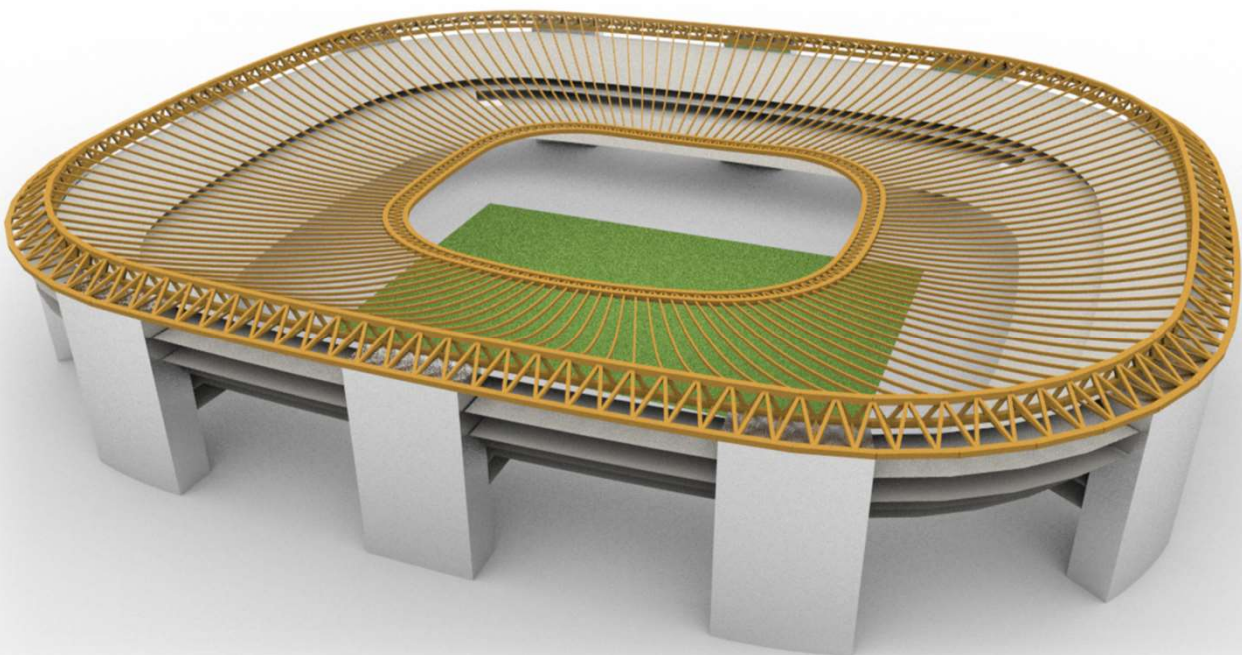


Long-span Timber Roof Structure

for The New Feyenoord stadium

Master Thesis
Structural Engineering

L.C. Bauer



Long-span Timber Roof Structure

for the New Feyenoord stadium

by

L.C. Bauer

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Student number: 4171144
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Thesis committee: Prof. dr. ir. J. W. G. van de Kuilen, TU Delft
Dr. ir. K. C. Terwel, TU Delft
Ir. P. A. de Vries, TU Delft
Ir. J. Arts, Royal HaskoningDHV

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Abstract

Timber structures are experiencing a revelation in the build environment due to new technologies and production techniques. However, the longest spans created with timber structures still don't compete with the span widths of structures made of steel. This thesis provides a preliminary design for the long-span football stadium roof structure of The New Feyenoord stadium, which extends far beyond the existing maximum spans of timber structures. The aim is to provide insight in the feasibility of a long-span structure in timber.

New Feyenoord stadium

The city of Rotterdam is planning to built a new football stadium for the football club Feyenoord with a magnificent roof structure. This stadium is used to determine initial boundary conditions for the preliminary design in this thesis. A perfect bowl with an elegant flat, almost floating, roof that allows unobstructed viewing is desired. The playing field needs to be exposed to the weather conditions for the natural quality of the grass. The stadium will consist of three tiers with a total height of 40 metres, that need to be covered by a roof structure with a width of 205 metres, and length of 245 metres.

Structural timber for a grand roof

Timber has a high strength-to-weight ratio that is beneficial for long-span structures in which the self-weight of the structure is a significant part of the load. It has natural durability, a good performance in fire conditions, ease of workability, and a high ratio of prefabrication. The latter two result in an ease of construction and less human induced errors. Engineered wood products make it possible to create complex timber structures with more reliability than wood in its natural shape. The most promising are laminated veneer lumber (LVL) and glued laminated timber (Glulam), where LVL made of beech hardwood has the highest strength properties. Furthermore, new types of connections diminish the impact that connections traditionally have on the structural performance of timber structures. The enormous size of this stadium means that multiple connections are necessary due to dimensional restrictions for transport. Which reduces the amount of prefabrication.

Structural systems for long-span stadium roofs

Promising configurations for a long-span stadium roof structure are the single span, stress ribbon, shell roof system, and tension / compression ring. First, several existing stadium roof structures are presented to gain better insight on the possibilities of these systems. These stadiums are De Kuip, preliminary design of The New Feyenoord stadium by RHDHV, the Allianz Riviera stadium, the Wanda Metropolitano stadium, the Estadio Municipal, and the Tokyo National stadium. Initial designs are made for each structural system applicable to the case of the New Feyenoord stadium. These initial designs are explored on their structural behaviour and benefits for the case of the New Feyenoord stadium in combination with the material behaviour of timber.

Structural forms for long-span timber structures

Several structural forms are applied in long-span timber structures showing their applicability. These forms are the arch, box girder, truss, shell structure, space frame, and stress ribbon. This is explored by means of respective reference projects. These projects are the Multi-use Arena in Lisbon, Trade Fair Hall 11 in Frankfurt, the Anaklia-Ganmukhuri bridge in the Georgian Republic, the Maicasagi bridge in Nord-du-Québec, the geodesic domes in Brindisi, the Elephant House in Zurich, the Allianz-Riviera stadium in Nice, the Grandview Heights Aquatics Centre in Surrey, and the Essing bridge in Essing. These existing long-span timber structures mainly span only half of the required length of the New Feyenoord stadium.

Possibility to create an efficient structural design for a long-span stadium timber roof structure

After a rough assessment of the design concepts it is found that a combination between a tension / compression ring structure and a stress ribbon configuration shows the greatest potential for an efficient roof structure for the New Feyenoord stadium. There are still many uncertainties for this type of structure for such a long span. There is no reference project and thus there can not be learned from mistakes. Also, the system is mathematically very complex because of its non-linear behaviour. Consequently, the structural system is modelled and verified in the parametric FEM environment of Grasshopper and Karamba3D. The found structural behaviour is verified by the strength verifications for timber structures.

Tension / compression ring with radial stress ribbons

The design consists of a triangular truss for the outer ring and a flat truss for the inner ring with radial stress ribbons in between, as seen in figure 1. The structural design shows a feasible solution for the highest downward load, namely loads instigated by wind. The strength verification is performed on the maximum occurring force combination within a element group. Only the strength verification of shear stiffness in the bottom chord at the inside perimeter of the outer truss ring is not met. Which is a very localised effect and hence can be strengthened. Furthermore, the element groups of the rings show a undesirable utilisation distribution due to the non-circumferential perimeter of The New Feyenoord stadium. Large cross sections are required to provide stiffness to the entire structural system. The ribbons make use of their inherent bending stiffness to carry the loads in the long and short straight sides of the stadium. An elegant and stiff tensile force flow is found for the ribbons in the corners. At last, the designed connections for the ribbons which are attached to the timber fulfil the strength verifications. These connections consist of self-drilling dowels with slotted in steel plates, self-drilling screws, and glued in steel rods. Many steel fasteners are used to increase the efficiency factor of the joint. Suggested connections for the complex nodes in the ring trusses consist of slotted in steel plates with bolts and dowels, glued-in rods, and cast steel parts. The structural system is supported on roller bearings.

The presented structural design only takes downward loading into account. Potential solutions for the severe upward loading are increasing the weight of the structural elements in the loaded area, tensile ties in the long and short side of the stadium, or tensile cables attached to the bottom side of the ribbons in the long and short side. An more in depth analysis of its stability against uplift forces and asymmetrical loading is needed to verify the proposed solutions. Recommendations are made to improve the design for The New Feyenoord stadium, for general possibilities for the chosen structural system, and for improvements and advice on the feasibility of special timber structures.

It is concluded that a timber long-span stadium roof structure consisting of the chosen structural system shows potential to be a feasible solution for the New Feyenoord stadium. It will be a grand architectural statement that makes a stadium iconic, being the only timber tension / compression ring stress ribbon roof structure spanning with an exceptional distance.

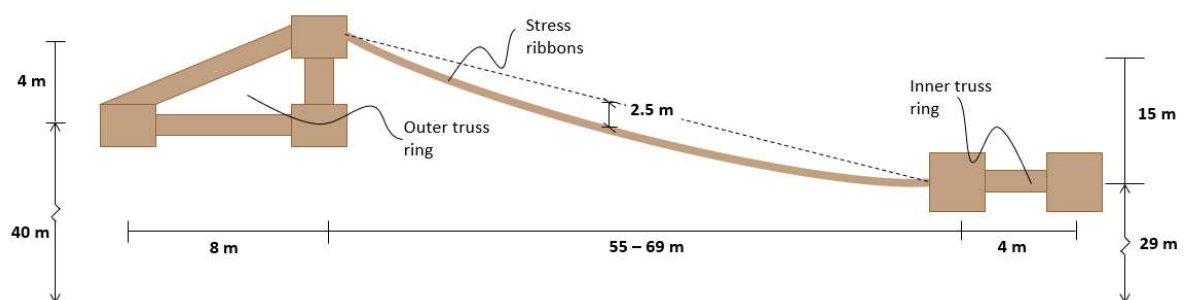


Figure 1: Side view of the structural system

Preface

Before you lies my master thesis report, which is the final part of my Master of Science in Structural engineering of the faculty of Civil Engineering at Delft University of Technology. This graduation thesis is a collaboration with Royal HaskoningDHV. The thesis describes the research I carried out to explore the possibilities of using structural timber in a long-span stadium roof structure for The New Feyenoord stadium. Prof. Dr. Ir. van de Kuilen, Dr. Ir. Terwel, and Ir. de Vries of Delft University of Technology supervised the project.

Throughout this project, I have combined several of my main interest in Structural Engineering to design a special timber structure. The topic of long-span timber roof structures is still novel, but is getting more attention in the build environment. I believe that this combination of a architectural, sustainable, and design engineering perspective fits the final chapter to my career at the TU Delft.

I want to express my gratitude to every one involved in supporting me. Firstly, Janko Arts from Royal HaskoningDHV for his commitment and dedication in guiding me during the process, his practical approach, discussing my findings, and positive feedback. Furthermore, I would like to thank Karel Terwel for both his academic and practical approach, being critical, knowledge on special structures, and his guidance. Next, I would like to thank Peter de Vries for his knowledge on timber structures, critical comments, willingness to discuss several concepts and answering my questions. Lastly, I would like to thank Jan-Willem van de Kuilen for his expertise regarding timber structures and role as chairman of the graduation committee.

I would also like to thank my colleagues at Royal HaskoningDHV for the time and effort they took to answer my questions and the pleasant working environment.

As a child of a civil engineering father and a architectural mother, my upcoming is perfectly expressed in the design for this thesis. I would like to express my gratitude to my father for being extremely supportive, critical, and reviewing all my work. Lastly, I would like to thank my mother for trying to let me value the process of hard work to come to a satisfying conclusion. Its a shame you are not here to witness the end result. I dedicate this final chapter of my Master's programme to you.

L.C. Bauer
Delft, September 2019

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Nomenclature

CAD	Computational aided design
CHS	Circular hollow section
CLT	Cross laminated timber
CNC	Computer numerical control
EWP	Engineered wood product
FEM	Finite element method
FIFA	Fédération Internationale de Football Association
FRP	Fibre reinforced polymer
Glulam /GLT	Glued laminated timber
Grasshopper	Algorithmic modelling for Rhinoceros
Kangaroo	Live physics engine for Grasshopper
Karamba3D	Parametric structural engineering tool for Grasshopper
LVL	Laminated veneer lumber
NEN	Nederlandse Norm
OMA	Office for Metropolitan Architecture
PC-panels	Polycarbonate panels
PSL	Parallel stranded lumber
RHDHV	Royal HaskoningDHV
Rhinoceros	3D CAD modeling software
SCIA Engineer	Structural analysis software
UEFA	Union of European Football Associations

1

Introduction

The introduction gives insight into the research subject and its relevance. The collaboration with Royal HaskoningDHV is introduced. The gap of knowledge is identified from which the research questions and objectives follow that define the scope of this thesis. At the end of this chapter a description of the research methodology is presented.

1.1 Prologue

Timber structures are seeing a revelation in the build environment due to new technologies and production techniques. However, the longest spans created with timber structures still don't compete with the lengths of structures made of steel. The long spans result in heavy supporting structures, which have a very big carbon footprint due to the usage of traditional building materials like steel and concrete. Timber is a lightweight material that works as a carbon storage, making it an interesting alternative for steel. The downside of implementing timber in a structural design is the anisotropic behaviour that has great influence in the design choices. This influence is particularly relevant for its long-term behaviour and its limitations for connections. It is interesting to investigate the possibilities of new timber products in combination with wide structures to search for a new limit of structural timber. Football stadiums are that kind of structures, which can be most open minded to a structural change. They are designed as an architectural beacon that can put aesthetics above costs. Something that might be required to choose for an unconventional structural system consisting of wood. This thesis provides a preliminary design for a long-span football stadium roof structure, which extends far beyond the existing maximum spans of timber structures.

1.2 Relevance

Football stadiums are situated in every big city around the world. This is no surprise being that football is the most popular sport in the world. An estimated 4.0 billion people around the globe enjoy and follow the game. [56] As the population grows these numbers are growing and clubs will get a larger fan base as a result. Resulting in the demand for bigger football stadiums. Furthermore, the rules and demands on football stadiums to host big international matches, like the Champions League final, are changing as well. This is due to the fact that it is the objective of the UEFA to improve the quality of both new and existing stadiums in Europe. As is defined by the general secretary of the UEFA, Gianni Infantino(2011): "In this sense, everything that we can do as UEFA to help support, nurture and encourage good and conscientious stadium design and building will be of enormous benefit to football and to local communities" [64]

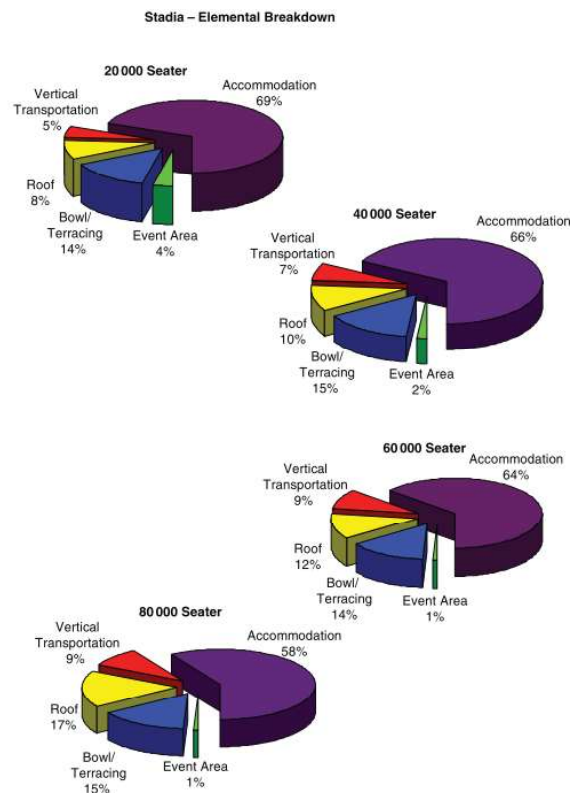


Figure 1.1: Elemental breakdown of stadia construction projects in typical percentage values [38]

Long-span football stadium roof structures

The demand for larger and modern stadiums is reflected by the ongoing high investments into newly developed stadiums which has seen an increase in Europe during the last years. [65] Football stadiums are regarded as architectural icons for their city that have a massive impact on the surrounding communities and infrastructure. [64] A proper design regarding economic and environmental sustainability, without jeopardising their sports nature and architectural qualities is therefore one of their liabilities. [58] These are very costly projects, where a big part of the initial costs is due to the roof structure, the supporting structure of the roof and the grandstands. [38] The price of the roof even has an exponential increase when the stadium becomes bigger as can be seen in 1.1.

The roof structure needs to span a longer distance when a stadium becomes bigger, which results in a considerable increase in weight of the roof. This weight has to be carried to the soil by the supporting pillars that will become more robust and heavier when the forces insinuated by the roof grow in magnitude. A lighter roof structure for large football stadiums is therefore very beneficial for the overall costs and performance of such a large scale project.

Steel is the structural material for long span roof structures of football stadiums. This can be seen for all football clubs of moderate size around the world. However, the UEFA guide to quality stadiums is saying the following about stadium design and sustainability.

‘Many might argue that the cost of designing and building an environmentally friendly building outweighs the benefits. However, this is not always the case, and all stadium designers should be encouraged to incorporate as many sustainable solutions into the whole design as possible. Designing a football stadium with higher benefits is possible, it simply requires a more careful and conscientious design and thought process.’ [64]

Structural timber

Steel is a very heavy material and needs a lot of maintenance and protection against influences from the weather. Therefore, it isn't the most beneficial material for such open and bulky structures as stadium roofs. However, the choice for steel as structural material is so often made, because it is very strong and has uniform material behaviour that makes it very reliable. Keeping in mind the above stated quotation it is interesting to explore what is out there to create a lighter, more sustainable and visually appealing roof structure. An forgotten material for large structures, but that is experiencing more interest, is structural timber. Timber has a high strength-to-weight ratio making it a particularly structurally efficient material for long-span structures where a large proportion of the load to be resisted is the self weight of the structure. Furthermore, timber is largely prefabricated and brought to site for rapid assembly, which can have economic benefits for construction. [53] [5] It is a natural and renewable building material that acts as a carbon storage making it ideal for sustainable building. It can be fully returned to the ecological cycle without producing any non-degradable waste and extremely low energy is required for processing and production. [37] [5] Wood from sustainable managed forests as an alternative to other materials is a great way to fight global warming.[1] [30] Lastly, new timber technology and engineering creates more potential and opportunities to explore the natural organic beauty of this most environmentally friendly building material. [12]

Steel and concrete rule the structural world of football stadiums, but Zaha Hadid Architects have recently won an international competition with their design of the world's only football stadium purely built from wood. [57] [6] The benefit of structural timber in football stadium design is expressed in the following way by the web article of WorldBuild365.

‘Timber as a building material is returning to the architectural spotlight, but this is the first sporting facility to take advantage of the durability, sustainability and beauty of natural wood.’[6]

This stadium, which is relatively small in size, has yet to be constructed. It is however already opening up a new way of thinking about structural design for football stadiums. The real design challenges arise when timber is used as a structural material for bigger stadium structures. Stadium structures that are built with three tiers or more including a magnificent roof structure or are being covered up by long-span roof structures are of interest.

1.3 Royal HaskoningDHV

The city of Rotterdam is planning to build such a new football stadium for the football club Feyenoord Rotterdam. Architectural firm OMA made a design for this large modern stadium which will have a capacity of 63,000 seats divided over three tiers. Due to financial challenges they came up with a business model for the entire area around the new stadium, trying to create a feasible project. [47] Royal HaskoningDHV is currently working on the structural design for this new stadium. Their material choice for the roof structure has been automatically for steel from the fact that conventionally this would be the best choice of material for such a long-span roof structure. Their initial roof designs are however quite heavy, and a lot of supporting structure is needed to carry the roof. Therefore, the lead designer of this project from Royal HaskoningDHV has interest in a roof structure made of timber as it theoretically could be lighter than the proposed steel solutions and reduce the total costs of the stadium. Their design process includes parametric modelling tools for the structural optimisation of the roof structure. These tools create more design flexibility, and efficiency during the design stage. Structural assessment like an optimal force distribution in their structural layout and an optimisation for reducing costs by reducing material are incorporated in their design due to these programs.



Figure 1.2: Sectional view of the architectural design made by OMA.[47]



Figure 1.3: front view of the architectural design made by OMA.[47]

1.4 State-of-art

In this section it will be shortly addressed what the main focus is of the different literature reviews that are carried out in the analysis phase. The demand for certain literature to establish a good research question is expressed. The fields of focus are football stadiums and particularly on roof design, on existing long-span timber structures and what can be concluded from those reference projects and the new possibilities of engineered wood products and more efficient connections are investigated.

Football stadium roof design

As mentioned there is a need for new football stadium roof structures that are feasible but aesthetically appealing as well. There are several structural solutions for roof structures but a lot of them don't fulfil the design vision of architectural firm OMA. They want the perfect bowl with an flat almost floating roof covering it. Furthermore, the playing field needs to be exposed to the weather conditions so it can be of great natural quality. The stadium will consist of three tiers that will have a total height of 40 metres. The span length of the stadium roof structure is 205 metres by 245 metres. Promising configurations are the single span, stress-ribbon, shell roof system and tension / compression ring. It is not greatly known what their specific structural behaviour is for the case of the New Feyenoord stadium. Several existing stadium roof structures are explored to gain better insight on what is possible and how it is made possible. These stadiums are De Kuip in Rotterdam, the preliminary design of The New Feyenoord stadium by RHDHV, the Allianz Riviera stadium in Nice, the Wanda Metropolitano stadium in Madrid, the Estadio Municipal in Braga, and the Tokyo National stadium in Tokyo.

Long-span timber structures

Timber has a high strength-to-weight ratio that is beneficial for long-span structures in which the self-weight of the structure is a significant part of the load. However, wood is an anisotropic material and its properties are influenced by multiple factors making it complex to create a good structural design. It is beneficial for the design if the elements are prefabricated as much as possible and this is restricted by the allowed maximum dimensions for transport. A relevant design issue for long-span timber roof structures is uplift wind loading. Other specific timber related design aspects are the serviceability limit state and fire loading. Several structural forms are applied in long-span timber structures showing their applicability for these type of structures. These forms are the arch, truss, box girder, shell, spaceframe, and stress-ribbon. Several reference projects are explored to gain better insight in the applicability and limitation of these structural systems. These projects are the Multi-use Arena in Lisbon, the Trade Fair Hall 11 in Frankfurt, the Anaklia-Ganmukhuri bridge in the Georgian Republic, the Maicasagi bridge in Nord-du-Québec, the geodesic domes in Brindisi, the Elephant House in Zurich, the Allianz-Riviera stadium in Nice, the Grandview Heights Aquatics Centre in Surrey, and the Essing bridge in Essing.

Structural timber

Engineered wood products make it possible to create complex timber structures with more reliability than with wood in its natural shape. The most promising EWP's for long-span timber structures are glulam and LVL. Where LVL made of locally sourced beech has the highest strength properties. There are also several options of improving these structural materials. However, a lack of knowledge and cost efficient applicability of these improvements are still a boundary for their implementation. Furthermore, New types of connections are nowadays on the market that diminish the original impact that connections had on timber structures. These connections are self drilling screws, self drilling dowels, steel fasteners fixed with adhesives, the HSK frame connection, the Sherpa system, the BVD system, the Hess Limitless splice joint, and the elastic glue joint. Several forms of strengthening the connections have been explored in the past as well, but might be unnecessary for these modern connections.

1.5 Research definition

1.5.1 Problem statement

A football stadium is a magnificent building project. A lot of design aspects have a significant impact on the initial costs and the environmental impact of such a structure. One of the most notable features of this design is the stadium roof structure. Nowadays, there are no long-span football stadium structural roof systems around the world made of solely timber. Steel is the most common material for almost all of these roof systems although the fact that timber can be a better alternative. Wood as a structural material has excellent specific strength and stiffness. Theoretically, it can make a lighter roof structure than steel. Less weight can result in a material reduction in the supporting structure of the roof. Lastly, correctly using timber as a structural material will result in a more sustainable structure. It is a renewable and recyclable resource which acts as a carbon store. A timber roof structure for long-span football stadiums can help to play a role in reducing carbon emissions of the building industry. It is, however, a complicated material to design with and therefore requires more thought to be successful. The lack of knowledge on long-span timber structures of the magnitude of a large football stadium is a limitation. Whereby, the design of the connections can be governing for the structural system and need to be safely designed for short- and long-term behaviour. Royal HaskoningDHV is currently working on the structural design of the New Feyenoord Stadium in Rotterdam. They are working with parametric tools to get to the most optimal design to make the project feasible for the city of Rotterdam. Their material choice for the roof structure has been steel from the beginning, but it results in a heavy roof- and supporting structure. It is therefore interesting if a long-span timber stadium roof structure is possible, and if it indeed can result in a more optimal design than with a steel roof. The limitations of structural timber can be governing in some places and than it is of interest if the roof can be made by implementing as much timber as is possible. The New Feyenoord Stadium gives good framework conditions as a case study for a long-span timber stadium roof structure.

1.5.2 Objective

Reading the problem statement, the following main research objective can be identified:

‘Create a structural design with as much structural timber as is feasible to create a light-weight roof structure for the New Feyenoord stadium.’

1.5.3 Aim

The following main research question is formulated:

‘To what extent can a timber long-span stadium roof structure be feasible for the New Feyenoord stadium?’

In order to answer the main research question, sub questions have been formulated:

- What are the design criteria of the New Feyenoord stadium?
- Which structural configurations are used for long-span stadium roofs?
- What are the benefits of structural timber for such a big roof
- How can timber be used as a structural product for long-span roof structures?
- Which structural forms are present for long-span timber structures?
- What are governing restrictions for a huge timber structure like a long-span roof?
- Are there any examples of long-span timber structures and how do they solve their problems?
- Is it possible to create an efficient structural design of a long-span stadium roof structure?
- Is a timber long-span roof structure efficient and attractive to be used for the New Feyenoord stadium?

