11,1	2,07E+11	2740	2032	9,5	1,67E+11	2598	2083	9,5	1,62E+11	2365	2083	11,1	1,73E+11
CS8 Y	3098	1930	9,5	1,79E+11	3027	1981	9,5	1,8E+11	2911	1981	11,1	2,02E+11	2740	2083	9,5	1,71E+11	2598	2134	9,5	1,66E+11	2365	2134	10,3	1,64E+11
CS9 chord bottom	628	1930	9,5	3,64E+10	628	1930	9,5	3,64E+10	628	1930	9,5	3,64E+10	628	1880	9,5	3,54E+10	628	1727	10,3	3,53E+10	628	1727	10,3	3,53E+10
CS10 chord outer	685	2083	9,5	4,28E+10	685	2083	9,5	4,28E+10	685	1778	11,1	4,27E+10	685	1778	11,1	4,27E+10	685	1778	11,1	4,27E+10	685	1778	11,1	4,27E+10
CS11 vert above col	180	2083	15,9	1,89E+10	180	2083	15,9	1,89E+10	180	2083	15,9	1,89E+10	180	2134	14,3	1,74E+10	180	2083	14,3	1,7E+10	180	2134	12,7	1,54E+10
CS12 inner ring	94	1829	11,1	6,03E+09	157	2134	9,5	1E+10	220	2134	9,5	1,41E+10	264	2134	9,5	1,69E+10	314	2134	9,5	2,01E+10				
Total volume (mm3)				6,47E+11	Total volume (mm3)				6,59E+11	Total volume (mm3)				7,12E+11	Total volume (mm3)				6,41E+11	Total volume (mm3)				6,32E+11
Total weight (kg)				5078946	Total weight (kg)				5170398	Total weight (kg)				5592081	Total weight (kg)				5030377	Total weight (kg)				4961972
Total reaction (kN)				4982446	Total reaction (kN)				5072161	Total reaction (kN)				5485832	Total reaction (kN)				49348	Total reaction (kN)				4867695
Reaction SCIA (kN)				4912788	Reaction SCIA (kN)				5001366	Reaction SCIA (kN)				5407184	Reaction SCIA (kN)				48670,71	Reaction SCIA (kN)				48005,64



E

Engineering Report final calculations

1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K1	109,000	0,000	50,000
K2	100,000	0,000	50,000
K3	100,000	0,000	41,000
K4	108,330	12,110	50,000
K5	99,380	11,110	50,000
K6	99,380	11,110	41,000
K7	106,270	24,220	50,000
K8	97,500	22,220	50,000
K9	97,500	22,220	41,000
K10	102,770	36,330	50,000
K11	94,280	33,330	50,000
K12	94,280	33,330	41,000
K13	97,640	48,440	50,000
K14	89,580	44,440	50,000
K15	89,580	44,440	41,000
K16	96,890	49,940	50,000
K17	88,890	45,810	50,000
K18	88,890	45,810	41,000
K19	90,630	60,560	50,000
K20	83,150	55,560	50,000
K21	83,150	55,560	41,000
K22	84,780	68,510	50,000
K23	77,780	62,850	50,000
K24	77,780	62,850	41,000
K25	81,240	72,670	50,000
K26	74,540	66,670	50,000
K27	74,540	66,670	41,000
K28	72,670	81,240	50,000
K29	66,670	74,540	50,000
K30	66,670	74,540	41,000
K31	68,510	84,780	50,000
K32	62,850	77,780	50,000
K33	62,850	77,780	41,000
K34	60,560	90,630	50,000
K35	55,560	83,150	50,000
K36	55,560	83,150	41,000
K37	49,940	96,890	50,000
K38	45,810	88,890	50,000
K39	45,810	88,890	41,000
K40	48,440	97,640	50,000
K41	44,440	89,580	50,000
K42	44,440	89,580	41,000
K43	36,330	102,770	50,000
K44	33,330	94,280	50,000
K45	33,330	94,280	41,000
K46	24,220	106,270	50,000
K47	22,220	97,500	50,000
K48	22,220	97,500	41,000
K49	12,110	108,330	50,000
K50	11,110	99,380	50,000
K51	11,110	99,380	41,000
K52	0,000	109,000	50,000
K53	0,000	100,000	50,000
K54	0,000	100,000	41,000
K55	-12,110	108,330	50,000
K56	-11,110	99,380	50,000
K57	-11,110	99,380	41,000
K58	-24,220	106,270	50,000
K59	-22,220	97,500	50,000
K60	-22,220	97,500	41,000
K61	-36,330	102,770	50,000
K62	-33,330	94,280	50,000
K63	-33,330	94,280	41,000
K64	-48,440	97,640	50,000
K65	-44,440	89,580	50,000
K66	-44,440	89,580	41,000
K67	-49,940	96,890	50,000
K68	-45,810	88,890	50,000
K69	-45,810	88,890	41,000
K70	-60,560	90,630	50,000
K71	-55,560	83,150	50,000
K72	-55,560	83,150	41,000
K73	-68,510	84,780	50,000
K74	-62,850	77,780	50,000
K75	-62,850	77,780	41,000

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K76	-72,670	81,240	50,000
K77	-66,670	74,540	50,000
K78	-66,670	74,540	41,000
K79	-81,240	72,670	50,000
K80	-74,540	66,670	50,000
K81	-74,540	66,670	41,000
K82	-84,780	68,510	50,000
K83	-77,780	62,850	50,000
K84	-77,780	62,850	41,000
K85	-90,630	60,560	50,000
K86	-83,150	55,560	50,000
K87	-83,150	55,560	41,000
K88	-96,890	49,940	50,000
K89	-88,890	45,810	50,000
K90	-88,890	45,810	41,000
K91	-97,640	48,440	50,000
K92	-89,580	44,440	50,000
K93	-89,580	44,440	41,000
K94	-102,770	36,330	50,000
K95	-94,280	33,330	50,000
K96	-94,280	33,330	41,000
K97	-106,270	24,220	50,000
K98	-97,500	22,220	50,000
K99	-97,500	22,220	41,000
K100	-108,330	12,110	50,000
K101	-99,380	11,110	50,000
K102	-99,380	11,110	41,000
K103	-109,000	0,000	50,000
K104	-100,000	0,000	50,000
K105	-100,000	0,000	41,000
K106	-108,330	-12,110	50,000
K107	-99,380	-11,110	50,000
K108	-99,380	-11,110	41,000
K109	-106,270	-24,220	50,000
K110	-97,500	-22,220	50,000
K111	-97,500	-22,220	41,000
K112	-102,770	-36,330	50,000
K113	-94,280	-33,330	50,000
K114	-94,280	-33,330	41,000
K115	-97,640	-48,440	50,000
K116	-89,580	-44,440	50,000
K117	-89,580	-44,440	41,000
K118	-96,890	-49,940	50,000
K119	-88,890	-45,810	50,000
K120	-88,890	-45,810	41,000
K121	-90,630	-60,560	50,000
K122	-83,150	-55,560	50,000
K123	-83,150	-55,560	41,000
K124	-84,780	-68,510	50,000
K125	-77,780	-62,850	50,000
K126	-77,780	-62,850	41,000
K127	-81,240	-72,670	50,000
K128	-74,540	-66,670	50,000
K129	-74,540	-66,670	41,000
K130	-72,670	-81,240	50,000
K131	-66,670	-74,540	50,000
K132	-66,670	-74,540	41,000
K133	-68,510	-84,780	50,000
K134	-62,850	-77,780	50,000
K135	-62,850	-77,780	41,000
K136	-60,560	-90,630	50,000
K137	-55,560	-83,150	50,000
K138	-55,560	-83,150	41,000
K139	-49,940	-96,890	50,000
K140	-45,810	-88,890	50,000
K141	-45,810	-88,890	41,000
K142	-48,440	-97,640	50,000
K143	-44,440	-89,580	50,000
K144	-44,440	-89,580	41,000
K145	-36,330	-102,770	50,000
K146	-33,330	-94,280	50,000
K147	-33,330	-94,280	41,000
K148	-24,220	-106,270	50,000
K149	-22,220	-97,500	50,000
K150	-22,220	-97,500	41,000

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K151	-12,110	-108,330	50,000
K152	-11,110	-99,380	50,000
K153	-11,110	-99,380	41,000
K154	0,000	-109,000	50,000
K155	0,000	-100,000	50,000
K156	0,000	-100,000	41,000
K157	12,110	-108,330	50,000
K158	11,110	-99,380	50,000
K159	11,110	-99,380	41,000
K160	24,220	-106,270	50,000
K161	22,220	-97,500	50,000
K162	22,220	-97,500	41,000
K163	36,330	-102,770	50,000
K164	33,330	-94,280	50,000
K165	33,330	-94,280	41,000
K166	48,440	-97,640	50,000
K167	44,440	-89,580	50,000
K168	44,440	-89,580	41,000
K169	49,940	-96,890	50,000
K170	45,810	-88,890	50,000
K171	45,810	-88,890	41,000
K172	60,560	-90,630	50,000
K173	55,560	-83,150	50,000
K174	55,560	-83,150	41,000
K175	68,510	-84,780	50,000
K176	62,850	-77,780	50,000
K177	62,850	-77,780	41,000
K178	72,670	-81,240	50,000
K179	66,670	-74,540	50,000
K180	66,670	-74,540	41,000
K181	81,240	-72,670	50,000
K182	74,540	-66,670	50,000
K183	74,540	-66,670	41,000
K184	84,780	-68,510	50,000
K185	77,780	-62,850	50,000
K186	77,780	-62,850	41,000
K187	90,630	-60,560	50,000
K188	83,150	-55,560	50,000
K189	83,150	-55,560	41,000
K190	96,890	-49,940	50,000
K191	88,890	-45,810	50,000
K192	88,890	-45,810	41,000
K193	97,640	-48,440	50,000
K194	89,580	-44,440	50,000
K195	89,580	-44,440	41,000
K196	102,770	-36,330	50,000
K197	94,280	-33,330	50,000
K198	94,280	-33,330	41,000
K199	106,270	-24,220	50,000
K200	97,500	-22,220	50,000
K201	97,500	-22,220	41,000
K202	108,330	-12,110	50,000
K203	99,380	-11,110	50,000
K204	99,380	-11,110	41,000
K205	100,000	0,000	0,000
K206	0,000	100,000	0,000
K207	-100,000	0,000	0,000
K208	0,000	-100,000	0,000
K209	66,960	87,270	0,000
K210	-87,270	66,960	0,000
K211	-66,960	-87,270	0,000
K212	87,270	-66,960	0,000
K213	87,270	66,960	0,000
K214	-66,960	87,270	0,000
K215	-87,270	-66,960	0,000
K216	66,960	-87,270	0,000
K217	44,440	-88,890	50,630
K218	33,330	-88,890	55,050
K219	22,220	-88,890	58,170
K220	11,110	-88,890	60,030
K221	0,000	-88,890	60,650
K222	-11,110	-88,890	60,030
K223	-22,220	-88,890	58,170
K224	-33,330	-88,890	55,050
K225	-44,440	-88,890	50,630
K226	66,670	66,670	55,670

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K227	55,560	66,670	62,500
K228	44,440	66,670	67,990
K229	33,330	66,670	72,210
K230	22,220	66,670	75,200
K231	11,110	66,670	76,980
K232	0,000	66,670	77,570
K233	-11,110	66,670	76,980
K234	-22,220	66,670	75,200
K235	-33,330	66,670	72,210
K236	-44,440	66,670	67,990
K237	-55,560	66,670	62,500
K238	-66,670	66,670	55,670
K239	77,780	55,560	54,420
K240	66,670	55,560	62,500
K241	55,560	55,560	69,210
K242	44,440	55,560	74,600
K243	33,330	55,560	78,750
K244	22,220	55,560	81,680
K245	11,110	55,560	83,430
K246	0,000	55,560	84,010
K247	-11,110	55,560	83,430
K248	-22,220	55,560	81,680
K249	-33,330	55,560	78,750
K250	-44,440	55,560	74,600
K251	-55,560	55,560	69,210
K252	-66,670	55,560	62,500
K253	-77,780	55,560	54,420
K254	55,560	-77,780	54,420
K255	44,440	-77,780	60,030
K256	33,330	-77,780	64,340
K257	22,220	-77,780	67,390
K258	11,110	-77,780	69,210
K259	0,000	-77,780	69,810
K260	-11,110	-77,780	69,210
K261	-22,220	-77,780	67,390
K262	-33,330	-77,780	64,340
K263	-44,440	-77,780	60,030
K264	-55,560	-77,780	54,420
K265	88,890	44,440	50,630
K266	77,780	44,440	60,030
K267	66,670	44,440	67,990
K268	55,560	44,440	74,600
K269	44,440	44,440	79,920
K270	33,330	44,440	84,010
K271	22,220	44,440	86,900
K272	11,110	44,440	88,620
K273	0,000	44,440	89,190
K274	-11,110	44,440	88,620
K275	-22,220	44,440	86,900
K276	-33,330	44,440	84,010
K277	-44,440	44,440	79,920
K278	-55,560	44,440	74,600
K279	-66,670	44,440	67,990
K280	-77,780	44,440	60,030
K281	-88,890	44,440	50,630
K282	88,890	33,330	55,050
K283	77,780	33,330	64,340
K284	66,670	33,330	72,210
K285	55,560	33,330	78,750
K286	44,440	33,330	84,010
K287	33,330	33,330	88,040
K288	22,220	33,330	90,900
K289	11,110	33,330	92,600
K290	0,000	33,330	93,170
K291	-11,110	33,330	92,600
K292	-22,220	33,330	90,900
K293	-33,330	33,330	88,040
K294	-44,440	33,330	84,010
K295	-55,560	33,330	78,750
K296	-66,670	33,330	72,210
K297	-77,780	33,330	64,340
K298	-88,890	33,330	55,050
K299	88,890	-11,110	60,030
K300	77,780	-11,110	69,210
K301	66,670	-11,110	76,980
K302	55,560	-11,110	83,430

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K303	44,440	-11,110	88,620
K304	33,330	-11,110	92,600
K305	22,220	-11,110	95,420
K306	11,110	-11,110	97,100
K307	0,000	-11,110	97,660
K308	-11,110	-11,110	97,100
K309	-22,220	-11,110	95,420
K310	-33,330	-11,110	92,600
K311	-44,440	-11,110	88,620
K312	-55,560	-11,110	83,430
K313	-66,670	-11,110	76,980
K314	-77,780	-11,110	69,210
K315	-88,890	-11,110	60,030
K316	88,890	-44,440	50,630
K317	77,780	-44,440	60,030
K318	66,670	-44,440	67,990
K319	55,560	-44,440	74,600
K320	44,440	-44,440	79,920
K321	33,330	-44,440	84,010
K322	22,220	-44,440	86,900
K323	11,110	-44,440	88,620
K324	0,000	-44,440	89,190
K325	-11,110	-44,440	88,620
K326	-22,220	-44,440	86,900
K327	-33,330	-44,440	84,010
K328	-44,440	-44,440	79,920
K329	-55,560	-44,440	74,600
K330	-66,670	-44,440	67,990
K331	-77,780	-44,440	60,030
K332	-88,890	-44,440	50,630
K333	88,890	0,000	60,650
K334	77,780	0,000	69,810
K335	66,670	0,000	77,570
K336	55,560	0,000	84,010
K337	44,440	0,000	89,190
K338	33,330	0,000	93,170
K339	22,220	0,000	95,980
K340	11,110	0,000	97,660
K341	0,000	0,000	98,210
K342	-11,110	0,000	97,660
K343	-22,220	0,000	95,980
K344	-33,330	0,000	93,170
K345	-44,440	0,000	89,190
K346	-55,560	0,000	84,010
K347	-66,670	0,000	77,570
K348	-77,780	0,000	69,810
K349	-88,890	0,000	60,650
K350	44,440	88,890	50,630
K351	33,330	88,890	55,050
K352	22,220	88,890	58,170
K353	11,110	88,890	60,030
K354	0,000	88,890	60,650
K355	-11,110	88,890	60,030
K356	-22,220	88,890	58,170
K357	-33,330	88,890	55,050
K358	-44,440	88,890	50,630
K359	88,890	-22,220	58,170
K360	77,780	-22,220	67,390
K361	66,670	-22,220	75,200
K362	55,560	-22,220	81,680
K363	44,440	-22,220	86,900
K364	33,330	-22,220	90,900
K365	22,220	-22,220	93,730
K366	11,110	-22,220	95,420
K367	0,000	-22,220	95,980
K368	-11,110	-22,220	95,420
K369	-22,220	-22,220	93,730
K370	-33,330	-22,220	90,900
K371	-44,440	-22,220	86,900
K372	-55,560	-22,220	81,680
K373	-66,670	-22,220	75,200
K374	-77,780	-22,220	67,390
K375	-88,890	-22,220	58,170
K376	66,670	-66,670	55,670
K377	55,560	-66,670	62,500
K378	44,440	-66,670	67,990

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K379	33,330	-66,670	72,210
K380	22,220	-66,670	75,200
K381	11,110	-66,670	76,980
K382	0,000	-66,670	77,570
K383	-11,110	-66,670	76,980
K384	-22,220	-66,670	75,200
K385	-33,330	-66,670	72,210
K386	-44,440	-66,670	67,990
K387	-55,560	-66,670	62,500
K388	-66,670	-66,670	55,670
K389	88,890	22,220	58,170
K390	77,780	22,220	67,390
K391	66,670	22,220	75,200
K392	55,560	22,220	81,680
K393	44,440	22,220	86,900
K394	33,330	22,220	90,900
K395	22,220	22,220	93,730
K396	11,110	22,220	95,420
K397	0,000	22,220	95,980
K398	-11,110	22,220	95,420
K399	-22,220	22,220	93,730
K400	-33,330	22,220	90,900
K401	-44,440	22,220	86,900
K402	-55,560	22,220	81,680
K403	-66,670	22,220	75,200
K404	-77,780	22,220	67,390
K405	-88,890	22,220	58,170
K406	88,890	-33,330	55,050
K407	77,780	-33,330	64,340
K408	66,670	-33,330	72,210
K409	55,560	-33,330	78,750
K410	44,440	-33,330	84,010
K411	33,330	-33,330	88,040
K412	22,220	-33,330	90,900
K413	11,110	-33,330	92,600
K414	0,000	-33,330	93,170
K415	-11,110	-33,330	92,600
K416	-22,220	-33,330	90,900
K417	-33,330	-33,330	88,040
K418	-44,440	-33,330	84,010
K419	-55,560	-33,330	78,750
K420	-66,670	-33,330	72,210
K421	-77,780	-33,330	64,340
K422	-88,890	-33,330	55,050
K423	88,890	11,110	60,030
K424	77,780	11,110	69,210
K425	66,670	11,110	76,980
K426	55,560	11,110	83,430
K427	44,440	11,110	88,620
K428	33,330	11,110	92,600
K429	22,220	11,110	95,420
K430	11,110	11,110	97,100
K431	0,000	11,110	97,660
K432	-11,110	11,110	97,100
K433	-22,220	11,110	95,420
K434	-33,330	11,110	92,600
K435	-44,440	11,110	88,620
K436	-55,560	11,110	83,430
K437	-66,670	11,110	76,980
K438	-77,780	11,110	69,210
K439	-88,890	11,110	60,030
K440	55,560	77,780	54,420
K441	44,440	77,780	60,030
K442	33,330	77,780	64,340
K443	22,220	77,780	67,390
K444	11,110	77,780	69,210
K445	0,000	77,780	69,810
K446	-11,110	77,780	69,210
K447	-22,220	77,780	67,390
K448	-33,330	77,780	64,340
K449	-44,440	77,780	60,030
K450	-55,560	77,780	54,420
K451	77,780	-55,560	54,420
K452	66,670	-55,560	62,500
K453	55,560	-55,560	69,210
K454	44,440	-55,560	74,600

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K455	33,330	-55,560	78,750
K456	22,220	-55,560	81,680
K457	11,110	-55,560	83,430
K458	0,000	-55,560	84,010
K459	-11,110	-55,560	83,430
K460	-22,220	-55,560	81,680

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K461	-33,330	-55,560	78,750
K462	-44,440	-55,560	74,600
K463	-55,560	-55,560	69,210
K464	-66,670	-55,560	62,500
K465	-77,780	-55,560	54,420