1.5.4 Scope

Certain limitations and general conditions are stated in this section to create a manageable thesis in terms of time and scope. The scope is chosen in a smart way considering the main goal of creating a feasible structural design of a long-span football stadium roof structure made of structural timber.

Structural design	The structural design only concerns the roof structure and is based on the shape and dimensions of the reference stadium.
Reference stadium	The New Feyenoord stadium will be used as a basis for the design of the roof structure. The reason to use this stadium is that the design criteria are known through Royal HaskoningDHV.
Roof design	The roof design will be as compliant as possible to the architectural design made by OMA. This means an elegant roof structure which is as flat as possible while it looks like it floats above the grandstands.
Roof finishing	The roof panels are made of polycarbonate and will have a maximum span of 2.5 metres. Furthermore, when it is beneficially for the structure, solar panels will be incorporated in the design as well without blocking sunlight near the pitch.
Supports	The roof is supported on the cores around the stands as designed by OMA and Royal HaskoningDHV, assuming the stands can carry the loads. There are 12 supports along the outer perimeter of the stadium. The dimensions of these supports are $b \cdot l \cdot h = 8 \cdot 22 \cdot 40$ metres.
Grandstands	The grandstand configuration determines the outer perimeter of the roof and is taken from the design of OMA and Royal HaskoningDHV. It is designed as the 'perfect bowl' which has the shape of a super ellipse. Key aspects of this design is unobstructed viewing, which means no columns supporting the roof on the grandstands and that the Kuip atmosphere is kept. The maximum dimensions of the outer super ellipse is 205 by 245 metres. [47] [10]
Loads	Loading of the structure will consist of dead loads and live loads. An estimation of the wind loads on the roof structure is made by RHDHV according to Dutch regulations. Earthquake loads are not considered, because of the lack of heavy earthquakes in the Netherlands. Fire loading is not considered as well, because the timber elements will be very large and a roof structure is considered.
Calculations	The structural calculations are based on the Eurocode. When the Eurocode is not sufficient, use is made of the Dutch NEN building codes. However, the state-of-art on timber solutions is mostly provided by research and experience of subcontractors and suppliers. In this design these starting points might be used to get to the most feasible design of the timber roof structure.
Material	Basis for the design is that timber is used as a structural material to the utmost extent for the roof structure. Only when it is beneficial other materials are used like at the connections or at very high peak values of the stresses.
Feasible design	A feasible design, depends on three key elements: the structural design, the constructibility and the building costs of the structure. The problem of this thesis will be prioritised to the structural design of the long-span football stadium roof using structural timber.
Construction	The manufacturability and constructibility will be addressed in a relative simplified manner due to lack of experience on these subjects.
Costs	The building costs of the structure are not addressed in depth. This is due to a lack of data available on building costs of such a timber structure.
Retractable roof	The structural design in this thesis will not incorporate a retractable roof although this is stated as one of the criteria by Feyenoord.[47] [10] This is considered as a unrealistic approach for this research, considering the time limitations.

1.6 Readers guide

The methodology of this research is divided into four phases which are explained in the paragraphs below. The introduction finalises with a scheme showing the design strategy used in this thesis. This scheme is presented in figure 1.4.

Phase 1: Analysis

First, literature will be reviewed in order to retrieve more knowledge about the following three subjects: structural design of football stadium roof structures, long-span timber structures and current available research on engineered wood products and new types of connections.

Regarding structural design of football stadium roof structures, information is gathered about design aspects, structural roof design options and reference stadiums. Additionally, the design boundaries coming from the New Feyenoord stadium are incorporated in order to create boundaries for the rough design of the roof structure. Next, the structural form for long-span timber structures is explored. Appealing and structurally intriguing reference projects are presented. At last, the material behaviour of wood, promising engineered wood products and new types of connections are discussed for their applicability in the final design. All these findings are discussed and summarised, where-after global designs will be made for the next phase.

Phase 2: Global design

Several structural solutions are made with the conclusions drawn in the literature phase in mind. After an assessment of the different options, one option is chosen. The boundaries of the design are determined accordingly with the New Feyenoord stadium design made by RHDHV and OMA. Simplified load cases and material properties are incorporated into the global designs. There are four designs which have a different global structural system for the main girders of the roof. These four categories are single-span, tension / compression ring, stress ribbon and grid-shell of trusses.

The first assessment of these systems is done with similar dimensions to the global design of RHDHV. Hereafter, an optimisation is done on the different structural systems and they are evaluated on their behaviour and outcome of this optimisation. This optimisation is done mostly for the peak forces in the members and the amount of material used. The goal of these assessment is to choose one structural design for which an more in depth analysis is done in the next phase.

Phase 3: Structural design

Previous phase explored the structural system of the main girders. In this phase the chosen structural system is explored in more depth and optimisations are made in regard with the initial design. This optimisation is done for the utilisation of forces in the structural members, the peak forces in the members, the amount of material used and the distribution of material to create a structural system. In this phase the configuration of the secondary girders is explored, chosen and implemented to fulfil this optimisation. Whereafter, The real detailing of the structural elements and connections is done.

The best suitable connections are incorporated in the chosen structural system. These connections are assessed at the most critical locations. A detailed design of this implementation is made, and all structural components will be verified and checked. The use of parametric tools makes it is easy to have an iterative design process and is therefore, highly recommend to be used. Meaning that when the chosen system provides unknown problems, the whole system can be easily altered in the parametric design. After the structural checks are in order, the manufacturability of the entire structure is determined.

Phase 4: Finalisation

At the end of this research, the design results are presented. It is desired to have created a good alternative for the iron fist that steel roof systems have on football stadium roof structures. These results will contribute to the answer to the research question. After which conclusions and recommendations for future long-span timber structural roof systems will be given.

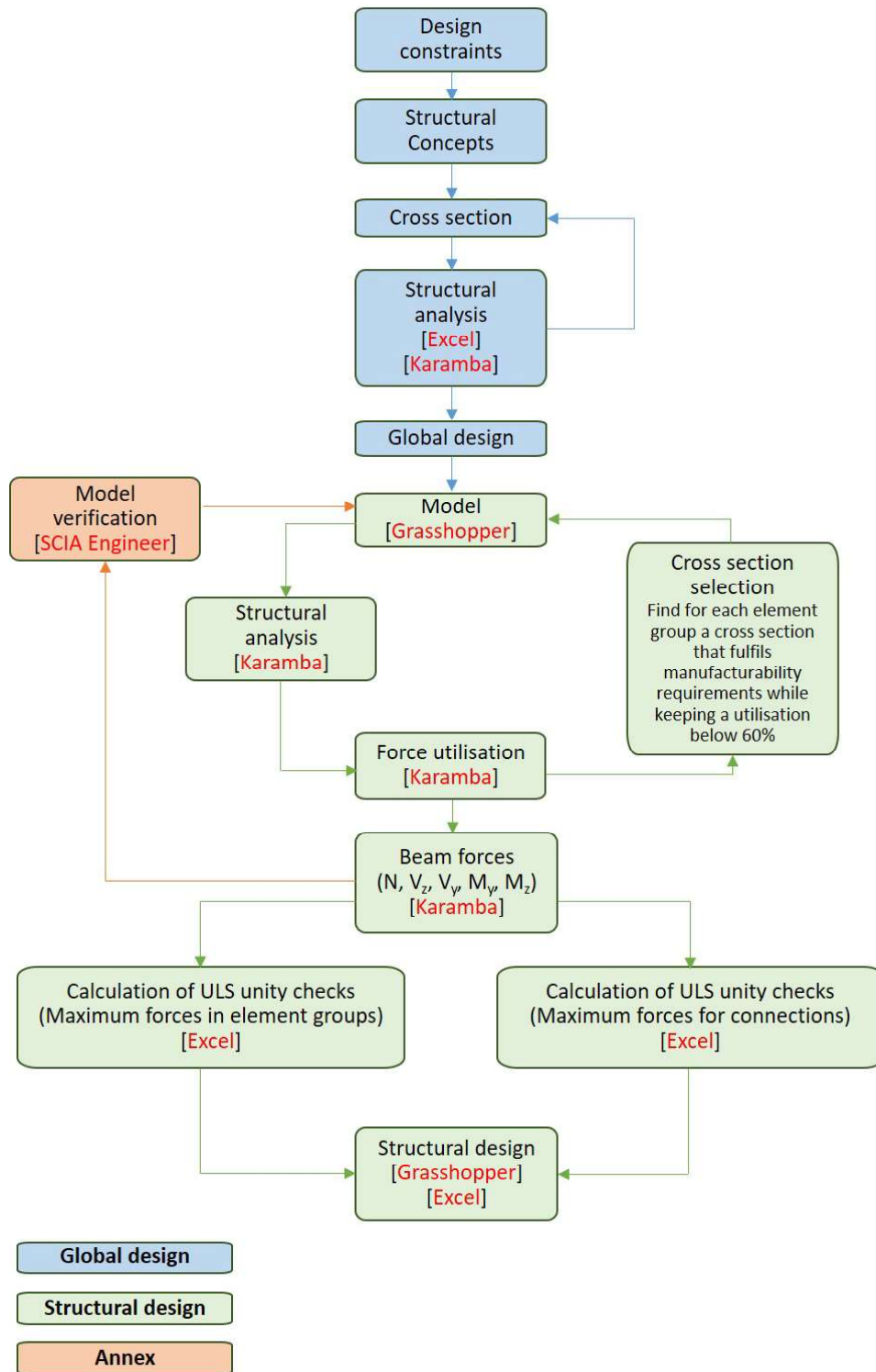


Figure 1.4: Design strategy

2

Analysis

The analysis gives insight into the design aspects of a long-span football stadium roof structure consisting of structural timber. It is divided into four sections. In the analysis of football stadium roof design are the designing background and structural options explained and accompanied with existing football stadiums. In the long-span timber structures analysis are the common used structural systems for long-span timber structures explained and accompanied with several reference projects. The structural timber analysis illustrates the material behaviour, modern engineered wood products and innovative connections. Finally, this chapter concludes in a summarising infographic with a summary, a list of demands and a defined scope of the project.

2.1 Structural design of football stadium roofs

The analysis of the roof design of a football stadium introduces the design aspects of a long-span stadium roof structure specialised for the New Feyenoord stadium. The best possible structural configurations are explained and the applicability of these configurations is presented by reference projects. In the next section, the structural possibilities of long-span timber structures will be analysed.

2.1.1 Design aspects

A few starting points for the stadium design of the New Feyenoord stadium are stated by OMA and RHDHV. These starting points result in the shape and aesthetics of the stadium. Furthermore, there are some starting points stated by the UEFA for modern stadium design. The most important design principles are presented in this section. These principles consider general comfort for the spectators, quality of the football pitch and aesthetics as formulated by Feyenoord and OMA. The FIFA stated that a modern stadium should not be built to only satisfy short-term demands, but rather in the hope that the facility will serve the requirements of the generations to come.[20]

Playing field

A playing field which is suitable for all matches at the top professional level and where major international and domestic games are played should have dimensions of 105 x 68 metres. Beside the playing field, additional flat areas are required with a minimum of 8.5 metre on the sides and 10 metre on the ends. This results in an overall playing field and additional area dimension of: length:125 metres, width: 85 metres as can be seen in figure 2.1. [20]

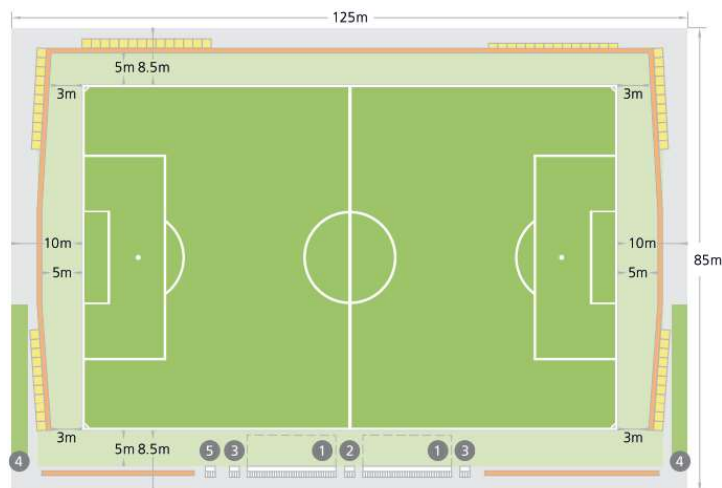


Figure 2.1: Playing field dimensions according to FIFA regulations

It is extremely difficult to maintain a grass pitch in a acceptable condition for the top level of football when the pitch is covered by a roof. For a natural grass pitch, it is critical that there is enough light and air movement to sustain the healthy growth of grass. The quality of a playing field is severely reduced if the stadium does not have an opening in the roof for natural wind flow. Furthermore, all sides of the playing field must receive a reasonable amount of direct sunlight. Therefore, the roof covering will consist of transparent polycarbonate sheeting. These translucent panels allow natural sunlight to enter the field and grandstand more easily, which is beneficial for the pitch and visual experience of the match.

Atmosphere

De Kuip atmosphere, meaning that the visitor is central to the design, needs to be preserved. 'Whether it's fans of football matches or visitors to a concert or event, in the new Feyenoord stadium you come to watch and experience.' [47] The goal is to have no obstructing columns on the tribunes and an unobstructed supporter perimeter around the pitch. This results in the tribune being an entity around the pitch, which will be entirely filled with supporters on a match day. An impression is seen in figure 2.2.



Figure 2.2: The unique atmosphere of De Kuip with unobstructed spectator viewing

Acoustics contribute to the atmospheric aspects of a football stadium. The structural surface and geometry influence acoustic reflections and should be designed to avoid hindrance. Therefore, The stadium design should provide an acoustical experience in the stands and on the field that contributes maximally to the best possible atmosphere. It is also required to limit the noise to the environment during matches and events.

Grandstand

The grandstand will consist of three tiers to accommodate the desired 63,000 seats for spectators. This will make it the biggest stadium of the Netherlands, hence a very ambitious plan. The first row of the first tier needs to be as close to the pitch as possible. Contributing to the spectator experience as stated in the previous section. [47] [10]

The atmosphere of the existing stadium of Feyenoord, namely De Kuip atmosphere, is of great importance and sets the ambition for the sight lines. This is translated into a spatial starting point based on one of De Kuip its most atmospheric elements, namely the unimpeded oval ring structure of the grandstand.

A very relevant aspect of stadium design is the unobstructed and complete view of the spectators. The quality of view from each seat is determined by the c-value, which is a variable that defines the quality of the spectator's line of vision over the head of the person in front. A higher c-value results in better visibility of the entire pitch. However, it can also result in an increase in the overall height and width of the stadium which is unwanted. An recommended minimal for the c-value is 90 and an optimal value is 120 as can be seen in figure 2.3a. [20] [64] [47] [10]

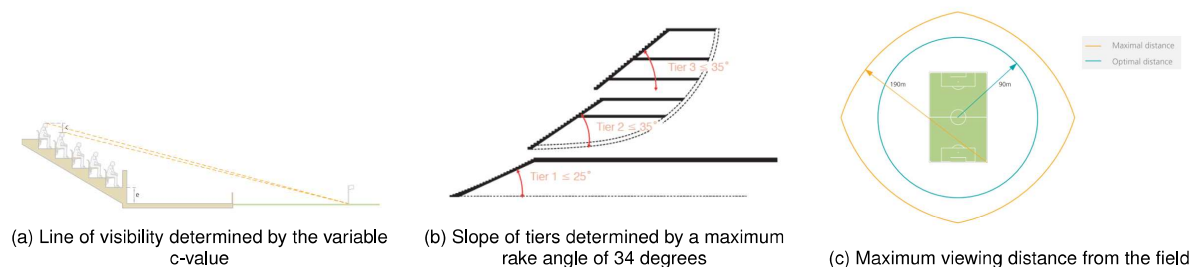


Figure 2.3: Grandstand design

To satisfy a certain level of comfort on the grandstands there is a maximum angle for the tiers related to the pitch. This is called the rake angle and should not exceed an angle of 34 degrees (35 degrees for the worst case) as can be seen in figure 2.3b. [20] [64] [47] [10]

The FIFA recommends that the majority of seats is located inside a seating perimeter of maximum 90 / 190 metres from the field. Within this margin spectators have a clear view of the playing field from all seats. This distance is measured from the most distant corner of the playing field as can be seen in figure 2.3c. [20] [64] [47] [10]

All the above stated aspects are used to find the ideal shape of the grandstand. This is done by means of an interactive process between OMA and RHDHV using parametric form finding that resulted in the 'bowl' configuration. The spectator's experience is maximised for all seats in the stadium hence the name "the perfect bowl". The perfect bowl is an intricate balance between the unobstructed oval ring structure of the Kuip, the sightlines of the spectators, the rake angle of the three tiers and the distance to the field. The resulting outer perimeter of the stadium is in the shape of a super ellipse with width and length of 205 by 245 metres.

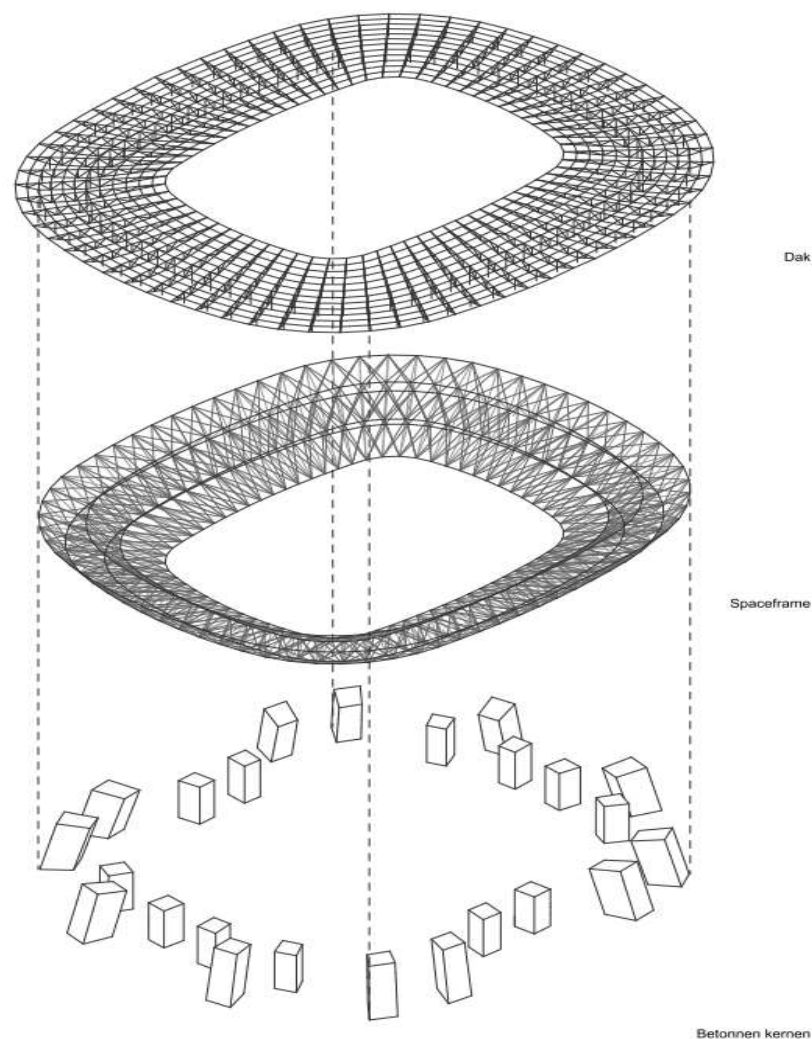


Figure 2.4: Schematisation of the floating roof over the perfect bowl which is supported in the outer perimeter by several cores

Roof design

A modern stadium should provide a roof over all spectators when it is located in cold, wet climates. Feyenoord would like to have its stadium completely covered by a retractable roof, which is out of the scope of this thesis. Furthermore, it is extremely difficult to maintain the grass pitches in an acceptable condition with this kind of roof. It is chosen to have an opening above the pitch with almost the same dimensions as the playing field.

It is the design vision of OMA to have a flat roof that looks like it is floating above the grandstands as can be seen in figure 2.4. The idea is to search for roof configurations that provide a flat shell like impression. Only perimeter supports will support the roof that needs to be able to resist the implemented forces through an internal force flow. All this will create a very visually elegant roof structure.

The roof is supported at the outer perimeter by twelve cores made of concrete walls. The cores will have the following dimensions: height = 40 metres, length = 22 metres and the width = 8 metres. The cores will be almost evenly distributed along the circumference of the stadium.

It is stated in the UEFA stadium Infrastructure Regulations that no object may be located less than 21 metres above the field of play. [66]

2.1.2 Structural roof systems

There are a few commonly used structural roof systems for football stadiums. Those configurations that fulfil the design vision of OMA are presented whereby their advantages and disadvantages are mentioned. The focus of these configurations is on a medium sized football stadium. Of importance is the possibility to create a structural system out of structural timber that can produce the required span. As a result, a lot of systems are already abandoned.

Single-span

The entire stadium is spanned from façade to façade in one go creating the possibility for long spans. In previously realised stadiums, this span is made over the short side as well as over the long side of the stadium. The latter is less common as the larger span leads to a less efficient primary structure. The main beams, which are usually two, carry almost the entire roof structure and therefore require a robust and solid support. The flexural beam system and the arch system are often used structural systems for this configuration, where the latter results in a more efficient material usage. These linear systems span the entire length of a grandstand while transferring loads through bending, because of this principle the rigid elements often consist of truss girders which are heavy and bulky. Complete unobstructed viewing is provided for any grandstand configuration, unless the relatively high construction along the roof opening forms an obstacle to sight-lines in the stadium. Most of the material for the roof structure is thus concentrated around the field plan: therefore, a high degree of flexibility is present in this zone for special loads during events. The single span girders have such long lengths that they need to be supported during erection or the entire span might be hoisted in to place from the building site, depending on the weight of the structure and space of the building site available. A schematisation of this roof configuration can be seen in 2.5

Advantages

- Unobstructed spectator viewing for all stadium configurations
- Aesthetically appealing due to floating roof
- High stiffness in elements close to pitch, benefit for special loads
- Large spans are possible
- Highly efficient structural shape when arches are used

Disadvantages

- Main girders are often bulky and heavy
- High support reactions
- Supporting structure during construction might be needed



Figure 2.5: Single span system

Stress-ribbon

The stress-ribbon configuration also spans the entire stadium in one go. The difference with the single span is that the primary forces in this roof system are taken by members solely in tension. The system is very economical in material usage in relation to other structural configurations due to a reduction of the cross sections of the elements. In contrary to the single span definition above having two main girders, the stress ribbon system consists of multiple cable like elements spanning the stadium. This results in a more evenly utilised structural system, which is also very slender. Tension cables need to be stabilized and restraint against deformation that cause them to go into compression. A stress-ribbon roof structure is a very light structure which is a very economical method of spanning wide areas especially. An other structural system which is entirely in tension is the cable-net structure. A schematisation of the stress-ribbon roof configuration can be seen in 2.6.

Advantages

- Unobstructed spectator viewing for all stadium configurations
- Economic solution for wide spans
- Very lightweight structure
- optimum cross section utilisation with high span widths
- elegant and harmonious design
- Straight forward prefabrication due to similar elements

Disadvantages

- Sophisticated design is needed
- Heavy supporting structure to restrain the cables
- post- or pre-tensioning might be required



Figure 2.6: Stress ribbon system

shell roof system

A shell roof system spans the entire stadium in two directions on only perimeter supports. They resist the applied loads through their inherent three dimensional shape. This is mainly done by membrane forces, where several forms belong to this type of load bearing structures. It is a structurally efficient form as it carries the loads by way of tension, compression and shear forces in the plane of the shell resulting in thin structural members. These systems are structurally ideal when they form an enclosed roof. They are mostly difficult to design and might need certain shape restrictions to work properly. Furthermore, the fabrication and installation of shell structures is time consuming. The possibility for very elegant roof forms is offered, but requires a certain amount of height to have an in-plane force distribution. A schematisation of this roof configuration can be seen in 2.7.

Advantages

- Potential for great visual elegance
- Suitable for large spans due to high-strength-to-weight ratio from the double curvature

Disadvantages

- Increased labour time in fabrication and installation
- Specialist designers needed, as the mathematics involved are advanced
- Structurally ideal when fully enclosed
- High structures to create the double curvature



Figure 2.7: Shell roof system

Tension / compression ring

A tensile / compression ring is derived from the spoke wheel principle. With the spoke wheel principle, a lightweight, cost-efficient roof structure can be made. Such a roof consists of an inner tension ring and an outer compression ring, the two being connected by radial members which maintain the geometry of the structure and carry the roof covering. This typology can be applied on almost all ellipse or circular shaped stadiums, when the circle is fully closed. It is a balanced supporting system which is suitable for very large canopies. The ring action determines the efficiency of the tension/compression ring. The ring action is influenced by multiple factors: the curvature of the ring, the cross section of the ring and the loads acting on the ring. A schematisation of this roof configuration can be seen in 2.8.

Advantages

- Lightweight structure that has a very modest appearance when seen from the outside
- Special supporting structure is not needed
- Relatively low building costs

Disadvantages

- Structural system can only be used for bowl-shaped stadiums
- Irregular curvature of the rings weakens the system
- Stiff inner and outer ring

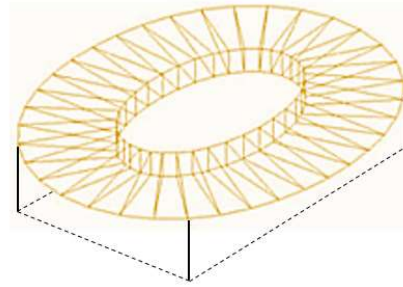


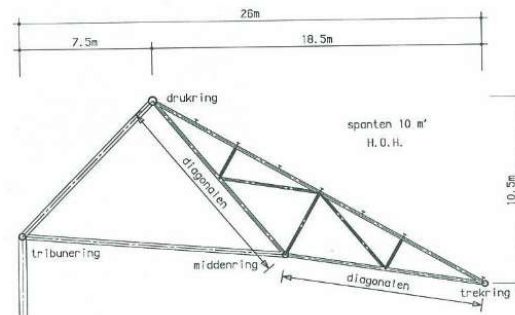
Figure 2.8: Tension / compression ring roof system

2.1.3 Reference football stadiums**De Kuip, Rotterdam**

In 1994 the Feyenoord stadium, also called 'De Kuip', was renovated whereby a canopy roof was added to the stadium. After this renovation De Kuip was left with a capacity of 51,117 seats. A governing condition for the roofing of the stadium was that the structure could not block the sightlines of the spectators. The roof structure had to be column free over the whole depth of the stands. Furthermore, the support system for the canopy was not allowed to dominate the facade and external appearance of the stadium. Therefore, a self-supporting canopy structure with a fixed cantilever is avoided. An important aesthetic demand was that the roof should appear to 'float', just like the second ring of the stadium appears to be floating above the first ring. Lastly, the roof had to follow the characteristic ground plan of De Kuip as seen in figure 2.9a, with heavy curvatures in the four corners and light curvatures along the sides.



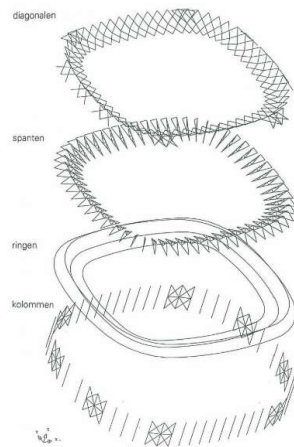
(a) The characteristic ground plan



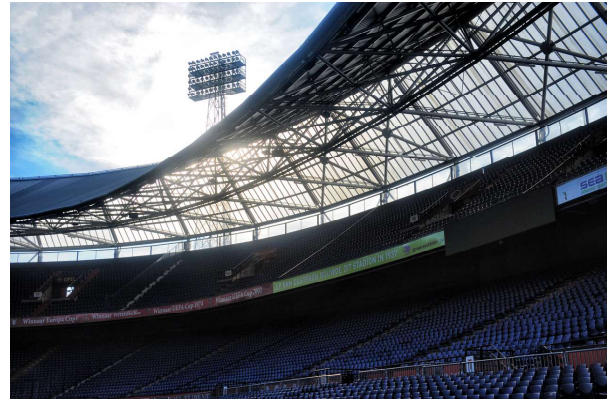
(b) Sectional view of the trussed cantilever

Figure 2.9: 'De Kuip', Rotterdam

This resulted in a spatially stable structure that transfers the loads three-dimensionally. The structural system consists of trussed cantilevers, figure 2.9b, placed at a distance of 10 metres apart connected by a compression ring, two tension rings and diagonals in two planes in between. The force transfer partially takes place through ring action and partially by beam action. The diagonals in at least two planes are essential for the spatial load transfer and form fixing of the structure. The structure deflects the most in the tension ring in the middle of the long side of the roof structure. These deflections were kept as low as possible by the placement of the diagonals in the lower level plane of the roof. The combined load transfer is highest in the long sides due to the lack of curvature to fully carry the loads by solely ring action. Due to limited deflection in the corners, the stiff and sloped placed truss will transfer the loads to the corners. At the wind bracing in the corners there are trusses placed at the lower and upper level to carry the horizontal loads due to wind as can be seen in figure 2.10a. The forces in the rings do not follow a uniform force flow at an equal load distribution due to the special shape of the stadium. Therefore, the diagonals are also needed for an more efficient load transfer at an equally distributed load. A bottom view of the roof structure is shown in figure 2.10b.



(a) Exploded drawing of the levels of the roof structure



(b) Bottom view of the roof showing the diagonals in the lower level plane

Figure 2.10: Schematic drawings of De Kuip, Rotterdam

The trusses were entirely prefabricated and transported as a whole to the building site. All other parts were transported as single elements to be erected on site. The roof units are assembled outside the stadium after which they were connected to each other during construction while being supported on temporary columns. The temporary columns were removed one by one after the rings were closed. After removing the temporary columns the measured deflections were compared with the calculated deflections. The largest deflection (500mm) occurred at the long side.

New Feyenoord Stadium, Rotterdam

The design for the New Feyenoord Stadium is ongoing and therefore still unelaborated. The dimensions and design vision for this stadium are stated in the previous section 2.1.1. The initial roof design as made by RHDHV will be elaborated in this sub section. The initial design for the roof construction is entirely made of steel and constructed from primary, secondary and tertiary lattice girders. The two primary lens-shaped trusses are box-shaped and bear directly on underlying concrete cores at the ends. Along the opening of the roof there are secondary trusses that bear on the primary trusses. Tertiary trusses from the outer edge of the stadium to the primary and secondary trusses form the rest of the roof construction. A parametric model is made for the fixed roof section based on design sketches of which various parameters can be varied to assess the effects for the total costs and visual aspects of the roof. The preliminary design of the roof structure can be seen in figure 2.11a and 2.11b.

[Confidential - Do not distribute to third parties]



(a) Overview of the initial design by RHDHV



(b) View from underneath the roof structure

Figure 2.11: Preliminary design of The New Feyenoord Stadium, Rotterdam

Important parameters were found to be the upward and downward bulge of the primary trusses as well as the height of the entire roof. These parameters are varied in order to fulfil the design vision of the architect. The facade height along the perimeter of the stadium is minimised. As there are no user spaces situated at the top level, a high roof increases the costs of a facade which has no function. It also has an unfavourable impact on the appearance of the stadium compared to the Master Plan of the architect. Furthermore, the upward bulge of the primary truss is minimised to respect the image stated by the architect and to reduce the surface area of the roof finishing. The construction height of the primary beams is set to 14 metres as this is the initial found optimum with the lowest possible construction height between the criteria for strength and rigidity. At last the entire roof structure is lifted by a few meters to prevent that parts of the structure will obstruct the view from the stands at the top. An overview of these dimension for the primary roof truss can be seen in figure 2.12a. The limitation of the construction height for the primary girders to 14 metres results in large forces in the edge bars of the truss resulting in CHS with dimensions of 1321x65 mm, steel S355 which are utilised up to 95%. This high degree of utilisation can complicate the design of the connections as these will weaken the structure and ask for some margin in utilisation. Therefore, RHDHV anticipates that the nodes of the primary and secondary trusses will have to be made of gusset plates as shown in figure 2.12b. All above stated design aspect are still open for optimisation as this is a preliminary design. For instance, a higher construction height of the primary girders will result in lower internal forces and therefore less steel. This is an intrigued process were multiple factors influence each other and finding an optimal solution will take time and good cooperation between client, architect and engineers involved. [10]

[Confidential - Do not distribute to third parties]

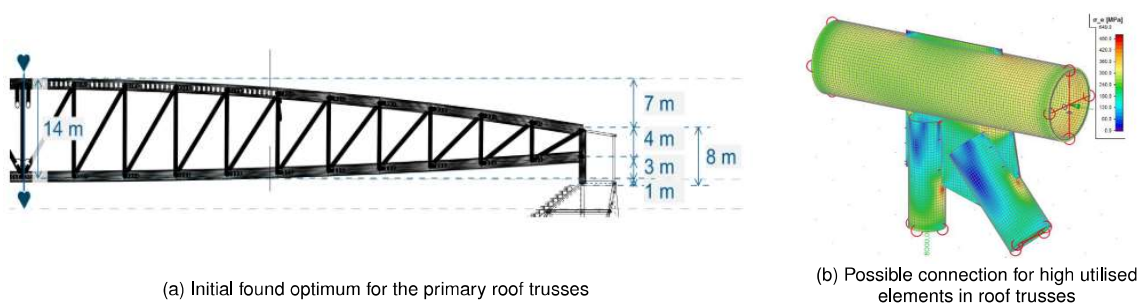


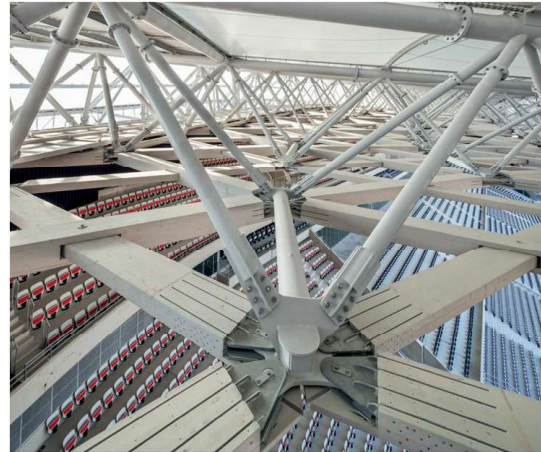
Figure 2.12: Preliminary detail designs for The New Feyenoord Stadium, Rotterdam

Allianz Riviera stadium, Nice

The multifunction Allianz Riviera Stadium in Nice, which has a capacity of 35 000, is regarded as a show-piece in terms of sustainability. It is the largest timber-and-steel lattice structure consisting of two levels made up of a timber lattice and steel space frame. The basis of this space frame are cantilevering timber-and-steel canopies, which provide a roof with different roof covering materials as can be seen in figure 2.13a. The stadium consists of curving half-frames with an inner arch (intrados) of criss-crossing glulam members to carry the compressive forces and a curving outer arch (extrados) made of steel circular hollow sections to carry the tensile forces. An intermediate structure made of steel circular hollow sections arranged in pyramidal form links the two arches to form a spatial truss. An exploded view of this principle is seen in 2.14a. A detail of this space frame configuration is seen in figure 2.13b. To resist the axial forces, the designers chose intersecting glulam members of different thicknesses laid up in parallel blocks. The middle of the thicker cross-section includes a slot through which the other, thinner cross-section is 'threaded'. One bolt connects the members at each intersection. This 'threaded' connection halved the number of structural nodes compared with the original plan. The individual members with lengths varying between 7 and 10 metres brace each other and thus halve their buckling lengths; on their own they would be at risk of buckling. To join the steel and timber members to form a lattice, the timber contractor developed a butterfly-shaped steel connecting plate combined with a tubular steel 'purlin'. Four timber members (via plates let into the timber) plus the steel pyramids are then joined at every node. This principle is seen in figure 2.13b.



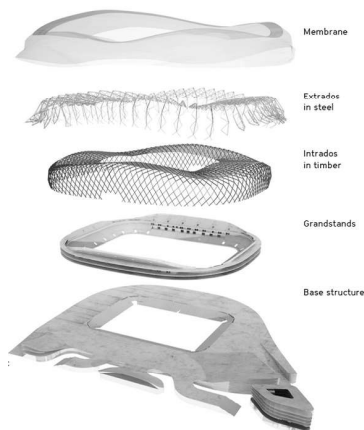
(a) Roof membrane existing of photovoltaic panels for generating electricity, PVC foil for shade, and ETFE foil for transparency



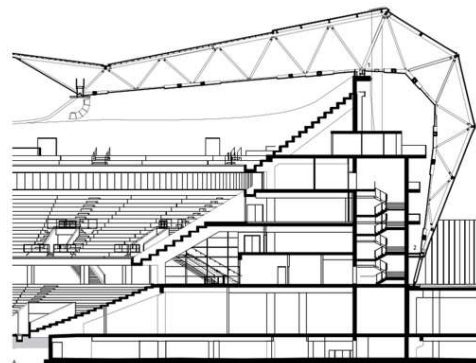
(b) Detail of the roof cantilevers

Figure 2.13: Allianz Riviera Arena, Nice

A total of sixty timber-and-steel frames cantilever 46 metres out over the grandstands at a height of 30 metres above the pitch. They are supported at two points only: at the top of the grandstands and at the bottom of the rear wall to the grandstands. Steel beams running around the whole stadium serve as supports. The lower, 800 metre long steel 'wailing' is supported on V-columns that are anchored in the base structure and tie the cantilevering half-frames back to this. There are also horizontal steel beams that tie the circumferential beam back to the concrete walls. The zigzags take the tension and compression due to wind, snow, and earthquake loads. This principle is seen in figure 2.14b. Additional structural calculations had to be performed for each curved section, which, owing to the complexity of the geometry, were difficult even with the help of a computer. Besides the geometry and the applied loads, the calculations also had to take into account the different properties of the building materials, e.g. strength and stiffness, plus external influences such as humidity and temperature, which affect timber, steel, and concrete (creep and shrinkage) differently. [37].



(a) Exploded drawing of the levels of the construction



(b) Section showing the half-frames cantilevers

Figure 2.14: Schematic drawings of the Allianz Riviera Arena, Nice

Wanda Metropolitano, Madrid

The Wanda Metropolitano stadium in Madrid is the new home for Spanish football club Atlético Madrid and is able to accommodate over 67.000 spectators. The roof structure has approximate dimensions of 286 metres in length and 248 metres in width. The main structure of the roof consists of an exterior double compressing ring of steel and an interior double tension ring connected to each other by two groups of radial cables as seen in figure 2.15a and figure 2.15b. The cables are alternately connected between the lower and upper part of the compression and tension rings, spanning 57.00 meters between them. The necessary height of the roof is divided into two by means of using a double ring

for both rings. The design of the roof includes a sound amplification by reflecting the sounds coming from the stands in the lower part of the structure. The result is a light roof structure protecting 96% of the spectators of the stadium which floats above the stands like a big blanket. The structure does not cover the playing field in order for more sun to reach the grass. At night the best possible lightning is provided by integrated floodlights into the interior ring of the roof. The roof has a minimum height of 45 metres and a maximum height of 57 metres, weighing around 6,300 tons. This roof is going to be responsible for a big part of the stadium's image, where the stands have a solid, almost fortress-like appearance, the roof images a very stable, slender and elegant structure that has the shape of a wave as indicated by the architect and the engineer. [43] [44] [42]

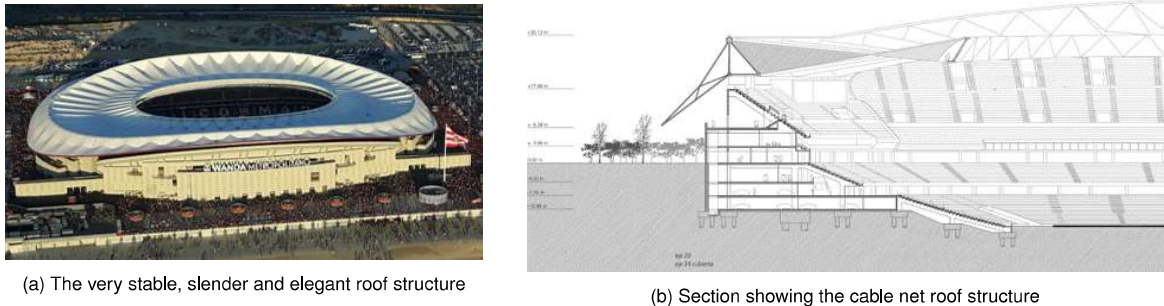


Figure 2.15: The Wanda Metropolitano Stadium, Madrid

The cable net, consisting of 96 radial cables and 2 tension rings, was first assembled on ground level and connected to the compression ring/ tension ring. After which the hydraulic jack was installed onto the tension ring side of the radial cables, a method unique to this project. Then, the hydraulic jack was used to lift the cable net into the air. A more detailed view of the cables is seen in figure 2.16a and the compression and tension rings are visible in figure 2.16b.



Figure 2.16: Structural system and the building process

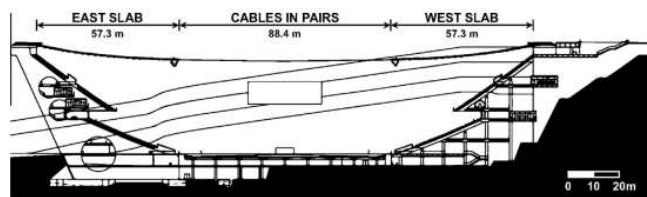
Estadio Municipal de Braga, Braga

The Estadio Municipal De Braga is an 30.000 seat stadium integrated in its rocky environment. The stadium has a suspended cable roof consisting of 34 pairs of full locked coil cables with diameters varying between 80 and 86 mm, spaced 3.75 metre apart from each other. The roof has a span of 202 metres with two concrete slabs on the first 57.3 metres in each end covering the two stands of the stadium. These concrete slabs are supported by continuing cables from tribune to tribune where the middle 88.4 metre part is uncovered to allow enough light for the pitch. This can be seen in figure 2.17a as well as figure 2.17b. The concrete slabs, with a height of 0.245 metres, are only supported by the cables in their normal direction to allow for relative tangential movements between two elements. The cables in the roof have a varying length to create a slight slope for the efficient drainage of rainwater. A transversal triangular truss is suspended from the inner border of each slab acting as a stiffness beam and simultaneously accommodating the floodlights and loudspeakers. All this is supported by large, very stiff beams at the top of the tribunes to allow for the transition between the roof cables and the supporting uprights. The east stand is structurally formed by sixteen 50 meters high concrete walls, all

of which are 1 metre thick. Their geometry is defined by the goal to minimize the unbalanced moments, due to the high forces transmitted by the roof cables and the self-weight of the tribunes, at the level of the foundation. The cable forces at the West stand are directly distributed to the foundation which is anchored in the rock by prestressing tendons embedded in concrete. The anchoring connection of the cables can be seen in figure 2.18b. This roof shape has an expected proneness to dynamic effects induced by wind. This resulted in extensive studies during the design phase to define the design loads and to evaluate the corresponding static and dynamic behaviour of the structure. The initial weight of the roof was chosen to counter the uplift forces of the wind by using a static approach of the wind. Thereafter, more sophisticated wind analysis was done by means of computational modelling and physical models of the structure and location. These tests demonstrated the aerodynamic stability of the total roof configuration. However, in the central zone of the roof some dynamic behaviour can be expected. Therefore, a decision was made to connect pairs of cables to each other with dampers in between. [45] [23]



(a) Overview of the stadium



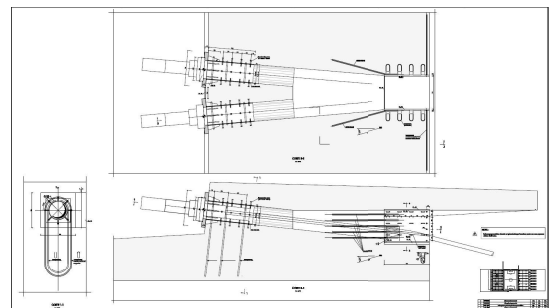
(b) Longitudinal section of the stadium

Figure 2.17: Estadio Municipal de Braga, Braga

The roof construction process entailed three fundamental problems: the detailing of the covers, the assembly system and the effect of the assembly process on the final shape of the suspended cables. The final connection of the roof slabs has many advantages concerned with the response to thermal actions, to the shrinkage of the concrete roof slab and to the dynamic load of the wind. The prefabricated elements were assembled over the cables, on top of the stands. Each new piece is linked to the previous piece with bolts and the pieces were slid along the cables using gravity. When all the elements were in position the transversal and longitudinal joints between the panels were concreted to minimize problems generated by different deflections. The response of the whole system, in terms of stresses and deformations, is carefully monitored to identify and analyse any deviations from the predicted behaviour. This guarantees the good structural performance in the final geometry intended for the system of cables that give the roof its unique form. [45] [23] A more detailed view of the cables can be seen in figure 2.18a.



(a) Detail of the cables in the finalised roof structure



(b) Detail of the cable anchoring to the stiff beam at the ends of the roof

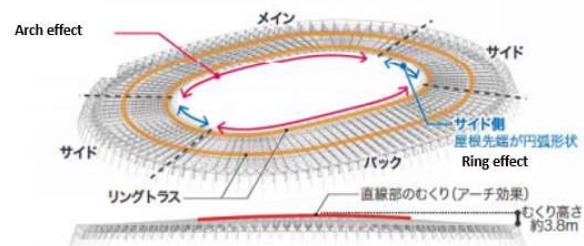
Figure 2.18: Detailed views of the cable configuration

Tokyo National stadium, Tokyo

The Tokyo National stadium has a capacity of 68.000 seats, which can be extended to 80.000 seats. It is under construction right now and is planned to be finished at the end of 2019. The traditional Japanese culture is expressed by actively using domestic timber for the stadium roof, which is possible due to modern technologies. The roof has a hybrid structure that makes use of the strengths of wood and steel. It uses highly durable wood that has been pressure-injected for durability. The demand for a reduction in construction time resulted in a cantilever roof spaceframe system with simple cross sections. The repeated cantilevered roof frame is a triangular truss that connects two upper chords and one lower chord with seven lattices in a 'tree like manner'. The top chord is made of steel and the bracing and bottom chord are made of laminated wood. The steel frames will carry the long-term load of the roof including lighting, speakers, etc. necessary for the stadiums performance. In the case of short-term loads, such as uplift forces induced by wind, the wood and steel frame work together to reduce deformations. The height of the entire stadium is 50 metres and the cantilever is 60 metres in length. The roof also consist of a central ring truss of steel in the middle of the cantilever and a smaller ring truss at the end of the cantilever. Both ring trusses are located at places where the equipment such as lighting and speakers is placed. The roof end at the long sides is lifted to create an arch effect and integrated with the ring truss to create better form resistance of the whole roof since these sides are straight. This can be seen in figure 2.19a and 2.19b.



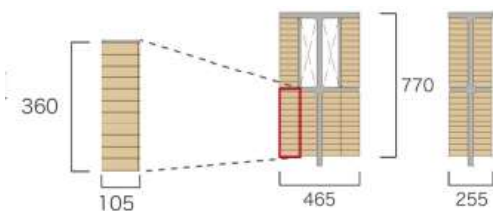
(a) View of the hybrid roof structure



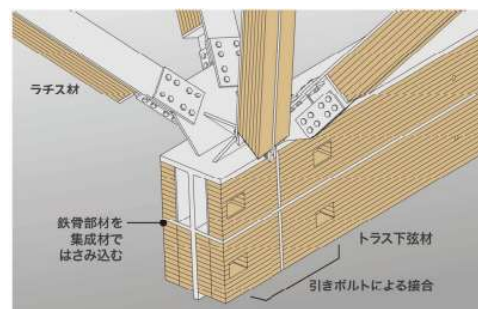
(b) Schematisation of the ring trusses for the ring effect and the slight lift of the long sides for the arch effect of the whole roof

Figure 2.19: Tokyo National Stadium, Tokyo

The cross sections of the lower chords of the trusses can be seen in figure 2.20a. left in the image is the outer laminated piece of wood shown with its dimensions. The total cross section can be seen in the middle, which has dimension 770 mm x 465 mm that includes the inserted steel plate. On the right is the middle section of the cross section shown to distinguish the different dimensions that contribute to the total width of the cross section. The connection between the lower chord and the lattice girders is seen in figure 2.20b. The steel frame and the timber are integrated in the axial direction of the timber by pulling bolts so that the stiffness of the timber works for tension and compression.



(a) Sectional view of the lower chord material



(b) Member configuration of the lower chord with inserted steel plates and the lattice beams

Figure 2.20: Detail of the lower chord of the space truss

The installation of the roof is done by pre-assembling truss units in the field from factory made elements (slotted steel plates and timber elements are already connected in the factory). This results in a simple construction that can be done sequentially in the circumferential direction with a repeated frame to reduce cost and construction time. A roof truss frame made of steel and wood will be erected in two pieces on temporary supports and then connected. A finished frame will also be connected to the adjoining frame with high-strength bolts in the upper chord. The temporary supports can then be removed, as the truss is able to cantilever on its own, after which an early construction of the seats can begin. After the roof is completed, the entire roof is integrated with a ring truss to enhance the co-operability of the roof to resist short-term loads such as earthquake-, snow- and wind loading. This process is schematically shown in figure 2.21.



Figure 2.21: Construction sequence of the roof trusses

2.1.4 Concluding

The design vision of OMA for the New Feyenoord roof structure is defined as an elegant, floating and thin structure putting the primary focus on the experience of the spectators, being a unobstructed group. To fulfil this vision there are four known roof configurations applicable, namely the single span, stress-ribbon, shell roof and the tension/compression ring systems. There are multiple interesting stadiums around the world making use of these systems. All in their own unique way with their one points of attention to create a feasible project. This illustrates that although multiple design restrictions are in place there are still several possibilities to create a feasible project. A further evaluation of these systems in combination with design restrictions of the New Feyenoord stadium is needed. Furthermore, the material aspects of timber need to be incorporated into these evaluations. The next section explores the possibilities of timber as a structural material for long-span structures.

2.2 Long-span timber structures

The analysis of long-span timber structures introduces the design aspects of such structures. The most suitable structural forms are explained and their applicability is presented by accompanying reference projects. In the next section, structural timber will be analysed.

2.2.1 Design aspects

A few relevant aspects of long-span timber structures are stated. These aspects consider the design issues of constructing with the material wood for long-spans. General consideration of creating a realistic design with timber as well as the problems that arise when these spans become very long are addressed.