2. Staven

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Cw1	CS13 - CHS (1010; 56)	S 355	42,932	K209	K30	Kolom (100)
Cw2	CS13 - CHS (1010; 56)	S 355	42,932	K210	K81	Kolom (100)
Cw3	CS13 - CHS (1010; 56)	S 355	42,932	K211	K132	Kolom (100)
Cw4	CS13 - CHS (1010; 56)	S 355	42,932	K212	K183	Kolom (100)
Cw5	CS13 - CHS (1010; 56)	S 355	42,932	K213	K27	Kolom (100)
Cw6	CS13 - CHS (1010; 56)	S 355	42,932	K214	K78	Kolom (100)
Cw7	CS13 - CHS (1010; 56)	S 355	42,932	K215	K129	Kolom (100)
Cw8	CS13 - CHS (1010; 56)	S 355	42,932	K216	K180	Kolom (100)
Cw9	CS13 - CHS (1010; 56)	S 355	46,162	K213	K18	Kolom (100)
Cw10	CS13 - CHS (1010; 56)	S 355	46,162	K214	K69	Kolom (100)
Cw11	CS13 - CHS (1010; 56)	S 355	46,162	K215	K120	Kolom (100)
Cw12	CS13 - CHS (1010; 56)	S 355	46,162	K216	K171	Kolom (100)
Cw13	CS13 - CHS (1010; 56)	S 355	46,162	K209	K39	Kolom (100)
Cw14	CS13 - CHS (1010; 56)	S 355	46,162	K210	K90	Kolom (100)
Cw15	CS13 - CHS (1010; 56)	S 355	46,162	K211	K141	Kolom (100)
Cw16	CS13 - CHS (1010; 56)	S 355	46,162	K212	K192	Kolom (100)
Cv1	CS14 - CHS (880; 19)	S 355	41,000	K205	K3	Kolom (100)
Cv2	CS14 - CHS (880; 19)	S 355	41,000	K206	K54	Kolom (100)
Cv3	CS14 - CHS (880; 19)	S 355	41,000	K207	K105	Kolom (100)
Cv4	CS14 - CHS (880; 19)	S 355	41,000	K208	K156	Kolom (100)
Cv5	CS22 - CHS (830; 20)	S 355	9,000	K30	K29	Kolom (100)
Cv6	CS22 - CHS (830; 20)	S 355	9,000	K81	K80	Kolom (100)
Cv7	CS22 - CHS (830; 20)	S 355	9,000	K132	K131	Kolom (100)
Cv8	CS22 - CHS (830; 20)	S 355	9,000	K183	K182	Kolom (100)
Cv9	CS22 - CHS (830; 20)	S 355	9,000	K27	K26	Kolom (100)
Cv10	CS22 - CHS (830; 20)	S 355	9,000	K78	K77	Kolom (100)
Cv11	CS22 - CHS (830; 20)	S 355	9,000	K129	K128	Kolom (100)
Cv12	CS22 - CHS (830; 20)	S 355	9,000	K180	K179	Kolom (100)
Cv13	CS22 - CHS (830; 20)	S 355	9,000	K3	K2	Kolom (100)
Cv14	CS22 - CHS (830; 20)	S 355	9,000	K54	K53	Kolom (100)
Cv15	CS22 - CHS (830; 20)	S 355	9,000	K105	K104	Kolom (100)
Cv16	CS22 - CHS (830; 20)	S 355	9,000	K156	K155	Kolom (100)
Cv17	CS22 - CHS (830; 20)	S 355	9,000	K18	K17	Kolom (100)
Cv18	CS22 - CHS (830; 20)	S 355	9,000	K69	K68	Kolom (100)
Cv19	CS22 - CHS (830; 20)	S 355	9,000	K120	K119	Kolom (100)
Cv20	CS22 - CHS (830; 20)	S 355	9,000	K171	K170	Kolom (100)
Cv21	CS22 - CHS (830; 20)	S 355	9,000	K39	K38	Kolom (100)
Cv22	CS22 - CHS (830; 20)	S 355	9,000	K90	K89	Kolom (100)
Cv23	CS22 - CHS (830; 20)	S 355	9,000	K141	K140	Kolom (100)
Cv24	CS22 - CHS (830; 20)	S 355	9,000	K192	K191	Kolom (100)
Chordi1	CS15 - CHS (600; 17)	S 355	11,127	K2	K5	Vakwerkligger (95)
Chordi2	CS15 - CHS (600; 17)	S 355	11,268	K5	K8	Vakwerkligger (95)
Chordi3	CS15 - CHS (600; 17)	S 355	11,567	K8	K11	Vakwerkligger (95)
Chordi4	CS15 - CHS (600; 17)	S 355	12,063	K11	K14	Vakwerkligger (95)
Chordi5	CS15 - CHS (600; 17)	S 355	1,534	K14	K17	Vakwerkligger (95)
Chordi6	CS15 - CHS (600; 17)	S 355	11,314	K17	K20	Vakwerkligger (95)
Chordi7	CS15 - CHS (600; 17)	S 355	9,054	K20	K23	Vakwerkligger (95)
Chordi8	CS15 - CHS (600; 17)	S 355	5,009	K23	K26	Vakwerkligger (95)
Chordi9	CS15 - CHS (600; 17)	S 355	11,130	K26	K29	Vakwerkligger (95)
Chordi10	CS15 - CHS (600; 17)	S 355	5,009	K29	K32	Vakwerkligger (95)
Chordi11	CS15 - CHS (600; 17)	S 355	9,054	K32	K35	Vakwerkligger (95)
Chordi12	CS15 - CHS (600; 17)	S 355	11,314	K35	K38	Vakwerkligger (95)
Chordi13	CS15 - CHS (600; 17)	S 355	1,534	K38	K41	Vakwerkligger (95)
Chordi14	CS15 - CHS (600; 17)	S 355	12,063	K41	K44	Vakwerkligger (95)
Chordi15	CS15 - CHS (600; 17)	S 355	11,567	K44	K47	Vakwerkligger (95)
Chordi16	CS15 - CHS (600; 17)	S 355	11,268	K47	K50	Vakwerkligger (95)
Chordi17	CS15 - CHS (600; 17)	S 355	11,127	K50	K53	Vakwerkligger (95)
Chordi18	CS15 - CHS (600; 17)	S 355	11,127	K53	K56	Vakwerkligger (95)
Chordi19	CS15 - CHS (600; 17)	S 355	11,268	K56	K59	Vakwerkligger (95)
Chordi20	CS15 - CHS (600; 17)	S 355	11,567	K59	K62	Vakwerkligger (95)
Chordi21	CS15 - CHS (600; 17)	S 355	12,063	K62	K65	Vakwerkligger (95)
Chordi22	CS15 - CHS (600; 17)	S 355	1,534	K65	K68	Vakwerkligger (95)
Chordi23	CS15 - CHS (600; 17)	S 355	11,314	K68	K71	Vakwerkligger (95)
Chordi24	CS15 - CHS (600; 17)	S 355	9,054	K71	K74	Vakwerkligger (95)
Chordi25	CS15 - CHS (600; 17)	S 355	5,009	K74	K77	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Chordi26	CS15 - CHS (600; 17)	S 355	11,130	K77	K80	Vakwerkligger (95)
Chordi27	CS15 - CHS (600; 17)	S 355	5,009	K80	K83	Vakwerkligger (95)
Chordi28	CS15 - CHS (600; 17)	S 355	9,054	K83	K86	Vakwerkligger (95)
Chordi29	CS15 - CHS (600; 17)	S 355	11,314	K86	K89	Vakwerkligger (95)
Chordi30	CS15 - CHS (600; 17)	S 355	1,534	K89	K92	Vakwerkligger (95)
Chordi31	CS15 - CHS (600; 17)	S 355	12,063	K92	K95	Vakwerkligger (95)
Chordi32	CS15 - CHS (600; 17)	S 355	11,567	K95	K98	Vakwerkligger (95)
Chordi33	CS15 - CHS (600; 17)	S 355	11,268	K98	K101	Vakwerkligger (95)
Chordi34	CS15 - CHS (600; 17)	S 355	11,127	K101	K104	Vakwerkligger (95)
Chordi35	CS15 - CHS (600; 17)	S 355	11,127	K104	K107	Vakwerkligger (95)
Chordi36	CS15 - CHS (600; 17)	S 355	11,268	K107	K110	Vakwerkligger (95)
Chordi37	CS15 - CHS (600; 17)	S 355	11,567	K110	K113	Vakwerkligger (95)
Chordi38	CS15 - CHS (600; 17)	S 355	12,063	K113	K116	Vakwerkligger (95)
Chordi39	CS15 - CHS (600; 17)	S 355	1,534	K116	K119	Vakwerkligger (95)
Chordi40	CS15 - CHS (600; 17)	S 355	11,314	K119	K122	Vakwerkligger (95)
Chordi41	CS15 - CHS (600; 17)	S 355	9,054	K122	K125	Vakwerkligger (95)
Chordi42	CS15 - CHS (600; 17)	S 355	5,009	K125	K128	Vakwerkligger (95)
Chordi43	CS15 - CHS (600; 17)	S 355	11,130	K128	K131	Vakwerkligger (95)
Chordi44	CS15 - CHS (600; 17)	S 355	5,009	K131	K134	Vakwerkligger (95)
Chordi45	CS15 - CHS (600; 17)	S 355	9,054	K134	K137	Vakwerkligger (95)
Chordi46	CS15 - CHS (600; 17)	S 355	11,314	K137	K140	Vakwerkligger (95)
Chordi47	CS15 - CHS (600; 17)	S 355	1,534	K140	K143	Vakwerkligger (95)
Chordi48	CS15 - CHS (600; 17)	S 355	12,063	K143	K146	Vakwerkligger (95)
Chordi49	CS15 - CHS (600; 17)	S 355	11,567	K146	K149	Vakwerkligger (95)
Chordi50	CS15 - CHS (600; 17)	S 355	11,268	K149	K152	Vakwerkligger (95)
Chordi51	CS15 - CHS (600; 17)	S 355	11,127	K152	K155	Vakwerkligger (95)
Chordi52	CS15 - CHS (600; 17)	S 355	11,127	K155	K158	Vakwerkligger (95)
Chordi53	CS15 - CHS (600; 17)	S 355	11,268	K158	K161	Vakwerkligger (95)
Chordi54	CS15 - CHS (600; 17)	S 355	11,567	K161	K164	Vakwerkligger (95)
Chordi55	CS15 - CHS (600; 17)	S 355	12,063	K164	K167	Vakwerkligger (95)
Chordi56	CS15 - CHS (600; 17)	S 355	1,534	K167	K170	Vakwerkligger (95)
Chordi57	CS15 - CHS (600; 17)	S 355	11,314	K170	K173	Vakwerkligger (95)
Chordi58	CS15 - CHS (600; 17)	S 355	9,054	K173	K176	Vakwerkligger (95)
Chordi59	CS15 - CHS (600; 17)	S 355	5,009	K176	K179	Vakwerkligger (95)
Chordi60	CS15 - CHS (600; 17)	S 355	11,130	K179	K182	Vakwerkligger (95)
Chordi61	CS15 - CHS (600; 17)	S 355	5,009	K182	K185	Vakwerkligger (95)
Chordi62	CS15 - CHS (600; 17)	S 355	9,054	K185	K188	Vakwerkligger (95)
Chordi63	CS15 - CHS (600; 17)	S 355	11,314	K188	K191	Vakwerkligger (95)
Chordi64	CS15 - CHS (600; 17)	S 355	1,534	K191	K194	Vakwerkligger (95)
Chordi65	CS15 - CHS (600; 17)	S 355	12,063	K194	K197	Vakwerkligger (95)
Chordi66	CS15 - CHS (600; 17)	S 355	11,567	K197	K200	Vakwerkligger (95)
Chordi67	CS15 - CHS (600; 17)	S 355	11,268	K200	K203	Vakwerkligger (95)
Chordi68	CS15 - CHS (600; 17)	S 355	11,127	K203	K2	Vakwerkligger (95)
Vert1	CS16 - CHS (360; 8)	S 355	9,000	K1	K2	Vakwerkligger (95)
Vert2	CS16 - CHS (360; 8)	S 355	9,006	K4	K5	Vakwerkligger (95)
Vert3	CS16 - CHS (360; 8)	S 355	12,728	K1	K3	Vakwerkligger (95)
Vert4	CS16 - CHS (360; 8)	S 355	12,732	K4	K6	Vakwerkligger (95)
Vert5	CS16 - CHS (360; 8)	S 355	9,000	K6	K5	Vakwerkligger (95)
Vert6	CS16 - CHS (360; 8)	S 355	8,995	K7	K8	Vakwerkligger (95)
Vert7	CS16 - CHS (360; 8)	S 355	12,724	K7	K9	Vakwerkligger (95)
Vert8	CS16 - CHS (360; 8)	S 355	9,000	K9	K8	Vakwerkligger (95)
Vert9	CS16 - CHS (360; 8)	S 355	9,004	K10	K11	Vakwerkligger (95)
Vert10	CS16 - CHS (360; 8)	S 355	12,731	K10	K12	Vakwerkligger (95)
Vert11	CS16 - CHS (360; 8)	S 355	9,000	K12	K11	Vakwerkligger (95)
Vert12	CS16 - CHS (360; 8)	S 355	8,998	K13	K14	Vakwerkligger (95)
Vert13	CS16 - CHS (360; 8)	S 355	12,726	K13	K15	Vakwerkligger (95)
Vert14	CS16 - CHS (360; 8)	S 355	9,000	K15	K14	Vakwerkligger (95)
Vert15	CS16 - CHS (360; 8)	S 355	9,003	K16	K17	Vakwerkligger (95)
Vert16	CS16 - CHS (360; 8)	S 355	12,730	K16	K18	Vakwerkligger (95)
Vert17	CS16 - CHS (360; 8)	S 355	8,997	K19	K20	Vakwerkligger (95)
Vert18	CS16 - CHS (360; 8)	S 355	12,726	K19	K21	Vakwerkligger (95)
Vert19	CS16 - CHS (360; 8)	S 355	9,000	K21	K20	Vakwerkligger (95)
Vert20	CS16 - CHS (360; 8)	S 355	9,002	K22	K23	Vakwerkligger (95)
Vert21	CS16 - CHS (360; 8)	S 355	12,729	K22	K24	Vakwerkligger (95)
Vert22	CS16 - CHS (360; 8)	S 355	9,000	K24	K23	Vakwerkligger (95)
Vert23	CS16 - CHS (360; 8)	S 355	8,994	K25	K26	Vakwerkligger (95)
Vert24	CS16 - CHS (360; 8)	S 355	12,724	K25	K27	Vakwerkligger (95)
Vert25	CS16 - CHS (360; 8)	S 355	8,994	K28	K29	Vakwerkligger (95)
Vert26	CS16 - CHS (360; 8)	S 355	12,724	K28	K30	Vakwerkligger (95)
Vert27	CS16 - CHS (360; 8)	S 355	9,002	K31	K32	Vakwerkligger (95)
Vert28	CS16 - CHS (360; 8)	S 355	12,729	K31	K33	Vakwerkligger (95)
Vert29	CS16 - CHS (360; 8)	S 355	9,000	K33	K32	Vakwerkligger (95)
Vert30	CS16 - CHS (360; 8)	S 355	8,997	K34	K35	Vakwerkligger (95)
Vert31	CS16 - CHS (360; 8)	S 355	12,726	K34	K36	Vakwerkligger (95)
Vert32	CS16 - CHS (360; 8)	S 355	9,000	K36	K35	Vakwerkligger (95)
Vert33	CS16 - CHS (360; 8)	S 355	9,003	K37	K38	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Vert34	CS16 - CHS (360; 8)	S 355	12,730	K37	K39	Vakwerkligger (95)
Vert35	CS16 - CHS (360; 8)	S 355	8,998	K40	K41	Vakwerkligger (95)
Vert36	CS16 - CHS (360; 8)	S 355	12,726	K40	K42	Vakwerkligger (95)
Vert37	CS16 - CHS (360; 8)	S 355	9,000	K42	K41	Vakwerkligger (95)
Vert38	CS16 - CHS (360; 8)	S 355	9,004	K43	K44	Vakwerkligger (95)
Vert39	CS16 - CHS (360; 8)	S 355	12,731	K43	K45	Vakwerkligger (95)
Vert40	CS16 - CHS (360; 8)	S 355	9,000	K45	K44	Vakwerkligger (95)
Vert41	CS16 - CHS (360; 8)	S 355	8,995	K46	K47	Vakwerkligger (95)
Vert42	CS16 - CHS (360; 8)	S 355	12,724	K46	K48	Vakwerkligger (95)
Vert43	CS16 - CHS (360; 8)	S 355	9,000	K48	K47	Vakwerkligger (95)
Vert44	CS16 - CHS (360; 8)	S 355	9,006	K49	K50	Vakwerkligger (95)
Vert45	CS16 - CHS (360; 8)	S 355	12,732	K49	K51	Vakwerkligger (95)
Vert46	CS16 - CHS (360; 8)	S 355	9,000	K51	K50	Vakwerkligger (95)
Vert47	CS16 - CHS (360; 8)	S 355	9,000	K52	K53	Vakwerkligger (95)
Vert48	CS16 - CHS (360; 8)	S 355	12,728	K52	K54	Vakwerkligger (95)
Vert49	CS16 - CHS (360; 8)	S 355	9,006	K55	K56	Vakwerkligger (95)
Vert50	CS16 - CHS (360; 8)	S 355	12,732	K55	K57	Vakwerkligger (95)
Vert51	CS16 - CHS (360; 8)	S 355	9,000	K57	K56	Vakwerkligger (95)
Vert52	CS16 - CHS (360; 8)	S 355	8,995	K58	K59	Vakwerkligger (95)
Vert53	CS16 - CHS (360; 8)	S 355	12,724	K58	K60	Vakwerkligger (95)
Vert54	CS16 - CHS (360; 8)	S 355	9,000	K60	K59	Vakwerkligger (95)
Vert55	CS16 - CHS (360; 8)	S 355	9,004	K61	K62	Vakwerkligger (95)
Vert56	CS16 - CHS (360; 8)	S 355	12,731	K61	K63	Vakwerkligger (95)
Vert57	CS16 - CHS (360; 8)	S 355	9,000	K63	K62	Vakwerkligger (95)
Vert58	CS16 - CHS (360; 8)	S 355	8,998	K64	K65	Vakwerkligger (95)
Vert59	CS16 - CHS (360; 8)	S 355	12,726	K64	K66	Vakwerkligger (95)
Vert60	CS16 - CHS (360; 8)	S 355	9,000	K66	K65	Vakwerkligger (95)
Vert61	CS16 - CHS (360; 8)	S 355	9,003	K67	K68	Vakwerkligger (95)
Vert62	CS16 - CHS (360; 8)	S 355	12,730	K67	K69	Vakwerkligger (95)
Vert63	CS16 - CHS (360; 8)	S 355	8,997	K70	K71	Vakwerkligger (95)
Vert64	CS16 - CHS (360; 8)	S 355	12,726	K70	K72	Vakwerkligger (95)
Vert65	CS16 - CHS (360; 8)	S 355	9,000	K72	K71	Vakwerkligger (95)
Vert66	CS16 - CHS (360; 8)	S 355	9,002	K73	K74	Vakwerkligger (95)
Vert67	CS16 - CHS (360; 8)	S 355	12,729	K73	K75	Vakwerkligger (95)
Vert68	CS16 - CHS (360; 8)	S 355	9,000	K75	K74	Vakwerkligger (95)
Vert69	CS16 - CHS (360; 8)	S 355	8,994	K76	K77	Vakwerkligger (95)
Vert70	CS16 - CHS (360; 8)	S 355	12,724	K76	K78	Vakwerkligger (95)
Vert71	CS16 - CHS (360; 8)	S 355	8,994	K79	K80	Vakwerkligger (95)
Vert72	CS16 - CHS (360; 8)	S 355	12,724	K79	K81	Vakwerkligger (95)
Vert73	CS16 - CHS (360; 8)	S 355	9,002	K82	K83	Vakwerkligger (95)
Vert74	CS16 - CHS (360; 8)	S 355	12,729	K82	K84	Vakwerkligger (95)
Vert75	CS16 - CHS (360; 8)	S 355	9,000	K84	K83	Vakwerkligger (95)
Vert76	CS16 - CHS (360; 8)	S 355	8,997	K85	K86	Vakwerkligger (95)
Vert77	CS16 - CHS (360; 8)	S 355	12,726	K85	K87	Vakwerkligger (95)
Vert78	CS16 - CHS (360; 8)	S 355	9,000	K87	K86	Vakwerkligger (95)
Vert79	CS16 - CHS (360; 8)	S 355	9,003	K88	K89	Vakwerkligger (95)
Vert80	CS16 - CHS (360; 8)	S 355	12,730	K88	K90	Vakwerkligger (95)
Vert81	CS16 - CHS (360; 8)	S 355	8,998	K91	K92	Vakwerkligger (95)
Vert82	CS16 - CHS (360; 8)	S 355	12,726	K91	K93	Vakwerkligger (95)
Vert83	CS16 - CHS (360; 8)	S 355	9,000	K93	K92	Vakwerkligger (95)
Vert84	CS16 - CHS (360; 8)	S 355	9,004	K94	K95	Vakwerkligger (95)
Vert85	CS16 - CHS (360; 8)	S 355	12,731	K94	K96	Vakwerkligger (95)
Vert86	CS16 - CHS (360; 8)	S 355	9,000	K96	K95	Vakwerkligger (95)
Vert87	CS16 - CHS (360; 8)	S 355	8,995	K97	K98	Vakwerkligger (95)
Vert88	CS16 - CHS (360; 8)	S 355	12,724	K97	K99	Vakwerkligger (95)
Vert89	CS16 - CHS (360; 8)	S 355	9,000	K99	K98	Vakwerkligger (95)
Vert90	CS16 - CHS (360; 8)	S 355	9,006	K100	K101	Vakwerkligger (95)
Vert91	CS16 - CHS (360; 8)	S 355	12,732	K100	K102	Vakwerkligger (95)
Vert92	CS16 - CHS (360; 8)	S 355	9,000	K102	K101	Vakwerkligger (95)
Vert93	CS16 - CHS (360; 8)	S 355	9,000	K103	K104	Vakwerkligger (95)
Vert94	CS16 - CHS (360; 8)	S 355	12,728	K103	K105	Vakwerkligger (95)
Vert95	CS16 - CHS (360; 8)	S 355	9,006	K106	K107	Vakwerkligger (95)
Vert96	CS16 - CHS (360; 8)	S 355	12,732	K106	K108	Vakwerkligger (95)
Vert97	CS16 - CHS (360; 8)	S 355	9,000	K108	K107	Vakwerkligger (95)
Vert98	CS16 - CHS (360; 8)	S 355	8,995	K109	K110	Vakwerkligger (95)
Vert99	CS16 - CHS (360; 8)	S 355	12,724	K109	K111	Vakwerkligger (95)
Vert100	CS16 - CHS (360; 8)	S 355	9,000	K111	K110	Vakwerkligger (95)
Vert101	CS16 - CHS (360; 8)	S 355	9,004	K112	K113	Vakwerkligger (95)
Vert102	CS16 - CHS (360; 8)	S 355	12,731	K112	K114	Vakwerkligger (95)
Vert103	CS16 - CHS (360; 8)	S 355	9,000	K114	K113	Vakwerkligger (95)
Vert104	CS16 - CHS (360; 8)	S 355	8,998	K115	K116	Vakwerkligger (95)
Vert105	CS16 - CHS (360; 8)	S 355	12,726	K115	K117	Vakwerkligger (95)
Vert106	CS16 - CHS (360; 8)	S 355	9,000	K117	K116	Vakwerkligger (95)
Vert107	CS16 - CHS (360; 8)	S 355	9,003	K118	K119	Vakwerkligger (95)
Vert108	CS16 - CHS (360; 8)	S 355	12,730	K118	K120	Vakwerkligger (95)
Vert109	CS16 - CHS (360; 8)	S 355	8,997	K121	K122	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Vert110	CS16 - CHS (360; 8)	S 355	12,726	K121	K123	Vakwerkligger (95)
Vert111	CS16 - CHS (360; 8)	S 355	9,000	K123	K122	Vakwerkligger (95)
Vert112	CS16 - CHS (360; 8)	S 355	9,002	K124	K125	Vakwerkligger (95)
Vert113	CS16 - CHS (360; 8)	S 355	12,729	K124	K126	Vakwerkligger (95)
Vert114	CS16 - CHS (360; 8)	S 355	9,000	K126	K125	Vakwerkligger (95)
Vert115	CS16 - CHS (360; 8)	S 355	8,994	K127	K128	Vakwerkligger (95)
Vert116	CS16 - CHS (360; 8)	S 355	12,724	K127	K129	Vakwerkligger (95)
Vert117	CS16 - CHS (360; 8)	S 355	8,994	K130	K131	Vakwerkligger (95)
Vert118	CS16 - CHS (360; 8)	S 355	12,724	K130	K132	Vakwerkligger (95)
Vert119	CS16 - CHS (360; 8)	S 355	9,002	K133	K134	Vakwerkligger (95)
Vert120	CS16 - CHS (360; 8)	S 355	12,729	K133	K135	Vakwerkligger (95)
Vert121	CS16 - CHS (360; 8)	S 355	9,000	K135	K134	Vakwerkligger (95)
Vert122	CS16 - CHS (360; 8)	S 355	8,997	K136	K137	Vakwerkligger (95)
Vert123	CS16 - CHS (360; 8)	S 355	12,726	K136	K138	Vakwerkligger (95)
Vert124	CS16 - CHS (360; 8)	S 355	9,000	K138	K137	Vakwerkligger (95)
Vert125	CS16 - CHS (360; 8)	S 355	9,003	K139	K140	Vakwerkligger (95)
Vert126	CS16 - CHS (360; 8)	S 355	12,730	K139	K141	Vakwerkligger (95)
Vert127	CS16 - CHS (360; 8)	S 355	8,998	K142	K143	Vakwerkligger (95)
Vert128	CS16 - CHS (360; 8)	S 355	12,726	K142	K144	Vakwerkligger (95)
Vert129	CS16 - CHS (360; 8)	S 355	9,000	K144	K143	Vakwerkligger (95)
Vert130	CS16 - CHS (360; 8)	S 355	9,004	K145	K146	Vakwerkligger (95)
Vert131	CS16 - CHS (360; 8)	S 355	12,731	K145	K147	Vakwerkligger (95)
Vert132	CS16 - CHS (360; 8)	S 355	9,000	K147	K146	Vakwerkligger (95)
Vert133	CS16 - CHS (360; 8)	S 355	8,995	K148	K149	Vakwerkligger (95)
Vert134	CS16 - CHS (360; 8)	S 355	12,724	K148	K150	Vakwerkligger (95)
Vert135	CS16 - CHS (360; 8)	S 355	9,000	K150	K149	Vakwerkligger (95)
Vert136	CS16 - CHS (360; 8)	S 355	9,006	K151	K152	Vakwerkligger (95)
Vert137	CS16 - CHS (360; 8)	S 355	12,732	K151	K153	Vakwerkligger (95)
Vert138	CS16 - CHS (360; 8)	S 355	9,000	K153	K152	Vakwerkligger (95)
Vert139	CS16 - CHS (360; 8)	S 355	9,000	K154	K155	Vakwerkligger (95)
Vert140	CS16 - CHS (360; 8)	S 355	12,728	K154	K156	Vakwerkligger (95)
Vert141	CS16 - CHS (360; 8)	S 355	9,006	K157	K158	Vakwerkligger (95)
Vert142	CS16 - CHS (360; 8)	S 355	12,732	K157	K159	Vakwerkligger (95)
Vert143	CS16 - CHS (360; 8)	S 355	9,000	K159	K158	Vakwerkligger (95)
Vert144	CS16 - CHS (360; 8)	S 355	8,995	K160	K161	Vakwerkligger (95)
Vert145	CS16 - CHS (360; 8)	S 355	12,724	K160	K162	Vakwerkligger (95)
Vert146	CS16 - CHS (360; 8)	S 355	9,000	K162	K161	Vakwerkligger (95)
Vert147	CS16 - CHS (360; 8)	S 355	9,004	K163	K164	Vakwerkligger (95)
Vert148	CS16 - CHS (360; 8)	S 355	12,731	K163	K165	Vakwerkligger (95)
Vert149	CS16 - CHS (360; 8)	S 355	9,000	K165	K164	Vakwerkligger (95)
Vert150	CS16 - CHS (360; 8)	S 355	8,998	K166	K167	Vakwerkligger (95)
Vert151	CS16 - CHS (360; 8)	S 355	12,726	K166	K168	Vakwerkligger (95)
Vert152	CS16 - CHS (360; 8)	S 355	9,000	K168	K167	Vakwerkligger (95)
Vert153	CS16 - CHS (360; 8)	S 355	9,003	K169	K170	Vakwerkligger (95)
Vert154	CS16 - CHS (360; 8)	S 355	12,730	K169	K171	Vakwerkligger (95)
Vert155	CS16 - CHS (360; 8)	S 355	8,997	K172	K173	Vakwerkligger (95)
Vert156	CS16 - CHS (360; 8)	S 355	12,726	K172	K174	Vakwerkligger (95)
Vert157	CS16 - CHS (360; 8)	S 355	9,000	K174	K173	Vakwerkligger (95)
Vert158	CS16 - CHS (360; 8)	S 355	9,002	K175	K176	Vakwerkligger (95)
Vert159	CS16 - CHS (360; 8)	S 355	12,729	K175	K177	Vakwerkligger (95)
Vert160	CS16 - CHS (360; 8)	S 355	9,000	K177	K176	Vakwerkligger (95)
Vert161	CS16 - CHS (360; 8)	S 355	8,994	K178	K179	Vakwerkligger (95)
Vert162	CS16 - CHS (360; 8)	S 355	12,724	K178	K180	Vakwerkligger (95)
Vert163	CS16 - CHS (360; 8)	S 355	8,994	K181	K182	Vakwerkligger (95)
Vert164	CS16 - CHS (360; 8)	S 355	12,724	K181	K183	Vakwerkligger (95)
Vert165	CS16 - CHS (360; 8)	S 355	9,002	K184	K185	Vakwerkligger (95)
Vert166	CS16 - CHS (360; 8)	S 355	12,729	K184	K186	Vakwerkligger (95)
Vert167	CS16 - CHS (360; 8)	S 355	9,000	K186	K185	Vakwerkligger (95)
Vert168	CS16 - CHS (360; 8)	S 355	8,997	K187	K188	Vakwerkligger (95)
Vert169	CS16 - CHS (360; 8)	S 355	12,726	K187	K189	Vakwerkligger (95)
Vert170	CS16 - CHS (360; 8)	S 355	9,000	K189	K188	Vakwerkligger (95)
Vert171	CS16 - CHS (360; 8)	S 355	9,003	K190	K191	Vakwerkligger (95)
Vert172	CS16 - CHS (360; 8)	S 355	12,730	K190	K192	Vakwerkligger (95)
Vert173	CS16 - CHS (360; 8)	S 355	8,998	K193	K194	Vakwerkligger (95)
Vert174	CS16 - CHS (360; 8)	S 355	12,726	K193	K195	Vakwerkligger (95)
Vert175	CS16 - CHS (360; 8)	S 355	9,000	K195	K194	Vakwerkligger (95)
Vert176	CS16 - CHS (360; 8)	S 355	9,004	K196	K197	Vakwerkligger (95)
Vert177	CS16 - CHS (360; 8)	S 355	12,731	K196	K198	Vakwerkligger (95)
Vert178	CS16 - CHS (360; 8)	S 355	9,000	K198	K197	Vakwerkligger (95)
Vert179	CS16 - CHS (360; 8)	S 355	8,995	K199	K200	Vakwerkligger (95)
Vert180	CS16 - CHS (360; 8)	S 355	12,724	K199	K201	Vakwerkligger (95)
Vert181	CS16 - CHS (360; 8)	S 355	9,000	K201	K200	Vakwerkligger (95)
Vert182	CS16 - CHS (360; 8)	S 355	9,006	K202	K203	Vakwerkligger (95)
Vert183	CS16 - CHS (360; 8)	S 355	12,732	K202	K204	Vakwerkligger (95)
Vert184	CS16 - CHS (360; 8)	S 355	9,000	K204	K203	Vakwerkligger (95)
Diag1	CS17 - CHS (630; 15)	S 355	17,235	K4	K3	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Diag2	CS17 - CHS (630; 15)	S 355	14,696	K1	K5	Vakwerkligger (95)
Diag3	CS17 - CHS (630; 15)	S 355	14,311	K3	K5	Vakwerkligger (95)
Diag4	CS17 - CHS (630; 15)	S 355	17,335	K4	K9	Vakwerkligger (95)
Diag5	CS17 - CHS (630; 15)	S 355	14,810	K7	K5	Vakwerkligger (95)
Diag6	CS17 - CHS (630; 15)	S 355	14,421	K9	K5	Vakwerkligger (95)
Diag7	CS17 - CHS (630; 15)	S 355	17,546	K10	K9	Vakwerkligger (95)
Diag8	CS17 - CHS (630; 15)	S 355	15,058	K7	K11	Vakwerkligger (95)
Diag9	CS17 - CHS (630; 15)	S 355	14,656	K9	K11	Vakwerkligger (95)
Diag10	CS17 - CHS (630; 15)	S 355	17,909	K10	K15	Vakwerkligger (95)
Diag11	CS17 - CHS (630; 15)	S 355	15,479	K13	K11	Vakwerkligger (95)
Diag12	CS17 - CHS (630; 15)	S 355	15,051	K15	K11	Vakwerkligger (95)
Diag13	CS17 - CHS (630; 15)	S 355	12,833	K16	K15	Vakwerkligger (95)
Diag14	CS17 - CHS (630; 15)	S 355	9,137	K13	K17	Vakwerkligger (95)
Diag15	CS17 - CHS (630; 15)	S 355	9,130	K15	K17	Vakwerkligger (95)
Diag16	CS17 - CHS (630; 15)	S 355	17,360	K16	K21	Vakwerkligger (95)
Diag17	CS17 - CHS (630; 15)	S 355	14,852	K19	K17	Vakwerkligger (95)
Diag18	CS17 - CHS (630; 15)	S 355	14,457	K21	K17	Vakwerkligger (95)
Diag19	CS17 - CHS (630; 15)	S 355	15,854	K22	K21	Vakwerkligger (95)
Diag20	CS17 - CHS (630; 15)	S 355	13,052	K19	K23	Vakwerkligger (95)
Diag21	CS17 - CHS (630; 15)	S 355	12,766	K21	K23	Vakwerkligger (95)
Diag22	CS17 - CHS (630; 15)	S 355	13,757	K22	K27	Vakwerkligger (95)
Diag23	CS17 - CHS (630; 15)	S 355	10,412	K25	K23	Vakwerkligger (95)
Diag24	CS17 - CHS (630; 15)	S 355	10,300	K27	K23	Vakwerkligger (95)
Diag25	CS17 - CHS (630; 15)	S 355	17,227	K28	K27	Vakwerkligger (95)
Diag26	CS17 - CHS (630; 15)	S 355	14,690	K25	K29	Vakwerkligger (95)
Diag27	CS17 - CHS (630; 15)	S 355	14,313	K27	K29	Vakwerkligger (95)
Diag28	CS17 - CHS (630; 15)	S 355	13,762	K28	K33	Vakwerkligger (95)
Diag29	CS17 - CHS (630; 15)	S 355	10,404	K31	K29	Vakwerkligger (95)
Diag30	CS17 - CHS (630; 15)	S 355	10,300	K33	K29	Vakwerkligger (95)
Diag31	CS17 - CHS (630; 15)	S 355	15,855	K34	K33	Vakwerkligger (95)
Diag32	CS17 - CHS (630; 15)	S 355	13,052	K31	K35	Vakwerkligger (95)
Diag33	CS17 - CHS (630; 15)	S 355	12,766	K33	K35	Vakwerkligger (95)
Diag34	CS17 - CHS (630; 15)	S 355	17,366	K34	K39	Vakwerkligger (95)
Diag35	CS17 - CHS (630; 15)	S 355	14,845	K37	K35	Vakwerkligger (95)
Diag36	CS17 - CHS (630; 15)	S 355	14,457	K39	K35	Vakwerkligger (95)
Diag37	CS17 - CHS (630; 15)	S 355	12,825	K40	K39	Vakwerkligger (95)
Diag38	CS17 - CHS (630; 15)	S 355	9,148	K37	K41	Vakwerkligger (95)
Diag39	CS17 - CHS (630; 15)	S 355	9,130	K39	K41	Vakwerkligger (95)
Diag40	CS17 - CHS (630; 15)	S 355	17,905	K40	K45	Vakwerkligger (95)
Diag41	CS17 - CHS (630; 15)	S 355	15,484	K43	K41	Vakwerkligger (95)
Diag42	CS17 - CHS (630; 15)	S 355	15,051	K45	K41	Vakwerkligger (95)
Diag43	CS17 - CHS (630; 15)	S 355	17,543	K46	K45	Vakwerkligger (95)
Diag44	CS17 - CHS (630; 15)	S 355	15,062	K43	K47	Vakwerkligger (95)
Diag45	CS17 - CHS (630; 15)	S 355	14,656	K45	K47	Vakwerkligger (95)
Diag46	CS17 - CHS (630; 15)	S 355	17,330	K46	K51	Vakwerkligger (95)
Diag47	CS17 - CHS (630; 15)	S 355	14,816	K49	K47	Vakwerkligger (95)
Diag48	CS17 - CHS (630; 15)	S 355	14,421	K51	K47	Vakwerkligger (95)
Diag49	CS17 - CHS (630; 15)	S 355	17,233	K52	K51	Vakwerkligger (95)
Diag50	CS17 - CHS (630; 15)	S 355	14,698	K49	K53	Vakwerkligger (95)
Diag51	CS17 - CHS (630; 15)	S 355	14,311	K51	K53	Vakwerkligger (95)
Diag52	CS17 - CHS (630; 15)	S 355	17,233	K52	K57	Vakwerkligger (95)
Diag53	CS17 - CHS (630; 15)	S 355	14,698	K55	K53	Vakwerkligger (95)
Diag54	CS17 - CHS (630; 15)	S 355	14,311	K57	K53	Vakwerkligger (95)
Diag55	CS17 - CHS (630; 15)	S 355	17,330	K58	K57	Vakwerkligger (95)
Diag56	CS17 - CHS (630; 15)	S 355	14,816	K55	K59	Vakwerkligger (95)
Diag57	CS17 - CHS (630; 15)	S 355	14,421	K57	K59	Vakwerkligger (95)
Diag58	CS17 - CHS (630; 15)	S 355	17,543	K58	K63	Vakwerkligger (95)
Diag59	CS17 - CHS (630; 15)	S 355	15,062	K61	K59	Vakwerkligger (95)
Diag60	CS17 - CHS (630; 15)	S 355	14,656	K63	K59	Vakwerkligger (95)
Diag61	CS17 - CHS (630; 15)	S 355	17,905	K64	K63	Vakwerkligger (95)
Diag62	CS17 - CHS (630; 15)	S 355	15,484	K61	K65	Vakwerkligger (95)
Diag63	CS17 - CHS (630; 15)	S 355	15,051	K63	K65	Vakwerkligger (95)
Diag64	CS17 - CHS (630; 15)	S 355	12,825	K64	K69	Vakwerkligger (95)
Diag65	CS17 - CHS (630; 15)	S 355	9,148	K67	K65	Vakwerkligger (95)
Diag66	CS17 - CHS (630; 15)	S 355	9,130	K69	K65	Vakwerkligger (95)
Diag67	CS17 - CHS (630; 15)	S 355	17,366	K70	K69	Vakwerkligger (95)
Diag68	CS17 - CHS (630; 15)	S 355	14,845	K67	K71	Vakwerkligger (95)
Diag69	CS17 - CHS (630; 15)	S 355	14,457	K69	K71	Vakwerkligger (95)
Diag70	CS17 - CHS (630; 15)	S 355	15,855	K70	K75	Vakwerkligger (95)
Diag71	CS17 - CHS (630; 15)	S 355	13,052	K73	K71	Vakwerkligger (95)
Diag72	CS17 - CHS (630; 15)	S 355	12,766	K75	K71	Vakwerkligger (95)
Diag73	CS17 - CHS (630; 15)	S 355	13,762	K76	K75	Vakwerkligger (95)
Diag74	CS17 - CHS (630; 15)	S 355	10,404	K73	K77	Vakwerkligger (95)
Diag75	CS17 - CHS (630; 15)	S 355	10,300	K75	K77	Vakwerkligger (95)
Diag76	CS17 - CHS (630; 15)	S 355	17,227	K76	K81	Vakwerkligger (95)
Diag77	CS17 - CHS (630; 15)	S 355	14,690	K79	K77	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Diag78	CS17 - CHS (630; 15)	S 355	14,313	K81	K77	Vakwerkligger (95)
Diag79	CS17 - CHS (630; 15)	S 355	13,757	K82	K81	Vakwerkligger (95)
Diag80	CS17 - CHS (630; 15)	S 355	10,412	K79	K83	Vakwerkligger (95)
Diag81	CS17 - CHS (630; 15)	S 355	10,300	K81	K83	Vakwerkligger (95)
Diag82	CS17 - CHS (630; 15)	S 355	15,854	K82	K87	Vakwerkligger (95)
Diag83	CS17 - CHS (630; 15)	S 355	13,052	K85	K83	Vakwerkligger (95)
Diag84	CS17 - CHS (630; 15)	S 355	12,766	K87	K83	Vakwerkligger (95)
Diag85	CS17 - CHS (630; 15)	S 355	17,360	K88	K87	Vakwerkligger (95)
Diag86	CS17 - CHS (630; 15)	S 355	14,852	K85	K89	Vakwerkligger (95)
Diag87	CS17 - CHS (630; 15)	S 355	14,457	K87	K89	Vakwerkligger (95)
Diag88	CS17 - CHS (630; 15)	S 355	12,833	K88	K93	Vakwerkligger (95)
Diag89	CS17 - CHS (630; 15)	S 355	9,137	K91	K89	Vakwerkligger (95)
Diag90	CS17 - CHS (630; 15)	S 355	9,130	K93	K89	Vakwerkligger (95)
Diag91	CS17 - CHS (630; 15)	S 355	17,909	K94	K93	Vakwerkligger (95)
Diag92	CS17 - CHS (630; 15)	S 355	15,479	K91	K95	Vakwerkligger (95)
Diag93	CS17 - CHS (630; 15)	S 355	15,051	K93	K95	Vakwerkligger (95)
Diag94	CS17 - CHS (630; 15)	S 355	17,546	K94	K99	Vakwerkligger (95)
Diag95	CS17 - CHS (630; 15)	S 355	15,058	K97	K95	Vakwerkligger (95)
Diag96	CS17 - CHS (630; 15)	S 355	14,656	K99	K95	Vakwerkligger (95)
Diag97	CS17 - CHS (630; 15)	S 355	17,335	K100	K99	Vakwerkligger (95)
Diag98	CS17 - CHS (630; 15)	S 355	14,810	K97	K101	Vakwerkligger (95)
Diag99	CS17 - CHS (630; 15)	S 355	14,421	K99	K101	Vakwerkligger (95)
Diag100	CS17 - CHS (630; 15)	S 355	17,235	K100	K105	Vakwerkligger (95)
Diag101	CS17 - CHS (630; 15)	S 355	14,696	K103	K101	Vakwerkligger (95)
Diag102	CS17 - CHS (630; 15)	S 355	14,311	K105	K101	Vakwerkligger (95)
Diag103	CS17 - CHS (630; 15)	S 355	17,235	K106	K105	Vakwerkligger (95)
Diag104	CS17 - CHS (630; 15)	S 355	14,696	K103	K107	Vakwerkligger (95)
Diag105	CS17 - CHS (630; 15)	S 355	14,311	K105	K107	Vakwerkligger (95)
Diag106	CS17 - CHS (630; 15)	S 355	17,335	K106	K111	Vakwerkligger (95)
Diag107	CS17 - CHS (630; 15)	S 355	14,810	K109	K107	Vakwerkligger (95)
Diag108	CS17 - CHS (630; 15)	S 355	14,421	K111	K107	Vakwerkligger (95)
Diag109	CS17 - CHS (630; 15)	S 355	17,546	K112	K111	Vakwerkligger (95)
Diag110	CS17 - CHS (630; 15)	S 355	15,058	K109	K113	Vakwerkligger (95)
Diag111	CS17 - CHS (630; 15)	S 355	14,656	K111	K113	Vakwerkligger (95)
Diag112	CS17 - CHS (630; 15)	S 355	17,909	K112	K117	Vakwerkligger (95)
Diag113	CS17 - CHS (630; 15)	S 355	15,479	K115	K113	Vakwerkligger (95)
Diag114	CS17 - CHS (630; 15)	S 355	15,051	K117	K113	Vakwerkligger (95)
Diag115	CS17 - CHS (630; 15)	S 355	12,833	K118	K117	Vakwerkligger (95)
Diag116	CS17 - CHS (630; 15)	S 355	9,137	K115	K119	Vakwerkligger (95)
Diag117	CS17 - CHS (630; 15)	S 355	9,130	K117	K119	Vakwerkligger (95)
Diag118	CS17 - CHS (630; 15)	S 355	17,360	K118	K123	Vakwerkligger (95)
Diag119	CS17 - CHS (630; 15)	S 355	14,852	K121	K119	Vakwerkligger (95)
Diag120	CS17 - CHS (630; 15)	S 355	14,457	K123	K119	Vakwerkligger (95)
Diag121	CS17 - CHS (630; 15)	S 355	15,854	K124	K123	Vakwerkligger (95)
Diag122	CS17 - CHS (630; 15)	S 355	13,052	K121	K125	Vakwerkligger (95)
Diag123	CS17 - CHS (630; 15)	S 355	12,766	K123	K125	Vakwerkligger (95)
Diag124	CS17 - CHS (630; 15)	S 355	13,757	K124	K129	Vakwerkligger (95)
Diag125	CS17 - CHS (630; 15)	S 355	10,412	K127	K125	Vakwerkligger (95)
Diag126	CS17 - CHS (630; 15)	S 355	10,300	K129	K125	Vakwerkligger (95)
Diag127	CS17 - CHS (630; 15)	S 355	17,227	K130	K129	Vakwerkligger (95)
Diag128	CS17 - CHS (630; 15)	S 355	14,690	K127	K131	Vakwerkligger (95)
Diag129	CS17 - CHS (630; 15)	S 355	14,313	K129	K131	Vakwerkligger (95)
Diag130	CS17 - CHS (630; 15)	S 355	13,762	K130	K135	Vakwerkligger (95)
Diag131	CS17 - CHS (630; 15)	S 355	10,404	K133	K131	Vakwerkligger (95)
Diag132	CS17 - CHS (630; 15)	S 355	10,300	K135	K131	Vakwerkligger (95)
Diag133	CS17 - CHS (630; 15)	S 355	15,855	K136	K135	Vakwerkligger (95)
Diag134	CS17 - CHS (630; 15)	S 355	13,052	K133	K137	Vakwerkligger (95)
Diag135	CS17 - CHS (630; 15)	S 355	12,766	K135	K137	Vakwerkligger (95)
Diag136	CS17 - CHS (630; 15)	S 355	17,366	K136	K141	Vakwerkligger (95)
Diag137	CS17 - CHS (630; 15)	S 355	14,845	K139	K137	Vakwerkligger (95)
Diag138	CS17 - CHS (630; 15)	S 355	14,457	K141	K137	Vakwerkligger (95)
Diag139	CS17 - CHS (630; 15)	S 355	12,825	K142	K141	Vakwerkligger (95)
Diag140	CS17 - CHS (630; 15)	S 355	9,148	K139	K143	Vakwerkligger (95)
Diag141	CS17 - CHS (630; 15)	S 355	9,130	K141	K143	Vakwerkligger (95)
Diag142	CS17 - CHS (630; 15)	S 355	17,905	K142	K147	Vakwerkligger (95)
Diag143	CS17 - CHS (630; 15)	S 355	15,484	K145	K143	Vakwerkligger (95)
Diag144	CS17 - CHS (630; 15)	S 355	15,051	K147	K143	Vakwerkligger (95)
Diag145	CS17 - CHS (630; 15)	S 355	17,543	K148	K147	Vakwerkligger (95)
Diag146	CS17 - CHS (630; 15)	S 355	15,062	K145	K149	Vakwerkligger (95)
Diag147	CS17 - CHS (630; 15)	S 355	14,656	K147	K149	Vakwerkligger (95)
Diag148	CS17 - CHS (630; 15)	S 355	17,330	K148	K153	Vakwerkligger (95)
Diag149	CS17 - CHS (630; 15)	S 355	14,816	K151	K149	Vakwerkligger (95)
Diag150	CS17 - CHS (630; 15)	S 355	14,421	K153	K149	Vakwerkligger (95)
Diag151	CS17 - CHS (630; 15)	S 355	17,233	K154	K153	Vakwerkligger (95)
Diag152	CS17 - CHS (630; 15)	S 355	14,698	K151	K155	Vakwerkligger (95)
Diag153	CS17 - CHS (630; 15)	S 355	14,311	K153	K155	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Diag154	CS17 - CHS (630; 15)	S 355	17,233	K154	K159	Vakwerkligger (95)
Diag155	CS17 - CHS (630; 15)	S 355	14,698	K157	K155	Vakwerkligger (95)
Diag156	CS17 - CHS (630; 15)	S 355	14,311	K159	K155	Vakwerkligger (95)
Diag157	CS17 - CHS (630; 15)	S 355	17,330	K160	K159	Vakwerkligger (95)
Diag158	CS17 - CHS (630; 15)	S 355	14,816	K157	K161	Vakwerkligger (95)
Diag159	CS17 - CHS (630; 15)	S 355	14,421	K159	K161	Vakwerkligger (95)
Diag160	CS17 - CHS (630; 15)	S 355	17,543	K160	K165	Vakwerkligger (95)
Diag161	CS17 - CHS (630; 15)	S 355	15,062	K163	K161	Vakwerkligger (95)
Diag162	CS17 - CHS (630; 15)	S 355	14,656	K165	K161	Vakwerkligger (95)
Diag163	CS17 - CHS (630; 15)	S 355	17,905	K166	K165	Vakwerkligger (95)
Diag164	CS17 - CHS (630; 15)	S 355	15,484	K163	K167	Vakwerkligger (95)
Diag165	CS17 - CHS (630; 15)	S 355	15,051	K165	K167	Vakwerkligger (95)
Diag166	CS17 - CHS (630; 15)	S 355	12,825	K166	K171	Vakwerkligger (95)
Diag167	CS17 - CHS (630; 15)	S 355	9,148	K169	K167	Vakwerkligger (95)
Diag168	CS17 - CHS (630; 15)	S 355	9,130	K171	K167	Vakwerkligger (95)
Diag169	CS17 - CHS (630; 15)	S 355	17,366	K172	K171	Vakwerkligger (95)
Diag170	CS17 - CHS (630; 15)	S 355	14,845	K169	K173	Vakwerkligger (95)
Diag171	CS17 - CHS (630; 15)	S 355	14,457	K171	K173	Vakwerkligger (95)
Diag172	CS17 - CHS (630; 15)	S 355	15,855	K172	K177	Vakwerkligger (95)
Diag173	CS17 - CHS (630; 15)	S 355	13,052	K175	K173	Vakwerkligger (95)
Diag174	CS17 - CHS (630; 15)	S 355	12,766	K177	K173	Vakwerkligger (95)
Diag175	CS17 - CHS (630; 15)	S 355	13,762	K178	K177	Vakwerkligger (95)
Diag176	CS17 - CHS (630; 15)	S 355	10,404	K175	K179	Vakwerkligger (95)
Diag177	CS17 - CHS (630; 15)	S 355	10,300	K177	K179	Vakwerkligger (95)
Diag178	CS17 - CHS (630; 15)	S 355	17,227	K178	K183	Vakwerkligger (95)
Diag179	CS17 - CHS (630; 15)	S 355	14,690	K181	K179	Vakwerkligger (95)
Diag180	CS17 - CHS (630; 15)	S 355	14,313	K183	K179	Vakwerkligger (95)
Diag181	CS17 - CHS (630; 15)	S 355	13,757	K184	K183	Vakwerkligger (95)
Diag182	CS17 - CHS (630; 15)	S 355	10,412	K181	K185	Vakwerkligger (95)
Diag183	CS17 - CHS (630; 15)	S 355	10,300	K183	K185	Vakwerkligger (95)
Diag184	CS17 - CHS (630; 15)	S 355	15,854	K184	K189	Vakwerkligger (95)
Diag185	CS17 - CHS (630; 15)	S 355	13,052	K187	K185	Vakwerkligger (95)
Diag186	CS17 - CHS (630; 15)	S 355	12,766	K189	K185	Vakwerkligger (95)
Diag187	CS17 - CHS (630; 15)	S 355	17,360	K190	K189	Vakwerkligger (95)
Diag188	CS17 - CHS (630; 15)	S 355	14,852	K187	K191	Vakwerkligger (95)
Diag189	CS17 - CHS (630; 15)	S 355	14,457	K189	K191	Vakwerkligger (95)
Diag190	CS17 - CHS (630; 15)	S 355	12,833	K190	K195	Vakwerkligger (95)
Diag191	CS17 - CHS (630; 15)	S 355	9,137	K193	K191	Vakwerkligger (95)
Diag192	CS17 - CHS (630; 15)	S 355	9,130	K195	K191	Vakwerkligger (95)
Diag193	CS17 - CHS (630; 15)	S 355	17,909	K196	K195	Vakwerkligger (95)
Diag194	CS17 - CHS (630; 15)	S 355	15,479	K193	K197	Vakwerkligger (95)
Diag195	CS17 - CHS (630; 15)	S 355	15,051	K195	K197	Vakwerkligger (95)
Diag196	CS17 - CHS (630; 15)	S 355	17,546	K196	K201	Vakwerkligger (95)
Diag197	CS17 - CHS (630; 15)	S 355	15,058	K199	K197	Vakwerkligger (95)
Diag198	CS17 - CHS (630; 15)	S 355	14,656	K201	K197	Vakwerkligger (95)
Diag199	CS17 - CHS (630; 15)	S 355	17,335	K202	K201	Vakwerkligger (95)
Diag200	CS17 - CHS (630; 15)	S 355	14,810	K199	K203	Vakwerkligger (95)
Diag201	CS17 - CHS (630; 15)	S 355	14,421	K201	K203	Vakwerkligger (95)
Diag202	CS17 - CHS (630; 15)	S 355	17,235	K202	K3	Vakwerkligger (95)
Diag203	CS17 - CHS (630; 15)	S 355	14,696	K1	K203	Vakwerkligger (95)
Diag204	CS17 - CHS (630; 15)	S 355	14,311	K3	K203	Vakwerkligger (95)
X1	CS18 - CHS (800; 16)	S 355	1,508	K170	K217	Dakligger (90)
X2	CS18 - CHS (800; 16)	S 355	11,957	K217	K218	Dakligger (90)
X3	CS18 - CHS (800; 16)	S 355	11,540	K218	K219	Dakligger (90)
X4	CS18 - CHS (800; 16)	S 355	11,265	K219	K220	Dakligger (90)
X5	CS18 - CHS (800; 16)	S 355	11,127	K220	K221	Dakligger (90)
X6	CS18 - CHS (800; 16)	S 355	11,127	K221	K222	Dakligger (90)
X7	CS18 - CHS (800; 16)	S 355	11,265	K222	K223	Dakligger (90)
X8	CS18 - CHS (800; 16)	S 355	11,540	K223	K224	Dakligger (90)
X9	CS18 - CHS (800; 16)	S 355	11,957	K224	K225	Dakligger (90)
X10	CS18 - CHS (800; 16)	S 355	1,508	K225	K140	Dakligger (90)
X11	CS18 - CHS (800; 16)	S 355	9,700	K26	K226	Dakligger (90)
X12	CS18 - CHS (800; 16)	S 355	13,042	K226	K227	Dakligger (90)
X13	CS18 - CHS (800; 16)	S 355	12,401	K227	K228	Dakligger (90)
X14	CS18 - CHS (800; 16)	S 355	11,884	K228	K229	Dakligger (90)
X15	CS18 - CHS (800; 16)	S 355	11,505	K229	K230	Dakligger (90)
X16	CS18 - CHS (800; 16)	S 355	11,252	K230	K231	Dakligger (90)
X17	CS18 - CHS (800; 16)	S 355	11,126	K231	K232	Dakligger (90)
X18	CS18 - CHS (800; 16)	S 355	11,126	K232	K233	Dakligger (90)
X19	CS18 - CHS (800; 16)	S 355	11,252	K233	K234	Dakligger (90)
X20	CS18 - CHS (800; 16)	S 355	11,505	K234	K235	Dakligger (90)
X21	CS18 - CHS (800; 16)	S 355	11,884	K235	K236	Dakligger (90)
X22	CS18 - CHS (800; 16)	S 355	12,401	K236	K237	Dakligger (90)
X23	CS18 - CHS (800; 16)	S 355	13,042	K237	K238	Dakligger (90)
X24	CS18 - CHS (800; 16)	S 355	9,700	K238	K30	Dakligger (90)
X25	CS18 - CHS (800; 16)	S 355	6,955	K20	K239	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
X26	CS18 - CHS (800; 16)	S 355	13,737	K239	K240	Dakligger (90)
X27	CS18 - CHS (800; 16)	S 355	12,979	K240	K241	Dakligger (90)
X28	CS18 - CHS (800; 16)	S 355	12,357	K241	K242	Dakligger (90)
X29	CS18 - CHS (800; 16)	S 355	11,860	K242	K243	Dakligger (90)
X30	CS18 - CHS (800; 16)	S 355	11,490	K243	K244	Dakligger (90)
X31	CS18 - CHS (800; 16)	S 355	11,247	K244	K245	Dakligger (90)
X32	CS18 - CHS (800; 16)	S 355	11,125	K245	K246	Dakligger (90)
X33	CS18 - CHS (800; 16)	S 355	11,125	K246	K247	Dakligger (90)
X34	CS18 - CHS (800; 16)	S 355	11,247	K247	K248	Dakligger (90)
X35	CS18 - CHS (800; 16)	S 355	11,490	K248	K249	Dakligger (90)
X36	CS18 - CHS (800; 16)	S 355	11,860	K249	K250	Dakligger (90)
X37	CS18 - CHS (800; 16)	S 355	12,357	K250	K251	Dakligger (90)
X38	CS18 - CHS (800; 16)	S 355	12,979	K251	K252	Dakligger (90)
X39	CS18 - CHS (800; 16)	S 355	13,737	K252	K253	Dakligger (90)
X40	CS18 - CHS (800; 16)	S 355	6,955	K253	K86	Dakligger (90)
X41	CS18 - CHS (800; 16)	S 355	8,525	K176	K254	Dakligger (90)
X42	CS18 - CHS (800; 16)	S 355	12,455	K254	K255	Dakligger (90)
X43	CS18 - CHS (800; 16)	S 355	11,917	K255	K256	Dakligger (90)
X44	CS18 - CHS (800; 16)	S 355	11,521	K256	K257	Dakligger (90)
X45	CS18 - CHS (800; 16)	S 355	11,258	K257	K258	Dakligger (90)
X46	CS18 - CHS (800; 16)	S 355	11,126	K258	K259	Dakligger (90)
X47	CS18 - CHS (800; 16)	S 355	11,126	K259	K260	Dakligger (90)
X48	CS18 - CHS (800; 16)	S 355	11,258	K260	K261	Dakligger (90)
X49	CS18 - CHS (800; 16)	S 355	11,521	K261	K262	Dakligger (90)
X50	CS18 - CHS (800; 16)	S 355	11,917	K262	K263	Dakligger (90)
X51	CS18 - CHS (800; 16)	S 355	12,455	K263	K264	Dakligger (90)
X52	CS18 - CHS (800; 16)	S 355	8,525	K264	K134	Dakligger (90)
X53	CS18 - CHS (800; 16)	S 355	0,934	K14	K265	Dakligger (90)
X54	CS18 - CHS (800; 16)	S 355	14,553	K265	K266	Dakligger (90)
X55	CS18 - CHS (800; 16)	S 355	13,667	K266	K267	Dakligger (90)
X56	CS18 - CHS (800; 16)	S 355	12,928	K267	K268	Dakligger (90)