General design

Timber is less stiff and strong than steel, but has a low density compared with conventional structural materials. This results in efficiency for long-span or tall structures, in which the self-weight of the structure is a significant part of the total load. Especially if these loads are purely resisted in tension or compression, timber performs similar to steel. The strength-to-weight or elastic modulus-to-weight ratio is an example of the mass of material required to achieve a structure in the stated load transfer. Figure 2.22 shows these aspects for conventional structural materials. [53]

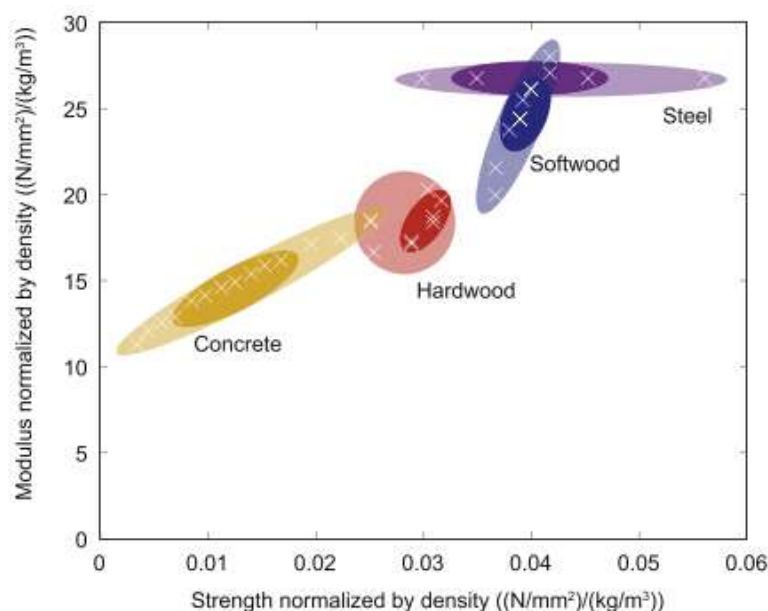


Figure 2.22: Compression strength and modulus of construction materials normalised by density. Design values of strength and stiffness based on the Eurocode design standards for the given materials. [53]

Wood is an anisotropic material where the physical values depend on the fibre directions. The compression and tension strength of timber parallel to the grain is much greater than that perpendicular to the grain, due to the natural characteristics of these cell structures. The material properties are characterised by moisture content, temperature and duration of load as well as inhomogeneity that reduce strength such as knots, deviated grain and fissures. These inhomogeneous factors determine that timber has no or very little ductility in the tensile area, while in compression linear elastic–plastic behaviour can be assumed. [11] [46] [5] [62]

The strength of sawn timber is also a function of species, density, size and form of the member. Strength grading methods have been devised to classify timber using either visual strength grading or machine strength grading methods. The characteristic strength value for all materials is normally defined as the 5% fractile in the distribution of strength. Timbers of similar strength properties are grouped together into a series of strength classes which are defined in the Eurocode or, when the

strength exceeds these classes, by the supplier. This simplifies the design and specification process by enabling designs to be based on defined strength class limits without the need to identify and source a particular species and grade combination. Strength classes are defined for softwoods, hardwoods and engineered wood products. [5] [4] [46]

The durability of a timber structure is, just as the mechanical strength and stability, an essential part for the structures design. The common major risk factors associated with timber are moisture, insects, fungi and ultra violet light. They can instigate a range of durability issues which include deformation of members, premature breakdown of surfaces, fungal and insect decay resulting in a decline of structural performance. Design solutions should prevent the above stated to happen, which can be done by protective detailing, adequate ventilation, protection from moisture, details to accommodate timber movement, and appropriate preservative treatment and finishes. [46] [11] [13] Based on the expected ambient climate to which the members will be exposed throughout its period of use, classification is made into service class 2 for this roof structure. (roofed outdoor) [4]

Transport limits

The Lower weight of timber elements is beneficial for transport and construction. However, restrictions on dimensions for transport negatively influence the structural unity of a timber building. The reduced dimensions of the elements means that the design need to incorporate connections for the individual parts. Each node constitutes a weak point in the cross-section of a timber element. It is not possible to transfer the entire load capacity of the member to the connection. In addition, connections are quite expensive and also affect the deformation behaviour of the structure.[52] The Dutch government sets requirements for the dimensions of objects that are transported on the road. The maximum permitted dimensions of indivisible cargo for which no exemption is required are: [54]

- Length = 22 metres
- Width = 3 metres
- Height = 4 metres

The timber structure should be prefabricated as much as possible for the best structural performance as well as a rapid and easy assembly on site. Timber structures are faster to build and cause less disruption and less waste than a concrete building characterized by the same size. It also takes less energy to create structural timber than that is required for the creation of other structural materials such as steel and concrete. [5] Wood has become one of the newest and most innovative constructive materials thanks to the use of mass timber technologies. Wood elements represent an promising construction material not only for their high strength-to-weight ratio but also for their elasticity that permits an easier site assembly without the need for complex worksite infrastructures.[11] This easy processability is further exploited by the industry by the use of three-dimensional CAD models coupled to robot production by CNC machines. This innovation helps to achieve maximum precision and the ability for automated production of complex geometries. Creating a very high standard for timber structures in terms of accuracy and speed of production. [37]

Design issues for long-span timber roof structures

Fire safety is the first thing people bear in mind when thinking of timber structures. Despite the common opinion, wood has better performance towards fire than other materials such as steel. This is due to its mechanical properties that do not change with high temperature. Although timber is classified as combustible material, a properly designed timber structure performs very well in case of fire. Heavy timber construction has good inherent fire resistance because a char layer is formed during fire that retards the further heat penetration. [46] [11] Although this effect of wood is structurally beneficial, proper design for fire loading is still required due to its combustible nature.

Another aspect of a wooden long-span football stadium roof structure is its lightweight nature that makes it more prone to wind suction than roof structures made up of heavier materials. The wind peak forces are high at the height of the stadium roof and especially as these structures are relatively unobstructed by adjacent buildings. Uplift forces due to wind can be severe for a long-span roof structure made of structural timber. These forces depend on the configuration of the roof system and will be addressed in more depth in chapter 3.2.

Lastly, the young's modulus of wood can generate possible problems connected to vibrations, buckling, instability phenomenon and deformability. Therefore, for timber structures the fulfilment of the service limit state can be more restrictive than the ultimate limit state requirements. [11] The restrictions of these aspects for football stadium roofs might be less severe as the serviceability criteria have less influence in such a roof. The serviceability criteria are based on standards for the appearance of the building, the comfort of users or the functioning of the structure. As the spectators are not in direct contact with the roof and there is a significant amount of space between the two, changes in vibration and deformation are less noticed. The serviceability criteria will be mostly governing for the roof finishing and detailing, not for the structure. [2]

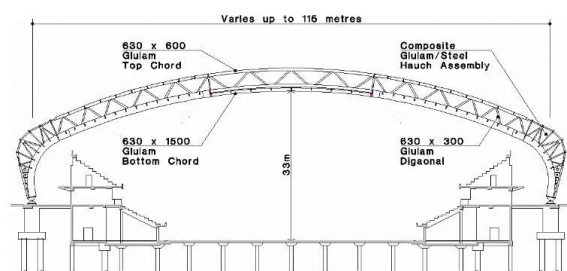
2.2.2 Structural form

Structures are more efficient when loads cause axial forces in the system rather than bending. A uniform internal stress distribution is the result which provides for all of the material in the structural member to be stressed to the limit. Moreover, structures predominantly subjected to axial forces are also more efficient at resisting deformations. The deformations that result from bending forces are commonly larger than those resulting from purely axial forces. [19] Several structural forms are actively used in existing long-span timber structures and therefore underscore their applicability for these structures. These structural forms are presented in this section and accompanied with reference projects to show their applied limits.

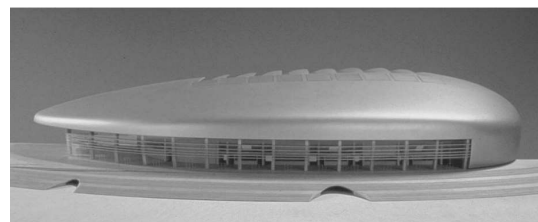
Arch

Arches are structurally beneficial and can span large areas by resolving forces into compressive stresses. It is very suitable for execution in timber, as it can be economically produced in curved forms and with varying depth. Mostly solid sections of constant depth are made, but composite sections of I- or box-shape are beneficial for large spans. The form of the arch should be chosen in such a way that the bending moments are minimised. High horizontal support reactions are a result of this configuration and need to be resisted. Larger spans usually demand that the arch is manufactured and transported in three or more parts, which are joined rigidly on site. In such a case, a system with hinges placed only at the abutments is chosen (two-hinged arch). The possibility of both in-plane and out-of-plane buckling failure is high for slender arches. [19]

An example of a long-span timber arch is the Lisbon Multi-use Arena which can be seen in figure 2.23a. The roof structure of the main hall has the longest one-way span of glued laminated timber in the world with a maximum of 115 metres between bearings. This arena consists of sixteen glued laminated timber two-hinged, arched, portal truss frames spaced 9.0 metres apart. The central trussed section of each portal frame is completed by glued laminated timber diagonal members in a Warren truss configuration. The main hall roof structure is completed by longitudinal glulam purlins, diagonal glulam braces and a plywood and subframe diaphragm in the roof surface to distribute lateral wind and seismic loads. The arches don't follow a pure parabolic or circular form due to architectural reasons. There is chosen for a sort of portal arch to reduce the forces on the supports.



(a) Structural schematic of the Arena



(b) External view of the Arena

Figure 2.23: Lisbon Multi-use Arena, Lisbon

Box girder

This form works similar to a spatial truss, but has a higher in-plane stability. When subjected to bending moments the top part of the box works in compression, the bottom part in tension and the sides are subjected to in-plane shear forces. The benefit of a box girder over a spatial truss is its torsional resistance. If asymmetric loading over its short axis is applied, a box girder is more rigid than an open configuration. Furthermore, wide flanges allow for a large span-to-depth ratio, but will result in more structural material. A closed box girder allows for better protection of the structural components by using a covering material on the outside of the box.

The Maicasagi bridge in Nord-du-Québec is the longest single-span wooden bridge in the world. It has a bold structural system that combines CLT with glulam to create two huge box girders. This innovative structure enables 1800kN logging trucks to travel over a 68 metre long clear span. Due to manufacturing (maximum glulam length of 24 metres) and shipping limitations two joints are added at one third of the span to avoid a joint in the centre of the bridge. The primary structure consists of two large box beams that are each made up of block-glued (4 members) glulam flanges and CLT webs as can be seen in figure 2.24b. The transportation joints were made with self-tapping wood-steel screws for the bottom flange (tension) and bolts for the top flange (compression).

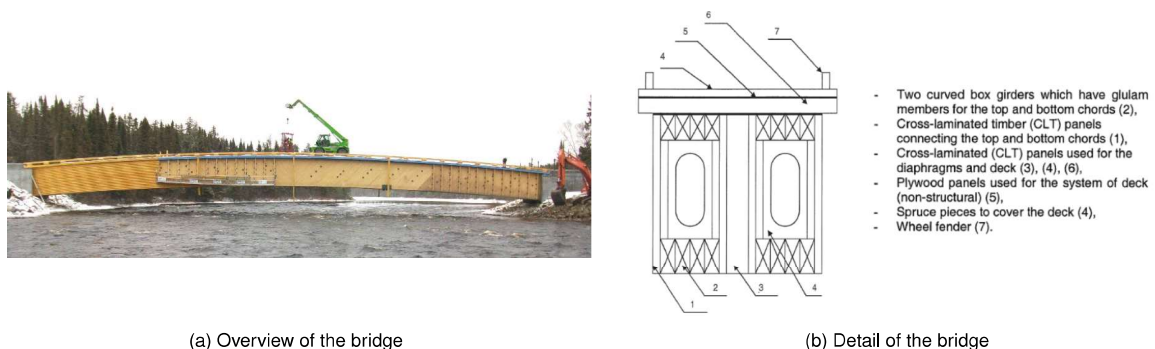


Figure 2.24: Maicasagi bridge, Nord-du-Québec

Truss

Trusses adopt the use of wood members in tension and compression connected in a single plane to create a rigid structure. Space trusses have members and nodes that extend in three dimensions, also with only tensile or compressive forces in the members. The height between the top and bottom chord of a truss is what makes it an efficient structural form and reduces the required material substantially in contrast to a solid beam of equal strength. However, a deeper truss will require extra material for the diagonals so an optimum depth of the truss will maximize the efficiency. Different configuration of the diagonals influence the internal force to be either compression or tension. This is a variable that can be adjusted to achieve the desired forces in the connections. Truss structures are generally characterised by a high number of connections which are expensive and time consuming. Another aspect is to have a low slenderness ratio of the compression members to reduce their risk to buckle. The elements in compression should therefore be made as short as possible or braced at regular intervals. The great advantage of a truss is that the individual members can be easily fabricated from single elements, namely solid timber sections with small dimensions. [37] [19] [18]

The Trade Fair Hall 11 in Frankfurt is one of the most impressive examples of the use of timber for long-span designs. The roof to hall 11 is a simply supported structure with a span of 79 metres and a 17.4 metres cantilever on each side. The Trusses are made from glulam and are max. 6.6 metres deep. They have round steel bars as diagonals and smaller bars in the opposite direction as these are only required for the uplift forces, which can be seen in figure 2.25b. The top chords of the trusses are linked by secondary beams made of timber and span 10.4 metres. Each top chord was prefabricated in two 39 metres long segments and assembled on site. Three segments were necessary for the bottom chord of which the middle one was 50 metres long. The timber grade that is used is GL 32c, which is a strength grade of glulam. The connection to transfer the tensile forces consists of steel splice plates and diagonal bolts. The next section elaborates more over this grading and connections. [36]

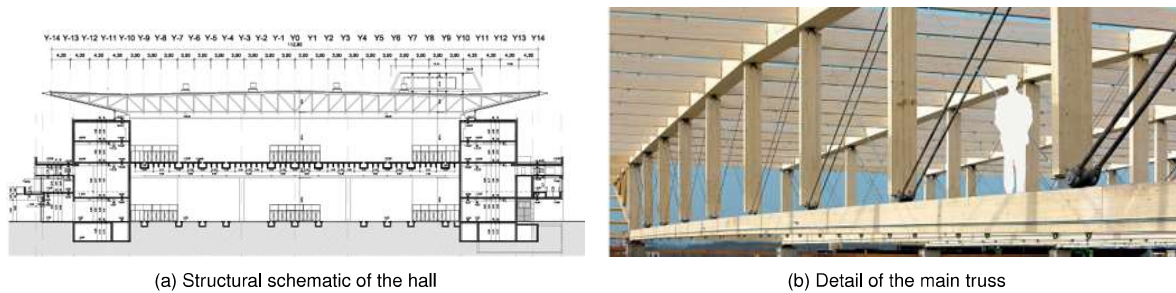


Figure 2.25: Trade Fair Hall 11, Frankfurt am Main

The Anaklia-Ganmukhuri bridge in Georgian Republic is another impressive example of a long-span truss configuration. It is one of the longest pedestrian timber bridges in Europe, with total length of 505 metres. The primary bridge structure is a triangulated truss, roughly 3.5-metres-high, constructed with glulam chords and diagonals and a 65mm thick timber floor diaphragm. The complex node that enables this configuration is seen in figure 2.27b. The bridge consists of ten spans, with two spans cable supported from concrete pylons and beam lengths of up to 48 metres and the longest free span is 84 metres. The patented 'Hess Limitless' connection (see section 2.3.2) was implemented at all chord splices which results in 141 joints with this connection. This made it possible to reduce the Glulam parts to a maximum length of 13.5 metres, so that expensive special transportation could be avoided. The decision was made to prefabricate all timber components and connections in Germany to ensure quality construction on site and ease of shipping. All other truss connections consisted of standard dowel and slotted steel plate connectors augmented with confining bolts as can be seen in figure 2.27a. [32]

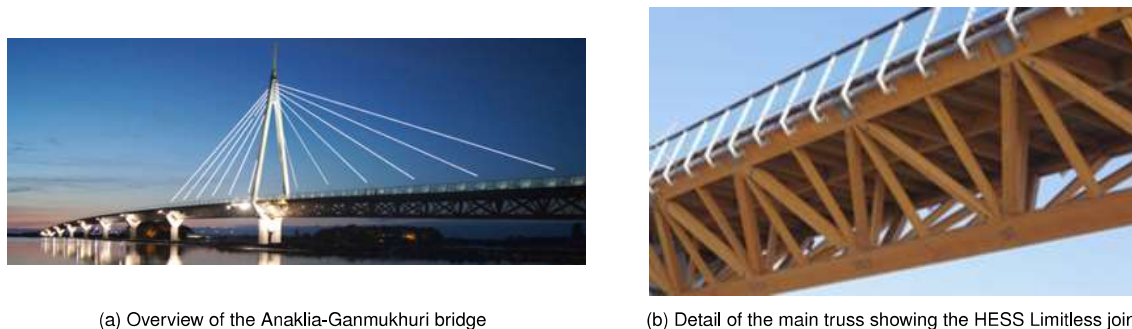


Figure 2.26: Anaklia-Ganmukhuri bridge, Anaklia

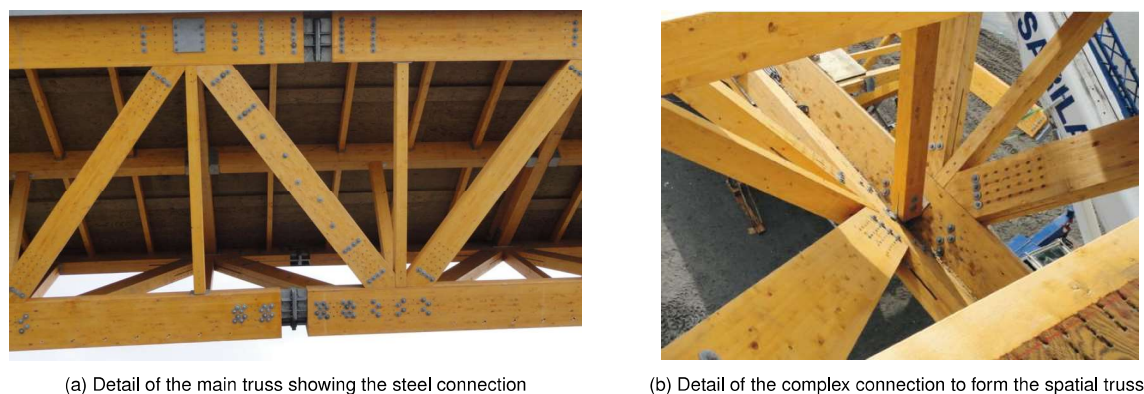
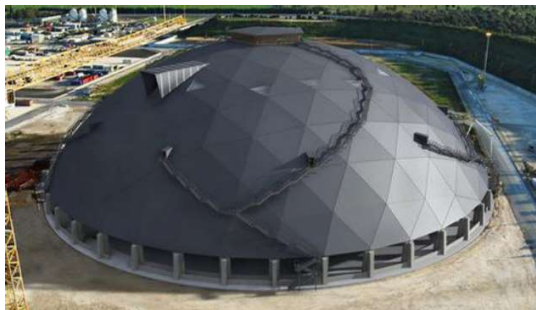


Figure 2.27: Detailing of the Anaklia-Ganmukhuri bridge, Anaklia

Shell

True shell structures are not possible in timber due to their anisotropic behaviour and are therefore always made up from three dimensional framework. Common configurations for long-span timber systems are the gridshell and dome structure. These are spatial structures made of an assemblage of linear members interconnected to each other, which resist applied loads along their lengths and at the connections. Another interesting spatial structure for long-span football stadium roof structures is the cable net, but has never been established in timber. A typical dome structure is the radial dome which consists of a number of curved members arranged radially that are hinged at their ends. The members are laterally supported by members arranged circumferentially (circular hoops). Mostly, a compression ring is placed at the top of the dome and the bottom points of the ribs are connected to a tension ring, typically made of pre-stressed concrete. Modern domes are designed according to a geodesic geometry, equal triangles that lie on the surface of a sphere or a hemisphere. The triangular elements have local rigidity and therefore do not need any additional bracing. Geodesic domes are in general more economic, easier to erect and have superior load-carrying capacity over similar radial rib domes. Timber gridshells have irregular complex double curved shapes created from standard linear elements in contrary to the regular shape of a geodesic dome. It is possible to create this complex shape from a flat grid due to the low torsional stiffness of timber. [19] [29]



(a) Overview of one of the two geodesic dome structures



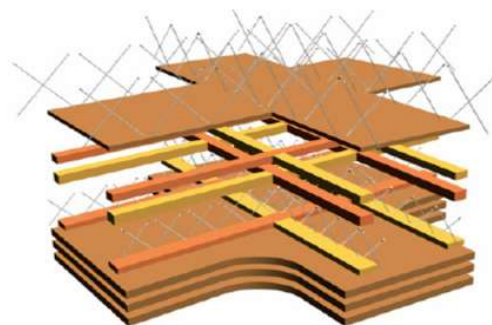
(b) Construction phase of one of the two geodesic dome structures

Figure 2.28: Geodesic domes, Brindisi

The geodesic domes in Brindisi are the largest timber structures of their type in Europe consisting of glulam ribs. Both domes are 143 metres in diameter and 46 metres in height. The erection occurred without the need of building a temporary supporting tower in the centre of the dome area, which is typical for radial rib dome structures. The joints, which consists of steel plates and inclined self-tapping screws, can resist bending moments and shear forces that arise during the construction phase, when the members of the triangulated dome are cantilevering out. Figure 2.28b shows the erection of the dome, which happens unsupported due to the steel connections.



(a) Overview of the shell roof with the skylights



(b) Detailing of the roof cross-section

Figure 2.29: Elephant House, Zurich

The Elephant house in Zurich has a free-form timber shell roof with a diameter of about 80 metres, see figure 2.29a. It corresponds to a shallow, “deformed”, parabolic dome with an off centre crown about 18 metres above the ground. It is a highly sophisticated shell structure using CLT, timber ribs, grouted steel plates, self drilling screws and LVL, resulting in a honeycomb-type hollow box as can be seen in figure 2.29b. It is described as a seven-part, flexible, composite cross-section with a total depth of 540 mm. This free-form, flat roof design requires no additional columns inside for support and has 271 different shaped skylights. The primary loads from the huge timber frame and shell are absorbed circumferentially by a prestressed, free running concrete beam with a total length of approx. 270 metres. Its cross-section is approximately 480 mm x 2000 mm and adopted to the curvature of the structure.

Spaceframe

A space frame is a grid of structural members which is three-dimensional in shape and stable in three dimensions. The difference with the above stated shell structures is the configuration of the elements. The shell structures have their structural elements in plane and a spaceframe consists of a top grid and a bottom grid interconnected with diagonals. The top and bottom side of the grid are usually flat and supported at its edges only. These structures consist of a large amount of nodes with lots of elements meeting each other.

The Allianz Riviera stadium in Nice makes use of a cantilevering spaceframe as explained in section 2.1.3. The bottom grid, which is in compression, is made out of timber elements and the top grid, which is in tension, is constructed with steel elements. The diagonals connecting the two grids are also made out of steel. Large amounts of timber in the compression zone of the roof structure brings the advantage of timbers high compressive strength in relation to its self-weight, and thus reducing the dead load of the entire structure. The nodes of the timber lattice are not rigid and permit a certain amount of play. A detail view of the spaceframe is seen in figure 2.30.



Figure 2.30: Detail view of the spaceframe of the Allianz Riviera stadium in Nice

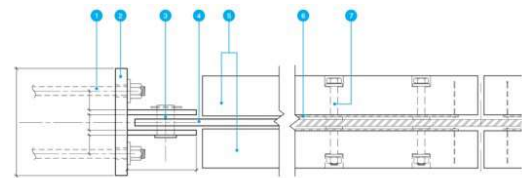
Stress-ribbon

A stress-ribbon configuration making use of timber cables reduce the risk of going into unstable compression as they provide geometrical stability. Therefore, it is normally not necessary to stiffen the cable-shaped structure by adding mass or pre-tensioned cables. However, as the structural efficiency results in slender elements it is still a point of interest. Also, if the timber elements follow a curved configuration they are initially prestressed during manufacturing (bending of the laminates) to create the curvature. Timber members possess inherent bending stiffness and have excellent resistance to tensile stresses. Especially in terms of the ratio between tensile strength and material density, which make it particularly adequate for cable-shaped structures. More of the material can be used to carry the loads instead of carrying its own self-weight. The more complex tension system, a cable-net structure, made in timber would look like an inverted gridshell. The connections necessary to create a cable-net structure made up out of timber elements are very complex. [19]

The Grandview Heights Aquatics Centre in western Canada takes advantage of wood's flexural ability by means of a long-span suspended roof. The continuous glulam beams show the result of a combination between form and function creating a slender and lightweight roof. This slender catenary roof spans the 55 metre and 45 metre gaps with pairs of 130 mm x 266 mm GLT 'cables' at 800 mm centres, reducing the effective structural depth of a steel truss solution by 90%. A central support is located between the gaps and at each end of the building are seven reinforced concrete columns. Reinforced concrete edge strips pick up the tensile forces from the suspension members and transfer them into the central and outer supports. The wood cables were sized to have sufficient strength to resist snow loads and self-weight in tension, and just enough strength to resist wind uplift as skinny compression arches for a perfect balance. In addition, a shear-transmitting connection of the double layered plywood boards with the Glulam members provides composite action. The warped roof geometry, as well as the damping effect of glued roof insulation would sufficiently mitigate the potential for resonance caused by dynamic wind excitation. Due to transport restrictions and to save time, longitudinal joints consisting of 22-mm-thick steel plates were each connected to two pairs of Glulam members by a total of six bolts. A lifting frame was used for the short span while the long-span beams were lifted with two cranes. The whole roof including plywood layers was erected in 12 days. The cables during construction can be seen in figure 2.31a and the schematic drawing of the details is seen in figure 2.31b.



(a) The roof structure during construction



1. $\varnothing 29\text{mm} \times 1500\text{mm}$ steel rod (typ. 2) anchored into concrete slab and 25mm nonshrink grout (not shown)
2. 38mm steel plate
3. $\varnothing 57\text{mm}$ pin and 16mm plate (typ. 2)
4. 22mm \times 200mm steel plate
5. 5" \times 10" deep Douglas fir glulam
6. 220mm \times 800mm nailing plate (typ. 2)
7. $\varnothing 25\text{mm}$ ASTM A325 bolts, hot-dipped galvanized (220mm o.c.) with wood plugs

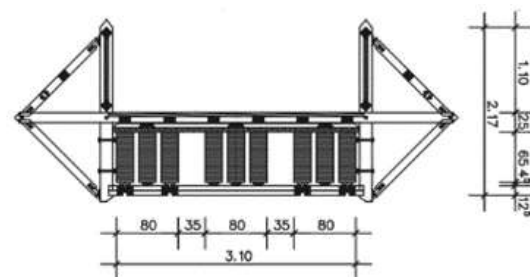
(b) Detail of the connection at the supports and the internal connection of the cables

Figure 2.31: Grandview Heights Aquatics Centre, Surrey

Another stress-ribbon example exploiting the inherent geometrical stiffness of timber cables, is the timber bridge crossing the Rhein-Main-Donau Canal in Essing. It consists of nine continuous Glulam beams of 220 mm x 650 mm over four spans, of which the longest is 73 metres long. The shape is determined by keeping the maximum normal force approximately the same in each span under a uniformly distributed load. The amount of occurring bending moments is minimised by a low height-width ratio of the bridge cross-section so that almost only internal tensile forces occur. The cross-section can be seen in figure 2.32b. The Glulam beams were produced in lengths of 40-45 metres and connected on site by means of finger joints. Lateral stiffness of the bridge is provided by a system of diagonal bracing members underneath the bridge deck as can be seen in figure 2.32a.



(a) Bottom view of the stress ribbon bridge with diagonal lateral bracing



(b) Cross sectional view of the bridge

Figure 2.32: Rhein-Main-Donau Canal Bridge, Essing

2.2.3 Concluding

Additional requirements for wooden structures are defined in Eurocode 5. The design and calculation models for different limit states should, where applicable, take into account the following points.

- Different material properties per direction (such as strength and stiffness)
- Different time-dependent behaviour of the materials (load duration and creep)
- Different climate conditions (temperature and moisture variations)
- Different design situations (construction phases and changes of supporting conditions)

Due to these differences in wood and in its circumstances, there are modification and material factors that accommodate for this behaviour. The quantitative values of these factors are presented in the next chapter.

Though for the initial structural design it is concluded from the reference projects and the already applied structural systems in timber, that all systems are interesting and a further analysis is required to distinguish the most promising solution.

2.3 Structural timber

The analysis of structural timber introduces the most suitable engineered wood products for a long-span football stadium roof structure. The best possible new type of connections are presented to create the roof system. In the next section, the analysis phase will be concluded.

2.3.1 Engineered wood products

Structural timber makes it possible to build longer spans. This due to its higher strength and stiffness properties and the fact that these products can go beyond the dimensional limits of tree trunks themselves. Modern methods of production and new wood-based materials create possibilities for even longer spans and more efficient load-bearing structures in timber. They have good dimensional stability, durability and better fire resistance than other construction materials. The most common timber products on the market suitable for long-span timber structures are glued-laminated timber, laminated veneer lumber, and hybrid products. The advantages of engineered wood products as structural material are summarised below: [61] [63]

- Enhanced strength and stiffness
- Increased size and scope of application
- Reduced moisture content
- Dimensional consistency
- Low energy usage in manufacturing
- Easy workability by both hand and mechanical tools
- good fire resistance

Glued-laminated timber

Glued-laminated timber (GLT or Glulam in short) is made by gluing together a number of graded timber laminations with their grain parallel to the longitudinal axis of the section. Laminations are typically 25 mm or 45 mm thick, but smaller laminations can be used for specialised sections. Large defects that reduce strength in the individual laminations are taken away before bonding, creating a more homogeneous material. Within the manufacturing process it is possible to create beams and columns which are straight, curved, double curved or twisted. Furthermore, The beam sizes can have unlimited width and length due to the continuous laminations by means of finger-jointing, but are restricted by transportation limitations. Normally, the laminates are dried to around 12-15% moisture content before being machined and assembled to diminish damage caused by shrinkage. After the drying, the glue is applied and the beam is pressurised at right angles to the glue line and held until curing of the adhesive is complete. Thereafter they are cut and shaped before the required preservative and finishing treatment is applied. All this is done in a fully automated process for standard dimensions, which are than very accurate. [13] [16]



Figure 2.33: Glued-laminated timber

Glued-laminated timber is divided into strength classes depending on the individual strength properties of the laminations. There are multiple glulam products available for service class 2. The biggest distinction is between laminations from softwood and hardwood. The first one is more widely available, easier to process and therefore cheaper than the latter, which can result in its turn in very slender elements due to its higher mechanical properties. In table 2.1 are the strength properties for GL32h (softwood)[27] GL35c (HESS Limitless) and GL75 (hardwood) presented.

Table 2.1: Characteristic strength and stiffness properties according to NEN-EN 14080:2013 and HESS TIMBER GmbH

Property			Softwood		Baubuche
			GL 32h	GL 35c	GL 75
Bending strength	$f_{m,g,k}$	N/mm^2	32	35	75 ^a
Tensile strength	$f_{t,0,g,k}$	N/mm^2	25.6	19.5	60 ^b
	$f_{t,90,g,k}$	N/mm^2	0.5	0.5	0.6
Compression strength	$f_{c,0,g,k}$	N/mm^2	32	26.5	49.5 ^c
	$f_{c,90,g,k}$	N/mm^2	2.5	3.0	12.3 ^c
Shear strength	$f_{v,g,k}$	N/mm^2	3.5	2.5	4.5 ^d
Modulus of elasticity	$E_{0,g,mean}$	N/mm^2	14200	13700	16800
	$E_{0,g,05}$	N/mm^2	11800	11400	15300
	$E_{90,g,mean}$	N/mm^2	300	420	470
Shear-modulus	$G_{g,mean}$	N/mm^2	650	780	850
Density	$\rho_{g,k}$	kg/m^3	440	410	730
	$\rho_{g,mean}$	kg/m^3	480		800
^a For flatwise bending, the characteristic strength value may be multiplied with factor $k_{h,m} = (600/h)^{0.1}$, where h is the height of the beam cross-section in mm.					
^b The characteristic tensile strength may be multiplied with factor $k_{h,t} = (600/h)^{0.1}$, where h is the larger side length of the beam cross-section perpendicular to the longitudinal axis in mm.					
^c The characteristic compressive strength may be increased for $n > 3$ with $k_{c,o} = \min(0.0009 * h + 0.892; 1.18)$, where h is the height of the beam cross-section in mm and n is the number of lamellas of LVL.					
^d The characteristic shear strength may be multiplied with factor $k_{h,v} = (600/h)^{0.13}$, where h is the height of the beam cross-section in mm.					

The available dimensions of glulam depend on the manufacturer. The Hasslacher Group is such a manufacturer and they provide the following dimensions for strength class GL32h and GL75, which are stated below. [27] [28] [59]

	GL32h	GL75
Heights	80 to 1280 mm in 40 mm steps. Special components up to 4000 mm.	80 to 600 mm in 40 mm steps. Special components up to 2500 mm.
Widths	60 to 200 mm in 20 mm steps. Any desired extension possible through block bonding.	50 to 300 mm. Extension up to 1200 mm through block bonding.
Lengths	Up to 27 m. Or up to 40 m as special components.	Up to 18 m. Or up to 36 m as special components.

Several innovations that have taken place, or are taking place in the field of GLT are HESS Limitless, Hybrid glulam, FRP glulam, and Block glulam. HESS TIMBER GmbH has developed the **HESS Limitless** splice joint, a patented high-strength on site beam joint (GL35c) of which its strength characteristics can be seen in table 2.1. This joint is further elaborated in section 2.3.2. **Hybrid glulam** combines higher grade material with lower grade laminates leading to a better utilisation of the used material in a cross-section. Also stronger beams can be obtained than the leading strength classes by making use of hardwood or LVL. **FRP Glulam** uses the high tension resistance of fibre composites

to strengthen glulam beams at locations where high tensile forces occur. Other advantages of this combined gluing are the good force transfer from the fibres to a large area of the timber resulting in higher stiffness and more ductile behaviour. Lastly, **Block glulam** extends the dimensional and load carrying capacity limits of GLT by bonding GLT beams together creating even greater cross-sections. [37] [61] [8] [19] [33] [39]

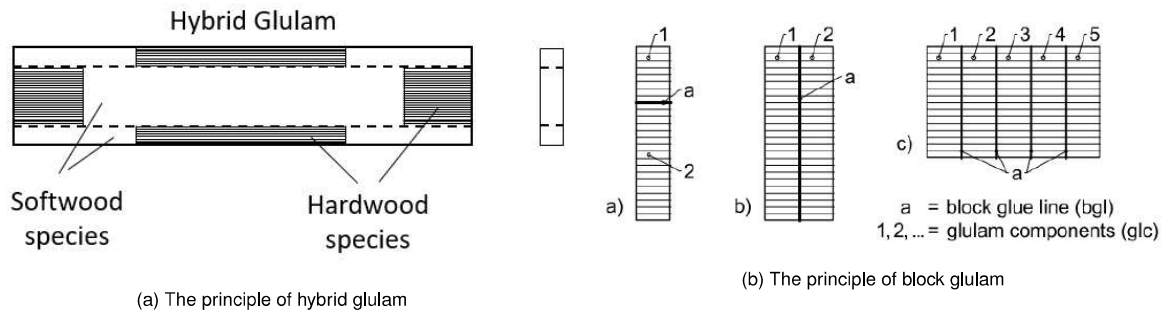


Figure 2.34: Strength improvements for glued-laminated timber

Laminated Veneer Lumber

Laminated veneer lumber (LVL) is a structural member manufactured by bonding together thin vertical softwood veneers of max. 7 mm thick with their grain parallel to the longitudinal axis of the section, under heat and pressure. The good dimensional stability of LVL compared with solid wood and its optional moisture resistance have encouraged the use of this wood-based product in long-span roof structures in recent times. It is often used for high load applications to resist either flexural or axial loads or a combination of both. The LVL also has higher strength properties than glued-laminated wood and can be used in for example trusses. The peeling of logs, which is part of the production process, results in a better spread of defects in the material. This will give the product an increased strength, especially in the tensile resistance of the material.



Figure 2.35: Laminated Veneer Lumber

A common manufacturer of softwood LVL in Europe is Metsäwood. The softwood product is commonly known as Kerto, which is supplied in two types, namely Kerto-S® (Straight) and Kerto-Q® (Quer). The difference can be found in the directions in which the veneers are orientated. For Kerto-S® all veneers are orientated in the same direction, as for Kerto-Q®, approximately a fifth of the veneers is orientated in transverse direction. The latter is mostly used in plate material and the Kerto-S is used for beams and columns. The strength properties of Kerto-S can be found in 2.2.

Table 2.2: Characteristic strength and stiffness properties according to MetsäWood and Pollmeier

Property			Kerto-S	Baubuche-S
Bending strength	$f_{m,g,k}$	N/mm^2	44	75 ^a
Tensile strength	$f_{t,0,g,k}$	N/mm^2	35	60
	$f_{t,90,g,k}$	N/mm^2	0.8	1.5
Compression strength	$f_{c,0,g,k}$	N/mm^2	35	57.5
	$f_{c,90,g,k}$	N/mm^2	6.0	14
Shear strength	$f_{v,g,k}$	N/mm^2	4.1	8.0
Modulus of elasticity	$E_{0,g,mean}$	N/mm^2	13800	16800
	$E_{0,g,05}$	N/mm^2	11600	14900
	$E_{90,g,mean}$	N/mm^2	350	470
Shear-modulus	$G_{g,mean}$	N/mm^2	600	760
Density	$\rho_{g,k}$	kg/m^3	480	730
	$\rho_{g,mean}$	kg/m^3	510	800
^a Values valid for $h \leq 300$ mm. For $300 < h \leq 1000$ mm, the characteristic strength value must be multiplied with factor $k_h = (300/h)^{0,12}$, where h is the dimension of the cross-section relevant for bending stress in mm.				

The available dimensions of LVL depend on the manufacturer. MetsäWood and Pollmeier are such manufacturers and they provide the following dimensions for Kerto-S and Baubuche-S, which are stated below.

	Kerto-S	Baubuche-S
Heights	200 mm to 600 mm in approximately 40 mm steps.	100 to 1820 mm.
Widths	27 mm to 75 mm in 6 mm steps.	40 mm. 60 mm on request
Lengths	Up to 25 m.	up to 18 m.

Other configurations of LVL that are of interest are Hardwood LVL, and Parallel Stranded Lumber. **Hardwood LVL**, known as BauBuche, is a laminated veneer lumber made from locally sourced beech manufactured exclusively by Pollmeier of which its strength characteristics can be seen in table 2.2. The exceptionally high strength of BauBuche allows structures with significantly slimmer dimensions, compared to softwood materials. It is created with a cost-efficient manufacturing technology placing it at the same price level as conventional softwood structures. Pieces of this hardwood LVL are used as lamellae for the hardwood glulam stated in the previous section, giving it its superior properties. Furthermore, BauBuche has a high surface quality to make it ideally suited for visible construction elements. **Parallel Stranded Lumber (PSL)** is produced from strands, which are dried to a moisture content of 3% and cut into thin long strands. The defects that reduce strength characteristics are diminished creating a higher strength in compression and shear than Kerto LVL. However, PSL is not produced in Europe, but is instead imported from North America. [50] [37]

2.3.2 New connections

New specialised jointing methods make way for more reliable load-bearing structures in combination with efficient methods of connecting timber elements to create longer spans. To obtain long elements for large spans, either truss structures or splicing of elements are necessary. This is done by new gluing techniques and a constant further development in the use of three-dimensional CAD models coupled with digitally controlled robot production exploiting timbers easy processable nature. Hereby, achieving maximum precision and being ideal for automated production of complex geometries and repetitive structures. Resulting in a very high standard for timber structures in terms of accuracy and speed of production. The most interesting types of connections and jointing techniques for a long-span timber structure are self drilling fully threaded screws, self drilling dowels, the Sherpa system, the BVD system, the Hess Limitless splice joint, steel fasteners fixed with adhesives, and the elastic glue joint. Traditionally the strengthening of connections is a possibility as well, whereas new types

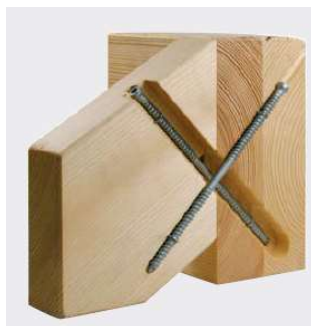
of strengthening are also under exploration. The above stated connections are divided into three categories: mechanical connections using steel, glued connections and strengthening connections. [37] [41]

Mechanical connections using steel

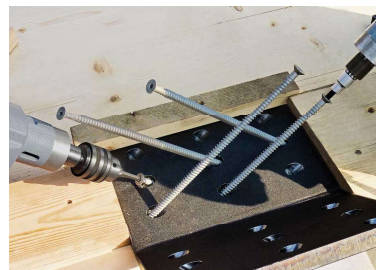
Laterally loaded joints with mechanical fasteners undergo significant deformation, unlike rigid glue joints. These displacements must be taken into account resulting in increased calculation effort, but a semi-rigid joint can be also deemed beneficial. It can withstand a higher degree of deformation before failure, resulting in more ductile failure and therefore more overall safety. Mechanical connections allow for plastic deformation for better load distribution or creep may result in this behaviour, which can help relieve highly stressed areas. The elastic-plastic behaviour of laterally loaded timber joints with mechanical fasteners is due to the interaction between the plastic deformation of the fasteners and the embedment of the wood surrounding the fastener. Particularly dowel-type fasteners have this behaviour if specific edge and end distances and spacings are maintained to avoid brittle failure. The required minimum distances guarantee that almost all brittle failure modes like splitting can be prevented. [16] [36] [13]

Self-drilling full-threaded screws

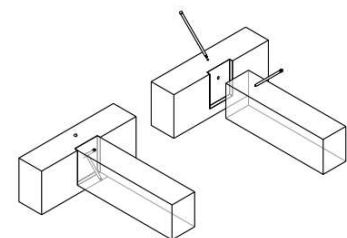
Fully threaded screws are a good option for timber connections as they are easy to install and exhibit a high shearing-off strength. The thread of the screws reduces the slip in the connection by pulling the structural elements together and the pull-out strength is increased. The screw connection can also be applied for connections in which large forces occur. The load bearing capacity is only limited by the potential tensile failure of the screw. They are for instance used to attach diagonal members to the top and bottom chords of trusses. The screws can be driven directly into wood or wood-based products using special tools with high torques. This makes them a structurally efficient connection for producing connections on site. The high-tension capacity (pull-out resistance) of the screws can be exploited when they are driven into the timber at an angle (permissible angles are currently 45°-90°). The outer thread diameter d is not less than 3.0 mm and not greater than 14.0 mm. The overall length of the screws is ranging from 18 mm to 1500 mm. [13] [37] [35] [21] [51]



(a) Self-drilling WT double threaded screw connection



(b) Self-drilling WR full-threaded screws



(c) Beam to beam connection using self-drilling screws at an angle to the grain

Figure 2.36: Self-drilling screw connections

Dowel type fasteners with slotted in steel plate

Timber elements are connected via slotted-in steel plates which are mounted to timber by dowel type fasteners. Incorporated steel plates into the timber cross-section have durability benefits due to a reduction of weathering exposure of the steel/timber interface. The steel plates form a more severe restraint condition for the dowel than the wood does. They have the effect that a plastic hinge is formed in the dowel where the steel plate is placed with an increased resistance of the joint as a result. For normal dowel and bolted connections with slotted steel sheets, the steel and timber elements are drilled separately, which must be done with great precision to reduce resulting additional deformations. Self-drilling dowels do away with the challenges of precision assembly required for other connection solutions. Furthermore, a self-drilling dowel has good fire safety as they are recessed into the timber and can be closed by wooden caps. The self-drilling dowels made in carbon steel are available with

diameters of 5 mm and 7 mm and can be driven in one go through the timber elements and up to three inserted 5 mm steel plates of S235. Generally, higher number of thinner plates and lower diameter of dowels provide a smoother stress distribution and more ductile behaviour. A joint using normal dowels is usually built up of several steel plates of thickness 8 or 10 mm and dowels of diameter 10, 12 or 16 mm. A bolted connection has greater possibilities for large diameters and lengths. [13] [18] [52] [35]

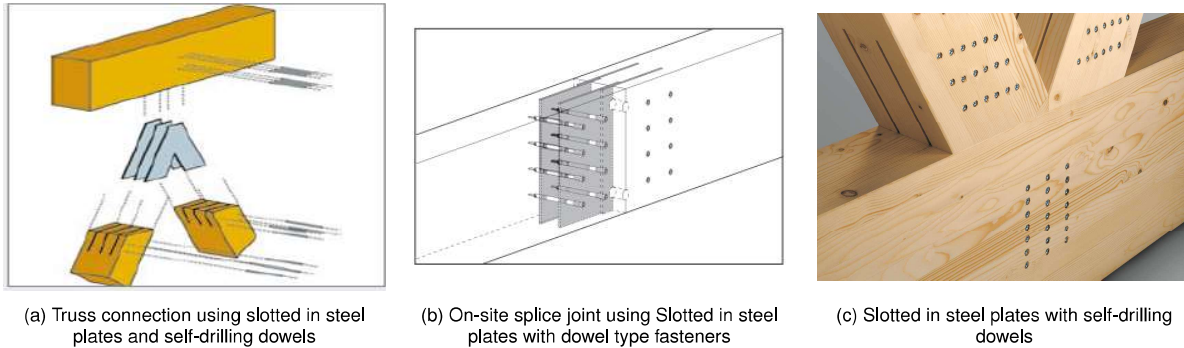


Figure 2.37: Dowel type fasteners with slotted in steel plates

BVD Anchor Bolt

A massive cylindrical steel or gusset anchor bolt with half circle side holes is inserted in a centric drilled hole in the timber parallel to the grain. The bolt is orthogonally anchored by dowel type fasteners with a diameter of 16 mm through these side holes. A rigid load transfer is achieved by injection of a high-strength, non-shrinking cement grout via infill holes in the anchor bolt that results in a gapless contact between anchor bolt and fasteners. Several anchor bolts can be placed in parallel in large joint; this is, however, not covered explicitly by the technical approval at present. The BVD anchor bolt may only be loaded in its axial direction by external loads, shear force due to self-weight of the timber is allowed. Depending on the magnitude of force, anchor bolts differ in length, diameter and amount of incorporated dowels (4-24 pieces). The two-directional placement of the dowels ensures a more effective load transfer than with slotted-in plates and dowels, resulting in a higher capacity. Furthermore, the net cross-sectional area is also larger since pre-drilled holes for anchor bolts occupy less area than slots for steel plates. The same joint efficiency is reached as for normal dowels connections, because of premature splitting of the timber. This can be enhanced by lateral self-drilling screws as is elaborated in 2.3.2 resulting in an efficiency factor of $\eta = 0.7 - 0.9$. This will exclude premature splitting and common failure modes are now fastener yielding, block shear or tension failure of the wood. The system is very efficient for high-tension connections, to carry direct axial loads. [18] [9]

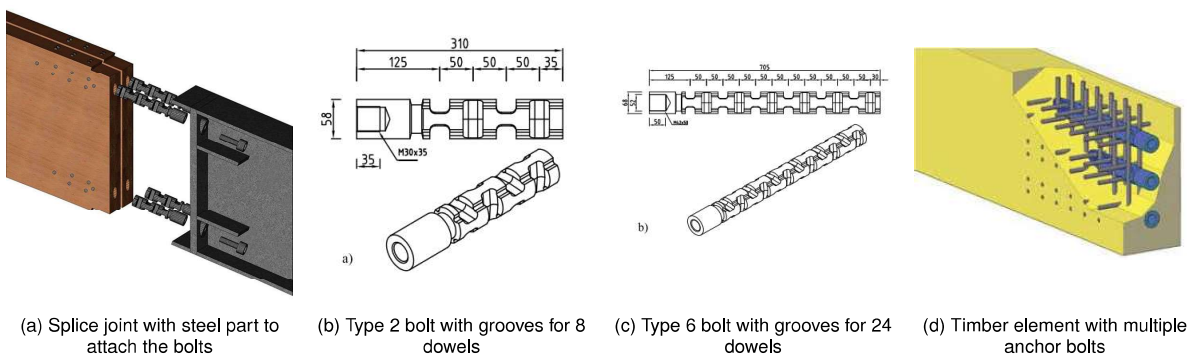


Figure 2.38: BVD anchor bolt

SHERPA system

The SHERPA system connectors consist of shaped aluminium or steel parts with milled tapering grooves or elongated holes in one part and matching pins or tongues on the other part. The metal system elements are secured with fully threaded or system screws to the timber elements. The system elements simply slot together on-site with a possibility to be further secured with screws. Highly efficient connections are obtained by driving the screws at an angle to the grain with a characteristic load-carrying capacity of up to 300 kN in shear and up to 60 kN in tension. The connection can also be protected against uplift by specially designed locking screws. Besides the Sherpa system there is the quite similar Megant system, which can carry up to 500 kN in shear and up to 60 kN in tension. [51] [25] [40]

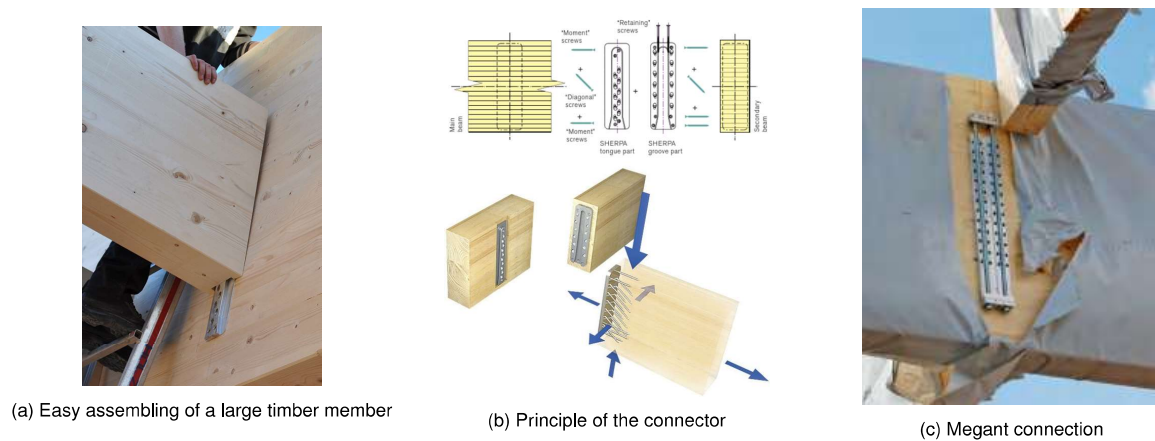


Figure 2.39: Sherpa system connector

Joist bearing

The joist bearing is a single-piece system connector suitable for invisible load-bearing timber-to-timber connections. The connectors are screwed or bolted to one of the timber members and then attached to the second member by means of a dowel-slotted plate connection. The connection is suitable for applications under various angles and uses a split dowel driven into the beam that engages in the slot in the connector bracket, holding the beam in place to ensure easy installation of the beams. The joist are available up to a length of 960 mm and have a plate thickness of up to 12 mm.

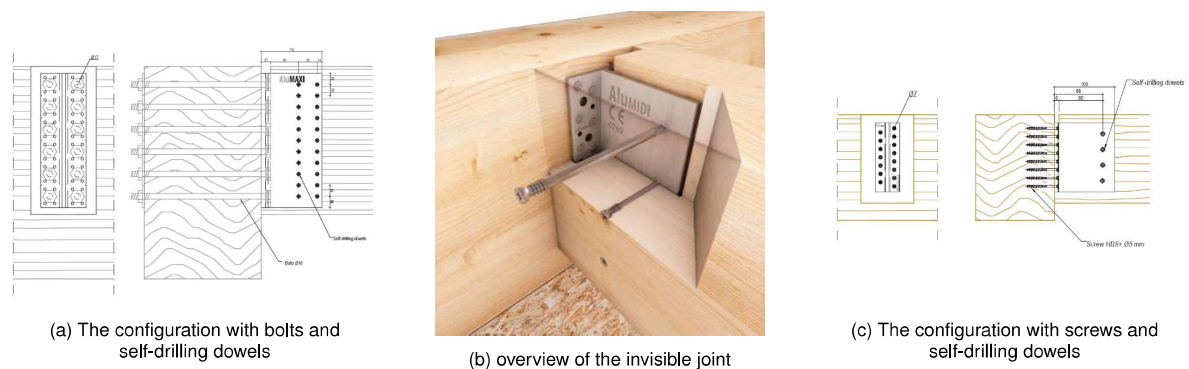


Figure 2.40: Joist bearing

Cast parts

Cast parts are suitable for larger numbers of individually designed connections. They are used in complex nodes transferring high forces, which makes them costly. However, for very big structures this effect is less severe.