X57	CS18 - CHS (800; 16)	S 355	12,327	K268	K269	Dakligger (90)
X58	CS18 - CHS (800; 16)	S 355	11,839	K269	K270	Dakligger (90)
X59	CS18 - CHS (800; 16)	S 355	11,480	K270	K271	Dakligger (90)
X60	CS18 - CHS (800; 16)	S 355	11,242	K271	K272	Dakligger (90)
X61	CS18 - CHS (800; 16)	S 355	11,125	K272	K273	Dakligger (90)
X62	CS18 - CHS (800; 16)	S 355	11,125	K273	K274	Dakligger (90)
X63	CS18 - CHS (800; 16)	S 355	11,242	K274	K275	Dakligger (90)
X64	CS18 - CHS (800; 16)	S 355	11,480	K275	K276	Dakligger (90)
X65	CS18 - CHS (800; 16)	S 355	11,839	K276	K277	Dakligger (90)
X66	CS18 - CHS (800; 16)	S 355	12,327	K277	K278	Dakligger (90)
X67	CS18 - CHS (800; 16)	S 355	12,928	K278	K279	Dakligger (90)
X68	CS18 - CHS (800; 16)	S 355	13,667	K279	K280	Dakligger (90)
X69	CS18 - CHS (800; 16)	S 355	14,553	K280	K281	Dakligger (90)
X70	CS18 - CHS (800; 16)	S 355	0,934	K281	K92	Dakligger (90)
X71	CS18 - CHS (800; 16)	S 355	7,386	K11	K282	Dakligger (90)
X72	CS18 - CHS (800; 16)	S 355	14,482	K282	K283	Dakligger (90)
X73	CS18 - CHS (800; 16)	S 355	13,615	K283	K284	Dakligger (90)
X74	CS18 - CHS (800; 16)	S 355	12,892	K284	K285	Dakligger (90)
X75	CS18 - CHS (800; 16)	S 355	12,301	K285	K286	Dakligger (90)
X76	CS18 - CHS (800; 16)	S 355	11,818	K286	K287	Dakligger (90)
X77	CS18 - CHS (800; 16)	S 355	11,472	K287	K288	Dakligger (90)
X78	CS18 - CHS (800; 16)	S 355	11,239	K288	K289	Dakligger (90)
X79	CS18 - CHS (800; 16)	S 355	11,125	K289	K290	Dakligger (90)
X80	CS18 - CHS (800; 16)	S 355	11,125	K290	K291	Dakligger (90)
X81	CS18 - CHS (800; 16)	S 355	11,239	K291	K292	Dakligger (90)
X82	CS18 - CHS (800; 16)	S 355	11,472	K292	K293	Dakligger (90)
X83	CS18 - CHS (800; 16)	S 355	11,818	K293	K294	Dakligger (90)
X84	CS18 - CHS (800; 16)	S 355	12,301	K294	K295	Dakligger (90)
X85	CS18 - CHS (800; 16)	S 355	12,892	K295	K296	Dakligger (90)
X86	CS18 - CHS (800; 16)	S 355	13,615	K296	K297	Dakligger (90)
X87	CS18 - CHS (800; 16)	S 355	14,482	K297	K298	Dakligger (90)
X88	CS18 - CHS (800; 16)	S 355	7,386	K298	K95	Dakligger (90)
X89	CS18 - CHS (800; 16)	S 355	14,513	K203	K299	Dakligger (90)
X90	CS18 - CHS (800; 16)	S 355	14,412	K299	K300	Dakligger (90)
X91	CS18 - CHS (800; 16)	S 355	13,557	K300	K301	Dakligger (90)
X92	CS18 - CHS (800; 16)	S 355	12,847	K301	K302	Dakligger (90)
X93	CS18 - CHS (800; 16)	S 355	12,272	K302	K303	Dakligger (90)
X94	CS18 - CHS (800; 16)	S 355	11,801	K303	K304	Dakligger (90)
X95	CS18 - CHS (800; 16)	S 355	11,462	K304	K305	Dakligger (90)
X96	CS18 - CHS (800; 16)	S 355	11,236	K305	K306	Dakligger (90)
X97	CS18 - CHS (800; 16)	S 355	11,124	K306	K307	Dakligger (90)
X98	CS18 - CHS (800; 16)	S 355	11,124	K307	K308	Dakligger (90)
X99	CS18 - CHS (800; 16)	S 355	11,236	K308	K309	Dakligger (90)
X100	CS18 - CHS (800; 16)	S 355	11,462	K309	K310	Dakligger (90)
X101	CS18 - CHS (800; 16)	S 355	11,801	K310	K311	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
X102	CS18 - CHS (800; 16)	S 355	12,272	K311	K312	Dakligger (90)
X103	CS18 - CHS (800; 16)	S 355	12,847	K312	K313	Dakligger (90)
X104	CS18 - CHS (800; 16)	S 355	13,557	K313	K314	Dakligger (90)
X105	CS18 - CHS (800; 16)	S 355	14,412	K314	K315	Dakligger (90)
X106	CS18 - CHS (800; 16)	S 355	14,513	K315	K107	Dakligger (90)
X107	CS18 - CHS (800; 16)	S 355	0,934	K194	K316	Dakligger (90)
X108	CS18 - CHS (800; 16)	S 355	14,553	K316	K317	Dakligger (90)
X109	CS18 - CHS (800; 16)	S 355	13,667	K317	K318	Dakligger (90)
X110	CS18 - CHS (800; 16)	S 355	12,928	K318	K319	Dakligger (90)
X111	CS18 - CHS (800; 16)	S 355	12,327	K319	K320	Dakligger (90)
X112	CS18 - CHS (800; 16)	S 355	11,839	K320	K321	Dakligger (90)
X113	CS18 - CHS (800; 16)	S 355	11,480	K321	K322	Dakligger (90)
X114	CS18 - CHS (800; 16)	S 355	11,242	K322	K323	Dakligger (90)
X115	CS18 - CHS (800; 16)	S 355	11,125	K323	K324	Dakligger (90)
X116	CS18 - CHS (800; 16)	S 355	11,125	K324	K325	Dakligger (90)
X117	CS18 - CHS (800; 16)	S 355	11,242	K325	K326	Dakligger (90)
X118	CS18 - CHS (800; 16)	S 355	11,480	K326	K327	Dakligger (90)
X119	CS18 - CHS (800; 16)	S 355	11,839	K327	K328	Dakligger (90)
X120	CS18 - CHS (800; 16)	S 355	12,327	K328	K329	Dakligger (90)
X121	CS18 - CHS (800; 16)	S 355	12,928	K329	K330	Dakligger (90)
X122	CS18 - CHS (800; 16)	S 355	13,667	K330	K331	Dakligger (90)
X123	CS18 - CHS (800; 16)	S 355	14,553	K331	K332	Dakligger (90)
X124	CS18 - CHS (800; 16)	S 355	0,934	K332	K116	Dakligger (90)
X125	CS18 - CHS (800; 16)	S 355	15,390	K2	K333	Dakligger (90)
X126	CS18 - CHS (800; 16)	S 355	14,399	K333	K334	Dakligger (90)
X127	CS18 - CHS (800; 16)	S 355	13,552	K334	K335	Dakligger (90)
X128	CS18 - CHS (800; 16)	S 355	12,842	K335	K336	Dakligger (90)
X129	CS18 - CHS (800; 16)	S 355	12,267	K336	K337	Dakligger (90)
X130	CS18 - CHS (800; 16)	S 355	11,801	K337	K338	Dakligger (90)
X131	CS18 - CHS (800; 16)	S 355	11,460	K338	K339	Dakligger (90)
X132	CS18 - CHS (800; 16)	S 355	11,236	K339	K340	Dakligger (90)
X133	CS18 - CHS (800; 16)	S 355	11,124	K340	K341	Dakligger (90)
X134	CS18 - CHS (800; 16)	S 355	11,124	K341	K342	Dakligger (90)
X135	CS18 - CHS (800; 16)	S 355	11,236	K342	K343	Dakligger (90)
X136	CS18 - CHS (800; 16)	S 355	11,460	K343	K344	Dakligger (90)
X137	CS18 - CHS (800; 16)	S 355	11,801	K344	K345	Dakligger (90)
X138	CS18 - CHS (800; 16)	S 355	12,267	K345	K346	Dakligger (90)
X139	CS18 - CHS (800; 16)	S 355	12,842	K346	K347	Dakligger (90)
X140	CS18 - CHS (800; 16)	S 355	13,552	K347	K348	Dakligger (90)
X141	CS18 - CHS (800; 16)	S 355	14,399	K348	K349	Dakligger (90)
X142	CS18 - CHS (800; 16)	S 355	15,390	K349	K104	Dakligger (90)
X143	CS18 - CHS (800; 16)	S 355	1,508	K38	K350	Dakligger (90)
X144	CS18 - CHS (800; 16)	S 355	11,957	K350	K351	Dakligger (90)
X145	CS18 - CHS (800; 16)	S 355	11,540	K351	K352	Dakligger (90)
X146	CS18 - CHS (800; 16)	S 355	11,265	K352	K353	Dakligger (90)
X147	CS18 - CHS (800; 16)	S 355	11,127	K353	K354	Dakligger (90)
X148	CS18 - CHS (800; 16)	S 355	11,127	K354	K355	Dakligger (90)
X149	CS18 - CHS (800; 16)	S 355	11,265	K355	K356	Dakligger (90)
X150	CS18 - CHS (800; 16)	S 355	11,540	K356	K357	Dakligger (90)
X151	CS18 - CHS (800; 16)	S 355	11,957	K357	K358	Dakligger (90)
X152	CS18 - CHS (800; 16)	S 355	1,508	K358	K68	Dakligger (90)
X153	CS18 - CHS (800; 16)	S 355	11,869	K200	K359	Dakligger (90)
X154	CS18 - CHS (800; 16)	S 355	14,437	K359	K360	Dakligger (90)
X155	CS18 - CHS (800; 16)	S 355	13,580	K360	K361	Dakligger (90)
X156	CS18 - CHS (800; 16)	S 355	12,862	K361	K362	Dakligger (90)
X157	CS18 - CHS (800; 16)	S 355	12,284	K362	K363	Dakligger (90)
X158	CS18 - CHS (800; 16)	S 355	11,808	K363	K364	Dakligger (90)
X159	CS18 - CHS (800; 16)	S 355	11,465	K364	K365	Dakligger (90)
X160	CS18 - CHS (800; 16)	S 355	11,238	K365	K366	Dakligger (90)
X161	CS18 - CHS (800; 16)	S 355	11,124	K366	K367	Dakligger (90)
X162	CS18 - CHS (800; 16)	S 355	11,124	K367	K368	Dakligger (90)
X163	CS18 - CHS (800; 16)	S 355	11,238	K368	K369	Dakligger (90)
X164	CS18 - CHS (800; 16)	S 355	11,465	K369	K370	Dakligger (90)
X165	CS18 - CHS (800; 16)	S 355	11,808	K370	K371	Dakligger (90)
X166	CS18 - CHS (800; 16)	S 355	12,284	K371	K372	Dakligger (90)
X167	CS18 - CHS (800; 16)	S 355	12,862	K372	K373	Dakligger (90)
X168	CS18 - CHS (800; 16)	S 355	13,580	K373	K374	Dakligger (90)
X169	CS18 - CHS (800; 16)	S 355	14,437	K374	K375	Dakligger (90)
X170	CS18 - CHS (800; 16)	S 355	11,869	K375	K110	Dakligger (90)
X171	CS18 - CHS (800; 16)	S 355	9,700	K182	K376	Dakligger (90)
X172	CS18 - CHS (800; 16)	S 355	13,042	K376	K377	Dakligger (90)
X173	CS18 - CHS (800; 16)	S 355	12,401	K377	K378	Dakligger (90)
X174	CS18 - CHS (800; 16)	S 355	11,884	K378	K379	Dakligger (90)
X175	CS18 - CHS (800; 16)	S 355	11,505	K379	K380	Dakligger (90)
X176	CS18 - CHS (800; 16)	S 355	11,252	K380	K381	Dakligger (90)
X177	CS18 - CHS (800; 16)	S 355	11,126	K381	K382	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
X178	CS18 - CHS (800; 16)	S 355	11,126	K382	K383	Dakligger (90)
X179	CS18 - CHS (800; 16)	S 355	11,252	K383	K384	Dakligger (90)
X180	CS18 - CHS (800; 16)	S 355	11,505	K384	K385	Dakligger (90)
X181	CS18 - CHS (800; 16)	S 355	11,884	K385	K386	Dakligger (90)
X182	CS18 - CHS (800; 16)	S 355	12,401	K386	K387	Dakligger (90)
X183	CS18 - CHS (800; 16)	S 355	13,042	K387	K388	Dakligger (90)
X184	CS18 - CHS (800; 16)	S 355	9,700	K388	K128	Dakligger (90)
X185	CS18 - CHS (800; 16)	S 355	11,869	K8	K389	Dakligger (90)
X186	CS18 - CHS (800; 16)	S 355	14,437	K389	K390	Dakligger (90)
X187	CS18 - CHS (800; 16)	S 355	13,580	K390	K391	Dakligger (90)
X188	CS18 - CHS (800; 16)	S 355	12,862	K391	K392	Dakligger (90)
X189	CS18 - CHS (800; 16)	S 355	12,284	K392	K393	Dakligger (90)
X190	CS18 - CHS (800; 16)	S 355	11,808	K393	K394	Dakligger (90)
X191	CS18 - CHS (800; 16)	S 355	11,465	K394	K395	Dakligger (90)
X192	CS18 - CHS (800; 16)	S 355	11,238	K395	K396	Dakligger (90)
X193	CS18 - CHS (800; 16)	S 355	11,124	K396	K397	Dakligger (90)
X194	CS18 - CHS (800; 16)	S 355	11,124	K397	K398	Dakligger (90)
X195	CS18 - CHS (800; 16)	S 355	11,238	K398	K399	Dakligger (90)
X196	CS18 - CHS (800; 16)	S 355	11,465	K399	K400	Dakligger (90)
X197	CS18 - CHS (800; 16)	S 355	11,808	K400	K401	Dakligger (90)
X198	CS18 - CHS (800; 16)	S 355	12,284	K401	K402	Dakligger (90)
X199	CS18 - CHS (800; 16)	S 355	12,862	K402	K403	Dakligger (90)
X200	CS18 - CHS (800; 16)	S 355	13,580	K403	K404	Dakligger (90)
X201	CS18 - CHS (800; 16)	S 355	14,437	K404	K405	Dakligger (90)
X202	CS18 - CHS (800; 16)	S 355	11,869	K405	K98	Dakligger (90)
X203	CS18 - CHS (800; 16)	S 355	7,386	K197	K406	Dakligger (90)
X204	CS18 - CHS (800; 16)	S 355	14,482	K406	K407	Dakligger (90)
X205	CS18 - CHS (800; 16)	S 355	13,615	K407	K408	Dakligger (90)
X206	CS18 - CHS (800; 16)	S 355	12,892	K408	K409	Dakligger (90)
X207	CS18 - CHS (800; 16)	S 355	12,301	K409	K410	Dakligger (90)
X208	CS18 - CHS (800; 16)	S 355	11,818	K410	K411	Dakligger (90)
X209	CS18 - CHS (800; 16)	S 355	11,472	K411	K412	Dakligger (90)
X210	CS18 - CHS (800; 16)	S 355	11,239	K412	K413	Dakligger (90)
X211	CS18 - CHS (800; 16)	S 355	11,125	K413	K414	Dakligger (90)
X212	CS18 - CHS (800; 16)	S 355	11,125	K414	K415	Dakligger (90)
X213	CS18 - CHS (800; 16)	S 355	11,239	K415	K416	Dakligger (90)
X214	CS18 - CHS (800; 16)	S 355	11,472	K416	K417	Dakligger (90)
X215	CS18 - CHS (800; 16)	S 355	11,818	K417	K418	Dakligger (90)
X216	CS18 - CHS (800; 16)	S 355	12,301	K418	K419	Dakligger (90)
X217	CS18 - CHS (800; 16)	S 355	12,892	K419	K420	Dakligger (90)
X218	CS18 - CHS (800; 16)	S 355	13,615	K420	K421	Dakligger (90)
X219	CS18 - CHS (800; 16)	S 355	14,482	K421	K422	Dakligger (90)
X220	CS18 - CHS (800; 16)	S 355	7,386	K422	K113	Dakligger (90)
X221	CS18 - CHS (800; 16)	S 355	14,513	K5	K423	Dakligger (90)
X222	CS18 - CHS (800; 16)	S 355	14,412	K423	K424	Dakligger (90)
X223	CS18 - CHS (800; 16)	S 355	13,557	K424	K425	Dakligger (90)
X224	CS18 - CHS (800; 16)	S 355	12,847	K425	K426	Dakligger (90)
X225	CS18 - CHS (800; 16)	S 355	12,272	K426	K427	Dakligger (90)
X226	CS18 - CHS (800; 16)	S 355	11,801	K427	K428	Dakligger (90)
X227	CS18 - CHS (800; 16)	S 355	11,462	K428	K429	Dakligger (90)
X228	CS18 - CHS (800; 16)	S 355	11,236	K429	K430	Dakligger (90)
X229	CS18 - CHS (800; 16)	S 355	11,124	K430	K431	Dakligger (90)
X230	CS18 - CHS (800; 16)	S 355	11,124	K431	K432	Dakligger (90)
X231	CS18 - CHS (800; 16)	S 355	11,236	K432	K433	Dakligger (90)
X232	CS18 - CHS (800; 16)	S 355	11,462	K433	K434	Dakligger (90)
X233	CS18 - CHS (800; 16)	S 355	11,801	K434	K435	Dakligger (90)
X234	CS18 - CHS (800; 16)	S 355	12,272	K435	K436	Dakligger (90)
X235	CS18 - CHS (800; 16)	S 355	12,847	K436	K437	Dakligger (90)
X236	CS18 - CHS (800; 16)	S 355	13,557	K437	K438	Dakligger (90)
X237	CS18 - CHS (800; 16)	S 355	14,412	K438	K439	Dakligger (90)
X238	CS18 - CHS (800; 16)	S 355	14,513	K439	K101	Dakligger (90)
X239	CS18 - CHS (800; 16)	S 355	8,525	K32	K440	Dakligger (90)
X240	CS18 - CHS (800; 16)	S 355	12,455	K440	K441	Dakligger (90)
X241	CS18 - CHS (800; 16)	S 355	11,917	K441	K442	Dakligger (90)
X242	CS18 - CHS (800; 16)	S 355	11,521	K442	K443	Dakligger (90)
X243	CS18 - CHS (800; 16)	S 355	11,258	K443	K444	Dakligger (90)
X244	CS18 - CHS (800; 16)	S 355	11,126	K444	K445	Dakligger (90)
X245	CS18 - CHS (800; 16)	S 355	11,126	K445	K446	Dakligger (90)
X246	CS18 - CHS (800; 16)	S 355	11,258	K446	K447	Dakligger (90)
X247	CS18 - CHS (800; 16)	S 355	11,521	K447	K448	Dakligger (90)
X248	CS18 - CHS (800; 16)	S 355	11,917	K448	K449	Dakligger (90)
X249	CS18 - CHS (800; 16)	S 355	12,455	K449	K450	Dakligger (90)
X250	CS18 - CHS (800; 16)	S 355	8,525	K450	K74	Dakligger (90)
X251	CS18 - CHS (800; 16)	S 355	6,955	K188	K451	Dakligger (90)
X252	CS18 - CHS (800; 16)	S 355	13,737	K451	K452	Dakligger (90)
X253	CS18 - CHS (800; 16)	S 355	12,979	K452	K453	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
X254	CS18 - CHS (800; 16)	S 355	12,357	K453	K454	Dakligger (90)
X255	CS18 - CHS (800; 16)	S 355	11,860	K454	K455	Dakligger (90)
X256	CS18 - CHS (800; 16)	S 355	11,490	K455	K456	Dakligger (90)
X257	CS18 - CHS (800; 16)	S 355	11,247	K456	K457	Dakligger (90)
X258	CS18 - CHS (800; 16)	S 355	11,125	K457	K458	Dakligger (90)
X259	CS18 - CHS (800; 16)	S 355	11,125	K458	K459	Dakligger (90)
X260	CS18 - CHS (800; 16)	S 355	11,247	K459	K460	Dakligger (90)
X261	CS18 - CHS (800; 16)	S 355	11,490	K460	K461	Dakligger (90)
X262	CS18 - CHS (800; 16)	S 355	11,860	K461	K462	Dakligger (90)
X263	CS18 - CHS (800; 16)	S 355	12,357	K462	K463	Dakligger (90)
X264	CS18 - CHS (800; 16)	S 355	12,979	K463	K464	Dakligger (90)
X265	CS18 - CHS (800; 16)	S 355	13,737	K464	K465	Dakligger (90)
X266	CS18 - CHS (800; 16)	S 355	6,955	K465	K122	Dakligger (90)
Y1	CS19 - CHS (800; 16)	S 355	1,508	K17	K265	Dakligger (90)
Y2	CS19 - CHS (800; 16)	S 355	11,957	K265	K282	Dakligger (90)
Y3	CS19 - CHS (800; 16)	S 355	11,540	K282	K389	Dakligger (90)
Y4	CS19 - CHS (800; 16)	S 355	11,265	K389	K423	Dakligger (90)
Y5	CS19 - CHS (800; 16)	S 355	11,127	K423	K333	Dakligger (90)
Y6	CS19 - CHS (800; 16)	S 355	11,127	K333	K299	Dakligger (90)
Y7	CS19 - CHS (800; 16)	S 355	11,265	K299	K359	Dakligger (90)
Y8	CS19 - CHS (800; 16)	S 355	11,540	K359	K406	Dakligger (90)
Y9	CS19 - CHS (800; 16)	S 355	11,957	K406	K316	Dakligger (90)
Y10	CS19 - CHS (800; 16)	S 355	1,508	K316	K191	Dakligger (90)
Y11	CS19 - CHS (800; 16)	S 355	14,513	K56	K355	Dakligger (90)
Y12	CS19 - CHS (800; 16)	S 355	14,412	K355	K446	Dakligger (90)
Y13	CS19 - CHS (800; 16)	S 355	13,557	K446	K233	Dakligger (90)
Y14	CS19 - CHS (800; 16)	S 355	12,847	K233	K247	Dakligger (90)
Y15	CS19 - CHS (800; 16)	S 355	12,272	K247	K274	Dakligger (90)
Y16	CS19 - CHS (800; 16)	S 355	11,801	K274	K291	Dakligger (90)
Y17	CS19 - CHS (800; 16)	S 355	11,462	K291	K398	Dakligger (90)
Y18	CS19 - CHS (800; 16)	S 355	11,236	K398	K432	Dakligger (90)
Y19	CS19 - CHS (800; 16)	S 355	11,124	K432	K342	Dakligger (90)
Y20	CS19 - CHS (800; 16)	S 355	11,124	K342	K308	Dakligger (90)
Y21	CS19 - CHS (800; 16)	S 355	11,236	K308	K368	Dakligger (90)
Y22	CS19 - CHS (800; 16)	S 355	11,462	K368	K415	Dakligger (90)
Y23	CS19 - CHS (800; 16)	S 355	11,801	K415	K325	Dakligger (90)
Y24	CS19 - CHS (800; 16)	S 355	12,272	K325	K459	Dakligger (90)
Y25	CS19 - CHS (800; 16)	S 355	12,847	K459	K383	Dakligger (90)
Y26	CS19 - CHS (800; 16)	S 355	13,557	K383	K260	Dakligger (90)
Y27	CS19 - CHS (800; 16)	S 355	14,412	K260	K222	Dakligger (90)
Y28	CS19 - CHS (800; 16)	S 355	14,513	K222	K152	Dakligger (90)
Y29	CS19 - CHS (800; 16)	S 355	8,525	K23	K239	Dakligger (90)
Y30	CS19 - CHS (800; 16)	S 355	12,455	K239	K266	Dakligger (90)
Y31	CS19 - CHS (800; 16)	S 355	11,917	K266	K283	Dakligger (90)
Y32	CS19 - CHS (800; 16)	S 355	11,521	K283	K390	Dakligger (90)
Y33	CS19 - CHS (800; 16)	S 355	11,258	K390	K424	Dakligger (90)
Y34	CS19 - CHS (800; 16)	S 355	11,126	K424	K334	Dakligger (90)
Y35	CS19 - CHS (800; 16)	S 355	11,126	K334	K300	Dakligger (90)
Y36	CS19 - CHS (800; 16)	S 355	11,258	K300	K360	Dakligger (90)
Y37	CS19 - CHS (800; 16)	S 355	11,521	K360	K407	Dakligger (90)
Y38	CS19 - CHS (800; 16)	S 355	11,917	K407	K317	Dakligger (90)
Y39	CS19 - CHS (800; 16)	S 355	12,455	K317	K451	Dakligger (90)
Y40	CS19 - CHS (800; 16)	S 355	8,525	K451	K185	Dakligger (90)
Y41	CS19 - CHS (800; 16)	S 355	15,390	K53	K354	Dakligger (90)
Y42	CS19 - CHS (800; 16)	S 355	14,399	K354	K445	Dakligger (90)
Y43	CS19 - CHS (800; 16)	S 355	13,552	K445	K232	Dakligger (90)
Y44	CS19 - CHS (800; 16)	S 355	12,842	K232	K246	Dakligger (90)
Y45	CS19 - CHS (800; 16)	S 355	12,267	K246	K273	Dakligger (90)
Y46	CS19 - CHS (800; 16)	S 355	11,801	K273	K290	Dakligger (90)
Y47	CS19 - CHS (800; 16)	S 355	11,460	K290	K397	Dakligger (90)
Y48	CS19 - CHS (800; 16)	S 355	11,236	K397	K431	Dakligger (90)
Y49	CS19 - CHS (800; 16)	S 355	11,124	K431	K341	Dakligger (90)
Y50	CS19 - CHS (800; 16)	S 355	11,124	K341	K307	Dakligger (90)
Y51	CS19 - CHS (800; 16)	S 355	11,236	K307	K367	Dakligger (90)
Y52	CS19 - CHS (800; 16)	S 355	11,460	K367	K414	Dakligger (90)
Y53	CS19 - CHS (800; 16)	S 355	11,801	K414	K324	Dakligger (90)
Y54	CS19 - CHS (800; 16)	S 355	12,267	K324	K458	Dakligger (90)
Y55	CS19 - CHS (800; 16)	S 355	12,842	K458	K382	Dakligger (90)
Y56	CS19 - CHS (800; 16)	S 355	13,552	K382	K259	Dakligger (90)
Y57	CS19 - CHS (800; 16)	S 355	14,399	K259	K221	Dakligger (90)
Y58	CS19 - CHS (800; 16)	S 355	15,390	K221	K155	Dakligger (90)
Y59	CS19 - CHS (800; 16)	S 355	9,700	K29	K226	Dakligger (90)
Y60	CS19 - CHS (800; 16)	S 355	13,042	K226	K240	Dakligger (90)
Y61	CS19 - CHS (800; 16)	S 355	12,401	K240	K267	Dakligger (90)
Y62	CS19 - CHS (800; 16)	S 355	11,884	K267	K284	Dakligger (90)
Y63	CS19 - CHS (800; 16)	S 355	11,505	K284	K391	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Y64	CS19 - CHS (800; 16)	S 355	11,252	K391	K425	Dakligger (90)
Y65	CS19 - CHS (800; 16)	S 355	11,126	K425	K335	Dakligger (90)
Y66	CS19 - CHS (800; 16)	S 355	11,126	K335	K301	Dakligger (90)
Y67	CS19 - CHS (800; 16)	S 355	11,252	K301	K361	Dakligger (90)
Y68	CS19 - CHS (800; 16)	S 355	11,505	K361	K408	Dakligger (90)
Y69	CS19 - CHS (800; 16)	S 355	11,884	K408	K318	Dakligger (90)
Y70	CS19 - CHS (800; 16)	S 355	12,401	K318	K452	Dakligger (90)
Y71	CS19 - CHS (800; 16)	S 355	13,042	K452	K376	Dakligger (90)
Y72	CS19 - CHS (800; 16)	S 355	9,700	K376	K179	Dakligger (90)
Y73	CS19 - CHS (800; 16)	S 355	6,955	K35	K440	Dakligger (90)
Y74	CS19 - CHS (800; 16)	S 355	13,737	K440	K227	Dakligger (90)
Y75	CS19 - CHS (800; 16)	S 355	12,979	K227	K241	Dakligger (90)
Y76	CS19 - CHS (800; 16)	S 355	12,357	K241	K268	Dakligger (90)
Y77	CS19 - CHS (800; 16)	S 355	11,860	K268	K285	Dakligger (90)
Y78	CS19 - CHS (800; 16)	S 355	11,490	K285	K392	Dakligger (90)
Y79	CS19 - CHS (800; 16)	S 355	11,247	K392	K426	Dakligger (90)
Y80	CS19 - CHS (800; 16)	S 355	11,125	K426	K336	Dakligger (90)
Y81	CS19 - CHS (800; 16)	S 355	11,125	K336	K302	Dakligger (90)
Y82	CS19 - CHS (800; 16)	S 355	11,247	K302	K362	Dakligger (90)
Y83	CS19 - CHS (800; 16)	S 355	11,490	K362	K409	Dakligger (90)
Y84	CS19 - CHS (800; 16)	S 355	11,860	K409	K319	Dakligger (90)
Y85	CS19 - CHS (800; 16)	S 355	12,357	K319	K453	Dakligger (90)
Y86	CS19 - CHS (800; 16)	S 355	12,979	K453	K377	Dakligger (90)
Y87	CS19 - CHS (800; 16)	S 355	13,737	K377	K254	Dakligger (90)
Y88	CS19 - CHS (800; 16)	S 355	6,955	K254	K173	Dakligger (90)
Y89	CS19 - CHS (800; 16)	S 355	7,386	K62	K357	Dakligger (90)
Y90	CS19 - CHS (800; 16)	S 355	14,482	K357	K448	Dakligger (90)
Y91	CS19 - CHS (800; 16)	S 355	13,615	K448	K235	Dakligger (90)
Y92	CS19 - CHS (800; 16)	S 355	12,892	K235	K249	Dakligger (90)
Y93	CS19 - CHS (800; 16)	S 355	12,301	K249	K276	Dakligger (90)
Y94	CS19 - CHS (800; 16)	S 355	11,818	K276	K293	Dakligger (90)
Y95	CS19 - CHS (800; 16)	S 355	11,472	K293	K400	Dakligger (90)
Y96	CS19 - CHS (800; 16)	S 355	11,239	K400	K434	Dakligger (90)
Y97	CS19 - CHS (800; 16)	S 355	11,125	K434	K344	Dakligger (90)
Y98	CS19 - CHS (800; 16)	S 355	11,125	K344	K310	Dakligger (90)
Y99	CS19 - CHS (800; 16)	S 355	11,239	K310	K370	Dakligger (90)
Y100	CS19 - CHS (800; 16)	S 355	11,472	K370	K417	Dakligger (90)
Y101	CS19 - CHS (800; 16)	S 355	11,818	K417	K327	Dakligger (90)
Y102	CS19 - CHS (800; 16)	S 355	12,301	K327	K461	Dakligger (90)
Y103	CS19 - CHS (800; 16)	S 355	12,892	K461	K385	Dakligger (90)
Y104	CS19 - CHS (800; 16)	S 355	13,615	K385	K262	Dakligger (90)
Y105	CS19 - CHS (800; 16)	S 355	14,482	K262	K224	Dakligger (90)
Y106	CS19 - CHS (800; 16)	S 355	7,386	K224	K146	Dakligger (90)
Y107	CS19 - CHS (800; 16)	S 355	0,934	K41	K350	Dakligger (90)
Y108	CS19 - CHS (800; 16)	S 355	14,553	K350	K441	Dakligger (90)
Y109	CS19 - CHS (800; 16)	S 355	13,667	K441	K228	Dakligger (90)
Y110	CS19 - CHS (800; 16)	S 355	12,928	K228	K242	Dakligger (90)
Y111	CS19 - CHS (800; 16)	S 355	12,327	K242	K269	Dakligger (90)
Y112	CS19 - CHS (800; 16)	S 355	11,839	K269	K286	Dakligger (90)
Y113	CS19 - CHS (800; 16)	S 355	11,480	K286	K393	Dakligger (90)
Y114	CS19 - CHS (800; 16)	S 355	11,242	K393	K427	Dakligger (90)
Y115	CS19 - CHS (800; 16)	S 355	11,125	K427	K337	Dakligger (90)
Y116	CS19 - CHS (800; 16)	S 355	11,125	K337	K303	Dakligger (90)
Y117	CS19 - CHS (800; 16)	S 355	11,242	K303	K363	Dakligger (90)
Y118	CS19 - CHS (800; 16)	S 355	11,480	K363	K410	Dakligger (90)
Y119	CS19 - CHS (800; 16)	S 355	11,839	K410	K320	Dakligger (90)
Y120	CS19 - CHS (800; 16)	S 355	12,327	K320	K454	Dakligger (90)
Y121	CS19 - CHS (800; 16)	S 355	12,928	K454	K378	Dakligger (90)
Y122	CS19 - CHS (800; 16)	S 355	13,667	K378	K255	Dakligger (90)
Y123	CS19 - CHS (800; 16)	S 355	14,553	K255	K217	Dakligger (90)
Y124	CS19 - CHS (800; 16)	S 355	0,934	K217	K167	Dakligger (90)
Y125	CS19 - CHS (800; 16)	S 355	11,869	K47	K352	Dakligger (90)
Y126	CS19 - CHS (800; 16)	S 355	14,437	K352	K443	Dakligger (90)
Y127	CS19 - CHS (800; 16)	S 355	13,580	K443	K230	Dakligger (90)
Y128	CS19 - CHS (800; 16)	S 355	12,862	K230	K244	Dakligger (90)
Y129	CS19 - CHS (800; 16)	S 355	12,284	K244	K271	Dakligger (90)
Y130	CS19 - CHS (800; 16)	S 355	11,808	K271	K288	Dakligger (90)
Y131	CS19 - CHS (800; 16)	S 355	11,465	K288	K395	Dakligger (90)
Y132	CS19 - CHS (800; 16)	S 355	11,238	K395	K429	Dakligger (90)
Y133	CS19 - CHS (800; 16)	S 355	11,124	K429	K339	Dakligger (90)
Y134	CS19 - CHS (800; 16)	S 355	11,124	K339	K305	Dakligger (90)
Y135	CS19 - CHS (800; 16)	S 355	11,238	K305	K365	Dakligger (90)
Y136	CS19 - CHS (800; 16)	S 355	11,465	K365	K412	Dakligger (90)
Y137	CS19 - CHS (800; 16)	S 355	11,808	K412	K322	Dakligger (90)
Y138	CS19 - CHS (800; 16)	S 355	12,284	K322	K456	Dakligger (90)
Y139	CS19 - CHS (800; 16)	S 355	12,862	K456	K380	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Y140	CS19 - CHS (800; 16)	S 355	13,580	K380	K257	Dakligger (90)
Y141	CS19 - CHS (800; 16)	S 355	14,437	K257	K219	Dakligger (90)
Y142	CS19 - CHS (800; 16)	S 355	11,869	K219	K161	Dakligger (90)
Y143	CS19 - CHS (800; 16)	S 355	7,386	K44	K351	Dakligger (90)
Y144	CS19 - CHS (800; 16)	S 355	14,482	K351	K442	Dakligger (90)
Y145	CS19 - CHS (800; 16)	S 355	13,615	K442	K229	Dakligger (90)
Y146	CS19 - CHS (800; 16)	S 355	12,892	K229	K243	Dakligger (90)
Y147	CS19 - CHS (800; 16)	S 355	12,301	K243	K270	Dakligger (90)
Y148	CS19 - CHS (800; 16)	S 355	11,818	K270	K287	Dakligger (90)
Y149	CS19 - CHS (800; 16)	S 355	11,472	K287	K394	Dakligger (90)
Y150	CS19 - CHS (800; 16)	S 355	11,239	K394	K428	Dakligger (90)
Y151	CS19 - CHS (800; 16)	S 355	11,125	K428	K338	Dakligger (90)
Y152	CS19 - CHS (800; 16)	S 355	11,125	K338	K304	Dakligger (90)
Y153	CS19 - CHS (800; 16)	S 355	11,239	K304	K364	Dakligger (90)
Y154	CS19 - CHS (800; 16)	S 355	11,472	K364	K411	Dakligger (90)
Y155	CS19 - CHS (800; 16)	S 355	11,818	K411	K321	Dakligger (90)
Y156	CS19 - CHS (800; 16)	S 355	12,301	K321	K455	Dakligger (90)
Y157	CS19 - CHS (800; 16)	S 355	12,892	K455	K379	Dakligger (90)
Y158	CS19 - CHS (800; 16)	S 355	13,615	K379	K256	Dakligger (90)
Y159	CS19 - CHS (800; 16)	S 355	14,482	K256	K218	Dakligger (90)
Y160	CS19 - CHS (800; 16)	S 355	7,386	K218	K164	Dakligger (90)
Y161	CS19 - CHS (800; 16)	S 355	0,934	K65	K358	Dakligger (90)
Y162	CS19 - CHS (800; 16)	S 355	14,553	K358	K449	Dakligger (90)
Y163	CS19 - CHS (800; 16)	S 355	13,667	K449	K236	Dakligger (90)
Y164	CS19 - CHS (800; 16)	S 355	12,928	K236	K250	Dakligger (90)
Y165	CS19 - CHS (800; 16)	S 355	12,327	K250	K277	Dakligger (90)
Y166	CS19 - CHS (800; 16)	S 355	11,839	K277	K294	Dakligger (90)
Y167	CS19 - CHS (800; 16)	S 355	11,480	K294	K401	Dakligger (90)
Y168	CS19 - CHS (800; 16)	S 355	11,242	K401	K435	Dakligger (90)
Y169	CS19 - CHS (800; 16)	S 355	11,125	K435	K345	Dakligger (90)
Y170	CS19 - CHS (800; 16)	S 355	11,125	K345	K311	Dakligger (90)
Y171	CS19 - CHS (800; 16)	S 355	11,242	K311	K371	Dakligger (90)
Y172	CS19 - CHS (800; 16)	S 355	11,480	K371	K418	Dakligger (90)
Y173	CS19 - CHS (800; 16)	S 355	11,839	K418	K328	Dakligger (90)
Y174	CS19 - CHS (800; 16)	S 355	12,327	K328	K462	Dakligger (90)
Y175	CS19 - CHS (800; 16)	S 355	12,928	K462	K386	Dakligger (90)
Y176	CS19 - CHS (800; 16)	S 355	13,667	K386	K263	Dakligger (90)
Y177	CS19 - CHS (800; 16)	S 355	14,553	K263	K225	Dakligger (90)
Y178	CS19 - CHS (800; 16)	S 355	0,934	K225	K143	Dakligger (90)
Y179	CS19 - CHS (800; 16)	S 355	1,508	K89	K281	Dakligger (90)
Y180	CS19 - CHS (800; 16)	S 355	11,957	K281	K298	Dakligger (90)
Y181	CS19 - CHS (800; 16)	S 355	11,540	K298	K405	Dakligger (90)
Y182	CS19 - CHS (800; 16)	S 355	11,265	K405	K439	Dakligger (90)
Y183	CS19 - CHS (800; 16)	S 355	11,127	K439	K349	Dakligger (90)
Y184	CS19 - CHS (800; 16)	S 355	11,127	K349	K315	Dakligger (90)
Y185	CS19 - CHS (800; 16)	S 355	11,265	K315	K375	Dakligger (90)
Y186	CS19 - CHS (800; 16)	S 355	11,540	K375	K422	Dakligger (90)
Y187	CS19 - CHS (800; 16)	S 355	11,957	K422	K332	Dakligger (90)
Y188	CS19 - CHS (800; 16)	S 355	1,508	K332	K119	Dakligger (90)
Y189	CS19 - CHS (800; 16)	S 355	9,700	K77	K238	Dakligger (90)
Y190	CS19 - CHS (800; 16)	S 355	13,042	K238	K252	Dakligger (90)
Y191	CS19 - CHS (800; 16)	S 355	12,401	K252	K279	Dakligger (90)
Y192	CS19 - CHS (800; 16)	S 355	11,884	K279	K296	Dakligger (90)
Y193	CS19 - CHS (800; 16)	S 355	11,505	K296	K403	Dakligger (90)
Y194	CS19 - CHS (800; 16)	S 355	11,252	K403	K437	Dakligger (90)
Y195	CS19 - CHS (800; 16)	S 355	11,126	K437	K347	Dakligger (90)
Y196	CS19 - CHS (800; 16)	S 355	11,126	K347	K313	Dakligger (90)
Y197	CS19 - CHS (800; 16)	S 355	11,252	K313	K373	Dakligger (90)
Y198	CS19 - CHS (800; 16)	S 355	11,505	K373	K420	Dakligger (90)
Y199	CS19 - CHS (800; 16)	S 355	11,884	K420	K330	Dakligger (90)
Y200	CS19 - CHS (800; 16)	S 355	12,401	K330	K464	Dakligger (90)
Y201	CS19 - CHS (800; 16)	S 355	13,042	K464	K388	Dakligger (90)
Y202	CS19 - CHS (800; 16)	S 355	9,700	K388	K131	Dakligger (90)
Y203	CS19 - CHS (800; 16)	S 355	11,869	K59	K356	Dakligger (90)
Y204	CS19 - CHS (800; 16)	S 355	14,437	K356	K447	Dakligger (90)
Y205	CS19 - CHS (800; 16)	S 355	13,580	K447	K234	Dakligger (90)
Y206	CS19 - CHS (800; 16)	S 355	12,862	K234	K248	Dakligger (90)
Y207	CS19 - CHS (800; 16)	S 355	12,284	K248	K275	Dakligger (90)
Y208	CS19 - CHS (800; 16)	S 355	11,808	K275	K292	Dakligger (90)
Y209	CS19 - CHS (800; 16)	S 355	11,465	K292	K399	Dakligger (90)
Y210	CS19 - CHS (800; 16)	S 355	11,238	K399	K433	Dakligger (90)
Y211	CS19 - CHS (800; 16)	S 355	11,124	K433	K343	Dakligger (90)
Y212	CS19 - CHS (800; 16)	S 355	11,124	K343	K309	Dakligger (90)
Y213	CS19 - CHS (800; 16)	S 355	11,238	K309	K369	Dakligger (90)
Y214	CS19 - CHS (800; 16)	S 355	11,465	K369	K416	Dakligger (90)
Y215	CS19 - CHS (800; 16)	S 355	11,808	K416	K326	Dakligger (90)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Y216	CS19 - CHS (800; 16)	S 355	12,284	K326	K460	Dakligger (90)
Y217	CS19 - CHS (800; 16)	S 355	12,862	K460	K384	Dakligger (90)
Y218	CS19 - CHS (800; 16)	S 355	13,580	K384	K261	Dakligger (90)
Y219	CS19 - CHS (800; 16)	S 355	14,437	K261	K223	Dakligger (90)
Y220	CS19 - CHS (800; 16)	S 355	11,869	K223	K149	Dakligger (90)
Y221	CS19 - CHS (800; 16)	S 355	14,513	K50	K353	Dakligger (90)
Y222	CS19 - CHS (800; 16)	S 355	14,412	K353	K444	Dakligger (90)
Y223	CS19 - CHS (800; 16)	S 355	13,557	K444	K231	Dakligger (90)
Y224	CS19 - CHS (800; 16)	S 355	12,847	K231	K245	Dakligger (90)
Y225	CS19 - CHS (800; 16)	S 355	12,272	K245	K272	Dakligger (90)
Y226	CS19 - CHS (800; 16)	S 355	11,801	K272	K289	Dakligger (90)
Y227	CS19 - CHS (800; 16)	S 355	11,462	K289	K396	Dakligger (90)
Y228	CS19 - CHS (800; 16)	S 355	11,236	K396	K430	Dakligger (90)
Y229	CS19 - CHS (800; 16)	S 355	11,124	K430	K340	Dakligger (90)
Y230	CS19 - CHS (800; 16)	S 355	11,124	K340	K306	Dakligger (90)
Y231	CS19 - CHS (800; 16)	S 355	11,236	K306	K366	Dakligger (90)
Y232	CS19 - CHS (800; 16)	S 355	11,462	K366	K413	Dakligger (90)
Y233	CS19 - CHS (800; 16)	S 355	11,801	K413	K323	Dakligger (90)
Y234	CS19 - CHS (800; 16)	S 355	12,272	K323	K457	Dakligger (90)
Y235	CS19 - CHS (800; 16)	S 355	12,847	K457	K381	Dakligger (90)
Y236	CS19 - CHS (800; 16)	S 355	13,557	K381	K258	Dakligger (90)
Y237	CS19 - CHS (800; 16)	S 355	14,412	K258	K220	Dakligger (90)
Y238	CS19 - CHS (800; 16)	S 355	14,513	K220	K158	Dakligger (90)
Y239	CS19 - CHS (800; 16)	S 355	6,955	K71	K450	Dakligger (90)
Y240	CS19 - CHS (800; 16)	S 355	13,737	K450	K237	Dakligger (90)
Y241	CS19 - CHS (800; 16)	S 355	12,979	K237	K251	Dakligger (90)
Y242	CS19 - CHS (800; 16)	S 355	12,357	K251	K278	Dakligger (90)
Y243	CS19 - CHS (800; 16)	S 355	11,860	K278	K295	Dakligger (90)
Y244	CS19 - CHS (800; 16)	S 355	11,490	K295	K402	Dakligger (90)
Y245	CS19 - CHS (800; 16)	S 355	11,247	K402	K436	Dakligger (90)
Y246	CS19 - CHS (800; 16)	S 355	11,125	K436	K346	Dakligger (90)
Y247	CS19 - CHS (800; 16)	S 355	11,125	K346	K312	Dakligger (90)
Y248	CS19 - CHS (800; 16)	S 355	11,247	K312	K372	Dakligger (90)
Y249	CS19 - CHS (800; 16)	S 355	11,490	K372	K419	Dakligger (90)
Y250	CS19 - CHS (800; 16)	S 355	11,860	K419	K329	Dakligger (90)
Y251	CS19 - CHS (800; 16)	S 355	12,357	K329	K463	Dakligger (90)
Y252	CS19 - CHS (800; 16)	S 355	12,979	K463	K387	Dakligger (90)
Y253	CS19 - CHS (800; 16)	S 355	13,737	K387	K264	Dakligger (90)
Y254	CS19 - CHS (800; 16)	S 355	6,955	K264	K137	Dakligger (90)
Y255	CS19 - CHS (800; 16)	S 355	8,525	K83	K253	Dakligger (90)
Y256	CS19 - CHS (800; 16)	S 355	12,455	K253	K280	Dakligger (90)
Y257	CS19 - CHS (800; 16)	S 355	11,917	K280	K297	Dakligger (90)
Y258	CS19 - CHS (800; 16)	S 355	11,521	K297	K404	Dakligger (90)
Y259	CS19 - CHS (800; 16)	S 355	11,258	K404	K438	Dakligger (90)
Y260	CS19 - CHS (800; 16)	S 355	11,126	K438	K348	Dakligger (90)
Y261	CS19 - CHS (800; 16)	S 355	11,126	K348	K314	Dakligger (90)
Y262	CS19 - CHS (800; 16)	S 355	11,258	K314	K374	Dakligger (90)
Y263	CS19 - CHS (800; 16)	S 355	11,521	K374	K421	Dakligger (90)
Y264	CS19 - CHS (800; 16)	S 355	11,917	K421	K331	Dakligger (90)
Y265	CS19 - CHS (800; 16)	S 355	12,455	K331	K465	Dakligger (90)
Y266	CS19 - CHS (800; 16)	S 355	8,525	K465	K125	Dakligger (90)
Chordl1	CS20 - CHS (910; 20)	S 355	11,127	K3	K6	Vakwerkligger (95)
Chordl2	CS20 - CHS (910; 20)	S 355	11,268	K6	K9	Vakwerkligger (95)
Chordl3	CS20 - CHS (910; 20)	S 355	11,567	K9	K12	Vakwerkligger (95)
Chordl4	CS20 - CHS (910; 20)	S 355	12,063	K12	K15	Vakwerkligger (95)
Chordl5	CS20 - CHS (910; 20)	S 355	1,534	K15	K18	Vakwerkligger (95)
Chordl6	CS20 - CHS (910; 20)	S 355	11,314	K18	K21	Vakwerkligger (95)
Chordl7	CS20 - CHS (910; 20)	S 355	9,054	K21	K24	Vakwerkligger (95)
Chordl8	CS20 - CHS (910; 20)	S 355	5,009	K24	K27	Vakwerkligger (95)
Chordl9	CS20 - CHS (910; 20)	S 355	11,130	K27	K30	Vakwerkligger (95)
Chordl10	CS20 - CHS (910; 20)	S 355	5,009	K30	K33	Vakwerkligger (95)
Chordl11	CS20 - CHS (910; 20)	S 355	9,054	K33	K36	Vakwerkligger (95)
Chordl12	CS20 - CHS (910; 20)	S 355	11,314	K36	K39	Vakwerkligger (95)
Chordl13	CS20 - CHS (910; 20)	S 355	1,534	K39	K42	Vakwerkligger (95)
Chordl14	CS20 - CHS (910; 20)	S 355	12,063	K42	K45	Vakwerkligger (95)
Chordl15	CS20 - CHS (910; 20)	S 355	11,567	K45	K48	Vakwerkligger (95)
Chordl16	CS20 - CHS (910; 20)	S 355	11,268	K48	K51	Vakwerkligger (95)
Chordl17	CS20 - CHS (910; 20)	S 355	11,127	K51	K54	Vakwerkligger (95)
Chordl18	CS20 - CHS (910; 20)	S 355	11,127	K54	K57	Vakwerkligger (95)
Chordl19	CS20 - CHS (910; 20)	S 355	11,268	K57	K60	Vakwerkligger (95)
Chordl20	CS20 - CHS (910; 20)	S 355	11,567	K60	K63	Vakwerkligger (95)
Chordl21	CS20 - CHS (910; 20)	S 355	12,063	K63	K66	Vakwerkligger (95)
Chordl22	CS20 - CHS (910; 20)	S 355	1,534	K66	K69	Vakwerkligger (95)
Chordl23	CS20 - CHS (910; 20)	S 355	11,314	K69	K72	Vakwerkligger (95)
Chordl24	CS20 - CHS (910; 20)	S 355	9,054	K72	K75	Vakwerkligger (95)
Chordl25	CS20 - CHS (910; 20)	S 355	5,009	K75	K78	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Chordl26	CS20 - CHS (910; 20)	S 355	11,130	K78	K81	Vakwerkligger (95)
Chordl27	CS20 - CHS (910; 20)	S 355	5,009	K81	K84	Vakwerkligger (95)
Chordl28	CS20 - CHS (910; 20)	S 355	9,054	K84	K87	Vakwerkligger (95)
Chordl29	CS20 - CHS (910; 20)	S 355	11,314	K87	K90	Vakwerkligger (95)
Chordl30	CS20 - CHS (910; 20)	S 355	1,534	K90	K93	Vakwerkligger (95)
Chordl31	CS20 - CHS (910; 20)	S 355	12,063	K93	K96	Vakwerkligger (95)
Chordl32	CS20 - CHS (910; 20)	S 355	11,567	K96	K99	Vakwerkligger (95)
Chordl33	CS20 - CHS (910; 20)	S 355	11,268	K99	K102	Vakwerkligger (95)
Chordl34	CS20 - CHS (910; 20)	S 355	11,127	K102	K105	Vakwerkligger (95)
Chordl35	CS20 - CHS (910; 20)	S 355	11,127	K105	K108	Vakwerkligger (95)
Chordl36	CS20 - CHS (910; 20)	S 355	11,268	K108	K111	Vakwerkligger (95)
Chordl37	CS20 - CHS (910; 20)	S 355	11,567	K111	K114	Vakwerkligger (95)
Chordl38	CS20 - CHS (910; 20)	S 355	12,063	K114	K117	Vakwerkligger (95)
Chordl39	CS20 - CHS (910; 20)	S 355	1,534	K117	K120	Vakwerkligger (95)
Chordl40	CS20 - CHS (910; 20)	S 355	11,314	K120	K123	Vakwerkligger (95)
Chordl41	CS20 - CHS (910; 20)	S 355	9,054	K123	K126	Vakwerkligger (95)
Chordl42	CS20 - CHS (910; 20)	S 355	5,009	K126	K129	Vakwerkligger (95)
Chordl43	CS20 - CHS (910; 20)	S 355	11,130	K129	K132	Vakwerkligger (95)
Chordl44	CS20 - CHS (910; 20)	S 355	5,009	K132	K135	Vakwerkligger (95)
Chordl45	CS20 - CHS (910; 20)	S 355	9,054	K135	K138	Vakwerkligger (95)
Chordl46	CS20 - CHS (910; 20)	S 355	11,314	K138	K141	Vakwerkligger (95)
Chordl47	CS20 - CHS (910; 20)	S 355	1,534	K141	K144	Vakwerkligger (95)
Chordl48	CS20 - CHS (910; 20)	S 355	12,063	K144	K147	Vakwerkligger (95)
Chordl49	CS20 - CHS (910; 20)	S 355	11,567	K147	K150	Vakwerkligger (95)
Chordl50	CS20 - CHS (910; 20)	S 355	11,268	K150	K153	Vakwerkligger (95)
Chordl51	CS20 - CHS (910; 20)	S 355	11,127	K153	K156	Vakwerkligger (95)
Chordl52	CS20 - CHS (910; 20)	S 355	11,127	K156	K159	Vakwerkligger (95)
Chordl53	CS20 - CHS (910; 20)	S 355	11,268	K159	K162	Vakwerkligger (95)
Chordl54	CS20 - CHS (910; 20)	S 355	11,567	K162	K165	Vakwerkligger (95)
Chordl55	CS20 - CHS (910; 20)	S 355	12,063	K165	K168	Vakwerkligger (95)
Chordl56	CS20 - CHS (910; 20)	S 355	1,534	K168	K171	Vakwerkligger (95)
Chordl57	CS20 - CHS (910; 20)	S 355	11,314	K171	K174	Vakwerkligger (95)
Chordl58	CS20 - CHS (910; 20)	S 355	9,054	K174	K177	Vakwerkligger (95)
Chordl59	CS20 - CHS (910; 20)	S 355	5,009	K177	K180	Vakwerkligger (95)
Chordl60	CS20 - CHS (910; 20)	S 355	11,130	K180	K183	Vakwerkligger (95)
Chordl61	CS20 - CHS (910; 20)	S 355	5,009	K183	K186	Vakwerkligger (95)
Chordl62	CS20 - CHS (910; 20)	S 355	9,054	K186	K189	Vakwerkligger (95)
Chordl63	CS20 - CHS (910; 20)	S 355	11,314	K189	K192	Vakwerkligger (95)
Chordi64	CS20 - CHS (910; 20)	S 355	1,534	K192	K195	Vakwerkligger (95)
Chordi65	CS20 - CHS (910; 20)	S 355	12,063	K195	K198	Vakwerkligger (95)
Chordi66	CS20 - CHS (910; 20)	S 355	11,567	K198	K201	Vakwerkligger (95)
Chordi67	CS20 - CHS (910; 20)	S 355	11,268	K201	K204	Vakwerkligger (95)
Chordi68	CS20 - CHS (910; 20)	S 355	11,127	K204	K3	Vakwerkligger (95)
Chordo1	CS21 - CHS (800; 19)	S 355	12,129	K1	K4	Vakwerkligger (95)
Chordo2	CS21 - CHS (800; 19)	S 355	12,284	K4	K7	Vakwerkligger (95)
Chordo3	CS21 - CHS (800; 19)	S 355	12,606	K7	K10	Vakwerkligger (95)
Chordo4	CS21 - CHS (800; 19)	S 355	13,152	K10	K13	Vakwerkligger (95)
Chordo5	CS21 - CHS (800; 19)	S 355	1,677	K13	K16	Vakwerkligger (95)
Chordo6	CS21 - CHS (800; 19)	S 355	12,328	K16	K19	Vakwerkligger (95)
Chordo7	CS21 - CHS (800; 19)	S 355	9,870	K19	K22	Vakwerkligger (95)
Chordo8	CS21 - CHS (800; 19)	S 355	5,462	K22	K25	Vakwerkligger (95)
Chordo9	CS21 - CHS (800; 19)	S 355	12,120	K25	K28	Vakwerkligger (95)
Chordo10	CS21 - CHS (800; 19)	S 355	5,462	K28	K31	Vakwerkligger (95)
Chordo11	CS21 - CHS (800; 19)	S 355	9,870	K31	K34	Vakwerkligger (95)
Chordo12	CS21 - CHS (800; 19)	S 355	12,328	K34	K37	Vakwerkligger (95)
Chordo13	CS21 - CHS (800; 19)	S 355	1,677	K37	K40	Vakwerkligger (95)
Chordo14	CS21 - CHS (800; 19)	S 355	13,152	K40	K43	Vakwerkligger (95)
Chordo15	CS21 - CHS (800; 19)	S 355	12,606	K43	K46	Vakwerkligger (95)
Chordo16	CS21 - CHS (800; 19)	S 355	12,284	K46	K49	Vakwerkligger (95)
Chordo17	CS21 - CHS (800; 19)	S 355	12,129	K49	K52	Vakwerkligger (95)
Chordo18	CS21 - CHS (800; 19)	S 355	12,129	K52	K55	Vakwerkligger (95)
Chordo19	CS21 - CHS (800; 19)	S 355	12,284	K55	K58	Vakwerkligger (95)
Chordo20	CS21 - CHS (800; 19)	S 355	12,606	K58	K61	Vakwerkligger (95)
Chordo21	CS21 - CHS (800; 19)	S 355	13,152	K61	K64	Vakwerkligger (95)
Chordo22	CS21 - CHS (800; 19)	S 355	1,677	K64	K67	Vakwerkligger (95)
Chordo23	CS21 - CHS (800; 19)	S 355	12,328	K67	K70	Vakwerkligger (95)
Chordo24	CS21 - CHS (800; 19)	S 355	9,870	K70	K73	Vakwerkligger (95)
Chordo25	CS21 - CHS (800; 19)	S 355	5,462	K73	K76	Vakwerkligger (95)
Chordo26	CS21 - CHS (800; 19)	S 355	12,120	K76	K79	Vakwerkligger (95)
Chordo27	CS21 - CHS (800; 19)	S 355	5,462	K79	K82	Vakwerkligger (95)
Chordo28	CS21 - CHS (800; 19)	S 355	9,870	K82	K85	Vakwerkligger (95)
Chordo29	CS21 - CHS (800; 19)	S 355	12,328	K85	K88	Vakwerkligger (95)
Chordo30	CS21 - CHS (800; 19)	S 355	1,677	K88	K91	Vakwerkligger (95)
Chordo31	CS21 - CHS (800; 19)	S 355	13,152	K91	K94	Vakwerkligger (95)
Chordo32	CS21 - CHS (800; 19)	S 355	12,606	K94	K97	Vakwerkligger (95)
Chordo33	CS21 - CHS (800; 19)	S 355	12,284	K97	K100	Vakwerkligger (95)