Besista rod system

BESISTA® stands for Betschart safety rod systems, which are high-grade tension and compression rod systems. They have a whole range of steel elements that ensure stability in structural systems. Due to the hot-dip galvanized finish a very high, long-term corrosion protection of the elements is ensued. Individual rod lengths of up to 15 metres are available for diameters greater than 16mm. Extension and tensioning sleeves are used for longer rods. Furthermore, they have compression rods made of timber with the same end connection of rod anchors to gusset plates as the other elements. The tension rod system is available with threads of M6 to M76. [22]

Glued connections

In contrast to mechanical fasteners, timber elements glued together provide a rigid joint, therefore creating better homogeneous components from smaller segments. Nowadays, the adhesive bond in the glue joint is stronger than the bond between the wood fibres themselves. The modern synthetic resin glues are inexpensive to produce, very easy to use, their setting time can be adjusted to the requirements, and enable glue joints to be produced on site. In principle, glue joints represent the most efficient way of forming a structural connection between two elements. However, due to the high stiffness of the connection, its failure will be brittle. The timber industry is provided with the possibility for endless elements, enabling the chance for timber to be an alternative to steel and concrete in long-span structures. If the benefit of creating long elements can be combined with a ductile behaviour in failure of the entire structural system, a very efficient structure can be created. [37] [24]

HESS Limitless

HESS TIMBER GmbH has developed a patented high-strength beam splice joint named HESS LIMITLESS. It is formed by a large finger joint combined with a wedge shaped fitting and high premium lamination in the contact zones. This enables a cost-efficient use of long glulam timber in any configuration for a project without complicated prior transport planning. It can create glulam beams with strength class GL35c, the related strength characteristics can be seen in table 2.1. It looks like a stronger beam type but the tension, compression and shear resistance are reduced. Furthermore, high precision in manufacturing is required for cutting and gluing of the large finger joint. The production on site therefore takes place in climate controlled and specially equipped tents. It can reach a factor of efficiency of the joint for in-plane bending and shear as high as $\mu = 1.0$. For tension or compression parallel to the grain, the wedge shaped fitting must be placed on both sides reaching a factor of efficiency $\mu = 0.9$ and $\mu = 0.85$ respectively. The on-site gluing is an expensive and difficult solution due to the specific environmental conditions and expertise required for realisation. This jointing technique is used in the Anaklia-Ganmukhuri bridge as mentioned in 2.2.2 and on-site regular finger jointing has been used in the Essing bridge as mentioned in 2.2.2. [33] [37] [18]

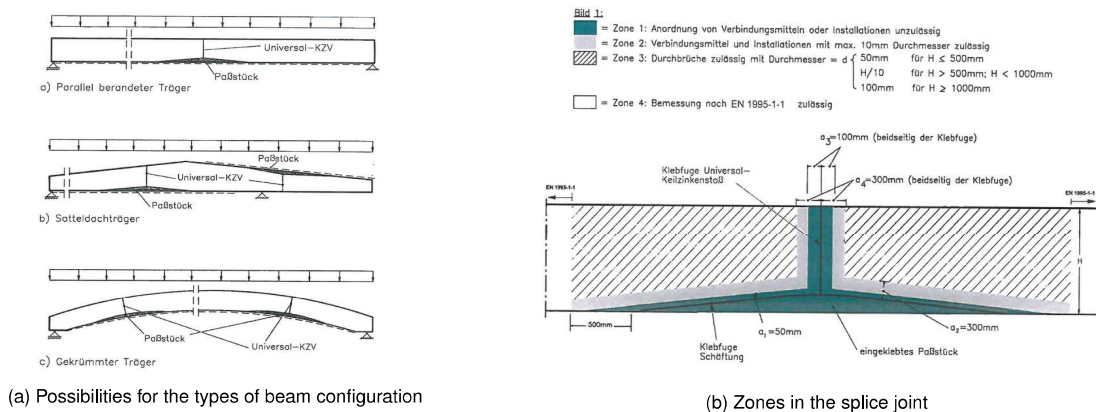


Figure 2.41: HESS Limitless splice joint

GSA-Technology

GSA® technology is a high-performance connection technology for modern timber engineering, in which threaded steel rods are glued to Glulam and hardwood with epoxy resin. It is developed by Neue Holzbau AG in order to realise highly efficient and standardised connections for timber construction. All connections are integrated into the timber cross-section providing invisible joints that create a aesthetically pleasing structure, and also good fire protection which improves the overall fire behaviour. This technology guarantees inherent ductility, stiffness, and load capacity of connections through an optimized design of the GSA®-rods and a special created strong epoxy glue. Accompanied with these beneficial aspects comes a reduced load capacity ($\eta * f_d$) with $\eta = 0.80$ of the connection. The GSA fasteners allow for efficient installation and alignment on site resulting in fast construction. Several GSA® systems are developed for splice joints, column-base joints, truss joints and the ability to apply prestressing on elements. Examples of these connections can be seen in figure 2.42. GSA technology is a good alternative to slotted steel sheets with dowel connectors in BauBuche trusses as seen in several example projects. This is due to the higher shearing strength of the material that results in a higher pull-out capacity of the glued-in rods. The manufacturer achieves a ductile failure mode by dimensioning the steel elements in such a way that they form the weakest link. The steel begins to expand under maximum load, which ensures that all the fasteners of the anchoring assembly are loaded equally and no brittle breaks can occur at the glued joints. [7] [52] [24] [48]

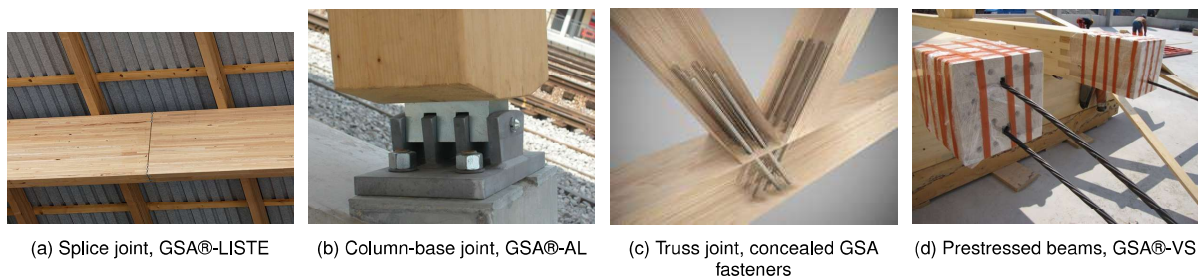


Figure 2.42: GSA® systems as presented by Neue Holzbau AG

Strengthening connections

The load-bearing capacity of the joint area of timber members is the primary weak point when considering the load-bearing behaviour of the overall timber construction. The effectiveness of dowel-type fasteners such as dowels or screws generally ranges between 40 to 60% when the load-bearing capacity is taken into account. This means that the structural joint detailing has a crucial impact on the performance and thus efficiency of a structure. Certain changes made to the structural detailing may improve the load-bearing capacity of the joint, thus reducing the required cross-section of the member. The above stated new connections might reach a higher efficiency, but still not the full capacity. Furthermore, the primary reason for reinforcing timber joints is to prevent brittle failure modes of timber due to splitting or shear and hence improve ductility. Another huge benefit is the improvement of the group effect, which has a particularly adverse effect on joints with multiple fasteners arranged in a row in the force and grain direction. [13] [18]

pre-stressed FRP laminates

Using pre-stressed bonded fibre-reinforced polymer (FRP) laminates for strengthening wooden structural members has been shown to be an effective and economical method. The most effective way of such a strengthening was to place reinforcement laminates on the tension side of flexural members, increasing the load-bearing capacity. Applying the pre-stressed laminates introduces a pre-camber and thus improves the serviceability limit state, which often governs the design. FRP materials have very high specific strength and stiffness, very good durability and fatigue performance and are very light weight. However, many uncertainties make this solution not reliable enough for nowadays construction and especially outdoors. The attachment of pre-stressed FRP laminates to timber results in high peak stresses near the edges, which are difficult to resist by clamping devices due to many factors. Furthermore, the long-term durability behaviour of the bonding between the laminate and wood in outdoor environments is unknown. [39]

Screws

The formation of plastic hinges in the dowel type steel fasteners requires large embedding deformation without preliminary timber failure due to splitting or shear failure. An easy way to reinforce connections in tensile members perpendicular to the grain is the use of self-drilling screws. The application is very fast and the screws are comparatively cheap leading to a cost effective way for the reinforcement of the member's joint areas. The screws, when placed in contact to the fastener, will increase the load-carrying capacity and the stiffness of the joint due to contribution of the screw acting as a "beam" supporting the fasteners. This kind of reinforcement creates a joint not at risk of splitting, and therefore additionally increases the load-bearing capacity. The screws need to be arranged perpendicular to the grain and at right angles to the actual dowel-type fasteners. The reinforcement eliminates the risk of the timber splitting, which results in a full utilisation of a dowel group, hence the reduction factor of effective number of fasteners doesn't apply. A prerequisite is that for each dowel row a fully threaded screw that has a capacity of 30% of the shear load acting on the dowels is screwed in. Perpendicular to grain reinforcement is not covered in Eurocode 5, but by the national annex of Germany. [14] [13] [35] [15]

Glued-in steel rods

Glued-in steel rods are used, besides the previous mentioned load transfer solutions, to prevent cracks due to tensile stresses perpendicular to the grain. The rods are also used for rehabilitation and repair of elements, during which they may be subject to lateral or axial load or a combination of both. They have the advantage of transmitting significant concentrated loads, be arranged in parallel with the grain, create very rigid joints in axial loading, and ensure effective fire resistance as they are protected by the surrounding wood. Instead of glued-in rods, steel rods with a thread that are driven into pre-drilled holes like screws are increasingly popular for reinforcements perpendicular to the grain. They create a very strong mechanical connection with the wood via the thread. The glued-in steel rods, however, are best suited for arrangements parallel to the grain. Rods with a diameter between 12 and 24 mm are generally used with drilled holes that are around 1 to 4 mm larger than the external diameter of the thread with a length up to 3000 mm that can be cut to the desired length. [13]

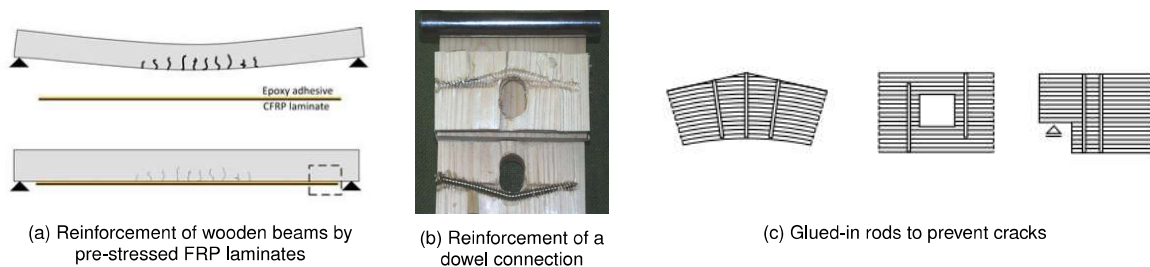


Figure 2.43: Ways of strengthening timber elements and connections

2.3.3 concluding

Due to the fact that all existing timber structures only span half the distance of the required distance of the New Feyenoord stadium it is wisely to choose the strongest EWP available. This is the Hardwood LVL as can be seen in section 2.3.1. Hardwood LVL is well suited to be used where high strength and dimensional stability is needed. However, almost all structures use glulam as this is more widely available and therefore cheaper. Larger cross-sections are possible with glulam due to the bigger laminations. Glulam produced from LVL lamellas is a possibility and will provide the best aspects of both options, but is probably more costly than regular glulam. It has a higher tangential resistance and embedment strength, which is beneficial for connections using steel fasteners. The final material will be chosen after the global design assessment in the next chapter. The entire football stadium roof structure will consist of a large amount of connections. If simple connections and repeating connections are used this will be beneficial for the end product. Furthermore, a high level of prefabricated connections will reduce the occurring errors during construction and result in more efficient nodes. The relevant connections will depend on the configuration of the roof and the type of internal load transfer. The components that will make up the stadium roof are larger than the allowed dimensions for transport and therefore will need to be made in smaller segments.

2.4 Conclusion of analysis phase

The architectural design vision for the roof structure of The New Feyenoord stadium is defined as an elegant, floating and thin structure putting the primary focus on an unobstructed experience for the spectators. The goal is to have no obstructing columns on the tribunes and an unobstructed supporter perimeter around the pitch. The supporter perimeter consists of a oval ring structure with three tiers to accommodate 63.000 spectators. The resulting outer perimeter at the top of the stadium is in the shape of a super ellipse with width and length of 205 by 245 metres located at a height of 40 metres. The roof will be supported on 12 cores at the outer perimeter. The quality of the pitch depends on the amount of direct sunlight and air movement on the natural turf. The roof finishing will hence consist of transparent polycarbonate sheeting.

There are several structural possibilities to fulfil the above stated criteria with a timber roof structure. To determine a structural system that will be feasible in timber for the enormous dimensions of the stadium and still conform to the stated design criteria an assessment is needed. The infographic presented in figure 2.44 shows the four structural systems that are of interest and the possibilities for timber configurations to create long spans. Also, the different techniques to connect timber members are presented with their main aspects. Global designs are made for the next phase by combining the structural systems with the possible configurations while bearing in mind the type of connections. The goal of this research is mentioned as well as points of attention for the used material, which then result in the focus of the next phase.

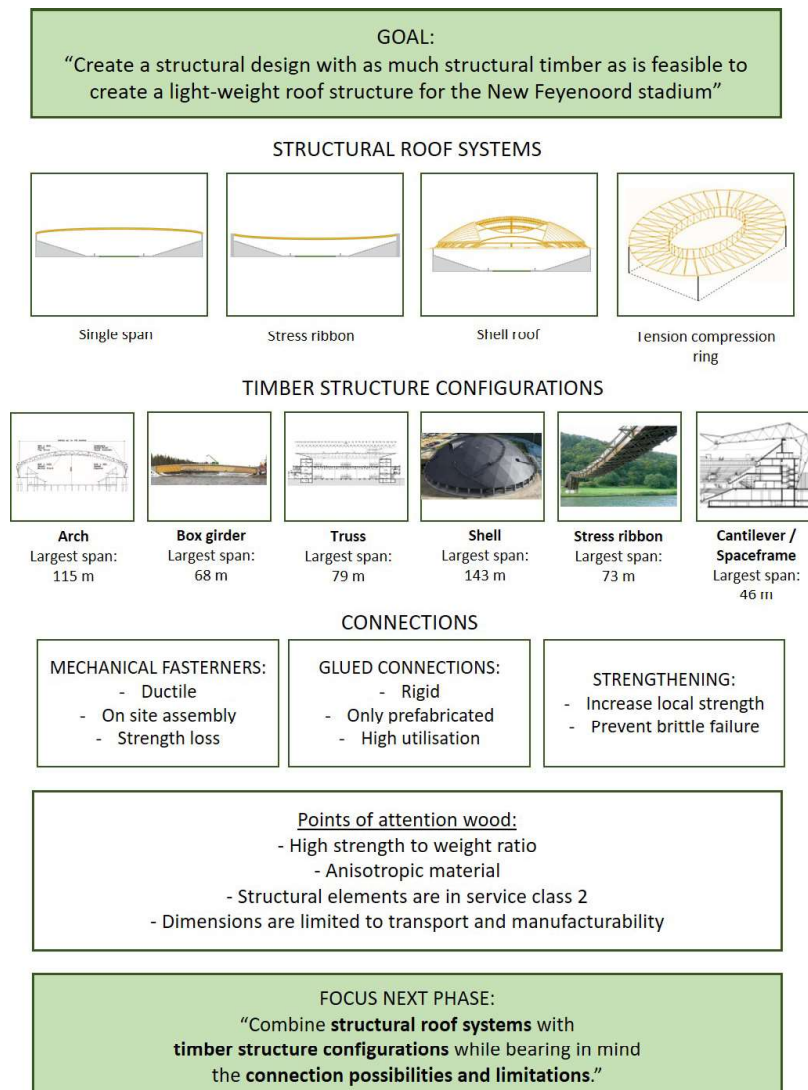


Figure 2.44: Infographic to conclude analysis phase

3

Global design

The global design gives insight into the primary structural system of the long-span stadium roof structure. It is divided into five sections. In the design aspects are the necessary variables presented to evaluate a timber structure on its primary forces. In the design strategy are the different steps in the design process presented in chronological order. The structural concepts illustrates the preliminary designs made for the primary structural system. In the concept assessment is an evaluation of these preliminary designs made by means of a Harris profile. Finally, this chapter concludes in a chosen primary structural system and a chosen secondary structural system.

3.1 Introduction

The analysis phase has presented many design possibilities to determine if a long-span timber football stadium roof structure might be feasible. It is difficult to conclude which structural system in combination with what configuration will allow a timber structure to extend far beyond the current long-span timber structures. This chapter presents the geometry of the stadium roof, the material and design properties for a timber roof structure for The New Feyenoord stadium, and the design loads and load combinations. Several structural concepts are introduced that follow from the analysis done in the previous section. These concepts are then assessed by means of a Harris profile to conclude on a chosen primary and secondary structural system. The assessment will include a rough calculation on the structural performance of the concepts to determine their potential for a more detailed design phase.

3.2 Design aspects

The design aspects of the roof structure for The New Feyenoord stadium are presented. The geometrical boundary conditions for the roof structure are visually shown. This will include the final outer perimeter, the roof opening, height of the stadium, and the size and location of the cores. The required safety and serviceability of the stadium depends on the service life and the design situation of the structure. For building structures and other common structures a indicative design working life of 50 years is classified, reliability class 3. A football stadium should be designed for consequence class 3 (CC3) as is stated in the Eurocode [2]. The design loads to assess the structural performance of the roof structure are determined and the corresponding load combinations for roof structures given.

Geometry of the stadium

As mentioned in the previous chapter the outer perimeter of the stadium has a width and length of 205 by 245 metres. The dimensions of the pitch are 125 by 85 metres. It is wanted to keep an opening the size of the pitch above the playing field, but this will depend on the chosen structural concept. The roof is supported by twelve cores with a height of 40 metres, a width of 8 metres and a length of 22 metres. These dimensions are shown in figure 3.1b and 3.1a. The supporting cores of the roof structure are evenly distributed around the outer perimeter of the stadium.

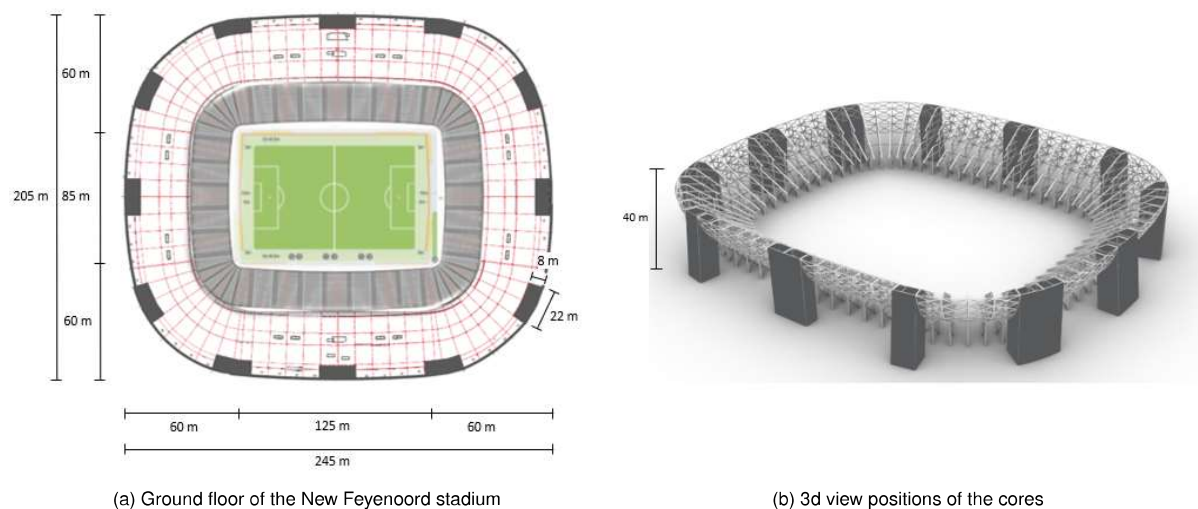


Figure 3.1: Geometry of the stadium

Material and design properties

For the global design analysis it is chosen to use GL32h, because it is widely available and therefore the most economic option. It is assumed that this material grade will result in the most feasible timber structure due to a cost reduction. However, the costs are excluded from this research and the focus is on the structural feasibility of a timber long-span roof structure. Hence if the material grade is not strong enough the choice will be made for the glulam constructed with Hardwood LVL lamellas. The structure falls under consequence class CC3 and service class 2. This is associated with the reliability and natural durability of the timber structure. A structure needs to fulfil a certain level of safety against deterioration. This is done by the appointment of the correct service class after which the appropriate timber species or product is chosen that has a proper natural durability within these demands.

Load-duration and moisture influence

The duration of load in combination with the present temperature of the surrounding and hygroscopic behaviour of wood strongly influences the mechanical design values of timber. The European standards consider this behaviour by reducing the design strength by two coefficients K_{mod} and K_{def} . The first one is used for the ultimate limit state and are the same for GLT and LVL, they are presented in table 3.1. The latter is used in serviceability limit state to evaluate the long term displacements under quasi permanent loading combination, they are shown in table 3.2. [11] [4]

Table 3.1: Values of k_{mod} for glue laminated timber and laminated veneer lumber

Material	Service Class	Load duration class				
		Permanent	Long term	Medium term	Short term	Instantaneous
GLT / LVL	1	0.60	0.70	0.80	0.90	1.10
	2	0.60	0.70	0.80	0.90	1.10
	3	0.50	0.55	0.65	0.70	0.90

Table 3.2: Values of k_{def} for glued laminated timber and laminated veneer lumber

Material	Service class		
	1	2	3
GLT / LVL	0.60	0.80	2.00

Loads and load combinations

The loads acting on the roof structure are presented in this section. They are established by RHDHV according to the Eurocode. RHDHV excluded asymmetric loading in their initial design phase due to the little impact it has on their structural configuration. However, depending on the structural system and the fact that timber roofs are more prone to wind loading, the asymmetric loading due to wind needs to be taken into account as well. The following permanent and variable loads are applied to the roof of the stadium.

Permanent loads

The permanent loads are constant present on the roof structure. The following permanent loads are taken into account; self-weight, secondary timber weight, event loading and architectural finishing. The loads will be explained individually, followed by an overview of the permanent loads and their design values.

Self-weight

The self-weight of the timber is determined by the material choice, which is GL32h in the global design analysis. The self-weight of the primary construction is 4.4 kN/m^3 .

Secondary timber self-weight

The configuration of the secondary timber structure is unknown as the global design phase explores multiple structural solutions. It is therefore left out of the loading for now. The weight of the secondary timber is the same as for the self-weight of the primary structure and will be added in the next phase.

Event loading

This loading takes into account the standard stadium facilities regarding light and sound. This loading is considered to be evenly distributed along the roof surface and weighs 0.01 kN/m^2 . This loading is also not regarded in the rough calculation to distinguish the most promising structural system and will be added in the next phase.

Architectural finishing

In the preliminary design of RHDHV the roof is designed with a multi-walled polycarbonate and rod system as finishing, this has in total a weight of approximately 0.15 kN/m^2 .

Variable loads

The variable loads can occur randomly on the retractable roof. Therefore it is important to take all possibilities into account. The following variable loads are considered; snow loading, suction and pressure caused by the wind and suction and pressure caused by the stadium. The loads will be explained individually, followed by an overview of the variable loads and their design values.

snow loading

Snow loading is calculated on the entire roof structure with the following value.

Char. snow load	$s_k =$		0.7 kN/m^2
	$\mu_i =$		0.8
	$C_e =$		1.0
	$C_t =$		1.0
Snow-load on roofs	$q_{Q,snow} =$	$\mu_i \times C_e \times C_t \times s_k$	0.56 kN/m^2

Wind loading

RHDHV has estimated the following values, awaiting a computational fluid dynamics analysis and a wind tunnel analysis, for the wind loading on the entire roof. These estimations are a conservative approximation based on the Eurocode. For wind loads, two situations have been observed, wind suction (up) and wind pressure (down).

Wind suction	$q_p (\uparrow) =$	$-0.7 * 1.32 =$	-0.987 kN/m^2
Wind pressure	$q_p (\downarrow) =$	$0.20 * 1.32 =$	0.282 kN/m^2

Internal pressure

RHDHV has estimated the following values, awaiting a computational fluid dynamics analysis and a wind tunnel analysis, for the internal pressure on the entire roof. For the internal pressure of the stadium, two situations have been maintained, upward pressure and downward pressure.

Upward pressure	$q_p (\uparrow) =$	$-0.2 * 1.32 =$	-0.282 kN/m^2
Downward pressure	$q_p (\downarrow) =$	$0.30 * 1.32 =$	0.423 kN/m^2

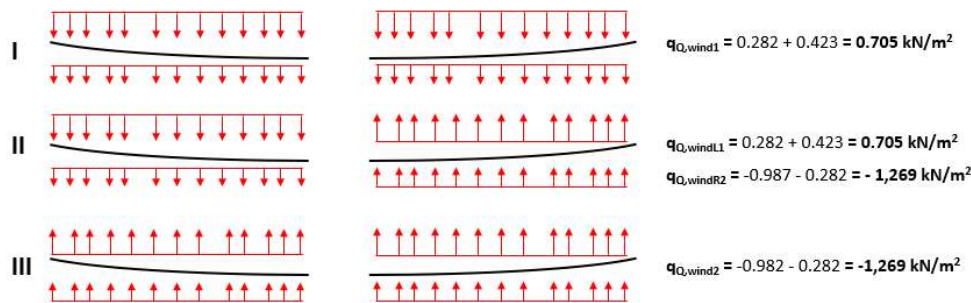
Snow loading is used for the simplicity of the rough calculations in the global design. However, as can be seen will the resulting force due to wind loading be greater and therefore leading.

Load combinations

The following load combinations are required to ensure structural safety of the roof structure. No combination between different variable loads is required for roof structures as stated in the Eurocode. Except for the different types of wind loading and internal pressure as these have the same instigator. The safety factors associated with ultimate limit state for the permanent loads is $\gamma_g = 0.9$ for favourable loading, $\gamma_g = 1.32$ for unfavourable loading, and for the variable loads is $\gamma_q = 1.65$. The safety factors for serviceability limit state are all equal to $\gamma = 1.00$. [2]

Lodcase ID	Name	[kN/m ²] ->	Self-weight	Secondary wood	Architectural finishing	Event load	Wind downwards	Wind upwards	Internal pressure downwards	Internal pressure upwards
				0.14	0.15	0.01	0.282	-0.987	0.423	-0.282
0	ULS-01		1.32	1.32	1.32	1.32	1.65		1.65	
1	ULS-02		0.90	0.90	0.90	0.90		1.65		1.65
2	ULS-03	Combination between LC 0 on the left part of the roof and LC 1 on the right part								
3	ULS-04		1.00	1.00	1.00	1.00	1.00		1.00	
4	SLS-02		1.00	1.00	1.00	1.00		1.00		1.00
5	SLS-03							1.00		1.00
6	SLS-04		1.00	1.00	1.00	1.00				
7	SLS-05				Only asymmetric wind load * 1.00					

(a) Load cases with their corresponding safety factors



(b) Schematisation of different combinations for wind loading

Figure 3.2: Load combinations for The New Feyenoord stadium

3.3 Structural concepts

The structural systems and reference projects that were investigated in the literature phase result in four global designs of the primary structural system for the roof of the New Feyenoord stadium. These four designs are respectively a single span roof, a tension/compression ring roof, a stress ribbon roof and a grid shell roof made up out of trusses. The four designs are the most promising global designs selected out of an assessment of different solutions. The four structural categories of the global designs are linked to the way the primary girders are distributed. Within these categories there are several options for the secondary girders, each resulting in slightly different solutions. However, for this first assessment all the secondary girders of each solution are kept the same as much as possible. By doing so the focus is more on the structural performance of the primary girders. The in depth design of the secondary girders will be done in a later stage. Now, each design is briefly introduced and explained.

3.3.1 Single span

Several football stadiums are built with the principle of a single girder spanning the entire width of the stadium. It is a rather conventional system creating efficient girders, but not necessarily efficient roof structures as a totality. The preliminary design of the New Feyenoord Stadium made by RHDHV consists of one variant like this as explained in 2.1.3. Other inspirational structures for this design are the Trade Fair hall 11 in Frankfurt and the Maicasagi bridge in Nord-du-Québec. As mentioned in the 2.2.2 section, the first one makes use of large trusses spanning the entire width and the latter consists of a box girder. Imagery of these reference projects are shown in the figures below.

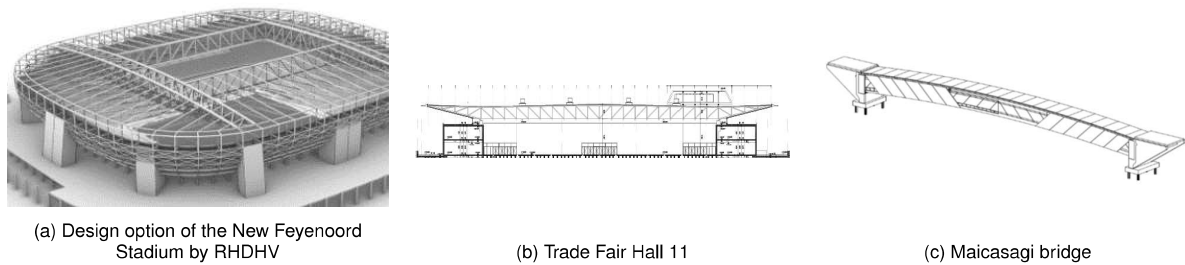


Figure 3.3: Inspirational structures for the single span concept

The idea of the single span is thus to span the width of the entire stadium with one big girder, either a truss- or a box girder spanning 206 meters. A secondary beam is resting on the main girder to create the opening in the middle of the roof. Tertiary girders are situated in between the primary and secondary girders and the outer perimeter to support the roof panels. The secondary beams will span 120 metres and are also made in one of the two mentioned configurations. The tertiary girders span 60 metres and are constructed as trusses.

Positive aspects:

- Strong, stiff main girder
- Good with torsion and buckling (box)
- Rectangular opening above the field
- High loads possible around roof opening for event loads

Negative aspects

- Bulky and heavy
- Fabrication / erection complex due to very big and heavy girders
- Structural efficiency
- High support reaction

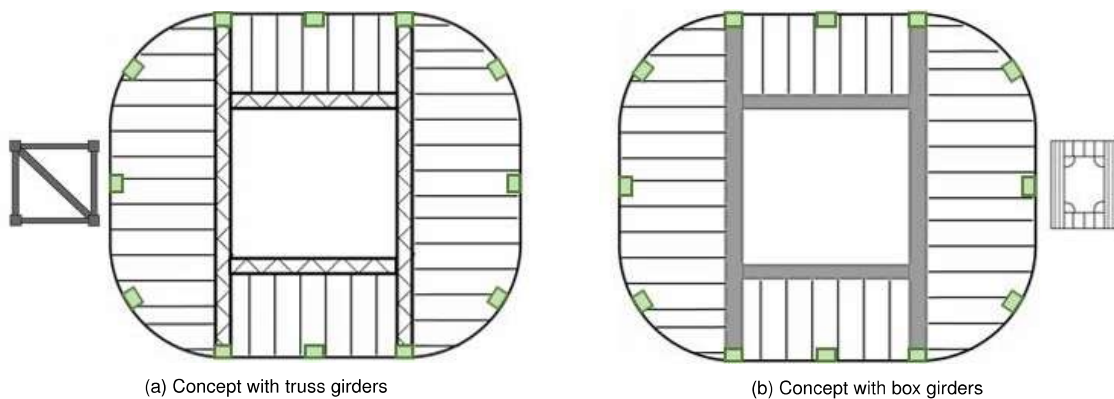


Figure 3.4: Global design of the single span concept; the locations of the supports are shown as green boxes

3.3.2 Tension / Compression ring

More modern stadiums are built with this structural principle, resulting in very efficient and lightweight roof structures. The problems that arise due to the super ellipse shape of the presented stadiums, were solved in their unique way. Two examples of these football stadium roof structures are De Kuip in Rotterdam and the Commerzbank Arena in Frankfurt. The way they solved their structural problems was presented in the previous chapter. Still the configurations of these example projects don't provide sufficient possibilities for a timber variant. However, due to development within computational modelling an more efficient design can be found which can be applicable in timber.

Another example of a roof structure with an outer tensile ring is that of The Elephant house in Zurich, seen in figure 3.5c. A interesting aspect of this roof structure is the grid that provides a shallow shell structure which is very stable. The configuration and size of the windows was made possible by computational modelling. This type of stable grid in between the tension ring is partly the inspiration for the configuration of the secondary girders in the second tensile / compression ring design, seen in figure 3.6b.

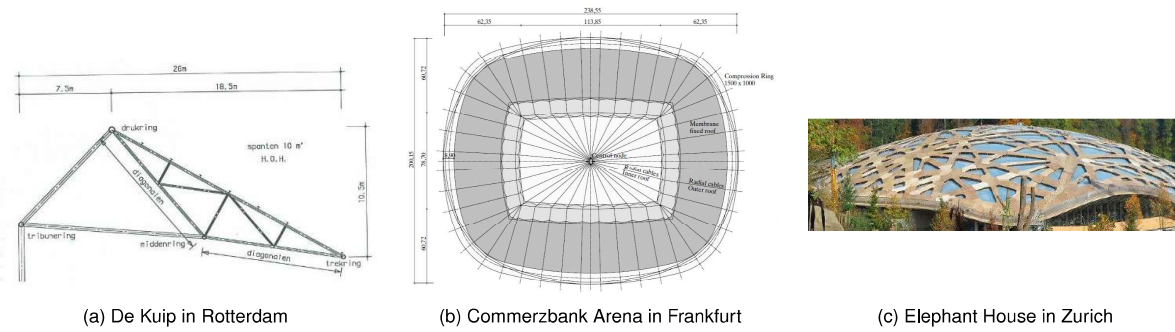


Figure 3.5: Inspirational structures for the tension compression ring concept

The tensile / compression ring concept hence consists of an outer ring in either tension or compression, depending on the relative position of the two rings to each other, and an inner ring with the reverse force flow. The two stiff rings are connected by an elegant light structure in between, consisting of either trusses or a space frame shell, which carry the roof panels. This typology is ideal when circular, but it can also be used for a super ellipse with additional structural elements redirecting the forces in the corners. Making use of these ring forces can create a more structural efficient roof than conventional structures.

Positive aspects:

- Lightweight
- Economic solution
- Ring and beam (or shell) action working together
- Can be made from smaller segments
- Low support reactions

Negative aspects

- Stiff inner and outer ring
- Ideal when circular

The two options can be seen in figure 3.6 where both options have an outer ring in tension and an inner ring in compression. The first option is in initial thought more easy to construct due to repetitive trusses as secondary girders. The second option is more beneficial for the force flow in the corners due to the none circular shape.

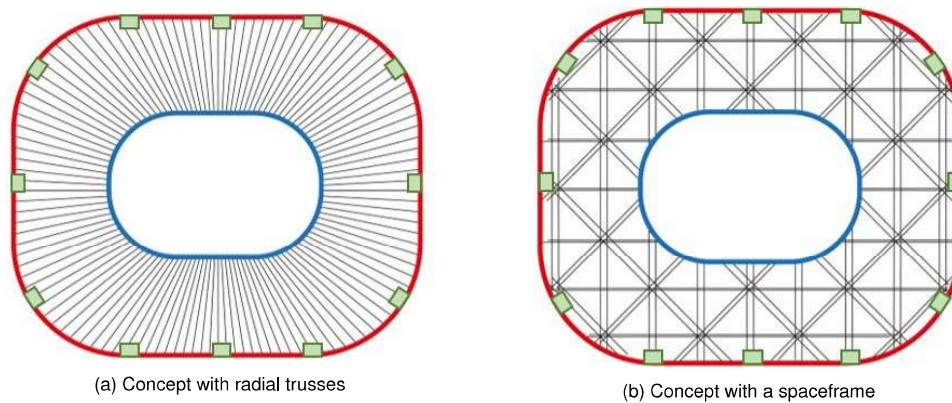


Figure 3.6: Global design of the tension/compression ring concept; the locations of the supports are shown as green boxes

3.3.3 Stress ribbon

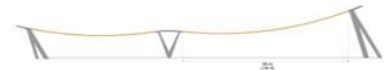
This is a somewhat unconventional option for a long-span stadium roof structure. The system is found in other roof structures and bridges, proven to be an elegant and slender (economic) solution. One of these examples is the timber stress ribbon bridge in Essing. This variant is also applied in a very slender roof made of concrete for the 1998 Portuguese National Pavilion in Lisbon, which is also really unconventional.



(a) Essing bridge in Essing



(b) 1998 Portuguese National Pavilion in Lisbon



(c) Grandview Heights Aquatic centre in Surrey

Figure 3.7: Inspirational structures for the stress ribbon concept

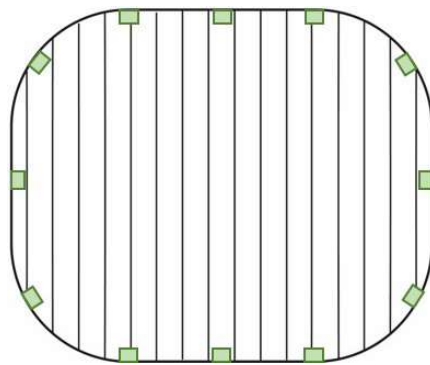
This roof system consists of timber beams working as tension cables with bending stiffness spanning the entire width of the stadium. This system takes advantage of wood's flexural ability and provides mass to handle dynamic wind forces. Furthermore, the whole cross section is stressed to its full potential as almost only tensile forces occur over its height, which can result in a very lightweight and efficient roof structure. However, it needs to be supported horizontally as well. To create the opening above the field, another option is to span several ribbons in the length of the pitch, connected between the other ribbons. When all these ribbons are laterally connected, it works as a shell hanging upside down.

Positive aspects:

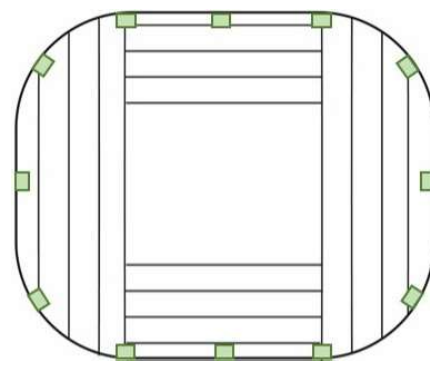
- Very lightweight
- Economic solution
- Full structural utilisation of the members cross section
- Elegant roof
- Weight of grandstands help supports

Negative aspects

- High support reactions
- Up-lift forces
- Serviceability limit state



(a) Concept with single span ribbons



(b) Concept with shell configuration ribbons

Figure 3.8: Global design of the stress ribbon concept; the locations of the supports are shown as green boxes

3.3.4 Grid shell of trusses

The entire stadium will be covered by a grid shell. The beam elements of the grid shell consist of trusses meeting each other in nodes. With this configuration a very stiff and robust shell structure is made. It is uncertain if an opening in the roof is possible with this solution. The curvature of the roof can be limited due to the bending stiffness of the trusses, creating an elegant roof from the outside. However, a strong outside perimeter is necessary to keep the shell in place as it is not a perfect round shape and the curvature of the shell is kept low. A grid shell made from timber beams is a regularly used structural system. However, a grid shell from trusses is not yet built. Below is the inspiration for this concept shown.



(a) Domes in Brindisi

(b) Elephant house in Zurich

(c) Louvre in Abu Dhabi

Figure 3.9: Inspirational structures for the grid shell of trusses concept

Positive aspects:

- Very stiff shell
- High strength from double curvature
- Repetition

Negative aspects:

- Constructability
- Structurally ideal when fully closed
- Complex connections
- Strong additional outside perimeter

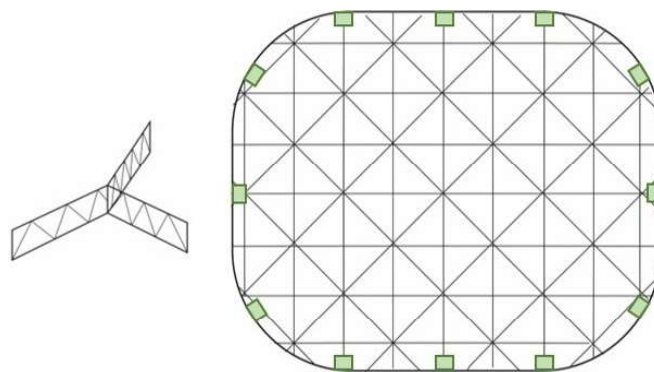


Figure 3.10: Concept for the grid shell of trusses design

3.4 Concept assessment

Due to a lack of available information on systems made up out of timber in this span range, an assessment is made between the four concepts. This is done on eight criteria originating from the analysis phase by making use of a Harris Profile. Several of these criteria require a rough calculation to make a substantiated decision.

3.4.1 Harris Profile criteria

The criteria in this assessment determine how much the different concepts fulfil the stated design restrictions and if they are feasible solutions for a long-span timber structure. It is chosen to give several criteria more weight than other criteria as these criteria are more valid than others to create an appealing and feasible structure.

Structural efficiency

The structural efficiency of the system is of importance to make sure that it has a certain level of economic and material effectiveness. The weight of the construction materials used to build a structural system is linked to the cost of these materials. It is cost effective to use the least amount of material necessary to provide a structure that can safely carry the applied loads. The most efficient structures are strong and lightweight.

Robustness

A guiding principle in the design of structural systems is to maximise its reliability. When a local failure occurs, it isn't acceptable that this results in the progressive collapse of the total structural system. It needs to be assessed if sufficient structural reliability is achieved by suitable measures like ensuring an appropriate degree of structural integrity.

Ease of construction/production

The ease of construction and production is of importance from an economic perspective but is also beneficial for timbers durability. A fast erection time reduces costs and also lowers the carbon footprint of a building. Furthermore, as timber needs to be protected against several climate conditions, it is beneficial to minimise its exposure to the elements.

Simplicity of connections

Connections are traditionally a weak point in timber structures and quite expensive. It is beneficial to minimise both aspects by reducing the complexity and quantity of the connections. However, more advanced connections are nowadays available, and the ability to implement those new connections can be beneficial.

Aesthetics/configuration

OMA wants a floating, almost unsupported roof. Such a roof is structurally impossible, but the question arises which system can reach this design vision as much as possible. The other point of interest is if the configuration of the structural elements is done in a visually pleasing manner.

Support reactions

The building site of the New Feyenoord stadium is very restricted due to the existing structures and other complications. It is therefore of great importance to keep the support reactions as low as possible. Lower support reactions of the roof results in a more slender substructure and these two combined have a severe impact on the foundation.

Optimisation possibilities

These are rough designs, where several problems might occur during detailing. Are the systems flexible in their configuration or can they reach more substantial potential than estimated at first? This category addresses the flexibility of a global design to achieve greater potential by making practical and economic adjustments.

Advantage of timber

Does this structural configuration make use of the benefits of the material? As wood has certain specific properties and behaviour, not all classical structural systems can be blindly copied to a timber variant. It is essential to exploit the benefits of structural timber and diminish its weaknesses.

3.4.2 Rough calculations

The rough calculations are based upon the situation with a variable snow load in combination with the self-weight of the structural elements. The dimensions of the structural elements and end-to-end distance is kept similar to the found dimensions by RHDHV for their preliminary design. These are tried to be kept similar between all four concepts to make a acceptable validation between the options. However, the structural concepts differ considerably in the amount of elements needed to construct the main girders. This is considered in the assessment presented in the next section. The chosen cross-sections and preconditions for the rough calculation is shown in figure 3.11.

Preconditions	
Strength class	GL32c
Loads	Selfweight + snow load
Supports	12
Material factor	$\gamma_M = 1,25$
Consequence class	CC 3: $\gamma_G=1,3$; $\gamma_Q=1,65$
Service class (load duration)	SC 2 (short) $k_{mod}=0.9$; $k_{def}=0.8$
Selfweight timber	$\rho_{G,tim} = 4.4 \text{ kN/m}^3$
Selfweight polycarbonate panel	$q_{G,pc} = 0.15 \text{ kN/m}^2$
Snow load	$q_{Q,snow}=0.56 \text{ kN/m}^2$
length	$l = 246\text{m}$
width	$b = 206\text{m}$
Longitudinal main girders	1500 x 1500 mm
Bracing main girders	600 x 600 mm
Longitudinal secondary girders	500 x 500 mm
Bracing secondary girders	200 x 200 mm

Figure 3.11: Cross sectional dimensions and pre conditions for the rough calculation

The single span and stress ribbon concept are calculated by means of a simplified calculation done by hand. The single span is assumed as a simple supported beam and the stress ribbon as a simple supported cable. The tension compression ring and the gridshell of trusses are simulated by means of a FEM program named Grasshopper and Karamba3D. These two concepts are complex to verify by hand and therefore computational modelling is required. The corresponding maximum forces are obtained and then verified by hand calculations. All verifications are done according to Eurocode 5: Timber structures and can be seen in Appendix A.

3.4.3 Calculation results

The results of the rough calculation are presented in figure 3.12 and 3.13. The top table in the top figure provides the calculated values for the case where all cross sections of similar elements are kept the same between the different concepts. The table at the bottom in figure 3.12 highlights the deviation from maximum allowed values per concept. A cell filled with the colour red indicates the maximum found value for a category between the concepts. A cell filled with green indicates the lowest found value for a category. As mentioned, the structural concepts differ between their structural behaviour and hence the elements required for their primary girders. The bottom figure shows the results of all concepts when their strength verifications is tried to be kept below 1.00. Below each concept is mentioned what is adjusted from the assessment in the top figure to obtain these results.

	#1	#2	#3	#4	
	Single-span	Ten/Comp ring	Stress ribbon	Gridshell of trusses	
Compression parallel to the grain [U.C.]	1.13	1.27	n.a.	0.89	
Tension parallel to the grain [U.C.]	1.41	0.83	1.04	0.40	
Maximum vertical support reaction [kN]	29752	12563	17829	14901	
Maximum horizontal support reaction [kN]	0	22765	183635	0	
Deflection at midspan [m]	2.31	1.06	2.68	1.6	
Total weight [kg]	1.11E+07	5.23E+06	5.47E+06	5.17E+06	

Procentuele waarden t.o.v. maxima:	Single-span	Ten/Comp ring	Stress ribbon	Gridshell of trusses	100%
Compression parallel to the grain [U.C.]	113	127	0	89	Maximum U.C. is 1.0
Tension parallel to the grain [U.C.]	141	83	104	40	Maximum U.C. is 1.0
Maximum vertical support reaction [kN]	100	42	60	50	V Single-span
Maximum horizontal support reaction [kN]	0	100	807	0	H Single-span
Deflection at midspan [m]	262	129	327	195	Maximum w is 0.82 m
Total weight [kg]	100	47	49	47	kg Single-span

Figure 3.12: The results from the rough calculations to support the assessment with the Harris profile where all the cross sections are kept similar

	#1	#2	#3	#4	
	Single-span	Ten/Comp ring	Stress ribbon	Gridshell of trusses	
Compression parallel to the grain [U.C.]	0.89	0.88	0	0.70	
Tension parallel to the grain [U.C.]	1.12	0.68	0.82	0.57	
Maximum vertical support reaction [kN]	30402	13828	8479	14329	
Maximum horizontal support reaction [kN]	0	17992	24260	0	
Deflection at midspan [m]	1.42	0.72	0.59	1.24	
Total weight [kg]	1.13E+07	5.71E+06	9.29E+05	4.82E+06	

h 1st girder 4m higher h lnRing 4m higher f 4m lower h*b girders top 0.7*0.7m
 h*b lnRing 1.9*1.9m h*b girders 0.4*0.4m h*b girders bottom 0.4*0.4m

Figure 3.13: Adapting the main girders to obtain a unity check below 1.0 for the rough calculations

The single span has its maximum moment at midspan, which results in a compression force in the top chord and a tensile force in the bottom chord. Increasing the height of the truss decreases the internal forces. This structural concept will have a very low average utilisation, because the two primary girders are subjected to high bending forces in the middle. Furthermore, it will encounter high shear forces near the supports, which is undesirable in timber structures. The bending moment results in a high deflection at midspan, although the truss configuration is a stiff form. The truss can be optimised by increasing its strength locally by means of a stronger timber material.

The tension compression ring is modelled with the radial trusses. Large forces occur in all elements near the supports in the corners. The average force utilisation is very low in the outer ring due to very high peak forces in the corners and the short sides of the stadium. Solutions for these peak forces is to implement double truss rings. The top inner ring has high internal compressive forces, while the bottom ring does not. This results in a low average utilisation of the elements, due to the non-circular shape of the rings. Another solution is to use a more circular structural shape of the rings, however the result is that less support points can be used. A different configuration of the secondary elements might reduce the weight and hence forces on the ring trusses.

The stress ribbon has uniform tensile forces in the 'cables', hence a very elegant force flow. The average utilisation is equal to the highest utilisation, which results in a very optimal structure. This elegant force flow allows for very slender elements. However, high horizontal support reactions are introduced at the cores. The slender ribbons will also have a problem with uplift forces from wind. A hole in the middle of the roof will reduce the total loading on the ribbons. Increasing the weight is also a option although the resulting horizontal forces will than increase as well. If the second configuration of the stress ribbon is used than the horizontal forces will be more evenly spread to all cores.

The gridshell of trusses result in a robust, but very heavy structure. The average utilisation is almost halve of the highest unity check. The deflection is high at midspan due to the large amount of elements in this concept. It is very difficult to model a smooth outer perimeter while keeping the triangles of the grid the same size. The non-circular outer perimeter cuts the pattern and leaves many small elements to fill the gaps. It is uncertain if the geometry can be optimised for the outer perimeter of The New Feyenoord stadium. This concept has many connections that will weaken the structure, which are also subjected to high forces in the middle of the roof.

3.4.4 Harris Profile

The Harris profile is filled in with the knowledge acquired from the analysis phase and the rough calculations. four categories count more heavily due to their increased impact on the feasibility of a timber roof structure for The New Feyenoord stadium. the total score is calculated by means of the following values -- = -2, - = -1, + = 1, ++ = 2, and * = 2x.