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Chordo34	CS21 - CHS (800; 19)	S 355	12,129	K100	K103	Vakwerkligger (95)
Chordo35	CS21 - CHS (800; 19)	S 355	12,129	K103	K106	Vakwerkligger (95)
Chordo36	CS21 - CHS (800; 19)	S 355	12,284	K106	K109	Vakwerkligger (95)
Chordo37	CS21 - CHS (800; 19)	S 355	12,606	K109	K112	Vakwerkligger (95)
Chordo38	CS21 - CHS (800; 19)	S 355	13,152	K112	K115	Vakwerkligger (95)
Chordo39	CS21 - CHS (800; 19)	S 355	1,677	K115	K118	Vakwerkligger (95)
Chordo40	CS21 - CHS (800; 19)	S 355	12,328	K118	K121	Vakwerkligger (95)
Chordo41	CS21 - CHS (800; 19)	S 355	9,870	K121	K124	Vakwerkligger (95)
Chordo42	CS21 - CHS (800; 19)	S 355	5,462	K124	K127	Vakwerkligger (95)
Chordo43	CS21 - CHS (800; 19)	S 355	12,120	K127	K130	Vakwerkligger (95)
Chordo44	CS21 - CHS (800; 19)	S 355	5,462	K130	K133	Vakwerkligger (95)
Chordo45	CS21 - CHS (800; 19)	S 355	9,870	K133	K136	Vakwerkligger (95)
Chordo46	CS21 - CHS (800; 19)	S 355	12,328	K136	K139	Vakwerkligger (95)
Chordo47	CS21 - CHS (800; 19)	S 355	1,677	K139	K142	Vakwerkligger (95)
Chordo48	CS21 - CHS (800; 19)	S 355	13,152	K142	K145	Vakwerkligger (95)
Chordo49	CS21 - CHS (800; 19)	S 355	12,606	K145	K148	Vakwerkligger (95)
Chordo50	CS21 - CHS (800; 19)	S 355	12,284	K148	K151	Vakwerkligger (95)
Chordo51	CS21 - CHS (800; 19)	S 355	12,129	K151	K154	Vakwerkligger (95)
Chordo52	CS21 - CHS (800; 19)	S 355	12,129	K154	K157	Vakwerkligger (95)
Chordo53	CS21 - CHS (800; 19)	S 355	12,284	K157	K160	Vakwerkligger (95)
Chordo54	CS21 - CHS (800; 19)	S 355	12,606	K160	K163	Vakwerkligger (95)
Chordo55	CS21 - CHS (800; 19)	S 355	13,152	K163	K166	Vakwerkligger (95)
Chordo56	CS21 - CHS (800; 19)	S 355	1,677	K166	K169	Vakwerkligger (95)
Chordo57	CS21 - CHS (800; 19)	S 355	12,328	K169	K172	Vakwerkligger (95)
Chordo58	CS21 - CHS (800; 19)	S 355	9,870	K172	K175	Vakwerkligger (95)
Chordo59	CS21 - CHS (800; 19)	S 355	5,462	K175	K178	Vakwerkligger (95)
Chordo60	CS21 - CHS (800; 19)	S 355	12,120	K178	K181	Vakwerkligger (95)
Chordo61	CS21 - CHS (800; 19)	S 355	5,462	K181	K184	Vakwerkligger (95)
Chordo62	CS21 - CHS (800; 19)	S 355	9,870	K184	K187	Vakwerkligger (95)
Chordo63	CS21 - CHS (800; 19)	S 355	12,328	K187	K190	Vakwerkligger (95)
Chordo64	CS21 - CHS (800; 19)	S 355	1,677	K190	K193	Vakwerkligger (95)
Chordo65	CS21 - CHS (800; 19)	S 355	13,152	K193	K196	Vakwerkligger (95)
Chordo66	CS21 - CHS (800; 19)	S 355	12,606	K196	K199	Vakwerkligger (95)
Chordo67	CS21 - CHS (800; 19)	S 355	12,284	K199	K202	Vakwerkligger (95)
Chordo68	CS21 - CHS (800; 19)	S 355	12,129	K202	K1	Vakwerkligger (95)

3. Scharnieren

Naam	Staaf	Positie	ux	uy	uz	phix	phiy	phiz
H1	X1	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H2	X11	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H3	X25	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H4	X41	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H5	X53	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H6	X71	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H7	X89	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H8	X107	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H9	X125	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H10	X143	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H11	X153	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H12	X171	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H13	X185	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H14	X203	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H15	X221	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H16	X239	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H17	X251	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H18	Y1	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H19	Y11	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H20	Y29	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H21	Y41	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H22	Y59	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H23	Y73	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H24	Y89	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H25	Y107	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H26	Y125	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H27	Y143	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H28	Y161	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H29	Y179	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H30	Y189	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H31	Y203	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H32	Y221	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H33	Y239	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H34	Y255	Begin	Vast	Vast	Vast	Vrij	Vrij	Vrij
H35	X10	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H36	X24	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H37	X40	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij

Naam	Staaf	Positie	ux	uy	uz	phix	phiy	phiz
H38	X52	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H39	X70	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H40	X88	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H41	X106	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H42	X124	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H43	X142	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H44	X152	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H45	X170	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H46	X184	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H47	X202	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H48	X220	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H49	X238	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H50	X250	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H51	X266	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H52	Y10	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H53	Y28	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H54	Y40	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H55	Y58	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H56	Y72	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H57	Y88	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H58	Y106	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H59	Y124	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H60	Y142	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H61	Y160	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H62	Y178	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H63	Y188	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H64	Y202	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H65	Y220	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H66	Y238	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H67	Y254	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H68	Y266	Eind	Vast	Vast	Vast	Vrij	Vrij	Vrij
H69	Chordi1	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H70	Chordi2	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H71	Chordi3	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H72	Chordi4	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H73	Chordi5	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H74	Chordi6	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H75	Chordi7	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H76	Chordi8	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H77	Chordi9	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H78	Chordi10	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H79	Chordi11	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H80	Chordi12	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H81	Chordi13	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H82	Chordi14	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H83	Chordi15	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H84	Chordi16	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H85	Chordi17	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H86	Chordi18	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H87	Chordi19	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H88	Chordi20	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H89	Chordi21	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H90	Chordi22	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H91	Chordi23	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H92	Chordi24	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H93	Chordi25	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H94	Chordi26	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H95	Chordi27	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H96	Chordi28	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H97	Chordi29	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H98	Chordi30	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H99	Chordi31	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H100	Chordi32	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H101	Chordi33	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H102	Chordi34	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H103	Chordi35	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H104	Chordi36	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H105	Chordi37	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H106	Chordi38	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H107	Chordi39	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H108	Chordi40	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H109	Chordi41	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H110	Chordi42	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H111	Chordi43	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H112	Chordi44	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H113	Chordi45	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H114	Chordi46	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij

Naam	Staaf	Positie	ux	uy	uz	phix	phiy	phiz
H115	Chordi47	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H116	Chordi48	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H117	Chordi49	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H118	Chordi50	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H119	Chordi51	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H120	Chordi52	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H121	Chordi53	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H122	Chordi54	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H123	Chordi55	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H124	Chordi56	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H125	Chordi57	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H126	Chordi58	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H127	Chordi59	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H128	Chordi60	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H129	Chordi61	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H130	Chordi62	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H131	Chordi63	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H132	Chordi64	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H133	Chordi65	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H134	Chordi66	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H135	Chordi67	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H136	Chordi68	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H137	Vert1	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H138	Vert2	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H139	Vert3	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H140	Vert4	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H141	Vert5	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H142	Vert6	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H143	Vert7	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H144	Vert8	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H145	Vert9	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H146	Vert10	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H147	Vert11	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H148	Vert12	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H149	Vert13	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H150	Vert14	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H151	Vert15	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H152	Vert16	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H153	Vert17	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H154	Vert18	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H155	Vert19	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H156	Vert20	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H157	Vert21	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H158	Vert22	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H159	Vert23	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H160	Vert24	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H161	Vert25	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H162	Vert26	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H163	Vert27	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H164	Vert28	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H165	Vert29	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H166	Vert30	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H167	Vert31	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H168	Vert32	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H169	Vert33	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H170	Vert34	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H171	Vert35	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H172	Vert36	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H173	Vert37	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H174	Vert38	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H175	Vert39	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H176	Vert40	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H177	Vert41	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H178	Vert42	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H179	Vert43	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H180	Vert44	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H181	Vert45	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H182	Vert46	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H183	Vert47	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H184	Vert48	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H185	Vert49	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H186	Vert50	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H187	Vert51	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H188	Vert52	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H189	Vert53	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H190	Vert54	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H191	Vert55	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij

Naam	Staaf	Positie	ux	uy	uz	phix	phiy	phiz
H192	Vert56	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H193	Vert57	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H194	Vert58	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H195	Vert59	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H196	Vert60	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H197	Vert61	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H198	Vert62	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H199	Vert63	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H200	Vert64	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H201	Vert65	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H202	Vert66	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H203	Vert67	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H204	Vert68	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H205	Vert69	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H206	Vert70	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H207	Vert71	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H208	Vert72	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H209	Vert73	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H210	Vert74	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H211	Vert75	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H212	Vert76	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H213	Vert77	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H214	Vert78	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H215	Vert79	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H216	Vert80	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H217	Vert81	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H218	Vert82	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H219	Vert83	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H220	Vert84	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H221	Vert85	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H222	Vert86	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H223	Vert87	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H224	Vert88	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H225	Vert89	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H226	Vert90	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H227	Vert91	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H228	Vert92	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H229	Vert93	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H230	Vert94	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H231	Vert95	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H232	Vert96	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H233	Vert97	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H234	Vert98	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H235	Vert99	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H236	Vert100	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H237	Vert101	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H238	Vert102	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H239	Vert103	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H240	Vert104	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H241	Vert105	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H242	Vert106	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H243	Vert107	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H244	Vert108	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H245	Vert109	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H246	Vert110	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H247	Vert111	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H248	Vert112	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H249	Vert113	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H250	Vert114	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H251	Vert115	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H252	Vert116	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H253	Vert117	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H254	Vert118	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H255	Vert119	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H256	Vert120	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H257	Vert121	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H258	Vert122	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H259	Vert123	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H260	Vert124	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H261	Vert125	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H262	Vert126	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H263	Vert127	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H264	Vert128	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H265	Vert129	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H266	Vert130	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H267	Vert131	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H268	Vert132	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij

Naam	Staaf	Positie	ux	uy	uz	phix	phiy	phiz
H269	Vert133	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H270	Vert134	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H271	Vert135	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H272	Vert136	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H273	Vert137	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H274	Vert138	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H275	Vert139	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H276	Vert140	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H277	Vert141	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H278	Vert142	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H279	Vert143	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H280	Vert144	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H281	Vert145	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H282	Vert146	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H283	Vert147	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H284	Vert148	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H285	Vert149	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H286	Vert150	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H287	Vert151	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H288	Vert152	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H289	Vert153	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H290	Vert154	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H291	Vert155	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H292	Vert156	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H293	Vert157	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H294	Vert158	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H295	Vert159	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H296	Vert160	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H297	Vert161	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H298	Vert162	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H299	Vert163	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H300	Vert164	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H301	Vert165	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H302	Vert166	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H303	Vert167	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H304	Vert168	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H305	Vert169	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H306	Vert170	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H307	Vert171	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H308	Vert172	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H309	Vert173	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H310	Vert174	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H311	Vert175	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H312	Vert176	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H313	Vert177	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H314	Vert178	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H315	Vert179	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H316	Vert180	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H317	Vert181	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H318	Vert182	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H319	Vert183	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H320	Vert184	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H321	Diag1	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H322	Diag2	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H323	Diag3	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H324	Diag4	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H325	Diag5	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H326	Diag6	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H327	Diag7	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H328	Diag8	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H329	Diag9	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H330	Diag10	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H331	Diag11	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H332	Diag12	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H333	Diag13	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H334	Diag14	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H335	Diag15	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H336	Diag16	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H337	Diag17	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H338	Diag18	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H339	Diag19	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H340	Diag20	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H341	Diag21	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H342	Diag22	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H343	Diag23	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H344	Diag24	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H345	Diag25	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij

[illegible]

[illegible]

Naam	Staaf	Positie	ux	uy	uz	phix	phiy	phiz
H500	Diag180	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H501	Diag181	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H502	Diag182	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H503	Diag183	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H504	Diag184	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H505	Diag185	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H506	Diag186	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H507	Diag187	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H508	Diag188	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H509	Diag189	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H510	Diag190	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H511	Diag191	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H512	Diag192	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H513	Diag193	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H514	Diag194	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H515	Diag195	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H516	Diag196	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H517	Diag197	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H518	Diag198	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H519	Diag199	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H520	Diag200	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H521	Diag201	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H522	Diag202	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H523	Diag203	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij
H524	Diag204	Beide	Vast	Vast	Vast	Vast	Vrij	Vrij

4. Knoopondersteuningen

Naam	Knoop	Systeem	Type	X	Y	Z	Rx	Ry	Rz
Sn1	K209	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn2	K210	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn3	K211	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn4	K208	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn5	K205	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn6	K206	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn7	K207	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn8	K216	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn9	K214	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn10	K215	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn11	K213	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn12	K212	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij

5. Doorsneden

Naam	Type	Onderdeelmateriaal	Bouwwijze	A [m ²]	A _y [m ²]	I _y [m ⁴]	W _{el,y} [m ³]	W _{pl,y} [m ³]	Kleur
	Uitgebreid				A _z [m ²]	I _z [m ⁴]	W _{el,z} [m ³]	W _{pl,z} [m ³]	
CS13	CHS	S 355	gewalst	1,6784e-01	1,0685e-01	1,9160e-02	3,7940e-02	5,1025e-02	
	1010; 56				1,0685e-01	1,9160e-02	3,7940e-02	5,1025e-02	
CS14	CHS	S 355	gewalst	5,1393e-02	3,2718e-02	4,7647e-03	1,0829e-02	1,4087e-02	
	880; 19				3,2718e-02	4,7647e-03	1,0829e-02	1,4087e-02	
CS15	CHS	S 355	gewalst	3,1136e-02	1,9822e-02	1,3240e-03	4,4133e-03	5,7798e-03	
	600; 17				1,9822e-02	1,3240e-03	4,4133e-03	5,7798e-03	
CS16	CHS	S 355	gewalst	8,8467e-03	5,6320e-03	1,3709e-04	7,6160e-04	9,9140e-04	
	360; 8				5,6320e-03	1,3709e-04	7,6160e-04	9,9140e-04	
CS17	CHS	S 355	gewalst	2,8981e-02	1,8450e-02	1,3710e-03	4,3524e-03	5,6745e-03	
	630; 15				1,8450e-02	1,3710e-03	4,3524e-03	5,6745e-03	
CS18	CHS	S 355	gewalst	3,9408e-02	2,5088e-02	3,0291e-03	7,5727e-03	9,8359e-03	
	800; 16				2,5088e-02	3,0291e-03	7,5727e-03	9,8359e-03	
CS19	CHS	S 355	gewalst	3,9408e-02	2,5088e-02	3,0291e-03	7,5727e-03	9,8359e-03	
	800; 16				2,5088e-02	3,0291e-03	7,5727e-03	9,8359e-03	
CS20	CHS	S 355	gewalst	5,5920e-02	3,5600e-02	5,5396e-03	1,2175e-02	1,5845e-02	
	910; 20				3,5600e-02	5,5396e-03	1,2175e-02	1,5845e-02	
CS21	CHS	S 355	gewalst	4,6618e-02	2,9678e-02	3,5565e-03	8,8913e-03	1,1592e-02	
	800; 19				2,9678e-02	3,5565e-03	8,8913e-03	1,1592e-02	
CS22	CHS	S 355	gewalst	5,0894e-02	3,2400e-02	4,1765e-03	1,0064e-02	1,3125e-02	
	830; 20				3,2400e-02	4,1765e-03	1,0064e-02	1,3125e-02	

6. Belastingsgevallen

Naam	Omschrijving	Actie type	Lastgroep	Richting	Duur	'Master' belastingsgeval
	Spec	Belastingtype				
BG2	Self-Weight steel	Permanent Eigen gewicht	LG1	-Z		
BG3	Imposed Loads	Variabel	LG2		Kort	Geen

Naam	Omschrijving	Actie type	Lastgroep	Richting	Duur	'Master' belastingsgeval
	Spec	Belastingtype				
BG5	Standaard	Statisch				
BG5	Upward Wind Standaard	Variabel	LG2		Kort	Geen
BG6	Downward Wind Standaard	Variabel	LG2		Kort	Geen
BG7	Snow Load i Standaard	Statisch				
BG7	Snow Load i Standaard	Variabel	LG2		Kort	Geen
BG8	Snow Load ii Standaard	Statisch				
BG8	Snow Load ii Standaard	Variabel	LG2		Kort	Geen
BG10	Superimposed Loads	Permanent	LG1			
		Standaard				

7. Niet-lineaire combinaties

7.1. Niet-lineaire combinaties - NLC1

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC1	Uiterste Grenstoestand	BG2 - Self-Weight steel	1,50
		BG10 - Superimposed Loads	1,50

7.1.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC1

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K101	NLC1	-71,4	4,7	-27,7
K5	NLC1	71,4	4,7	-27,7
K155	NLC1	0,0	-71,7	-19,2
K53	NLC1	0,0	71,7	-19,2
K306	NLC1	-0,8	0,9	-180,6
K209	NLC1	0,0	0,0	0,0

7.1.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC1

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC1	0,000	0,42	0,08	0,42
Cv3	CS14 - CHS	S 355	NLC1	0,000	0,63	0,26	0,63
Cv7	CS22 - CHS	S 355	NLC1	0,000	0,30	0,26	0,30
ChordI9	CS15 - CHS	S 355	NLC1	0,000	0,44	0,44	0,44
Vert48	CS16 - CHS	S 355	NLC1	12,728	0,67	0,25	0,67
Diag12	CS17 - CHS	S 355	NLC1	0,000	0,41	0,26	0,41
X1	CS18 - CHS	S 355	NLC1	1,508	0,28	0,28	0,24
Y1	CS19 - CHS	S 355	NLC1	1,508	0,28	0,28	0,24
ChordI30	CS20 - CHS	S 355	NLC1	0,000	0,46	0,41	0,46
ChordO1	CS21 - CHS	S 355	NLC1	4,851	0,33	0,33	0,30

7.1.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC1

Doorsnede : CS13 - CHS (1010; 56)

Staal	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC1	-4338,66	41,01	221,21	283,98	-820,25	409,31
Cw2	CS13 - CHS	42,932	NLC1	-1925,03	-5,70	-111,05	270,82	667,70	832,06
Cw14	CS13 - CHS	0,000	NLC1	-4170,76	-44,56	212,06	-249,97	-759,57	-342,38
Cw9	CS13 - CHS	0,000	NLC1	-4170,76	44,56	212,06	249,97	-759,57	342,38
Cw9	CS13 - CHS	46,162	NLC1	-3376,81	-1,24	-230,32	231,69	-1357,17	1403,15
Cw6	CS13 - CHS	0,000	NLC1	-2876,92	-17,11	167,66	-296,32	-537,32	-737,88
Cw1	CS13 - CHS	0,000	NLC1	-2876,92	17,11	167,66	296,32	-537,32	737,88
Cw10	CS13 - CHS	21,866	NLC1	-3963,56	21,90	6,05	275,40	1743,51	1089,90
Cw14	CS13 - CHS	46,162	NLC1	-3376,81	1,24	-230,32	-231,69	-1357,17	-1403,15

7.1.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC1

Steunpunt	BG	R _x [kN]	R _y [kN]	R _z [kN]	M _x [kNm]	M _y [kNm]	M _z [kNm]
Sn1/K209	NLC1	-1830,48	-556,50	6737,49	0,00	0,00	0,00
Sn2/K210	NLC1	513,86	-1757,16	6433,75	0,00	0,00	0,00
Sn3/K211	NLC1	1830,48	556,50	6737,49	0,00	0,00	0,00
Sn4/K208	NLC1	0,00	-6,14	4426,75	0,00	0,00	0,00
Sn5/K205	NLC1	5,82	0,00	4698,46	0,00	0,00	0,00
Sn6/K206	NLC1	0,00	6,14	4426,75	0,00	0,00	0,00
Sn7/K207	NLC1	-5,82	0,00	4698,46	0,00	0,00	0,00
Sn8/K216	NLC1	-1830,48	556,50	6737,49	0,00	0,00	0,00
Sn9/K214	NLC1	1830,48	-556,50	6737,49	0,00	0,00	0,00
Sn10/K215	NLC1	513,86	1757,16	6433,75	0,00	0,00	0,00
Sn11/K213	NLC1	-513,86	-1757,16	6433,75	0,00	0,00	0,00
Sn12/K212	NLC1	-513,86	1757,16	6433,75	0,00	0,00	0,00

7.2. Niet-lineaire combinaties - NLC2

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC2	Uiterste Grenstoestand	BG2 - Self-Weight steel	1,30
		BG3 - Imposed Loads	1,65
		BG10 - Superimposed Loads	1,30

7.2.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC2

Knoop	BG	U _x [mm]	U _y [mm]	U _z [mm]
K347	NLC2	-316,1	0,0	184,5
K335	NLC2	316,1	0,0	184,5
K382	NLC2	0,0	-316,5	182,6
K232	NLC2	0,0	316,5	182,6
K341	NLC2	0,0	0,0	-885,9

7.2.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC2

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC2	0,000	0,38	0,07	0,38
Cv3	CS14 - CHS	S 355	NLC2	0,000	0,94	0,38	0,94
Cv14	CS22 - CHS	S 355	NLC2	0,000	0,31	0,29	0,31
Chord11	CS15 - CHS	S 355	NLC2	0,000	0,52	0,52	0,52
Vert48	CS16 - CHS	S 355	NLC2	12,728	0,99	0,37	0,99
Diag3	CS17 - CHS	S 355	NLC2	0,000	0,47	0,32	0,47
X76	CS18 - CHS	S 355	NLC2	0,000	0,72	0,72	0,58
Y94	CS19 - CHS	S 355	NLC2	0,000	0,72	0,72	0,59
Chord17	CS20 - CHS	S 355	NLC2	11,127	0,45	0,34	0,45
Chordo1	CS21 - CHS	S 355	NLC2	4,851	0,47	0,47	0,45

7.2.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC2

Doorsnede : CS13 - CHS (1010; 56)

Staaft	css	dx [m]	BG	N [kN]	V _y [kN]	V _z [kN]	M _x [kNm]	M _y [kNm]	M _z [kNm]
Cw10	CS13 - CHS	0,000	NLC2	-4007,33	40,70	190,53	201,55	-689,29	313,51
Cw2	CS13 - CHS	42,932	NLC2	-572,32	2,51	-88,98	206,37	613,42	816,45
Cw14	CS13 - CHS	0,000	NLC2	-3730,59	-43,64	182,13	-163,38	-641,16	-250,93
Cw9	CS13 - CHS	0,000	NLC2	-3730,59	43,64	182,13	163,38	-641,16	250,93
Cw9	CS13 - CHS	46,162	NLC2	-3042,65	4,26	-198,82	149,05	-1161,61	1443,28
Cw6	CS13 - CHS	0,000	NLC2	-1939,64	-14,03	140,80	-231,62	-407,51	-627,95
Cw1	CS13 - CHS	0,000	NLC2	-1939,64	14,03	140,80	231,62	-407,51	627,95
Cw10	CS13 - CHS	21,866	NLC2	-3682,20	24,72	5,06	194,78	1511,28	1026,07
Cw14	CS13 - CHS	46,162	NLC2	-3042,65	-4,26	-198,82	-149,05	-1161,61	-1443,28

7.2.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC2

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC2	-1694,04	-305,27	5528,90	0,00	0,00	0,00
Sn2/K210	NLC2	109,95	-1565,83	4631,57	0,00	0,00	0,00
Sn3/K211	NLC2	1694,04	305,27	5528,90	0,00	0,00	0,00
Sn4/K208	NLC2	0,00	-11,75	6586,45	0,00	0,00	0,00
Sn5/K205	NLC2	11,59	0,00	6980,90	0,00	0,00	0,00
Sn6/K206	NLC2	0,00	11,75	6586,45	0,00	0,00	0,00
Sn7/K207	NLC2	-11,59	0,00	6980,90	0,00	0,00	0,00
Sn8/K216	NLC2	-1694,04	305,27	5528,90	0,00	0,00	0,00
Sn9/K214	NLC2	1694,04	-305,27	5528,90	0,00	0,00	0,00
Sn10/K215	NLC2	109,95	1565,83	4631,57	0,00	0,00	0,00
Sn11/K213	NLC2	-109,95	-1565,83	4631,57	0,00	0,00	0,00
Sn12/K212	NLC2	-109,95	1565,83	4631,57	0,00	0,00	0,00

7.3. Niet-lineaire combinaties - NLC3

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC3	Uiterste Grenstoestand	BG2 - Self-Weight steel	1,30
		BG6 - Downward Wind	1,65
		BG10 - Superimposed Loads	1,30

7.3.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC3

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K333	NLC3	-199,2	0,0	-302,8
K349	NLC3	199,2	0,0	-302,8
K445	NLC3	0,0	-225,3	-346,6
K259	NLC3	0,0	225,3	-346,6
K463	NLC3	118,5	93,1	-386,8
K209	NLC3	0,0	0,0	0,0

7.3.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC3

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC3	0,000	0,99	0,16	0,99
Cv1	CS14 - CHS	S 355	NLC3	0,000	0,89	0,30	0,89
Cv7	CS22 - CHS	S 355	NLC3	0,000	0,98	0,92	0,98
Chordi13	CS15 - CHS	S 355	NLC3	0,000	0,95	0,95	0,95
Vert29	CS16 - CHS	S 355	NLC3	0,000	0,98	0,69	0,98
Diag12	CS17 - CHS	S 355	NLC3	0,000	0,99	0,64	0,99
X22	CS18 - CHS	S 355	NLC3	12,401	0,78	0,72	0,78
Y1	CS19 - CHS	S 355	NLC3	1,508	0,92	0,92	0,82
Chord15	CS20 - CHS	S 355	NLC3	1,534	1,00	1,00	0,96
Chordo7	CS21 - CHS	S 355	NLC3	4,935	0,92	0,92	0,88

7.3.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC3

Doorsnede : CS13 - CHS (1010; 56)

Staal	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC3	-9272,87	143,43	259,44	773,31	-661,73	284,44
Cw1	CS13 - CHS	42,932	NLC3	-7299,46	-42,37	-153,25	700,31	2657,50	2025,29
Cw14	CS13 - CHS	0,000	NLC3	-9116,76	-149,19	231,62	-714,43	-465,54	-136,85
Cw9	CS13 - CHS	0,000	NLC3	-9116,76	149,19	231,62	714,43	-465,54	136,85
Cw10	CS13 - CHS	46,162	NLC3	-8585,12	-81,28	-249,67	737,08	-509,00	2381,94
Cw1	CS13 - CHS	0,000	NLC3	-7986,51	100,58	268,67	717,41	-642,62	434,94
Cw13	CS13 - CHS	0,000	NLC3	-9272,87	-143,43	259,44	-773,31	-661,73	-284,44
Cw9	CS13 - CHS	46,162	NLC3	-8429,28	-76,49	-242,39	678,70	-1081,52	2622,38
Cw1	CS13 - CHS	28,621	NLC3	-7530,68	14,25	4,80	706,01	3776,43	2248,05

Staafl	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw14	CS13 - CHS	34,014	NLC3	-8611,46	3,82	-144,48	-688,10	1342,97	-3141,39
Cw9	CS13 - CHS	34,014	NLC3	-8611,46	-3,82	-144,48	688,10	1342,97	3141,39

7.3.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC3

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC3	-4116,81	-1858,64	15999,66	0,00	0,00	0,00
Sn2/K210	NLC3	2041,50	-4059,23	16410,27	0,00	0,00	0,00
Sn3/K211	NLC3	4116,81	1858,64	15999,66	0,00	0,00	0,00
Sn4/K208	NLC3	0,00	-4,54	5022,04	0,00	0,00	0,00
Sn5/K205	NLC3	1,77	0,00	5384,83	0,00	0,00	0,00
Sn6/K206	NLC3	0,00	4,54	5022,04	0,00	0,00	0,00
Sn7/K207	NLC3	-1,77	0,00	5384,83	0,00	0,00	0,00
Sn8/K216	NLC3	-4116,81	1858,64	15999,66	0,00	0,00	0,00
Sn9/K214	NLC3	4116,81	-1858,64	15999,66	0,00	0,00	0,00
Sn10/K215	NLC3	2041,50	4059,23	16410,27	0,00	0,00	0,00
Sn11/K213	NLC3	-2041,50	-4059,23	16410,27	0,00	0,00	0,00
Sn12/K212	NLC3	-2041,50	4059,23	16410,27	0,00	0,00	0,00

7.4. Niet-lineaire combinaties - NLC4

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC4	Uiterste Grenstoestand	BG2 - Self-Weight steel	0,90
		BG5 - Upward Wind	1,65
		BG10 - Superimposed Loads	0,90

7.4.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC4

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K348	NLC4	-257,0	24,5	306,5
K334	NLC4	257,0	24,5	306,5
K31	NLC4	-117,7	-90,0	134,9
K445	NLC4	0,0	460,5	416,7
K431	NLC4	0,0	140,3	-228,6

7.4.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC4

Staafl	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC4	42,932	0,17	0,17	0,06
Cv4	CS14 - CHS	S 355	NLC4	41,000	0,13	0,13	0,07
Cv7	CS22 - CHS	S 355	NLC4	0,000	0,68	0,68	0,46
Chord138	CS15 - CHS	S 355	NLC4	9,651	0,99	0,76	0,99
Vert29	CS16 - CHS	S 355	NLC4	9,000	0,73	0,73	0,00
Diag42	CS17 - CHS	S 355	NLC4	0,000	0,54	0,37	0,54
X13	CS18 - CHS	S 355	NLC4	0,000	0,85	0,85	0,34
Y10	CS19 - CHS	S 355	NLC4	0,000	0,79	0,79	0,62
Chord139	CS20 - CHS	S 355	NLC4	1,534	0,43	0,43	0,36
Chord7	CS21 - CHS	S 355	NLC4	0,000	0,86	0,76	0,86

7.4.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC4

Doorsnede : CS13 - CHS (1010; 56)

Staafl	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw4	CS13 - CHS	0,000	NLC4	1614,77	-35,94	34,29	-210,59	-291,39	647,18
Cw5	CS13 - CHS	42,932	NLC4	9008,45	84,80	-164,28	196,20	-2253,72	1546,90
Cw11	CS13 - CHS	46,162	NLC4	6572,62	-90,15	-127,22	-224,74	-885,66	-2000,89
Cw16	CS13 - CHS	46,162	NLC4	6572,62	90,15	-127,22	224,74	-885,66	2000,89

Staaf	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw11	CS13 - CHS	0,000	NLC4	6096,50	-44,12	110,21	-229,10	-570,43	412,58
Cw10	CS13 - CHS	0,000	NLC4	5666,65	-33,82	96,78	-313,63	-467,84	280,17
Cw13	CS13 - CHS	0,000	NLC4	5666,65	33,82	96,78	313,63	-467,84	-280,17
Cw1	CS13 - CHS	42,932	NLC4	3620,77	-44,76	-150,82	-248,09	-2515,09	-980,43
Cw11	CS13 - CHS	21,866	NLC4	6322,16	-44,86	-0,52	-227,04	555,79	-494,58

7.4.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC4

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC4	2673,01	696,19	-7983,67	0,00	0,00	0,00
Sn2/K210	NLC4	-2410,94	1610,92	-11024,19	0,00	0,00	0,00
Sn3/K211	NLC4	-1675,60	-1707,34	-8944,78	0,00	0,00	0,00
Sn4/K208	NLC4	0,00	5,23	-2298,42	0,00	0,00	0,00
Sn5/K205	NLC4	-16,27	-1,63	-1957,20	0,00	0,00	0,00
Sn6/K206	NLC4	0,00	-16,06	-232,47	0,00	0,00	0,00
Sn7/K207	NLC4	16,27	-1,63	-1957,20	0,00	0,00	0,00
Sn8/K216	NLC4	1675,60	-1707,34	-8944,78	0,00	0,00	0,00
Sn9/K214	NLC4	-2673,01	696,19	-7983,67	0,00	0,00	0,00
Sn10/K215	NLC4	-228,05	-2874,38	-6892,03	0,00	0,00	0,00
Sn11/K213	NLC4	2410,94	1610,92	-11024,19	0,00	0,00	0,00
Sn12/K212	NLC4	228,05	-2874,38	-6892,03	0,00	0,00	0,00

7.5. Niet-lineaire combinaties - NLC5i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC5i	Uiterste Grenstoestand	BG2 - Self-Weight steel	1,30
		BG7 - Snow Load i	1,65
		BG10 - Superimposed Loads	1,30

7.5.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC5i

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K101	NLC5i	-109,9	7,9	-41,2
K5	NLC5i	109,9	7,9	-41,2
K155	NLC5i	0,0	-111,0	-28,0
K53	NLC5i	0,0	111,0	-28,0
K306	NLC5i	-1,4	1,6	-271,7
K210	NLC5i	0,0	0,0	0,0

7.5.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC5i

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC5i	0,000	0,56	0,10	0,56
Cv1	CS14 - CHS	S 355	NLC5i	0,000	0,97	0,37	0,97
Cv7	CS22 - CHS	S 355	NLC5i	0,000	0,46	0,42	0,46
Chord19	CS15 - CHS	S 355	NLC5i	0,000	0,66	0,66	0,66
Vert48	CS16 - CHS	S 355	NLC5i	12,728	0,85	0,32	0,85
Diag12	CS17 - CHS	S 355	NLC5i	0,000	0,54	0,35	0,54
X1	CS18 - CHS	S 355	NLC5i	1,508	0,45	0,45	0,39
Y1	CS19 - CHS	S 355	NLC5i	1,508	0,47	0,47	0,41
Chord130	CS20 - CHS	S 355	NLC5i	0,000	0,55	0,52	0,55
Chord01	CS21 - CHS	S 355	NLC5i	4,851	0,50	0,50	0,46

7.5.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC5i

Doorsnede : CS13 - CHS (1010; 56)

Staaf	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC5i	-5732,00	67,37	207,53	352,99	-683,54	287,15
Cw2	CS13 - CHS	42,932	NLC5i	-2563,43	1,30	-93,76	338,62	1204,17	1208,50
Cw14	CS13 - CHS	0,000	NLC5i	-5493,66	-71,09	193,68	-307,36	-598,07	-193,63
Cw9	CS13 - CHS	0,000	NLC5i	-5493,66	71,09	193,68	307,36	-598,07	193,63
Cw9	CS13 - CHS	46,162	NLC5i	-4805,88	-8,99	-207,95	287,42	-1119,99	1897,17
Cw6	CS13 - CHS	0,000	NLC5i	-3651,79	-34,98	170,06	-369,20	-453,31	-576,62
Cw1	CS13 - CHS	0,000	NLC5i	-3651,79	34,98	170,06	369,20	-453,31	576,62
Cw1	CS13 - CHS	28,621	NLC5i	-3193,97	10,23	-3,26	362,18	2024,60	1213,41
Cw14	CS13 - CHS	41,303	NLC5i	-4878,35	-1,72	-174,38	-289,52	-188,64	-1914,12
Cw9	CS13 - CHS	41,303	NLC5i	-4878,35	1,72	-174,38	289,52	-188,64	1914,12

7.5.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC5i

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC5i	-2485,96	-733,00	8701,28	0,00	0,00	0,00
Sn2/K210	NLC5i	620,04	-2379,82	8101,80	0,00	0,00	0,00
Sn3/K211	NLC5i	2485,96	733,00	8701,28	0,00	0,00	0,00
Sn4/K208	NLC5i	0,00	-8,61	6323,89	0,00	0,00	0,00
Sn5/K205	NLC5i	7,47	0,00	6705,83	0,00	0,00	0,00
Sn6/K206	NLC5i	0,00	8,61	6323,89	0,00	0,00	0,00
Sn7/K207	NLC5i	-7,47	0,00	6705,83	0,00	0,00	0,00
Sn8/K216	NLC5i	-2485,96	733,00	8701,28	0,00	0,00	0,00
Sn9/K214	NLC5i	2485,96	-733,00	8701,28	0,00	0,00	0,00
Sn10/K215	NLC5i	620,04	2379,82	8101,80	0,00	0,00	0,00
Sn11/K213	NLC5i	-620,04	-2379,82	8101,80	0,00	0,00	0,00
Sn12/K212	NLC5i	-620,04	2379,82	8101,80	0,00	0,00	0,00

7.6. Niet-lineaire combinaties - NLC5ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC5ii	Jiterste Grenstoestand	BG2 - Self-Weight steel	1,30
		BG8 - Snow Load ii	1,65
		BG10 - Superimposed Loads	1,30

7.6.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC5ii

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K95	NLC5ii	-138,4	-1,1	-71,3
K11	NLC5ii	138,4	-1,1	-71,3
K221	NLC5ii	0,0	-195,7	-9,3
K132	NLC5ii	93,4	42,9	-20,4
K289	NLC5ii	-2,6	-141,0	-302,6
K330	NLC5ii	-51,0	-88,9	10,3

7.6.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC5ii

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC5ii	0,000	0,77	0,14	0,77
Cv2	CS14 - CHS	S 355	NLC5ii	0,000	0,93	0,32	0,93
Cv7	CS22 - CHS	S 355	NLC5ii	0,000	0,55	0,44	0,55
Chord19	CS15 - CHS	S 355	NLC5ii	0,000	0,85	0,85	0,85
Vert140	CS16 - CHS	S 355	NLC5ii	12,728	0,95	0,36	0,95
Diag12	CS17 - CHS	S 355	NLC5ii	0,000	0,84	0,55	0,84
X54	CS18 - CHS	S 355	NLC5ii	0,000	0,57	0,53	0,57
Y1	CS19 - CHS	S 355	NLC5ii	1,508	0,60	0,60	0,53
Chord15	CS20 - CHS	S 355	NLC5ii	1,534	0,77	0,77	0,76
Chord03	CS21 - CHS	S 355	NLC5ii	0,000	0,60	0,60	0,54

7.6.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC5ii
Doorsnede : CS13 - CHS (1010; 56)

Staaf	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw9	CS13 - CHS	0,000	NLC5ii	-7823,77	104,80	210,45	412,91	-484,13	149,95
Cw7	CS13 - CHS	42,932	NLC5ii	-453,64	-4,92	-73,93	-352,80	1296,52	-920,12
Cw14	CS13 - CHS	0,000	NLC5ii	-7823,76	-104,80	210,45	-412,91	-484,13	-149,95
Cw9	CS13 - CHS	46,162	NLC5ii	-7136,40	-41,52	-227,19	387,79	-1213,26	2143,24
Cw5	CS13 - CHS	0,000	NLC5ii	-6933,24	-65,69	195,86	-422,67	-348,43	-356,80
Cw2	CS13 - CHS	0,000	NLC5ii	-6933,24	65,69	195,86	422,67	-348,43	356,80
Cw2	CS13 - CHS	26,236	NLC5ii	-6514,08	18,52	-0,31	414,75	2434,63	1526,63
Cw14	CS13 - CHS	36,444	NLC5ii	-7281,64	3,28	-156,47	-393,08	680,21	-2371,18
Cw9	CS13 - CHS	36,444	NLC5ii	-7281,64	-3,28	-156,47	393,08	680,21	2371,18

7.6.4. Reacties

Niet-lineaire berekening, Extreem : Knoop
Selectie : Alle
Niet-lineaire combinaties : NLC5ii

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC5ii	-2498,73	-135,39	6874,09	0,00	0,00	0,00
Sn2/K210	NLC5ii	1624,14	-3484,93	13681,83	0,00	0,00	0,00
Sn3/K211	NLC5ii	2049,71	1271,37	9436,78	0,00	0,00	0,00
Sn4/K208	NLC5ii	0,00	-18,56	6507,51	0,00	0,00	0,00
Sn5/K205	NLC5ii	6,51	-1,55	4473,26	0,00	0,00	0,00
Sn6/K206	NLC5ii	0,00	-1,08	5763,86	0,00	0,00	0,00
Sn7/K207	NLC5ii	-6,51	-1,55	4473,26	0,00	0,00	0,00
Sn8/K216	NLC5ii	-2049,70	1271,37	9436,76	0,00	0,00	0,00
Sn9/K214	NLC5ii	2498,73	-135,39	6874,10	0,00	0,00	0,00
Sn10/K215	NLC5ii	-4,85	2360,32	6080,33	0,00	0,00	0,00
Sn11/K213	NLC5ii	-1624,14	-3484,93	13681,84	0,00	0,00	0,00
Sn12/K212	NLC5ii	4,84	2360,31	6080,33	0,00	0,00	0,00

7.7. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Brikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG10 - Superimposed Loads	1,00

7.7.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal
Selectie : Alle
Niet-lineaire combinaties : NLC6

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K101	NLC6	-47,2	3,1	-18,4
K5	NLC6	47,2	3,1	-18,4
K155	NLC6	0,0	-47,4	-12,8
K53	NLC6	0,0	47,4	-12,8
K306	NLC6	-0,5	0,6	-119,7
K209	NLC6	0,0	0,0	0,0

7.7.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede
Selectie : Alle
Niet-lineaire combinaties : NLC6

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC6	0,000	0,27	0,05	0,27
Cv3	CS14 - CHS	S 355	NLC6	0,000	0,41	0,17	0,41
Cv7	CS22 - CHS	S 355	NLC6	0,000	0,20	0,18	0,20
ChordI9	CS15 - CHS	S 355	NLC6	0,000	0,29	0,29	0,29
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag12	CS17 - CHS	S 355	NLC6	0,000	0,27	0,17	0,27
X1	CS18 - CHS	S 355	NLC6	1,508	0,18	0,18	0,16
Y1	CS19 - CHS	S 355	NLC6	1,508	0,18	0,18	0,16
ChordI30	CS20 - CHS	S 355	NLC6	0,000	0,30	0,27	0,30
ChordI1	CS21 - CHS	S 355	NLC6	4,851	0,22	0,22	0,20

7.7.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC6

Doorsnede : CS13 - CHS (1010; 56)

Staaf	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC6	-2894,19	22,22	142,73	184,33	-533,81	280,78
Cw2	CS13 - CHS	42,932	NLC6	-1284,10	-1,28	-70,36	172,14	458,25	562,34
Cw14	CS13 - CHS	0,000	NLC6	-2782,99	-24,75	137,17	-162,81	-495,45	-237,41
Cw9	CS13 - CHS	0,000	NLC6	-2782,99	24,75	137,17	162,81	-495,45	237,41
Cw9	CS13 - CHS	46,162	NLC6	-2253,51	5,15	-150,26	155,11	-871,56	957,20
Cw6	CS13 - CHS	0,000	NLC6	-1918,77	-8,52	108,75	-185,91	-362,24	-481,65
Cw1	CS13 - CHS	0,000	NLC6	-1918,77	8,52	108,75	185,91	-362,24	481,65
Cw10	CS13 - CHS	21,866	NLC6	-2643,78	14,16	4,26	180,72	1106,94	674,15
Cw14	CS13 - CHS	46,162	NLC6	-2253,51	-5,15	-150,26	-155,11	-871,56	-957,20

7.7.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC6

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC6	-1218,38	-368,88	4494,46	0,00	0,00	0,00
Sn2/K210	NLC6	340,58	-1170,15	4292,60	0,00	0,00	0,00
Sn3/K211	NLC6	1218,38	368,88	4494,46	0,00	0,00	0,00
Sn4/K208	NLC6	0,00	-4,02	2945,80	0,00	0,00	0,00
Sn5/K205	NLC6	3,95	0,00	3125,22	0,00	0,00	0,00
Sn6/K206	NLC6	0,00	4,02	2945,80	0,00	0,00	0,00
Sn7/K207	NLC6	-3,95	0,00	3125,22	0,00	0,00	0,00
Sn8/K216	NLC6	-1218,38	368,88	4494,46	0,00	0,00	0,00
Sn9/K214	NLC6	1218,38	-368,88	4494,46	0,00	0,00	0,00
Sn10/K215	NLC6	340,58	1170,15	4292,60	0,00	0,00	0,00
Sn11/K213	NLC6	-340,58	-1170,15	4292,60	0,00	0,00	0,00
Sn12/K212	NLC6	-340,58	1170,15	4292,60	0,00	0,00	0,00

7.8. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG3 - Imposed Loads	1,00
		BG10 - Superimposed Loads	1,00

7.8.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC7

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K347	NLC7	-188,1	0,0	92,9
K335	NLC7	188,1	0,0	92,9
K382	NLC7	0,0	-188,6	91,9
K232	NLC7	0,0	188,6	91,9
K341	NLC7	0,0	0,0	-534,1

7.8.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC7	0,000	0,28	0,05	0,28
Cv3	CS14 - CHS	S 355	NLC7	0,000	0,64	0,27	0,64
Cv14	CS22 - CHS	S 355	NLC7	0,000	0,21	0,20	0,21
Chordi11	CS15 - CHS	S 355	NLC7	0,000	0,37	0,37	0,37
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag3	CS17 - CHS	S 355	NLC7	0,000	0,32	0,22	0,32
X76	CS18 - CHS	S 355	NLC7	0,000	0,43	0,43	0,36
Y94	CS19 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Chord17	CS20 - CHS	S 355	NLC7	11,127	0,31	0,24	0,31
Chord01	CS21 - CHS	S 355	NLC7	4,851	0,33	0,33	0,31

7.8.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC7

Doorsnede : CS13 - CHS (1010; 56)

Staaft	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC7	-3045,50	26,61	143,33	161,40	-522,94	256,93
Cw2	CS13 - CHS	42,932	NLC7	-642,75	1,79	-68,07	157,52	474,76	616,92
Cw14	CS13 - CHS	0,000	NLC7	-2856,07	-29,10	137,40	-134,79	-486,10	-211,08
Cw9	CS13 - CHS	0,000	NLC7	-2856,07	29,10	137,40	134,79	-486,10	211,08
Cw9	CS13 - CHS	46,162	NLC7	-2326,70	6,92	-150,79	126,66	-871,98	1089,88
Cw6	CS13 - CHS	0,000	NLC7	-1598,55	-9,39	107,52	-174,21	-329,98	-475,70
Cw1	CS13 - CHS	0,000	NLC7	-1598,55	9,39	107,52	174,21	-329,98	475,70
Cw10	CS13 - CHS	21,866	NLC7	-2795,16	17,66	4,09	157,57	1124,24	737,83
Cw14	CS13 - CHS	46,162	NLC7	-2326,70	-6,92	-150,79	-126,66	-871,98	-1089,88

7.8.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC7

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC7	-1284,90	-266,55	4322,46	0,00	0,00	0,00
Sn2/K210	NLC7	144,79	-1198,36	3744,55	0,00	0,00	0,00
Sn3/K211	NLC7	1284,90	266,55	4322,46	0,00	0,00	0,00
Sn4/K208	NLC7	0,00	-6,93	4572,62	0,00	0,00	0,00
Sn5/K205	NLC7	6,89	0,00	4847,36	0,00	0,00	0,00
Sn6/K206	NLC7	0,00	6,93	4572,62	0,00	0,00	0,00
Sn7/K207	NLC7	-6,89	0,00	4847,36	0,00	0,00	0,00
Sn8/K216	NLC7	-1284,90	266,55	4322,46	0,00	0,00	0,00
Sn9/K214	NLC7	1284,90	-266,55	4322,46	0,00	0,00	0,00
Sn10/K215	NLC7	144,79	1198,36	3744,55	0,00	0,00	0,00
Sn11/K213	NLC7	-144,79	-1198,36	3744,55	0,00	0,00	0,00
Sn12/K212	NLC7	-144,79	1198,36	3744,55	0,00	0,00	0,00

7.9. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG6 - Downward Wind	1,00
		BG10 - Superimposed Loads	1,00

7.9.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC8

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K334	NLC8	-107,4	0,0	-194,4
K348	NLC8	107,4	0,0	-194,4
K445	NLC8	0,0	-127,8	-223,1
K259	NLC8	0,0	127,8	-223,1
K258	NLC8	-3,8	123,1	-224,7
K209	NLC8	0,0	0,0	0,0

7.9.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC8	0,000	0,59	0,11	0,59
Cv1	CS14 - CHS	S 355	NLC8	0,000	0,62	0,23	0,62
Cv7	CS22 - CHS	S 355	NLC8	0,000	0,62	0,54	0,62
Chordi13	CS15 - CHS	S 355	NLC8	0,000	0,62	0,62	0,62
Vert29	CS16 - CHS	S 355	NLC8	0,000	0,61	0,43	0,61
Diag12	CS17 - CHS	S 355	NLC8	0,000	0,61	0,41	0,61
X1	CS18 - CHS	S 355	NLC8	1,508	0,50	0,50	0,45
Y1	CS19 - CHS	S 355	NLC8	1,508	0,59	0,59	0,53
Chord130	CS20 - CHS	S 355	NLC8	0,000	0,66	0,56	0,66
Chordo7	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,52

7.9.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC8

Doorsnede : CS13 - CHS (1010; 56)

Staat	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC8	-6217,40	71,18	172,30	465,37	-503,81	246,44
Cw1	CS13 - CHS	42,932	NLC8	-4606,56	-7,45	-78,35	420,84	1768,11	1379,17
Cw14	CS13 - CHS	0,000	NLC8	-6096,40	-75,58	156,61	-426,60	-391,21	-147,47
Cw9	CS13 - CHS	0,000	NLC8	-6096,40	75,58	156,61	426,60	-391,21	147,47
Cw9	CS13 - CHS	46,162	NLC8	-5567,27	-15,81	-166,08	410,47	-761,10	1890,12
Cw13	CS13 - CHS	0,000	NLC8	-6217,40	-71,18	172,30	-465,37	-503,81	-246,44
Cw1	CS13 - CHS	31,006	NLC8	-4754,25	11,34	-4,54	423,08	2274,56	1359,82
Cw14	CS13 - CHS	38,873	NLC8	-5651,03	-3,09	-125,19	-413,02	308,87	-1939,05
Cw9	CS13 - CHS	38,873	NLC8	-5651,03	3,09	-125,19	413,02	308,87	1939,05

7.9.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC8

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC8	-2739,72	-1165,31	10529,64	0,00	0,00	0,00
Sn2/K210	NLC8	1255,67	-2693,70	10689,82	0,00	0,00	0,00
Sn3/K211	NLC8	2739,72	1165,31	10529,64	0,00	0,00	0,00
Sn4/K208	NLC8	0,00	-5,90	3955,73	0,00	0,00	0,00
Sn5/K205	NLC8	4,77	0,00	4212,95	0,00	0,00	0,00
Sn6/K206	NLC8	0,00	5,90	3955,73	0,00	0,00	0,00
Sn7/K207	NLC8	-4,77	0,00	4212,95	0,00	0,00	0,00
Sn8/K216	NLC8	-2739,72	1165,31	10529,64	0,00	0,00	0,00
Sn9/K214	NLC8	2739,72	-1165,31	10529,64	0,00	0,00	0,00
Sn10/K215	NLC8	1255,67	2693,70	10689,82	0,00	0,00	0,00
Sn11/K213	NLC8	-1255,67	-2693,70	10689,82	0,00	0,00	0,00
Sn12/K212	NLC8	-1255,67	2693,70	10689,82	0,00	0,00	0,00

7.10. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG5 - Upward Wind	1,00
		BG10 - Superimposed Loads	1,00

7.10.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC9

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K348	NLC9	-187,0	14,5	185,9
K334	NLC9	187,0	14,5	185,9
K259	NLC9	0,0	-64,5	115,8
K445	NLC9	0,0	315,6	254,0
K431	NLC9	0,0	88,1	-209,4

7.10.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staat	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv6	CS22 - CHS	S 355	NLC9	0,000	0,29	0,27	0,29
Chord138	CS15 - CHS	S 355	NLC9	7,238	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,45	0,18	0,45
Diag11	CS17 - CHS	S 355	NLC9	12,899	0,34	0,21	0,34
X13	CS18 - CHS	S 355	NLC9	0,000	0,55	0,55	0,28
Y111	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,37
Chord117	CS20 - CHS	S 355	NLC9	11,127	0,23	0,19	0,23
Chord26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

7.10.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC9

Doorsnede : CS13 - CHS (1010; 56)

Staafl	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw7	CS13 - CHS	0,000	NLC9	213,38	20,68	64,90	75,42	-346,47	-598,61
Cw2	CS13 - CHS	42,932	NLC9	5036,31	-36,93	-109,64	-53,56	-1118,83	-665,81
Cw5	CS13 - CHS	42,932	NLC9	5036,31	36,93	-109,64	53,56	-1118,83	665,81
Cw10	CS13 - CHS	46,162	NLC9	2658,58	-22,38	-152,93	-127,37	-1251,87	-536,85
Cw11	CS13 - CHS	0,000	NLC9	2433,89	-24,72	124,91	-76,22	-567,10	395,79
Cw9	CS13 - CHS	46,162	NLC9	1263,21	-11,54	-148,72	-133,13	-1182,13	-213,03
Cw14	CS13 - CHS	46,162	NLC9	1263,21	11,54	-148,72	133,13	-1182,13	213,03
Cw1	CS13 - CHS	42,932	NLC9	1623,61	-21,57	-109,96	-97,36	-1302,26	-337,07
Cw11	CS13 - CHS	21,866	NLC9	2684,56	-22,84	-0,93	-75,85	746,95	-110,03
Cw11	CS13 - CHS	46,162	NLC9	2963,40	-30,00	-142,04	-75,45	-936,47	-724,01
Cw16	CS13 - CHS	46,162	NLC9	2963,40	30,00	-142,04	75,45	-936,47	724,01

7.10.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC9

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC9	1071,87	277,79	-2862,01	0,00	0,00	0,00
Sn2/K210	NLC9	-1356,08	448,49	-4878,16	0,00	0,00	0,00
Sn3/K211	NLC9	-461,23	-897,01	-3458,17	0,00	0,00	0,00
Sn4/K208	NLC9	0,00	2,69	66,89	0,00	0,00	0,00
Sn5/K205	NLC9	-5,30	0,04	409,39	0,00	0,00	0,00
Sn6/K206	NLC9	0,00	-3,53	1436,10	0,00	0,00	0,00
Sn7/K207	NLC9	5,30	0,04	409,39	0,00	0,00	0,00
Sn8/K216	NLC9	461,23	-897,01	-3458,17	0,00	0,00	0,00
Sn9/K214	NLC9	-1071,87	277,79	-2862,01	0,00	0,00	0,00
Sn10/K215	NLC9	-6,06	-1211,70	-2285,86	0,00	0,00	0,00
Sn11/K213	NLC9	1356,08	448,49	-4878,16	0,00	0,00	0,00
Sn12/K212	NLC9	6,06	-1211,70	-2285,86	0,00	0,00	0,00

7.11. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG7 - Snow Load i	1,00
		BG10 - Superimposed Loads	1,00

7.11.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K101	NLC10i	-76,1	5,4	-28,8
K5	NLC10i	76,1	5,4	-28,8
K155	NLC10i	0,0	-76,8	-19,6
K53	NLC10i	0,0	76,8	-19,6
K306	NLC10i	-1,0	1,1	-189,1
K209	NLC10i	0,0	0,0	0,0

7.11.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staafl	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv3	CS14 - CHS	S 355	NLC10i	0,000	0,64	0,26	0,64
Cv7	CS22 - CHS	S 355	NLC10i	0,000	0,32	0,29	0,32
Chord19	CS15 - CHS	S 355	NLC10i	0,000	0,46	0,46	0,46
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,23	0,59
Diag12	CS17 - CHS	S 355	NLC10i	0,000	0,38	0,25	0,38
X1	CS18 - CHS	S 355	NLC10i	1,508	0,31	0,31	0,27
Y1	CS19 - CHS	S 355	NLC10i	1,508	0,32	0,32	0,28

Staafl	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Chordl30	CS20 - CHS	S 355	NLC10i	0,000	0,40	0,36	0,40
Chordo1	CS21 - CHS	S 355	NLC10i	4,851	0,35	0,35	0,32

7.11.3. Interne krachten in staaf

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Doorsnede : CS13 - CHS (1010; 56)

Staafl	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw10	CS13 - CHS	0,000	NLC10i	-4088,85	39,67	151,30	248,22	-516,31	245,67
Cw2	CS13 - CHS	42,932	NLC10i	-1828,86	3,23	-67,56	231,57	839,27	856,33
Cw14	CS13 - CHS	0,000	NLC10i	-3922,26	-42,75	142,44	-217,73	-458,53	-181,68
Cw9	CS13 - CHS	0,000	NLC10i	-3922,26	42,75	142,44	217,73	-458,53	181,68
Cw9	CS13 - CHS	46,162	NLC10i	-3392,97	3,60	-154,43	207,28	-835,02	1379,48
Cw6	CS13 - CHS	0,000	NLC10i	-2622,14	-20,28	122,99	-250,36	-359,48	-443,44
Cw1	CS13 - CHS	0,000	NLC10i	-2622,14	20,28	122,99	250,36	-359,48	443,44
Cw1	CS13 - CHS	28,621	NLC10i	-2269,62	7,79	-1,94	246,59	1417,86	835,95
Cw14	CS13 - CHS	46,162	NLC10i	-3392,97	-3,60	-154,43	-207,28	-835,02	-1379,48

7.11.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC10i	-1762,63	-520,33	6230,29	0,00	0,00	0,00
Sn2/K210	NLC10i	446,45	-1689,26	5826,35	0,00	0,00	0,00
Sn3/K211	NLC10i	1762,63	520,33	6230,29	0,00	0,00	0,00
Sn4/K208	NLC10i	0,00	-6,52	4449,87	0,00	0,00	0,00
Sn5/K205	NLC10i	6,12	0,00	4716,64	0,00	0,00	0,00
Sn6/K206	NLC10i	0,00	6,52	4449,87	0,00	0,00	0,00
Sn7/K207	NLC10i	-6,12	0,00	4716,64	0,00	0,00	0,00
Sn8/K216	NLC10i	-1762,63	520,33	6230,29	0,00	0,00	0,00
Sn9/K214	NLC10i	1762,63	-520,33	6230,29	0,00	0,00	0,00
Sn10/K215	NLC10i	446,45	1689,26	5826,35	0,00	0,00	0,00
Sn11/K213	NLC10i	-446,45	-1689,26	5826,35	0,00	0,00	0,00
Sn12/K212	NLC10i	-446,45	1689,26	5826,35	0,00	0,00	0,00

7.12. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG8 - Snow Load ii	1,00
		BG10 - Superimposed Loads	1,00

7.12.1. Verplaatsing van knopen

Niet-lineaire berekening, Extreem : Globaal

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Knoop	BG	Ux [mm]	Uy [mm]	Uz [mm]
K95	NLC10ii	-90,3	0,0	-47,5
K11	NLC10ii	90,3	0,0	-47,5
K221	NLC10ii	0,0	-124,6	-11,0
K132	NLC10ii	63,8	34,4	-15,4
K289	NLC10ii	-1,7	-85,2	-208,4
K209	NLC10ii	0,0	0,0	0,0

7.12.2. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staafl	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,48	0,09	0,48
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,61	0,25	0,61
Cv7	CS22 - CHS	S 355	NLC10ii	0,000	0,37	0,30	0,37
Chordl9	CS15 - CHS	S 355	NLC10ii	0,000	0,57	0,57	0,57

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,55	0,36	0,55
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,36	0,36	0,34
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,40	0,40	0,36
Chord130	CS20 - CHS	S 355	NLC10ii	0,000	0,52	0,48	0,52
Chord3	CS21 - CHS	S 355	NLC10ii	0,000	0,40	0,40	0,36

7.12.3. Interne krachten in staaft

Niet-lineaire berekening, Extreem : Globaal, Systeem : Hoofd

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Doorsnede : CS13 - CHS (1010; 56)

Staaft	css	dx [m]	BG	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Cw9	CS13 - CHS	0,000	NLC10ii	-5327,47	57,31	148,46	273,26	-393,70	158,41
Cw7	CS13 - CHS	42,932	NLC10ii	-577,96	-3,13	-59,34	-238,54	874,60	-676,60
Cw14	CS13 - CHS	0,000	NLC10ii	-5327,47	-57,31	148,46	-273,26	-393,70	-158,41
Cw9	CS13 - CHS	46,162	NLC10ii	-4798,37	-6,89	-162,13	260,99	-861,78	1565,14
Cw10	CS13 - CHS	0,000	NLC10ii	-4135,02	42,33	151,27	260,91	-535,75	221,73
Cw6	CS13 - CHS	0,000	NLC10ii	-1434,19	-16,08	122,21	-277,63	-345,11	-456,16
Cw1	CS13 - CHS	0,000	NLC10ii	-1434,19	16,08	122,21	277,63	-345,11	456,16
Cw2	CS13 - CHS	26,236	NLC10ii	-4220,14	13,99	4,04	270,66	1588,04	961,61
Cw14	CS13 - CHS	41,303	NLC10ii	-4854,11	-1,95	-136,24	-262,28	-135,18	-1577,51
Cw9	CS13 - CHS	41,303	NLC10ii	-4854,12	1,95	-136,24	262,28	-135,18	1577,51

7.12.4. Reacties

Niet-lineaire berekening, Extreem : Knoop

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Steunpunt	BG	Rx [kN]	Ry [kN]	Rz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Sn1/K209	NLC10ii	-1772,83	-162,73	5138,73	0,00	0,00	0,00
Sn2/K210	NLC10ii	1039,64	-2350,51	9158,82	0,00	0,00	0,00
Sn3/K211	NLC10ii	1501,42	838,68	6656,53	0,00	0,00	0,00
Sn4/K208	NLC10ii	0,00	-9,86	4543,53	0,00	0,00	0,00
Sn5/K205	NLC10ii	4,92	-0,22	3384,88	0,00	0,00	0,00
Sn6/K206	NLC10ii	0,00	3,14	4143,63	0,00	0,00	0,00
Sn7/K207	NLC10ii	-4,92	-0,22	3384,88	0,00	0,00	0,00
Sn8/K216	NLC10ii	-1501,42	838,68	6656,51	0,00	0,00	0,00
Sn9/K214	NLC10ii	1772,83	-162,73	5138,74	0,00	0,00	0,00
Sn10/K215	NLC10ii	76,63	1678,14	4625,20	0,00	0,00	0,00
Sn11/K213	NLC10ii	-1039,64	-2350,51	9158,83	0,00	0,00	0,00
Sn12/K212	NLC10ii	-76,64	1678,14	4625,20	0,00	0,00	0,00

8. Kritische belastingcoëfficiënten

N	f []
Stabiliteitcombinatie : S1	
1	3,00
2	3,00
Stabiliteitcombinatie : S2	
1	2,00
2	2,00
Stabiliteitcombinatie : S3	
1	1,00
2	1,13
Stabiliteitcombinatie : S4	
1	16,00
2	16,39
Stabiliteitcombinatie : S5	
1	2,00
2	2,00
Stabiliteitcombinatie : S6	
1	2,00
2	2,07
Stabiliteitcombinatie : S7	
1	4,00
2	4,01
Stabiliteitcombinatie : S8	
1	2,00
2	2,00
Stabiliteitcombinatie : S9	

N	f
-	[]
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	31,00
2	31,43
Stabiliteitcombinatie : S11	
1	3,00
2	3,00
Stabiliteitcombinatie : S12	
1	3,00
2	3,29

9. Materiaallijst

Naam	Massa [kg]	Oppervlak [m ²]	Volume [m ³]
Totale resultaten :	4469860,8	31396,178	5,6941e+02

Verklaring van symbolen	
Oppervlak	Opmerking: Enkel een oppervlak van elke 2D element is in rekening gebracht voor de berekening van het oppervlak.

Doorsnede	Materiaal	Eenheidsmassa [kg/m]	Lengte [m]	Massa [kg]	Oppervlak [m ²]	Volumieke massa [kg/m ³]	Volume [m ³]
CS13 - CHS (1010; 56)	S 355	1317,2	712,752	938871,0	2261,452	7850,0	1,1960e+02
CS14 - CHS (880; 19)	S 355	403,4	164,000	66150,3	453,372	7850,0	8,4268e+00
CS15 - CHS (600; 17)	S 355	244,4	628,016	153468,7	1183,723	7850,0	1,9550e+01
CS16 - CHS (360; 8)	S 355	69,4	1909,492	132581,2	2159,475	7850,0	1,6889e+01
CS17 - CHS (630; 15)	S 355	227,5	2924,282	665146,0	5787,456	7850,0	8,4732e+01
CS18 - CHS (800; 16)	S 355	309,3	3096,858	957829,9	7782,858	7850,0	1,2202e+02
CS19 - CHS (800; 16)	S 355	309,3	3096,857	957830,1	7782,858	7850,0	1,2202e+02
CS20 - CHS (910; 20)	S 355	438,9	628,016	275627,4	1795,313	7850,0	3,5112e+01
CS21 - CHS (800; 19)	S 355	365,9	684,539	250457,3	1720,345	7850,0	3,1905e+01
CS22 - CHS (830; 20)	S 355	399,4	180,000	71898,4	469,330	7850,0	9,1590e+00

F

Calculation of Concrete Filled CHS Columns

$$D_o := 1200 \text{ mm} \quad R_o := \frac{D_o}{2} = 600 \text{ mm}$$

$$t := 20 \text{ mm}$$

$$D_i := D_o - 2 \cdot t = 1160 \text{ mm} \quad R_i := \frac{D_i}{2} = 580 \text{ mm}$$

$$M_x := 773.31 \text{ kN} \cdot \text{m} \quad M_y := 3776.43 \text{ kN} \cdot \text{m} \quad M_z := 3141.39 \text{ kN} \cdot \text{m} \quad N_{druk} := 9272.87 \text{ kN}$$

$$A_s := \pi \cdot (R_o^2 - R_i^2) = 74141.587 \text{ mm}^2 \quad I_s := 0.25 \cdot \pi \cdot (R_o^4 - R_i^4) = 12908050231.4 \text{ mm}^4$$

$$A_c := \pi \cdot R_i^2 = 1056831.8 \text{ mm}^2 \quad I_c := 0.25 \cdot \pi \cdot R_i^4 = 88879551744.9 \text{ mm}^4$$

$$A_r := 0.005 \cdot A_c = 5284.2 \text{ mm}^2 \quad I_r := A_r \cdot (0.35 \cdot 0.35 \cdot D_o^2) = 932125620 \text{ mm}^4$$

$$f_{yd} := 355 \frac{\text{N}}{\text{mm}^2}$$

$$f_{rd} := \frac{400}{1.15} \frac{\text{N}}{\text{mm}^2}$$

$$f_{ck} := 20 \frac{\text{N}}{\text{mm}^2}$$

$$f_{cd} := \frac{f_{ck}}{1.5} = 13.3 \frac{\text{N}}{\text{mm}^2}$$

$$E_s := 210000 \frac{\text{N}}{\text{mm}^2}$$

$$E_r := E_s = 210000 \frac{\text{N}}{\text{mm}^2}$$

$$E_c := 30000 \frac{\text{N}}{\text{mm}^2}$$

$$E_{c;eff} := E_c \cdot \frac{1}{1 + 0.5 \cdot 1.25} = 18461.5 \frac{\text{N}}{\text{mm}^2}$$

$$L := 45 \text{ m}$$

$$N_{pl;Rd} := A_s \cdot f_{yd} + A_r \cdot f_{rd} + A_c \cdot f_{cd} = 42249.3 \text{ kN}$$

$$EI_e := I_s \cdot E_s + 0.6 \cdot I_c \cdot E_c + I_r \cdot E_r = 4506268.9 \text{ m}^2 \cdot \text{kN}$$

$$N_{cr} := \pi^2 \cdot \frac{EI_e}{L^2} = 21963 \text{ kN}$$

$$\lambda_{strep} := \frac{N_{pl;Rd}}{N_{cr}} = 1.92366$$

$$\lambda < 2.0$$

OK

$$\delta := A_s \cdot \frac{f_{yd}}{N_{pl;Rd}} = 0.6$$

$$0.2 < \delta < 0.9$$

OK

$$h_n := A_c \cdot \frac{f_{cd}}{2 \cdot D_o \cdot f_{cd} + 4 \cdot t \cdot (2 \cdot f_{yd} - f_{cd})} = 0.2 \text{ m}$$

$$W_{ps} := \frac{\pi \cdot (D_o^4 - D_i^4)}{32 \cdot D_o} = 21513417.1 \text{ mm}^3$$

$$W_{psn} := 0$$

$$W_{pc} := \frac{\pi \cdot D_i^3}{32} = 153240606.5 \text{ mm}^3$$

$$W_{pcn} := \frac{8 \cdot \pi \cdot h_n^3}{32} = 3254091.6 \text{ mm}^3$$

$$W_{pr} := A_r \cdot 0.35 \cdot D_o = 2219346.7 \text{ mm}^3$$

$$W_{prn} := 0$$

$$M_{pl;Rd} := f_{yd} \cdot (W_{ps} - W_{psn}) + 0.5 \cdot f_{cd} \cdot (W_{pc} - W_{pcn}) + f_{rd} \cdot (W_{pr} - W_{prn}) = 9409.1 \text{ kN} \cdot \text{m}$$

$$N_{A;Rd} := N_{pl;Rd}$$

$$M_{A;Rd} := 0$$

$$N_{B;Rd} := 0$$

$$M_{B;Rd} := M_{pl;Rd}$$

$$N_{C;Rd} := A_c \cdot f_{cd}$$

$$M_{C;Rd} := M_{pl;Rd}$$

$$x := \frac{N_{C;Rd}}{N_{pl;Rd}} = 0.334$$

$$N_{D;Rd} := \frac{N_{C;Rd}}{2}$$

$$M_{D;Rd} := f_{yd} \cdot W_{ps} + 0.5 \cdot f_{cd} \cdot W_{pc} + f_{rd} \cdot W_{pr}$$

$$y := \frac{M_{D;Rd}}{M_{pl;Rd}} = 1.002$$

$$Axial := \frac{N_{druk}}{N_{pl;Rd}} = 0.219 \quad \mu := \frac{(1 - Axial)}{x} = 2.3$$

$$\frac{N_{druk}}{N_{pl;Rd}} + \frac{M_y}{\mu \cdot M_{pl;Rd}} + \frac{M_z}{\mu \cdot M_{pl;Rd}} + \frac{M_x}{\mu \cdot M_{pl;Rd}} = 0.569$$

$$U.C. < 1.0$$

OK

G

Engineering Report Safety calculations

G.1. Critical arch member

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC6	0,000	0,27	0,05	0,27
Cv1	CS14 - CHS	S 355	NLC6	0,000	0,42	0,17	0,42
Cv10	CS22 - CHS	S 355	NLC6	0,000	0,21	0,18	0,21
Chord19	CS15 - CHS	S 355	NLC6	0,000	0,30	0,30	0,30
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag195	CS17 - CHS	S 355	NLC6	0,000	0,27	0,17	0,27
X143	CS18 - CHS	S 355	NLC6	1,508	0,19	0,19	0,16
Y10	CS19 - CHS	S 355	NLC6	0,000	0,19	0,19	0,17
Chord164	CS20 - CHS	S 355	NLC6	0,000	0,31	0,27	0,31
Chordo1	CS21 - CHS	S 355	NLC6	4,851	0,22	0,22	0,20

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC7	0,000	0,28	0,05	0,28
Cv1	CS14 - CHS	S 355	NLC7	0,000	0,64	0,27	0,64
Cv14	CS22 - CHS	S 355	NLC7	0,000	0,21	0,20	0,21
Chord145	CS15 - CHS	S 355	NLC7	0,000	0,37	0,37	0,37
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag204	CS17 - CHS	S 355	NLC7	0,000	0,32	0,22	0,32
X208	CS18 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Y155	CS19 - CHS	S 355	NLC7	11,818	0,44	0,44	0,36
Chord117	CS20 - CHS	S 355	NLC7	11,127	0,31	0,24	0,31
Chordo1	CS21 - CHS	S 355	NLC7	4,851	0,33	0,33	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC8	0,000	0,61	0,11	0,61
Cv1	CS14 - CHS	S 355	NLC8	0,000	0,63	0,23	0,63
Cv10	CS22 - CHS	S 355	NLC8	0,000	0,64	0,56	0,64
Chord19	CS15 - CHS	S 355	NLC8	0,000	0,65	0,65	0,65
Vert29	CS16 - CHS	S 355	NLC8	0,000	0,94	0,66	0,94
Diag195	CS17 - CHS	S 355	NLC8	0,000	0,62	0,42	0,62
X29	CS18 - CHS	S 355	NLC8	11,860	0,60	0,60	0,48
Y110	CS19 - CHS	S 355	NLC8	0,000	0,75	0,75	0,55
Chord164	CS20 - CHS	S 355	NLC8	0,000	0,68	0,58	0,68
Chordo62	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,53

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG5 - Upward Wind	1,00
		BG10 - Superimposed Loads	1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw1	CS13 - CHS	S 355	NLC9	42,932	0,08	0,08	0,06
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv9	CS22 - CHS	S 355	NLC9	0,000	0,30	0,21	0,30
Chord165	CS15 - CHS	S 355	NLC9	4,825	0,51	0,39	0,51
Vert29	CS16 - CHS	S 355	NLC9	9,000	0,65	0,65	0,00
Diag11	CS17 - CHS	S 355	NLC9	12,899	0,35	0,22	0,35
X29	CS18 - CHS	S 355	NLC9	11,860	0,77	0,77	0,54
Y146	CS19 - CHS	S 355	NLC9	0,000	0,97	0,86	0,97
Chord117	CS20 - CHS	S 355	NLC9	0,000	0,22	0,18	0,22
Chordo26	CS21 - CHS	S 355	NLC9	9,696	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG7 - Snow Load i	1,00
		BG10 - Superimposed Loads	1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv1	CS14 - CHS	S 355	NLC10i	0,000	0,64	0,26	0,64
Cv10	CS22 - CHS	S 355	NLC10i	0,000	0,33	0,29	0,33
Chord19	CS15 - CHS	S 355	NLC10i	0,000	0,47	0,47	0,47
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,24	0,59
Diag195	CS17 - CHS	S 355	NLC10i	0,000	0,38	0,25	0,38
X152	CS18 - CHS	S 355	NLC10i	0,000	0,31	0,31	0,29
Y10	CS19 - CHS	S 355	NLC10i	0,000	0,32	0,32	0,30
Chord164	CS20 - CHS	S 355	NLC10i	0,000	0,40	0,36	0,40
Chordo1	CS21 - CHS	S 355	NLC10i	4,851	0,35	0,35	0,32

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG8 - Snow Load ii	1,00
		BG10 - Superimposed Loads	1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,48	0,10	0,48
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,61	0,25	0,61
Cv7	CS22 - CHS	S 355	NLC10ii	0,000	0,38	0,30	0,38
Chord19	CS15 - CHS	S 355	NLC10ii	0,000	0,59	0,59	0,59
Vert19	CS16 - CHS	S 355	NLC10ii	0,000	0,72	0,50	0,72
Diag93	CS17 - CHS	S 355	NLC10ii	0,000	0,54	0,36	0,54
X241	CS18 - CHS	S 355	NLC10ii	0,000	0,38	0,38	0,32
Y145	CS19 - CHS	S 355	NLC10ii	0,000	0,41	0,41	0,36
Chord130	CS20 - CHS	S 355	NLC10ii	0,000	0,52	0,48	0,52
Chordo3	CS21 - CHS	S 355	NLC10ii	0,000	0,39	0,39	0,36

2. Kritische belastingcoëfficiënten

N	f
	[]
Stabiliteitcombinatie : S1	
1	3,00
2	3,03
Stabiliteitcombinatie : S2	
1	2,00
2	2,01
Stabiliteitcombinatie : S3	
1	1,00
2	1,13
Stabiliteitcombinatie : S4	
1	16,00
2	16,30
Stabiliteitcombinatie : S5	
1	2,00
2	2,03
Stabiliteitcombinatie : S6	
1	2,00
2	2,06
Stabiliteitcombinatie : S7	
1	4,00
2	4,04
Stabiliteitcombinatie : S8	
1	2,00
2	2,01
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	31,00
2	31,20
Stabiliteitcombinatie : S11	
1	3,00
2	3,05
Stabiliteitcombinatie : S12	
1	3,00
2	3,29

G.2. Middle arch member

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC6	0,000	0,27	0,05	0,27
Cv3	CS14 - CHS	S 355	NLC6	0,000	0,41	0,17	0,41
Cv7	CS22 - CHS	S 355	NLC6	0,000	0,20	0,18	0,20
Chordi26	CS15 - CHS	S 355	NLC6	11,130	0,29	0,29	0,29
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag12	CS17 - CHS	S 355	NLC6	0,000	0,27	0,17	0,27
X10	CS18 - CHS	S 355	NLC6	0,000	0,18	0,18	0,17
Y1	CS19 - CHS	S 355	NLC6	1,508	0,19	0,19	0,16
Chordl64	CS20 - CHS	S 355	NLC6	0,000	0,30	0,27	0,30
Chordo34	CS21 - CHS	S 355	NLC6	7,277	0,22	0,22	0,20

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC7	0,000	0,28	0,05	0,28
Cv3	CS14 - CHS	S 355	NLC7	0,000	0,63	0,26	0,63
Cv14	CS22 - CHS	S 355	NLC7	0,000	0,21	0,20	0,21
Chordi24	CS15 - CHS	S 355	NLC7	9,054	0,37	0,37	0,37
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,64	0,26	0,64
Diag3	CS17 - CHS	S 355	NLC7	0,000	0,34	0,23	0,34
X83	CS18 - CHS	S 355	NLC7	11,818	0,42	0,42	0,35
Y94	CS19 - CHS	S 355	NLC7	0,000	0,43	0,43	0,35
Chordl51	CS20 - CHS	S 355	NLC7	11,127	0,31	0,24	0,31
Chordo34	CS21 - CHS	S 355	NLC7	7,277	0,33	0,33	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC8	0,000	0,59	0,11	0,59
Cv3	CS14 - CHS	S 355	NLC8	0,000	0,60	0,22	0,60
Cv7	CS22 - CHS	S 355	NLC8	0,000	0,62	0,54	0,62
Chordi22	CS15 - CHS	S 355	NLC8	1,534	0,62	0,62	0,62
Vert29	CS16 - CHS	S 355	NLC8	0,000	0,61	0,43	0,61
Diag12	CS17 - CHS	S 355	NLC8	0,000	0,62	0,41	0,62
X10	CS18 - CHS	S 355	NLC8	0,000	0,50	0,50	0,45
Y1	CS19 - CHS	S 355	NLC8	1,508	0,60	0,60	0,53
Chordl64	CS20 - CHS	S 355	NLC8	0,000	0,67	0,57	0,67
Chordo7	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,53

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG5 - Upward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw5	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,23	0,08	0,23
Cv9	CS22 - CHS	S 355	NLC9	0,000	0,29	0,27	0,29
Chordi38	CS15 - CHS	S 355	NLC9	7,238	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,46	0,18	0,46
Diag11	CS17 - CHS	S 355	NLC9	12,899	0,35	0,21	0,35
X13	CS18 - CHS	S 355	NLC9	0,000	0,55	0,55	0,29
Y111	CS19 - CHS	S 355	NLC9	0,000	0,49	0,49	0,38
Chordl17	CS20 - CHS	S 355	NLC9	0,000	0,23	0,19	0,23
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,51	0,44	0,51

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG7 - Snow Load i BG10 - Superimposed Loads	1,00 1,00 1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv3	CS14 - CHS	S 355	NLC10i	0,000	0,63	0,25	0,63
Cv7	CS22 - CHS	S 355	NLC10i	0,000	0,33	0,29	0,33
Chordi26	CS15 - CHS	S 355	NLC10i	11,130	0,46	0,46	0,46
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,58	0,23	0,58
Diag12	CS17 - CHS	S 355	NLC10i	0,000	0,38	0,25	0,38
X10	CS18 - CHS	S 355	NLC10i	0,000	0,31	0,31	0,29
Y1	CS19 - CHS	S 355	NLC10i	1,508	0,32	0,32	0,29
Chordl64	CS20 - CHS	S 355	NLC10i	0,000	0,40	0,36	0,40
Chordo34	CS21 - CHS	S 355	NLC10i	7,277	0,35	0,35	0,32

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG8 - Snow Load ii BG10 - Superimposed Loads	1,00 1,00 1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,48	0,09	0,48
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,60	0,25	0,60
Cv7	CS22 - CHS	S 355	NLC10ii	0,000	0,38	0,30	0,38
Chordi26	CS15 - CHS	S 355	NLC10ii	11,130	0,57	0,57	0,57
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,55	0,37	0,55
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,36	0,36	0,34
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,41	0,41	0,36
Chordl30	CS20 - CHS	S 355	NLC10ii	0,000	0,52	0,48	0,52
Chordo32	CS21 - CHS	S 355	NLC10ii	12,606	0,40	0,40	0,36

2. Kritische belastingcoëfficiënten

N	f
Stabiliteitcombinatie : S1	
1	3,00
2	3,01
Stabiliteitcombinatie : S2	
1	2,00
2	2,01
Stabiliteitcombinatie : S3	
1	1,00
2	1,13
Stabiliteitcombinatie : S4	
1	16,00
2	16,33
Stabiliteitcombinatie : S5	
1	2,00
2	2,01
Stabiliteitcombinatie : S6	
1	2,00
2	2,06
Stabiliteitcombinatie : S7	
1	4,00
2	4,01
Stabiliteitcombinatie : S8	
1	2,00
2	2,01
Stabiliteitcombinatie : S9	
1	1,00
2	1,13
Stabiliteitcombinatie : S10	
1	31,00
2	31,36
Stabiliteitcombinatie : S11	
1	3,00
2	3,01
Stabiliteitcombinatie : S12	
1	3,00
2	3,28

G.3. Edge arch member

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC6	0,000	0,27	0,05	0,27
Cv3	CS14 - CHS	S 355	NLC6	0,000	0,41	0,17	0,41
Cv7	CS22 - CHS	S 355	NLC6	0,000	0,21	0,18	0,21
Chordi9	CS15 - CHS	S 355	NLC6	0,000	0,29	0,29	0,29
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag12	CS17 - CHS	S 355	NLC6	0,000	0,28	0,18	0,28
X1	CS18 - CHS	S 355	NLC6	1,508	0,18	0,18	0,16
Y1	CS19 - CHS	S 355	NLC6	1,508	0,19	0,19	0,16
Chordl64	CS20 - CHS	S 355	NLC6	0,000	0,31	0,27	0,31
Chordo1	CS21 - CHS	S 355	NLC6	4,851	0,22	0,22	0,20

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC7	0,000	0,28	0,05	0,28
Cv3	CS14 - CHS	S 355	NLC7	0,000	0,64	0,26	0,64
Cv16	CS22 - CHS	S 355	NLC7	0,000	0,21	0,20	0,21
Chordi24	CS15 - CHS	S 355	NLC7	9,054	0,38	0,38	0,38
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag3	CS17 - CHS	S 355	NLC7	0,000	0,38	0,27	0,38
X83	CS18 - CHS	S 355	NLC7	11,818	0,43	0,43	0,36
Y148	CS19 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Chordl51	CS20 - CHS	S 355	NLC7	11,127	0,31	0,24	0,31
Chordo1	CS21 - CHS	S 355	NLC7	4,851	0,34	0,34	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC8	0,000	0,59	0,11	0,59
Cv3	CS14 - CHS	S 355	NLC8	0,000	0,61	0,23	0,61
Cv7	CS22 - CHS	S 355	NLC8	0,000	0,63	0,55	0,63
Chordi13	CS15 - CHS	S 355	NLC8	0,000	0,62	0,62	0,62
Vert68	CS16 - CHS	S 355	NLC8	0,000	0,61	0,43	0,61
Diag12	CS17 - CHS	S 355	NLC8	0,000	0,63	0,42	0,63
X1	CS18 - CHS	S 355	NLC8	1,508	0,51	0,51	0,45
Y1	CS19 - CHS	S 355	NLC8	1,508	0,60	0,60	0,53
Chordl64	CS20 - CHS	S 355	NLC8	0,000	0,68	0,58	0,68
Chordo28	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,52

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG5 - Upward Wind	1,00
		BG10 - Superimposed Loads	1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw5	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv9	CS22 - CHS	S 355	NLC9	0,000	0,29	0,27	0,29
Chordi65	CS15 - CHS	S 355	NLC9	4,825	0,51	0,39	0,51
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,45	0,18	0,45
Diag92	CS17 - CHS	S 355	NLC9	12,899	0,34	0,21	0,34
X22	CS18 - CHS	S 355	NLC9	12,401	0,55	0,55	0,29
Y165	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,37
Chordl17	CS20 - CHS	S 355	NLC9	0,000	0,22	0,18	0,22
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG7 - Snow Load i	1,00
		BG10 - Superimposed Loads	1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv3	CS14 - CHS	S 355	NLC10i	0,000	0,64	0,26	0,64
Cv7	CS22 - CHS	S 355	NLC10i	0,000	0,33	0,29	0,33
Chordi9	CS15 - CHS	S 355	NLC10i	0,000	0,46	0,46	0,46
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,24	0,59
Diag12	CS17 - CHS	S 355	NLC10i	0,000	0,39	0,26	0,39
X1	CS18 - CHS	S 355	NLC10i	1,508	0,31	0,31	0,27
Y1	CS19 - CHS	S 355	NLC10i	1,508	0,33	0,33	0,29
Chordl64	CS20 - CHS	S 355	NLC10i	0,000	0,41	0,37	0,41
Chordo1	CS21 - CHS	S 355	NLC10i	6,064	0,36	0,36	0,33

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG8 - Snow Load ii	1,00
		BG10 - Superimposed Loads	1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,49	0,10	0,49
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,61	0,25	0,61
Cv7	CS22 - CHS	S 355	NLC10ii	0,000	0,38	0,30	0,38
Chordi9	CS15 - CHS	S 355	NLC10ii	0,000	0,57	0,57	0,57
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,55	0,37	0,55
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,36	0,36	0,34
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,41	0,41	0,36
Chordl30	CS20 - CHS	S 355	NLC10ii	0,000	0,52	0,48	0,52
Chordo3	CS21 - CHS	S 355	NLC10ii	0,000	0,40	0,40	0,36

2. Kritische belastingcoëfficiënten

N	f
Stabiliteitcombinatie : S1	
1	3,00
2	3,14
Stabiliteitcombinatie : S2	
1	2,00
2	2,10
Stabiliteitcombinatie : S3	
1	1,00
2	1,13
Stabiliteitcombinatie : S4	
1	15,00
2	15,13
Stabiliteitcombinatie : S5	
1	2,00
2	2,09
Stabiliteitcombinatie : S6	
1	2,00
2	2,07
Stabiliteitcombinatie : S7	
1	4,00
2	4,18
Stabiliteitcombinatie : S8	
1	2,00
2	2,09
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	30,00
2	30,10
Stabiliteitcombinatie : S11	
1	3,00
2	3,15
Stabiliteitcombinatie : S12	
1	3,00
2	3,30

G.4. Upper inner chord

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC6	0,000	0,27	0,05	0,27
Cv3	CS14 - CHS	S 355	NLC6	0,000	0,47	0,19	0,47
Cv7	CS22 - CHS	S 355	NLC6	0,000	0,25	0,20	0,25
Chordi26	CS15 - CHS	S 355	NLC6	11,130	0,30	0,30	0,30
Vert140	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag195	CS17 - CHS	S 355	NLC6	0,000	0,27	0,17	0,27
X123	CS18 - CHS	S 355	NLC6	14,553	0,24	0,24	0,19
Y264	CS19 - CHS	S 355	NLC6	11,917	0,22	0,22	0,17
Chordl64	CS20 - CHS	S 355	NLC6	0,000	0,30	0,27	0,30
Chordo35	CS21 - CHS	S 355	NLC6	4,851	0,25	0,25	0,23

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC7	0,000	0,28	0,05	0,28
Cv3	CS14 - CHS	S 355	NLC7	0,000	0,71	0,29	0,71
Cv7	CS22 - CHS	S 355	NLC7	0,000	0,25	0,21	0,25
Chordi45	CS15 - CHS	S 355	NLC7	0,000	0,38	0,38	0,38
Vert140	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag105	CS17 - CHS	S 355	NLC7	0,000	0,37	0,25	0,37
X215	CS18 - CHS	S 355	NLC7	11,818	0,44	0,44	0,36
Y101	CS19 - CHS	S 355	NLC7	11,818	0,48	0,48	0,39
Chordl51	CS20 - CHS	S 355	NLC7	11,127	0,31	0,24	0,31
Chordo35	CS21 - CHS	S 355	NLC7	4,851	0,37	0,37	0,35

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC8	0,000	0,59	0,11	0,59
Cv3	CS14 - CHS	S 355	NLC8	0,000	0,83	0,30	0,83
Cv7	CS22 - CHS	S 355	NLC8	0,000	0,75	0,64	0,75
Chordi56	CS15 - CHS	S 355	NLC8	1,534	0,63	0,63	0,63
Vert104	CS16 - CHS	S 355	NLC8	0,000	0,79	0,51	0,79
Diag195	CS17 - CHS	S 355	NLC8	0,000	0,62	0,41	0,62
X123	CS18 - CHS	S 355	NLC8	14,553	0,55	0,55	0,55
Y10	CS19 - CHS	S 355	NLC8	0,000	0,61	0,61	0,55
Chordl64	CS20 - CHS	S 355	NLC8	0,000	0,67	0,57	0,67
Chordo41	CS21 - CHS	S 355	NLC8	0,000	0,63	0,63	0,58

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG5 - Upward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,23	0,08	0,23
Cv7	CS22 - CHS	S 355	NLC9	0,000	0,33	0,33	0,21
Chordi65	CS15 - CHS	S 355	NLC9	4,825	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,46	0,18	0,46
Diag11	CS17 - CHS	S 355	NLC9	12,899	0,34	0,21	0,34
X22	CS18 - CHS	S 355	NLC9	12,401	0,55	0,55	0,29
Y165	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,37
Chordl18	CS20 - CHS	S 355	NLC9	0,000	0,23	0,19	0,23
Chordo26	CS21 - CHS	S 355	NLC9	9,696	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG7 - Snow Load i BG10 - Superimposed Loads	1,00 1,00 1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv3	CS14 - CHS	S 355	NLC10i	0,000	0,75	0,30	0,75
Cv7	CS22 - CHS	S 355	NLC10i	0,000	0,40	0,33	0,40
Chordi26	CS15 - CHS	S 355	NLC10i	11,130	0,47	0,47	0,47
Vert140	CS16 - CHS	S 355	NLC10i	12,728	0,58	0,23	0,58
Diag195	CS17 - CHS	S 355	NLC10i	0,000	0,38	0,25	0,38
X123	CS18 - CHS	S 355	NLC10i	14,553	0,41	0,41	0,33
Y264	CS19 - CHS	S 355	NLC10i	11,917	0,38	0,38	0,29
Chordl64	CS20 - CHS	S 355	NLC10i	0,000	0,40	0,36	0,40
Chordo35	CS21 - CHS	S 355	NLC10i	4,851	0,40	0,40	0,38

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG8 - Snow Load ii BG10 - Superimposed Loads	1,00 1,00 1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,48	0,09	0,48
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,60	0,25	0,60
Cv7	CS22 - CHS	S 355	NLC10ii	0,000	0,45	0,34	0,45
Chordi26	CS15 - CHS	S 355	NLC10ii	11,130	0,58	0,58	0,58
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,54	0,36	0,54
X123	CS18 - CHS	S 355	NLC10ii	14,553	0,43	0,43	0,34
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,40	0,40	0,35
Chordl30	CS20 - CHS	S 355	NLC10ii	0,000	0,50	0,46	0,50
Chordo32	CS21 - CHS	S 355	NLC10ii	12,606	0,41	0,41	0,37

2. Kritische belastingcoëfficiënten

N	f
	[]
Stabiliteitcombinatie : S1	
1	2,00
2	2,23
Stabiliteitcombinatie : S2	
1	1,00
2	1,09
Stabiliteitcombinatie : S3	
1	1,00
2	1,12
Stabiliteitcombinatie : S4	
1	12,00
2	13,92
Stabiliteitcombinatie : S5	
1	1,00
2	1,12
Stabiliteitcombinatie : S6	
1	2,00
2	2,04
Stabiliteitcombinatie : S7	
1	4,00
2	4,46
Stabiliteitcombinatie : S8	
1	2,00
2	2,19
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	23,00
2	27,63
Stabiliteitcombinatie : S11	
1	2,00
2	2,23
Stabiliteitcombinatie : S12	
1	3,00
2	3,25

G.5. Lower chord

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC6	0,000	0,28	0,05	0,28
Cv1	CS14 - CHS	S 355	NLC6	0,000	0,58	0,22	0,58
Cv5	CS22 - CHS	S 355	NLC6	0,000	0,30	0,24	0,30
Chordi11	CS15 - CHS	S 355	NLC6	0,000	0,33	0,33	0,33
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,44	0,17	0,44
Diag204	CS17 - CHS	S 355	NLC6	0,000	0,27	0,18	0,27
X54	CS18 - CHS	S 355	NLC6	0,000	0,21	0,21	0,17
Y1	CS19 - CHS	S 355	NLC6	1,508	0,20	0,20	0,18
Chordl30	CS20 - CHS	S 355	NLC6	0,000	0,30	0,27	0,30
Chordo1	CS21 - CHS	S 355	NLC6	4,851	0,22	0,22	0,20

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC7	0,000	0,29	0,06	0,29
Cv1	CS14 - CHS	S 355	NLC7	0,000	0,80	0,31	0,80
Cv5	CS22 - CHS	S 355	NLC7	0,000	0,27	0,24	0,27
Chordi11	CS15 - CHS	S 355	NLC7	0,000	0,42	0,42	0,42
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,66	0,26	0,66
Diag204	CS17 - CHS	S 355	NLC7	0,000	0,38	0,27	0,38
X208	CS18 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Y155	CS19 - CHS	S 355	NLC7	11,818	0,44	0,44	0,36
Chordl17	CS20 - CHS	S 355	NLC7	11,127	0,32	0,24	0,32
Chordo1	CS21 - CHS	S 355	NLC7	4,851	0,33	0,33	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC8	0,000	0,60	0,11	0,60
Cv1	CS14 - CHS	S 355	NLC8	0,000	1,19	0,36	1,19
Cv5	CS22 - CHS	S 355	NLC8	0,000	0,86	0,76	0,86
Chordi11	CS15 - CHS	S 355	NLC8	0,000	0,78	0,78	0,78
Vert29	CS16 - CHS	S 355	NLC8	0,000	0,63	0,44	0,63
Diag93	CS17 - CHS	S 355	NLC8	0,000	0,61	0,41	0,61
X125	CS18 - CHS	S 355	NLC8	0,000	0,52	0,15	0,52
Y1	CS19 - CHS	S 355	NLC8	1,508	0,63	0,63	0,57
Chordl30	CS20 - CHS	S 355	NLC8	0,000	0,66	0,56	0,66
Chordo41	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,53

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG5 - Upward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv9	CS22 - CHS	S 355	NLC9	0,000	0,32	0,29	0,32
Chordi38	CS15 - CHS	S 355	NLC9	7,238	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,45	0,18	0,45
Diag11	CS17 - CHS	S 355	NLC9	12,899	0,35	0,22	0,35
X22	CS18 - CHS	S 355	NLC9	12,401	0,55	0,55	0,29
Y111	CS19 - CHS	S 355	NLC9	0,000	0,49	0,49	0,39
Chordl18	CS20 - CHS	S 355	NLC9	11,127	0,22	0,18	0,22
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG7 - Snow Load i BG10 - Superimposed Loads	1,00 1,00 1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv1	CS14 - CHS	S 355	NLC10i	0,000	0,90	0,33	0,90
Cv5	CS22 - CHS	S 355	NLC10i	0,000	0,44	0,38	0,44
Chordi11	CS15 - CHS	S 355	NLC10i	0,000	0,52	0,52	0,52
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,60	0,24	0,60
Diag204	CS17 - CHS	S 355	NLC10i	0,000	0,40	0,28	0,40
X54	CS18 - CHS	S 355	NLC10i	0,000	0,34	0,34	0,28
Y1	CS19 - CHS	S 355	NLC10i	1,508	0,35	0,35	0,31
Chordl30	CS20 - CHS	S 355	NLC10i	0,000	0,39	0,36	0,39
Chordo1	CS21 - CHS	S 355	NLC10i	4,851	0,35	0,35	0,31

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG8 - Snow Load ii BG10 - Superimposed Loads	1,00 1,00 1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC10ii	0,000	0,38	0,08	0,38
Cv1	CS14 - CHS	S 355	NLC10ii	0,000	0,77	0,28	0,77
Cv5	CS22 - CHS	S 355	NLC10ii	0,000	0,45	0,42	0,45
Chordi11	CS15 - CHS	S 355	NLC10ii	0,000	0,58	0,58	0,58
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,62	0,25	0,62
Diag93	CS17 - CHS	S 355	NLC10ii	0,000	0,54	0,36	0,54
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,46	0,46	0,44
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,44	0,44	0,39
Chordl30	CS20 - CHS	S 355	NLC10ii	0,000	0,52	0,48	0,52
Chordo3	CS21 - CHS	S 355	NLC10ii	0,000	0,41	0,41	0,37

2. Kritische belastingcoëfficiënten

N	f
Stabiliteitcombinatie : S1	
1	2,00
2	2,25
Stabiliteitcombinatie : S2	
1	1,00
2	1,13
Stabiliteitcombinatie : S3	
1	1,00
2	1,12
Stabiliteitcombinatie : S4	
1	16,00
2	16,39
Stabiliteitcombinatie : S5	
1	1,00
2	1,13
Stabiliteitcombinatie : S6	
1	1,00
2	1,12
Stabiliteitcombinatie : S7	
1	3,00
2	3,37
Stabiliteitcombinatie : S8	
1	2,00
2	2,25
Stabiliteitcombinatie : S9	
1	1,00
2	1,13
Stabiliteitcombinatie : S10	
1	31,00
2	31,49
Stabiliteitcombinatie : S11	
1	2,00
2	2,25
Stabiliteitcombinatie : S12	
1	2,00
2	2,25

G.6. Vertical

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC6	0,000	0,28	0,05	0,28
Cv2	CS14 - CHS	S 355	NLC6	0,000	0,55	0,15	0,55
Cv14	CS22 - CHS	S 355	NLC6	0,000	0,59	0,14	0,59
Chordi9	CS15 - CHS	S 355	NLC6	0,000	0,29	0,29	0,29
Vert140	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag12	CS17 - CHS	S 355	NLC6	0,000	0,27	0,17	0,27
X143	CS18 - CHS	S 355	NLC6	1,508	0,19	0,19	0,17
Y1	CS19 - CHS	S 355	NLC6	1,508	0,18	0,18	0,16
Chordl30	CS20 - CHS	S 355	NLC6	0,000	0,30	0,27	0,30
Chordo35	CS21 - CHS	S 355	NLC6	4,851	0,22	0,22	0,20

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC7	0,000	0,30	0,06	0,30
Cv2	CS14 - CHS	S 355	NLC7	21,706	0,89	0,23	0,89
Cv14	CS22 - CHS	S 355	NLC7	0,000	0,99	0,22	0,99
Chordi11	CS15 - CHS	S 355	NLC7	0,000	0,38	0,38	0,38
Vert140	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag3	CS17 - CHS	S 355	NLC7	0,000	0,32	0,22	0,32
X76	CS18 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Y94	CS19 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Chordl18	CS20 - CHS	S 355	NLC7	0,000	0,46	0,37	0,46
Chordo35	CS21 - CHS	S 355	NLC7	4,851	0,33	0,33	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC8	0,000	0,60	0,11	0,60
Cv2	CS14 - CHS	S 355	NLC8	24,118	0,82	0,20	0,82
Cv14	CS22 - CHS	S 355	NLC8	9,000	0,82	0,19	0,82
Chordi13	CS15 - CHS	S 355	NLC8	0,000	0,63	0,63	0,63
Vert121	CS16 - CHS	S 355	NLC8	0,000	0,61	0,43	0,61
Diag12	CS17 - CHS	S 355	NLC8	0,000	0,61	0,41	0,61
X143	CS18 - CHS	S 355	NLC8	1,508	0,51	0,51	0,46
Y1	CS19 - CHS	S 355	NLC8	1,508	0,59	0,59	0,53
Chordl30	CS20 - CHS	S 355	NLC8	0,000	0,66	0,56	0,66
Chordo7	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,53

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG5 - Upward Wind	1,00
		BG10 - Superimposed Loads	1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	41,000	0,32	0,09	0,32
Cv14	CS22 - CHS	S 355	NLC9	0,000	0,32	0,10	0,32
Chordi38	CS15 - CHS	S 355	NLC9	7,238	0,50	0,38	0,50
Vert29	CS16 - CHS	S 355	NLC9	9,000	0,42	0,42	0,00
Diag11	CS17 - CHS	S 355	NLC9	12,899	0,34	0,21	0,34
X13	CS18 - CHS	S 355	NLC9	0,000	0,55	0,55	0,29
Y111	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,38
Chordl18	CS20 - CHS	S 355	NLC9	0,000	0,34	0,25	0,34
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG7 - Snow Load i	1,00
		BG10 - Superimposed Loads	1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC10i	0,000	0,39	0,07	0,39
Cv2	CS14 - CHS	S 355	NLC10i	0,000	0,83	0,23	0,83
Cv14	CS22 - CHS	S 355	NLC10i	0,000	0,92	0,22	0,92
Chordi9	CS15 - CHS	S 355	NLC10i	0,000	0,46	0,46	0,46
Vert140	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,23	0,59
Diag12	CS17 - CHS	S 355	NLC10i	0,000	0,38	0,25	0,38
X143	CS18 - CHS	S 355	NLC10i	1,508	0,32	0,32	0,28
Y1	CS19 - CHS	S 355	NLC10i	1,508	0,32	0,32	0,29
Chordl18	CS20 - CHS	S 355	NLC10i	0,000	0,41	0,33	0,41
Chordo35	CS21 - CHS	S 355	NLC10i	4,851	0,35	0,35	0,32

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG8 - Snow Load ii	1,00
		BG10 - Superimposed Loads	1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,48	0,09	0,48
Cv2	CS14 - CHS	S 355	NLC10ii	16,882	0,78	0,21	0,78
Cv14	CS22 - CHS	S 355	NLC10ii	0,000	0,83	0,20	0,83
Chordi9	CS15 - CHS	S 355	NLC10ii	0,000	0,58	0,58	0,58
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,55	0,37	0,55
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,36	0,36	0,34
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,41	0,41	0,36
Chordl30	CS20 - CHS	S 355	NLC10ii	0,000	0,52	0,48	0,52
Chordo3	CS21 - CHS	S 355	NLC10ii	0,000	0,40	0,40	0,36

2. Kritische belastingcoëfficiënten

N	f
	[]
Stabiliteitcombinatie : S1	
1	3,00
2	3,00
Stabiliteitcombinatie : S2	
1	2,00
2	2,00
Stabiliteitcombinatie : S3	
1	1,00
2	1,13
Stabiliteitcombinatie : S4	
1	16,00
2	16,39
Stabiliteitcombinatie : S5	
1	2,00
2	2,00
Stabiliteitcombinatie : S6	
1	2,00
2	2,06
Stabiliteitcombinatie : S7	
1	4,00
2	4,01
Stabiliteitcombinatie : S8	
1	2,00
2	2,00
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	31,00
2	31,43
Stabiliteitcombinatie : S11	
1	3,00
2	3,00
Stabiliteitcombinatie : S12	
1	3,00
2	3,29

G.7. Diagonal

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC6	0,000	0,27	0,05	0,27
Cv1	CS14 - CHS	S 355	NLC6	0,000	0,46	0,20	0,46
Cv5	CS22 - CHS	S 355	NLC6	0,000	0,27	0,23	0,27
Chordi11	CS15 - CHS	S 355	NLC6	0,000	0,31	0,31	0,31
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag3	CS17 - CHS	S 355	NLC6	0,000	0,27	0,19	0,27
X54	CS18 - CHS	S 355	NLC6	0,000	0,25	0,25	0,21
Y31	CS19 - CHS	S 355	NLC6	0,000	0,24	0,24	0,18
Chordl5	CS20 - CHS	S 355	NLC6	1,534	0,38	0,30	0,38
Chordo68	CS21 - CHS	S 355	NLC6	7,277	0,22	0,22	0,20

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC7	0,000	0,29	0,05	0,29
Cv1	CS14 - CHS	S 355	NLC7	0,000	0,69	0,29	0,69
Cv5	CS22 - CHS	S 355	NLC7	0,000	0,26	0,23	0,26
Chordi11	CS15 - CHS	S 355	NLC7	0,000	0,41	0,41	0,41
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag3	CS17 - CHS	S 355	NLC7	0,000	0,40	0,28	0,40
X208	CS18 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Y148	CS19 - CHS	S 355	NLC7	0,000	0,49	0,49	0,40
Chordl5	CS20 - CHS	S 355	NLC7	1,534	0,37	0,31	0,37
Chordo68	CS21 - CHS	S 355	NLC7	7,277	0,33	0,33	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC8	0,000	0,60	0,11	0,60
Cv1	CS14 - CHS	S 355	NLC8	0,000	0,74	0,29	0,74
Cv5	CS22 - CHS	S 355	NLC8	0,000	0,78	0,70	0,78
Chordi11	CS15 - CHS	S 355	NLC8	9,054	0,66	0,66	0,66
Vert14	CS16 - CHS	S 355	NLC8	0,000	0,83	0,58	0,83
Diag114	CS17 - CHS	S 355	NLC8	0,000	0,61	0,41	0,61
X54	CS18 - CHS	S 355	NLC8	0,000	0,61	0,53	0,61
Y179	CS19 - CHS	S 355	NLC8	1,508	0,61	0,61	0,54
Chordl5	CS20 - CHS	S 355	NLC8	1,534	0,82	0,63	0,82
Chordo41	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,53

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG5 - Upward Wind	1,00
		BG10 - Superimposed Loads	1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw5	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,02
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv9	CS22 - CHS	S 355	NLC9	0,000	0,32	0,29	0,32
Chordi38	CS15 - CHS	S 355	NLC9	7,238	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,45	0,18	0,45
Diag11	CS17 - CHS	S 355	NLC9	10,319	0,35	0,22	0,35
X22	CS18 - CHS	S 355	NLC9	12,401	0,55	0,55	0,29
Y111	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,38
Chordl18	CS20 - CHS	S 355	NLC9	11,127	0,22	0,18	0,22
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG7 - Snow Load i	1,00
		BG10 - Superimposed Loads	1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC10i	0,000	0,38	0,07	0,38
Cv1	CS14 - CHS	S 355	NLC10i	0,000	0,71	0,29	0,71
Cv5	CS22 - CHS	S 355	NLC10i	0,000	0,42	0,37	0,42
Chordi11	CS15 - CHS	S 355	NLC10i	0,000	0,51	0,51	0,51
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,24	0,59
Diag3	CS17 - CHS	S 355	NLC10i	0,000	0,41	0,29	0,41
X54	CS18 - CHS	S 355	NLC10i	0,000	0,40	0,40	0,35
Y31	CS19 - CHS	S 355	NLC10i	0,000	0,38	0,38	0,30
Chordl5	CS20 - CHS	S 355	NLC10i	1,534	0,51	0,41	0,51
Chordo68	CS21 - CHS	S 355	NLC10i	6,064	0,35	0,35	0,33

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel	1,00
		BG8 - Snow Load ii	1,00
		BG10 - Superimposed Loads	1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,40	0,06	0,40
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,60	0,25	0,60
Cv17	CS22 - CHS	S 355	NLC10ii	0,000	0,47	0,46	0,47
Chordi9	CS15 - CHS	S 355	NLC10ii	0,000	0,60	0,60	0,60
Vert14	CS16 - CHS	S 355	NLC10ii	0,000	0,77	0,54	0,77
Diag93	CS17 - CHS	S 355	NLC10ii	0,000	0,54	0,36	0,54
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,56	0,56	0,55
Y179	CS19 - CHS	S 355	NLC10ii	1,508	0,42	0,42	0,37
Chordl5	CS20 - CHS	S 355	NLC10ii	1,534	0,68	0,55	0,68
Chordo32	CS21 - CHS	S 355	NLC10ii	12,606	0,39	0,39	0,36

2. Kritische belastingcoëfficiënten

N	f
	[]
Stabiliteitcombinatie : S1	
1	2,00
2	2,23
Stabiliteitcombinatie : S2	
1	1,00
2	1,09
Stabiliteitcombinatie : S3	
1	1,00
2	1,12
Stabiliteitcombinatie : S4	
1	16,00
2	16,40
Stabiliteitcombinatie : S5	
1	1,00
2	1,12
Stabiliteitcombinatie : S6	
1	1,00
2	1,02
Stabiliteitcombinatie : S7	
1	4,00
2	4,46
Stabiliteitcombinatie : S8	
1	2,00
2	2,19
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	31,00
2	31,43
Stabiliteitcombinatie : S11	
1	2,00
2	2,23
Stabiliteitcombinatie : S12	
1	3,00
2	3,18

G.8. Vertical column

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC6	0,000	0,40	0,09	0,40
Cv4	CS14 - CHS	S 355	NLC6	0,000	0,40	0,17	0,40
Cv17	CS22 - CHS	S 355	NLC6	0,000	0,26	0,17	0,26
Chordi4	CS15 - CHS	S 355	NLC6	12,063	0,40	0,40	0,40
Vert48	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag12	CS17 - CHS	S 355	NLC6	0,000	0,44	0,29	0,44
X54	CS18 - CHS	S 355	NLC6	0,000	0,25	0,25	0,22
Y1	CS19 - CHS	S 355	NLC6	1,508	0,34	0,34	0,30
Chordl64	CS20 - CHS	S 355	NLC6	0,000	0,60	0,48	0,60
Chordo34	CS21 - CHS	S 355	NLC6	7,277	0,21	0,21	0,19

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC7	0,000	0,51	0,11	0,51
Cv4	CS14 - CHS	S 355	NLC7	0,000	0,62	0,26	0,62
Cv17	CS22 - CHS	S 355	NLC7	0,000	0,34	0,22	0,34
Chordi4	CS15 - CHS	S 355	NLC7	12,063	0,55	0,55	0,55
Vert48	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag12	CS17 - CHS	S 355	NLC7	0,000	0,56	0,37	0,56
X76	CS18 - CHS	S 355	NLC7	0,000	0,53	0,53	0,44
Y148	CS19 - CHS	S 355	NLC7	0,000	0,56	0,56	0,46
Chordl64	CS20 - CHS	S 355	NLC7	0,000	0,71	0,63	0,71
Chordo17	CS21 - CHS	S 355	NLC7	4,851	0,32	0,32	0,30

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaf	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC8	0,000	0,83	0,16	0,83
Cv2	CS14 - CHS	S 355	NLC8	0,000	0,62	0,22	0,62
Cv7	CS22 - CHS	S 355	NLC8	0,000	0,68	0,58	0,68
Chordi4	CS15 - CHS	S 355	NLC8	12,063	0,88	0,88	0,88
Vert68	CS16 - CHS	S 355	NLC8	0,000	0,61	0,43	0,61
Diag195	CS17 - CHS	S 355	NLC8	0,000	0,88	0,59	0,88
X53	CS18 - CHS	S 355	NLC8	0,934	0,50	0,50	0,38
Y1	CS19 - CHS	S 355	NLC8	1,508	0,80	0,80	0,72
Chordl5	CS20 - CHS	S 355	NLC8	1,534	1,04	1,04	1,00
Chordo7	CS21 - CHS	S 355	NLC8	4,935	0,55	0,55	0,52

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG5 - Upward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv9	CS22 - CHS	S 355	NLC9	0,000	0,29	0,27	0,29
Chordi38	CS15 - CHS	S 355	NLC9	7,238	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,45	0,18	0,45
Diag92	CS17 - CHS	S 355	NLC9	12,899	0,34	0,21	0,34
X13	CS18 - CHS	S 355	NLC9	0,000	0,55	0,55	0,29
Y111	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,38
Chordl18	CS20 - CHS	S 355	NLC9	0,000	0,23	0,19	0,23
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG7 - Snow Load i BG10 - Superimposed Loads	1,00 1,00 1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10i	0,000	0,61	0,13	0,61
Cv4	CS14 - CHS	S 355	NLC10i	0,000	0,63	0,25	0,63
Cv17	CS22 - CHS	S 355	NLC10i	0,000	0,42	0,29	0,42
Chordi4	CS15 - CHS	S 355	NLC10i	12,063	0,65	0,65	0,65
Vert48	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,24	0,59
Diag12	CS17 - CHS	S 355	NLC10i	0,000	0,66	0,44	0,66
X54	CS18 - CHS	S 355	NLC10i	0,000	0,42	0,42	0,33
Y1	CS19 - CHS	S 355	NLC10i	1,508	0,56	0,56	0,49
Chordl64	CS20 - CHS	S 355	NLC10i	0,000	0,81	0,75	0,81
Chordo34	CS21 - CHS	S 355	NLC10i	7,277	0,33	0,33	0,31

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG8 - Snow Load ii BG10 - Superimposed Loads	1,00 1,00 1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,66	0,14	0,66
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,62	0,25	0,62
Cv17	CS22 - CHS	S 355	NLC10ii	0,000	0,44	0,32	0,44
Chordi9	CS15 - CHS	S 355	NLC10ii	0,000	0,59	0,59	0,59
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,75	0,50	0,75
X54	CS18 - CHS	S 355	NLC10ii	0,000	0,47	0,47	0,42
Y1	CS19 - CHS	S 355	NLC10ii	1,508	0,57	0,57	0,50
Chordl5	CS20 - CHS	S 355	NLC10ii	1,534	0,81	0,78	0,81
Chordo32	CS21 - CHS	S 355	NLC10ii	12,606	0,39	0,39	0,35

2. Kritische belastingcoëfficiënten

N	f
[]	
Stabiliteitcombinatie : S1	
1	3,00
2	3,00
Stabiliteitcombinatie : S2	
1	2,00
2	2,00
Stabiliteitcombinatie : S3	
1	1,00
2	1,12
Stabiliteitcombinatie : S4	
1	14,00
2	14,11
Stabiliteitcombinatie : S5	
1	2,00
2	2,00
Stabiliteitcombinatie : S6	
1	2,00
2	2,10
Stabiliteitcombinatie : S7	
1	4,00
2	4,01
Stabiliteitcombinatie : S8	
1	2,00
2	2,00
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	28,00
2	28,15
Stabiliteitcombinatie : S11	
1	3,00
2	3,00
Stabiliteitcombinatie : S12	
1	3,00
2	3,35

G.9. Outer angled column

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC6	0,000	0,26	0,05	0,26
Cv4	CS14 - CHS	S 355	NLC6	0,000	0,59	0,24	0,59
Cv12	CS22 - CHS	S 355	NLC6	0,000	0,28	0,23	0,28
Chordi43	CS15 - CHS	S 355	NLC6	0,000	0,41	0,41	0,41
Vert140	CS16 - CHS	S 355	NLC6	12,728	0,57	0,23	0,57
Diag12	CS17 - CHS	S 355	NLC6	0,000	0,28	0,18	0,28
X264	CS18 - CHS	S 355	NLC6	12,979	0,19	0,19	0,17
Y201	CS19 - CHS	S 355	NLC6	13,042	0,20	0,20	0,17
Chordl5	CS20 - CHS	S 355	NLC6	1,534	0,32	0,28	0,32
Chordo1	CS21 - CHS	S 355	NLC6	4,851	0,22	0,22	0,21

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw13	CS13 - CHS	S 355	NLC7	0,000	0,28	0,05	0,28
Cv4	CS14 - CHS	S 355	NLC7	0,000	0,83	0,33	0,83
Cv16	CS22 - CHS	S 355	NLC7	0,000	0,29	0,28	0,29
Chordi43	CS15 - CHS	S 355	NLC7	0,000	0,50	0,50	0,50
Vert140	CS16 - CHS	S 355	NLC7	12,728	0,80	0,32	0,80
Diag105	CS17 - CHS	S 355	NLC7	0,000	0,34	0,23	0,34
X83	CS18 - CHS	S 355	NLC7	11,818	0,45	0,45	0,36
Y94	CS19 - CHS	S 355	NLC7	0,000	0,45	0,45	0,37
Chordl52	CS20 - CHS	S 355	NLC7	0,000	0,39	0,30	0,39
Chordo1	CS21 - CHS	S 355	NLC7	4,851	0,33	0,33	0,31

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw7	CS13 - CHS	S 355	NLC8	0,000	0,63	0,16	0,63
Cv4	CS14 - CHS	S 355	NLC8	0,000	1,31	0,40	1,31
Cv12	CS22 - CHS	S 355	NLC8	0,000	0,79	0,70	0,79
Chordi43	CS15 - CHS	S 355	NLC8	0,000	0,92	0,92	0,92
Vert140	CS16 - CHS	S 355	NLC8	12,728	0,70	0,28	0,70
Diag126	CS17 - CHS	S 355	NLC8	0,000	0,67	0,58	0,67
X48	CS18 - CHS	S 355	NLC8	0,000	0,63	0,63	0,50
Y57	CS19 - CHS	S 355	NLC8	14,399	0,67	0,54	0,67
Chordl43	CS20 - CHS	S 355	NLC8	0,000	0,77	0,51	0,77
Chordo41	CS21 - CHS	S 355	NLC8	4,935	0,61	0,61	0,57

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG5 - Upward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw2	CS13 - CHS	S 355	NLC9	42,932	0,09	0,09	0,01
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,22	0,08	0,22
Cv7	CS22 - CHS	S 355	NLC9	0,000	0,29	0,29	0,18
Chord165	CS15 - CHS	S 355	NLC9	4,825	0,50	0,38	0,50
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,45	0,18	0,45
Diag92	CS17 - CHS	S 355	NLC9	12,899	0,35	0,21	0,35
X22	CS18 - CHS	S 355	NLC9	12,401	0,55	0,55	0,29
Y111	CS19 - CHS	S 355	NLC9	0,000	0,48	0,48	0,37
Chord118	CS20 - CHS	S 355	NLC9	11,127	0,22	0,18	0,22
Chordo26	CS21 - CHS	S 355	NLC9	12,120	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG7 - Snow Load i BG10 - Superimposed Loads	1,00 1,00 1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10i	0,000	0,37	0,07	0,37
Cv4	CS14 - CHS	S 355	NLC10i	0,000	0,98	0,36	0,98
Cv12	CS22 - CHS	S 355	NLC10i	0,000	0,43	0,37	0,43
Chord143	CS15 - CHS	S 355	NLC10i	0,000	0,65	0,65	0,65
Vert140	CS16 - CHS	S 355	NLC10i	12,728	0,79	0,31	0,79
Diag12	CS17 - CHS	S 355	NLC10i	0,000	0,39	0,26	0,39
X152	CS18 - CHS	S 355	NLC10i	0,000	0,32	0,32	0,29
Y179	CS19 - CHS	S 355	NLC10i	1,508	0,33	0,33	0,29
Chord15	CS20 - CHS	S 355	NLC10i	1,534	0,42	0,37	0,42
Chordo1	CS21 - CHS	S 355	NLC10i	4,851	0,35	0,35	0,33

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG8 - Snow Load ii BG10 - Superimposed Loads	1,00 1,00 1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,49	0,10	0,49
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,88	0,34	0,88
Cv12	CS22 - CHS	S 355	NLC10ii	0,000	0,47	0,37	0,47
Chord126	CS15 - CHS	S 355	NLC10ii	11,130	0,58	0,58	0,58
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,81	0,32	0,81
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,56	0,37	0,56
X69	CS18 - CHS	S 355	NLC10ii	14,553	0,37	0,37	0,34
Y179	CS19 - CHS	S 355	NLC10ii	1,508	0,41	0,41	0,36
Chord130	CS20 - CHS	S 355	NLC10ii	0,000	0,53	0,49	0,53
Chordo3	CS21 - CHS	S 355	NLC10ii	0,000	0,40	0,40	0,37

2. Kritische belastingcoëfficiënten

N	f
	[]
Stabiliteitcombinatie : S1	
1	2,00
2	2,25
Stabiliteitcombinatie : S2	
1	1,00
2	1,12
Stabiliteitcombinatie : S3	
1	1,00
2	1,12
Stabiliteitcombinatie : S4	
1	15,00
2	15,71
Stabiliteitcombinatie : S5	
1	1,00
2	1,12
Stabiliteitcombinatie : S6	
1	1,00
2	1,12
Stabiliteitcombinatie : S7	
1	3,00
2	3,37
Stabiliteitcombinatie : S8	
1	2,00
2	2,25
Stabiliteitcombinatie : S9	
1	1,00
2	1,12
Stabiliteitcombinatie : S10	
1	29,00
2	30,23
Stabiliteitcombinatie : S11	
1	2,00
2	2,25
Stabiliteitcombinatie : S12	
1	2,00
2	2,25

G.10. Inner angled column

1. Niet-lineaire combinaties

1.1. Niet-lineaire combinaties - NLC6

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC6	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG10 - Superimposed Loads	1,00 1,00

1.1.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC6

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw11	CS13 - CHS	S 355	NLC6	0,000	0,30	0,06	0,30
Cv1	CS14 - CHS	S 355	NLC6	0,000	0,43	0,18	0,43
Cv10	CS22 - CHS	S 355	NLC6	0,000	0,19	0,18	0,19
Chord19	CS15 - CHS	S 355	NLC6	0,000	0,31	0,31	0,31
Vert140	CS16 - CHS	S 355	NLC6	12,728	0,43	0,17	0,43
Diag114	CS17 - CHS	S 355	NLC6	0,000	0,32	0,21	0,32
X1	CS18 - CHS	S 355	NLC6	1,508	0,20	0,20	0,17
Y188	CS19 - CHS	S 355	NLC6	0,000	0,23	0,23	0,21
Chord139	CS20 - CHS	S 355	NLC6	1,534	0,38	0,33	0,38
Chordo34	CS21 - CHS	S 355	NLC6	7,277	0,23	0,23	0,21

1.2. Niet-lineaire combinaties - NLC7

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC7	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG3 - Imposed Loads BG10 - Superimposed Loads	1,00 1,00 1,00

1.2.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC7

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC7	0,000	0,30	0,06	0,30
Cv1	CS14 - CHS	S 355	NLC7	0,000	0,66	0,27	0,66
Cv14	CS22 - CHS	S 355	NLC7	0,000	0,21	0,20	0,21
Chord19	CS15 - CHS	S 355	NLC7	0,000	0,38	0,38	0,38
Vert140	CS16 - CHS	S 355	NLC7	12,728	0,65	0,26	0,65
Diag3	CS17 - CHS	S 355	NLC7	0,000	0,33	0,23	0,33
X208	CS18 - CHS	S 355	NLC7	0,000	0,44	0,44	0,36
Y155	CS19 - CHS	S 355	NLC7	11,818	0,44	0,44	0,36
Chord139	CS20 - CHS	S 355	NLC7	1,534	0,35	0,32	0,35
Chordo34	CS21 - CHS	S 355	NLC7	7,277	0,34	0,34	0,32

1.3. Niet-lineaire combinaties - NLC8

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC8	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG6 - Downward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.3.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC8

Staaft	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw11	CS13 - CHS	S 355	NLC8	0,000	0,76	0,15	0,76
Cv1	CS14 - CHS	S 355	NLC8	0,000	0,70	0,26	0,70
Cv10	CS22 - CHS	S 355	NLC8	0,000	0,56	0,52	0,56
Chord138	CS15 - CHS	S 355	NLC8	12,063	0,86	0,86	0,86
Vert68	CS16 - CHS	S 355	NLC8	0,000	0,59	0,42	0,59
Diag114	CS17 - CHS	S 355	NLC8	0,000	0,82	0,55	0,82
X1	CS18 - CHS	S 355	NLC8	1,508	0,55	0,55	0,49
Y188	CS19 - CHS	S 355	NLC8	0,000	0,73	0,73	0,65
Chord139	CS20 - CHS	S 355	NLC8	1,534	0,89	0,86	0,89
Chordo28	CS21 - CHS	S 355	NLC8	4,935	0,56	0,56	0,54

1.4. Niet-lineaire combinaties - NLC9

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC9	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG5 - Upward Wind BG10 - Superimposed Loads	1,00 1,00 1,00

1.4.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC9

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw15	CS13 - CHS	S 355	NLC9	46,162	0,12	0,12	0,10
Cv2	CS14 - CHS	S 355	NLC9	0,000	0,26	0,09	0,26
Cv11	CS22 - CHS	S 355	NLC9	0,000	0,32	0,26	0,32
Chord138	CS15 - CHS	S 355	NLC9	7,238	0,69	0,53	0,69
Vert48	CS16 - CHS	S 355	NLC9	12,728	0,48	0,19	0,48
Diag92	CS17 - CHS	S 355	NLC9	12,899	0,34	0,21	0,34
X22	CS18 - CHS	S 355	NLC9	12,401	0,56	0,56	0,30
Y165	CS19 - CHS	S 355	NLC9	0,000	0,49	0,49	0,40
Chord139	CS20 - CHS	S 355	NLC9	1,534	0,26	0,26	0,19
Chordo26	CS21 - CHS	S 355	NLC9	9,696	0,50	0,43	0,50

1.5. Niet-lineaire combinaties - NLC10i

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10i	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG7 - Snow Load i BG10 - Superimposed Loads	1,00 1,00 1,00

1.5.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10i

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw11	CS13 - CHS	S 355	NLC10i	0,000	0,42	0,09	0,42
Cv1	CS14 - CHS	S 355	NLC10i	0,000	0,67	0,27	0,67
Cv19	CS22 - CHS	S 355	NLC10i	0,000	0,30	0,24	0,30
Chord19	CS15 - CHS	S 355	NLC10i	0,000	0,49	0,49	0,49
Vert140	CS16 - CHS	S 355	NLC10i	12,728	0,59	0,24	0,59
Diag114	CS17 - CHS	S 355	NLC10i	0,000	0,47	0,31	0,47
X1	CS18 - CHS	S 355	NLC10i	1,508	0,33	0,33	0,29
Y188	CS19 - CHS	S 355	NLC10i	0,000	0,38	0,38	0,36
Chord139	CS20 - CHS	S 355	NLC10i	1,534	0,48	0,46	0,48
Chordo34	CS21 - CHS	S 355	NLC10i	7,277	0,37	0,37	0,34

1.6. Niet-lineaire combinaties - NLC10ii

Naam	Type	Belastingsgevallen	Coëff. [-]
NLC10ii	Bruikbaarheidsgrenstoestand	BG2 - Self-Weight steel BG8 - Snow Load ii BG10 - Superimposed Loads	1,00 1,00 1,00

1.6.1. Staalcontrole

Niet-lineaire berekening, Extreem : Doorsnede

Selectie : Alle

Niet-lineaire combinaties : NLC10ii

Staal	css	mat	BG	dx [m]	Algehele toetsing [-]	Doorsnedetoetsing [-]	Stabiliteittoetsing [-]
Cw9	CS13 - CHS	S 355	NLC10ii	0,000	0,46	0,09	0,46
Cv4	CS14 - CHS	S 355	NLC10ii	0,000	0,60	0,24	0,60
Cv6	CS22 - CHS	S 355	NLC10ii	0,000	0,33	0,33	0,31
Chord19	CS15 - CHS	S 355	NLC10ii	0,000	0,61	0,61	0,61
Vert140	CS16 - CHS	S 355	NLC10ii	12,728	0,63	0,25	0,63
Diag12	CS17 - CHS	S 355	NLC10ii	0,000	0,53	0,35	0,53
X123	CS18 - CHS	S 355	NLC10ii	14,553	0,37	0,37	0,30
Y188	CS19 - CHS	S 355	NLC10ii	0,000	0,41	0,41	0,38
Chord139	CS20 - CHS	S 355	NLC10ii	1,534	0,52	0,50	0,52
Chordo32	CS21 - CHS	S 355	NLC10ii	12,606	0,42	0,42	0,38

2. Kritische belastingcoëfficiënten

N	f
Stabiliteitcombinatie : S1	
1	2,00
2	2,12
Stabiliteitcombinatie : S2	
1	1,00
2	1,02
Stabiliteitcombinatie : S3	
1	1,00
2	1,12
Stabiliteitcombinatie : S4	
1	14,00
2	14,48
Stabiliteitcombinatie : S5	
1	2,00
2	2,14
Stabiliteitcombinatie : S6	
1	1,00
2	1,03
Stabiliteitcombinatie : S7	
1	4,00
2	4,25
Stabiliteitcombinatie : S8	
1	2,00
2	2,05
Stabiliteitcombinatie : S9	
1	1,00
2	1,11
Stabiliteitcombinatie : S10	
1	28,00
2	29,36
Stabiliteitcombinatie : S11	
1	2,00
2	2,12
Stabiliteitcombinatie : S12	
1	2,00
2	2,04