	Singlespan		Tension/Compression ring		Stress ribbon		Grid shell	
	--	-	+	++	--	-	+	++
* Structural efficiency								
Robustness								
Ease of construction/production								
* Simplicity of connections								
* Aesthetics/configuration								
Support reactions								
* Optimisation possibilities								
Advantage of timber								
Total score	-1	1	8	14	15	14	-4	

Figure 3.14: The final Harris profile for the global design assessment

3.5 Conclusion of global design

Every option has its own difficulties that are complex to address in an early stage without prior experience in the field of engineering special structures. Due to the limitations of erected structures of this size using structural timber, it is beneficial to search for an out-of-the-box solution that might create new possibilities. This has been done in combination with rough calculations to result in a realistic underpinning for the choices made. The goal is, as stated in chapter 1, to create a feasible structure using structural timber to the utmost extend. Therefore, this early assessment is a mixture between finding a new solution as well as a backed investigation by structural calculations. This resulted in the choice for a tension compression ring structural system for the primary girders and an stress-ribbon configuration for the secondary girders. These choices are elaborated in the following subsections.

Primary system

The tension compression ring system is a structural solution that is seen more and more in modern stadiums, mostly due to new possibilities using computational design and calculations. The ring action spreads the resulting forces of the structure on to all the cores in the outer perimeter, reducing its peak supporting forces. However, due to the irregular shape of the stadiums perimeter, the spokes wheel concept is not exploited to its utmost extend. This will result in peak stresses near the corners and a low utilisation of the long sides. A solution to reduce this impact is to reduce the weight of the secondary girders to reduce the internal forces. Furthermore, as is seen in the Tokyo National stadium in section 2.1.3, a slight arch in the inner ring at the long sides will be beneficial for deformations. Finally, the tension compression ring configuration allows for an open roof above the playing field which is highly beneficial for the quality of the pitch.

Outer ring

To spread the ring forces in the outer ring it is chosen in consultation with experts to have a triangular truss cross section. The triangle is chosen in a way that the longest element is in tension so it does not have to cope with buckling. The truss configuration will result in a material reduction of the outer ring, but still be a very strong and stiff solution. This is needed as the outer ring is only supported on twelve points along its perimeter and spans freely between them. Besides supporting the secondary structure through ring action, it has to supports its own self-weight as a beam. A triangle shape is a stable and strong configuration for the expected deflections in multiple directions. The connections making up this outer triangular truss will be of a complex form due to many elements meeting in one point.

Inner ring

There is chosen to construct the inner ring as a flat truss. It is expected that a large tensile force will be present in the inner ring as this is situated 60 metres out of the supports. Another possibility might be to incorporate a second inner ring, half way the cantilever to reduce deflections and take up some of the ring action of the inner ring. The elaboration of these possibilities will be done in the next phase.

Secondary girders

Since it is out of the scope of the project to assess all different types of secondary girders, a promising configuration is picked. Tension cables are chosen, because they make use of an elegant force flow as seen in the assessment. Furthermore, it is a very interesting configuration for a tension compression ring due to it being never done before and shows several advantages. Expected ease of construction, optimisation possibilities due to end-to-end distance optimisation, very slender elements reduce the required structural material of the secondary girders which in turn reduce the weight on the primary system and the minimisation of required nodes and connectors in the elements, which can reduce the rigidity of the system. All these aspects are considered to be required to obtain a feasible solution for such an enormous structural width. If timber would like to have the option to be feasible, the weight and amount of connections should be kept to a minimum. However, this configuration might have problems with upward wind loading because of its light weight. These aspects are explored in the next chapter.

4

Structural Design

The structural design showcases the primary and secondary structural system of the long-span stadium roof structure accompanied with important connections. It is divided into five sections. In the general geometry are the chosen boundaries presented to create a valid structural model. The analytical model of a cable system elaborates on the structural principles of the chosen primary and secondary system. The finite element model describes the modelling method, cross sections and configuration, model verification, results of the FEM, and a discussion on the results. The connections illustrates solutions for the occurring joints in the structural system. Finally, this chapter concludes on the findings.

4.1 Introduction

The chosen structural system will be designed and analysed in more depth in this chapter. The system as a whole, primary and secondary structure working together, will be elaborated on all the design loads and will be verified for all internal force combinations. The strength verifications were already at their limits in the global design analysis, therefore it is chosen to use the Glulam made of hardwood (GL75). The high local properties in different directions of this timber is also beneficial for a better load transfer of the connections. The goal of the structural design is to find cross-sections which are within the limits of the maximum manufacturing dimensions. This will be done by maximising the intended structural behaviour of the combined systems to obtain the most uniform force flow as possible in the different elements. After the strength verifications several design options will be presented for the required connections to build the structure.

4.2 General geometry

Initial constraints are chosen according to the structural grid of RHDHV and the general design aspects. For example, the outer perimeter of the inner ring stops above the field to place the roof above every spectator while it follows the outer shape of the stadium to avoid extra eccentricities in the ring forces. These design restrictions are all put into a parametric model and can therefore be endlessly modified to find solutions that fulfil other conditions, like architectural restrictions for instance. However, this thesis focusses on the internal and external force flow of the design. In figure 4.1 and 4.2 are the design constraints presented which influence the structural performance of the stadium roof.

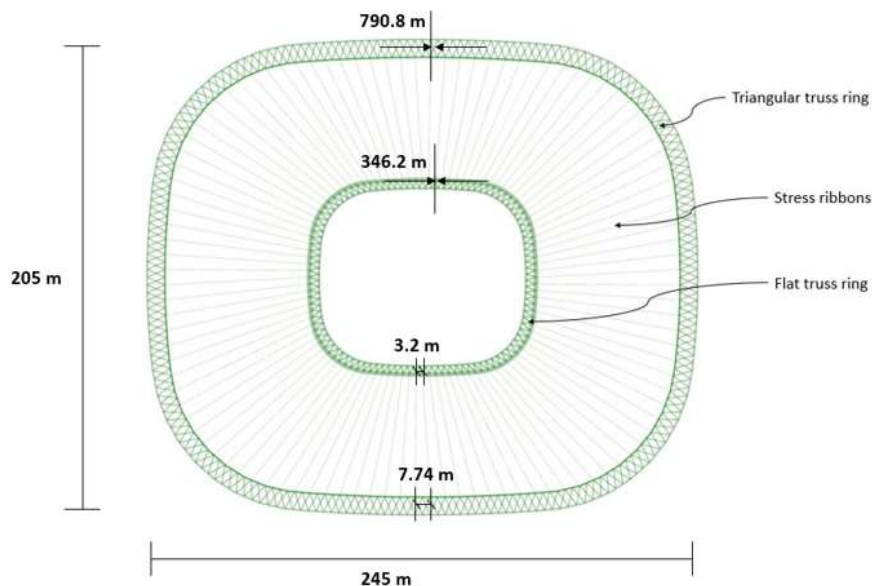


Figure 4.1: Design constraints of the timber stadium roof structure with 108 ribbons

4.2.1 Primary structural system

The primary structural system consists of a tension / compression ring configuration. The outer ring is in compression and follows the circumferential shape as chosen by OMA, which is not an ideal circle. The outer ring consist of a triangular truss to divide the internal forces along its chords and webs. In the global design the ring only had two chords, so it is expected that the triangular truss can cope with the non circular configuration in a stiffer and stronger manner. The webs follow the configuration of a v-shape truss to transform the tension force from the ribbons in a stable manner and create a stable shape in circumferential direction. The configuration of the triangle results in tension in the longest web element to avoid buckling behaviour. Furthermore, the roof has a visually smooth transition from the facade with this configuration. A combination between ring forces and beam forces is expected due to the non-circular shape and the span between the cores. The v-shape configuration will guide the forces through the truss ring to adjacent elements and hence reduce the supporting reactions.

The inner ring is in tension and follows the same circumferential shape as the outer ring. This ring is made as a flat truss to transfer the horizontal tensile forces coming from the ribbons in circumferential direction. The webs also follow a v-shape configuration. It is expected that large internal forces will occur in this ring because of the smaller circumferential length of the ring and due to it only consisting of two chords. Furthermore, large deflections are expected to initially stiffen the ribbons and rings by tensile forces and because of a lack of curvature in the long and short side of the stadium.

The inner ring will be placed at the required distance from the outer ring to provide shelter for all spectators. The location of the inner ring determines the length of the ribbons. It is wanted to place this ring just above the beginning of the stands to minimise the span of the ribbons. The flat truss is placed 15 metres lower than the top point of the outer truss. Resulting in a sag of the roof with the same lowest point as the stress ribbon roof in the global design phase. The influence of the sag of the roof on this new configuration is addressed in the next section. There is still space left for initial deformations to stiffen the roof structure before the minimum free height above the pitch is reached. This sag of the inner ring also influences the length of the ribbons. The length of the ribbons in turn determine the total roof area, which needs to be covered and is hence equal to the loaded area.

The expected structural behaviour of a non circular tension / compression ring configuration will be presented in the next section. The presented behaviour is valid for stiff spokes, which is not the case in this structural design. The latter will also be addressed in the next section.

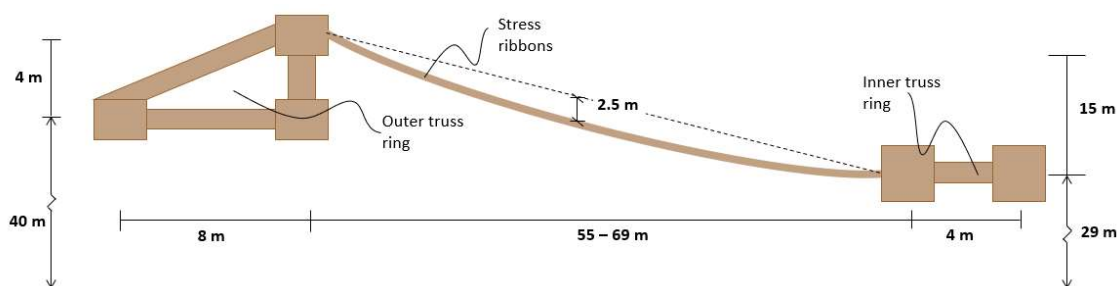


Figure 4.2: Design constraints of the distance between elements

4.2.2 Secondary structural system

The secondary structural system thus consists of stress ribbons. It is chosen to place the ribbons in a radial configuration. This results in more usage of the ring action by connecting the weaker points of the inner ring with the stiff corner ring elements. The amount of deformation in the straight sides of the stadium can be decreased by making more use of the available ring action.

The ribbons are assumed to be prefabricated with an curvature to ensure that it will behave as a cable. This will also reduce the initial expected residual stresses, which will occur during construction under its deflection due to self-weight. The sag of the ribbons corresponds with the chosen sag of the ribbons in the global design phase. This sag results in an inclination of the ribbon in the structural design as the lowest point is now supported by the inner ring. The new sag is determined by the curvature of the ribbon with respect to a straight inclined line between the highest and lowest point. The highest point of the ribbons is at the top of the outer truss, so that the inclination doesn't obstruct view lines of the spectators. The ribbons connect to the truss rings at the centre line of the v-shaped sloped diagonals to evenly direct the forces. These principles can be seen in figure 4.2 and 4.1.

In order to cover the gap between the ribbons, a tertiary system spanning transversally across the ribbons is required. Prefabricated timber elements will be connected in between the stress ribbons to support the roof panels. The roof covering will consist of PC-panels which can span up to 2.5 metres and thus is the end-to-end distance of the tertiary system 2.5 metres. Furthermore, the tertiary system must be able to work as a continuous diaphragm in order to provide the building with overall stability. They will strut the ribbons when they are subjected to uplift wind forces and horizontal wind forces. Besista rods will be placed as diagonals in line of the cores to transfer these forces to the substructure.

Furthermore, diagonals of Besista will be placed along the perimeter of the inner ring and outer ring to provide horizontal stability. These rods are successfully used in previous timber roof structures like the EXPO Stuttgart, Sevilla parasol and the Clamart sport centre. The effect of the tertiary system on the overall structural performance is not addressed in this thesis. The impact of their weight is included in the design loads.

4.3 Analytical model of cable system

The outer and inner ring trusses are carrying the horizontal force of the ribbons with ring action. The outer perimeter is non-circular and therefore part of the horizontal forces are also carried by beam action. The internal force of the ribbons is meant to be purely tensile just as in cable structures. However, due to the non-circular shape and loss of stiffness of the rings certain ribbons take up the transverse forces by bending as well. The schematisation of the structural system can be seen in figure 4.3. The basics of these principles are explained in this section.

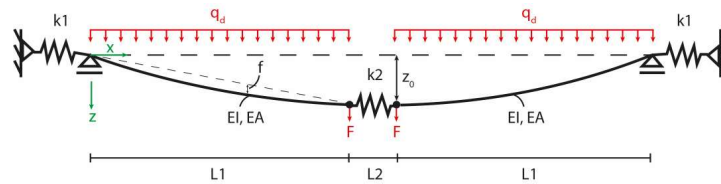


Figure 4.3: Schematisation of the analytical model consisting of two ring trusses and stress ribbons

4.3.1 Tension / compression ring

The tension compression ring system is originated from a spokes wheel. The principle of this system is based on the amount of ring action a wheel can provide. Transverse loading on the spokes will cause radial loads on the rings. These radial loads are caused by the vertical deformation of the central ring and the spokes and by possible pre-tensioning forces of the spokes. In order for the rings to provide ring action they need to be stressed. This is seen in figure 4.4.

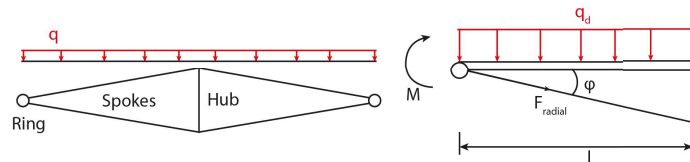
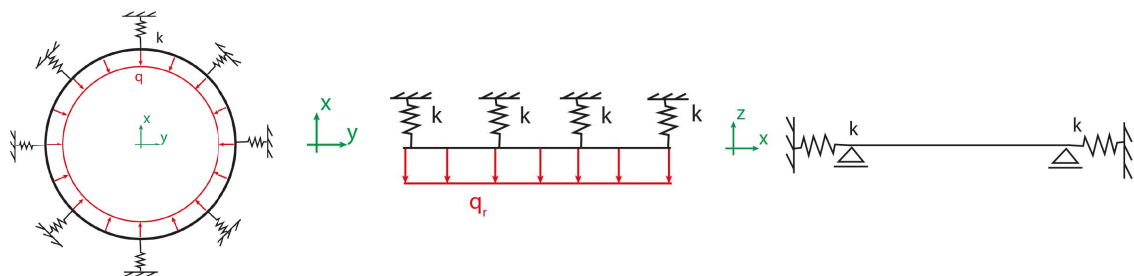


Figure 4.4: Resulting forces from transverse loading on a spoke wheel roof structure

Radial translations must be kept free for optimal ring action and thus only the transverse translation to the plane of the ring is blocked (z-direction). The stiffness of the ring itself provides resistance against radial translations. The translation resistance in the plane of the roof structure can be schematised by springs around its perimeter. The schematisation of a circular ring system is shown in figure 4.5. The spring stiffness is determined by the translation resistance of the ring in its plane.



(a) Schematisation of circular ring system (b) Schematisation of a ring element in the x,y-plane (c) Schematisation of a ring element in the x,z-plane

Figure 4.5: Schematisation of the outer ring in a circular tension / compression ring system

The acting radial load resulting from the spokes thus results in ring action. Equilibrium in the system is found by a balance between the force in the springs and the radial load. Where the spring force represents a constant spring force over the whole ring and the radial load is caused by a distributed load q on the roof, both in N/m. The spring constant is determined by $k = F_k/u$, which means the spring stiffness k is the spring force F_k divided by the deformation of the spring u . Equilibrium in the structure is found when $q = F_k = k * u$.

The deformation of the spring is equal to the extension of the radius of the ring. The spring constant can be expressed as a function of the radial load q . The spring stiffness $k = q/u$, where $u = \delta_r = \frac{(2\pi q r^2)/EA}{2\pi} = q r^2/EA$, results than in $k = EA/r^2$.

The strength and stiffness of the system will increase when a higher amount of ring action is present. The main influencing factors on the ring action are the radius of the ring, the loads acting on the ring, its extensional rigidity, and the translation of the outer ring. These factors are presented in figure 4.6. An increase of the radius has a negative influence on the spring constant. The ring will deform due to the loading in the ring. Tensile forces will result in extension of the ring and compressive forces will compress the ring. The extension of the ring increases when the radial load and radius have a higher value. This can then be countered by increasing the cross sectional area of the ring structure to increase its extensional rigidity.

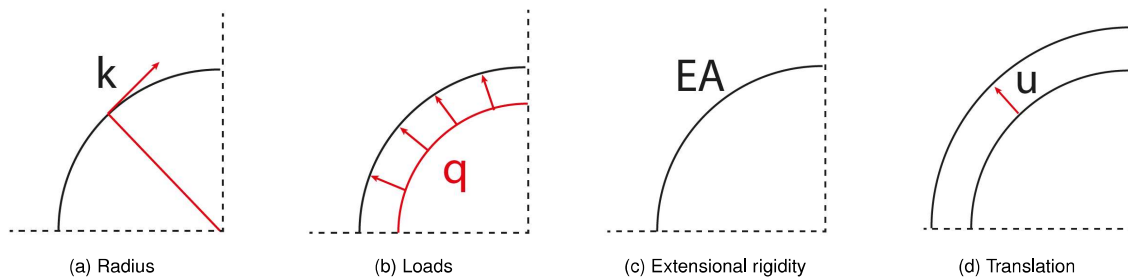


Figure 4.6: Factors that influence the ring action in a tension compression roof system

The New Feyenoord stadium has a non-circular shape, which has a negative influence on the very important structural characteristic of the tension compression ring system, namely the ring action. On the long and short side of the stadium the curvature is almost infinite, which greatly decreases the ring action. The difference in curvature results in a non constant amount of stress in the ring. Which in turn results in irregular ring action and extension in the rings. Besides the curvature difference, there is also a difference in the angle between the spokes and the ring elements. The force transfer between these elements is not equally efficient as can be seen in figure 4.7. The radial configuration of the spokes tries to minimise this effect, but it can not be eliminated due to the irregular outer perimeter shape of The New Feyenoord stadium. The angle between the spokes and the ring on the long side is wider than in the corners and the short sides.

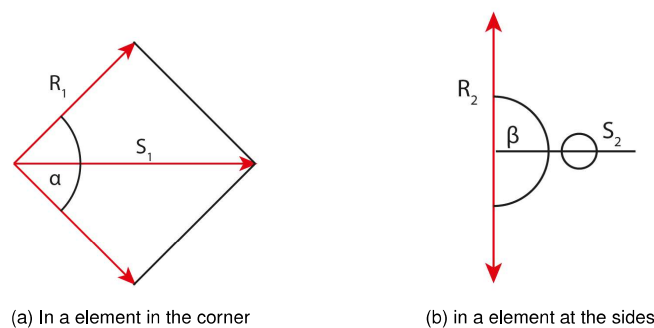


Figure 4.7: Polygon of forces

The angle between the spokes and the ring elements is thus not equal to each other. A smaller angle results in a better transfer of normal force. The angle difference will result in a difference between the normal forces in the spokes. The difference of normal forces has an effect on the transverse forces (V_y) and bending moments (M_x , torsion, and M_z) in the ring elements as well as in the spokes itself. An example of the consequence of this force difference for the outer ring is shown in figure 4.8. This example only shows the influence of the loading coming from the spokes. The outer ring structure is only supported in twelve points and spans around 60 metres between the cores. The outer ring itself is prone to beam action due to its own weight. These forces need to be added to the presented principle. It can be concluded that the corners which are supported by a core will provide most of the stiffness to the structure. Therefore the area of the elements in the rings need to have a certain amount of increased stiffness at the other points along its perimeter.

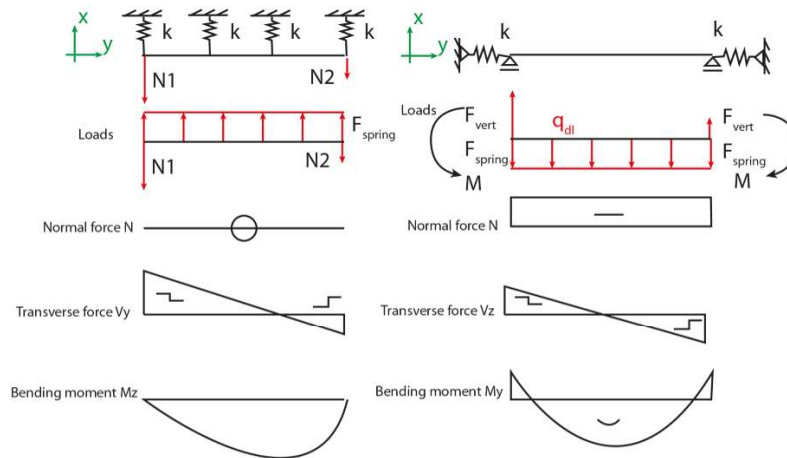


Figure 4.8: Structural behaviour of the outer ring in a non circular tension / compression ring

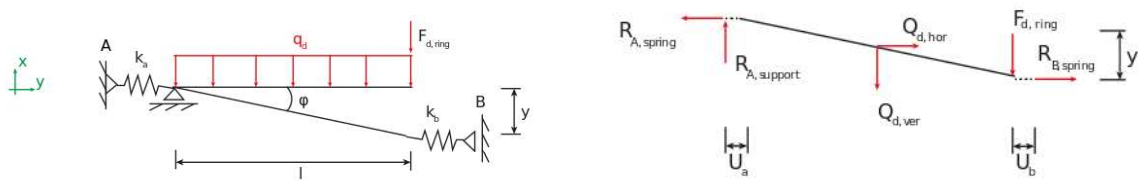
When the spokes are not pre-tensioned they will sag to the point of equilibrium when prone to any transverse load. The sag of the inner ring causes tensile forces in the lower spokes and compressive forces in the upper spokes. Now beam action will play a role to determine the strength and stiffness of the structure. As great deformations will arise in the long and short side of the stadium due to the lack of curvature, these bending moments will only increase more. Besides the earlier mentioned option of increasing the stiffness of the structural system it is also possible to increase the area of the ring that is affected by transverse loading of the spokes. This can be done by using stronger and stiffer ring elements to affect more spokes or by decreasing the centre-to-centre distance of the spokes by using more spokes. These options might be a better alternative to pre-tensioning as these induced tensile forces are normally high. High pre-stressing forces on the straight sides have a negative effect on its translation due to a lack of radial stiffness as can be seen in figure 4.9.



Figure 4.9: translation differences due to curvature in the ring elements prone to pre-tension

The stress distribution in the tension / compression ring is largely influenced by the type of connections used between the ring elements itself, and between the spokes and the rings. When hinges are used for these connections the translational stiffness is largely decreased as the ring elements in the straight sides can easily deflect in lateral direction. The ring action is used to the utmost extent when the ring elements are rigid connected in circumferential direction. The stiffness will help keeping the ring stable at locations with a lack of curvature.

In contrast, the connection between the spokes and the rings should be hinged to mimic the structural behaviour of a spokes wheel. The loading on the roof is mainly retained by normal forces due to the interaction between the spokes and the rings. High transverse forces and bending moments are minimised with the application of hinges, which result in an efficient load transfer. A schematisation of the hinged spokes between the outer and inner ring is seen in figure 4.10. The outer ring is placed on roll supports and the ring action is schematised by spring supports. The spokes can freely rotate around its nodes making it a kinematic indeterminate structure. The system stabilises itself due to the second order effect. This effect occurs in structures where considerable deflections can arise in which calculations from the structures original geometry are no longer sufficient any longer. The relation between load and deflections are in this theory no longer linear. The caused deformation of the spoke by the transverse loading is resisted by the spring forces of the rings. The resulting spring force of the inner ring will gradually decrease the sagging of the system and eventually cause equilibrium. The second order effect can be minimised by using stiff rings. This might result in structurally unnecessary material usage, which is only needed to provide stiffness instead of strength.



(a) deflection of the spoke due to hinged connecting and spring supports

(b) Equilibrium of the system

Figure 4.10: Large deformations of the spokes result in a second order effect

Horizontal translation of the ring elements is dependent on the amount of load acting on the structure. The transverse load ‘activates’ the ring action, which in turn provides a transverse and radial support. Furthermore, the extensional, bending and torsion rigidity of the rings in combination with its radius and transverse deformation as well as the bending and extension rigidity of the spokes influence the horizontal translations. These deformations are found after large vertical translations of the inner ring, which result in geometric non-linear behaviour. This complex behaviour in three dimensions can not be calculated by hand. [55] [31] [17]

4.3.2 Inclined cable

A cable system is very suitable for a spoke wheel roof structure. It resists the transverse loading through tensile forces only. As a result, very small cable members can be used for very large spans. The structural efficiency of a cable comes with a negative side effect, namely the resulting horizontal force induced at the supporting structure. These horizontal forces are restrained by the ring elements as explained in the previous section. Pure cables are not able to provide stiffness through beam action and fully rely on the ring action of their supporting structure. The lack of geometrical stiffness in a cable results in large deformations when prone to changes in loading. The geometrical stiffness can be increased by: adding extra weight, stiffening by external cables, and adding bending stiffness to the cable. These principles are shown in figure 4.11. A cable with inherent bending stiffness represents a stress ribbon, which is thus an improvement of the simply suspended cable system.

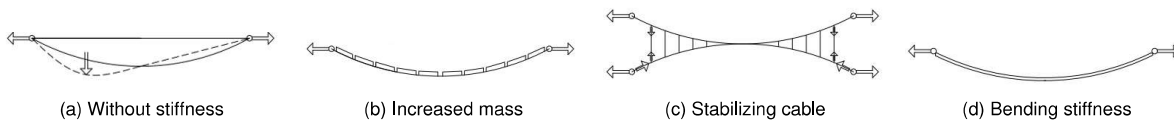


Figure 4.11: Different stiffening principles for cable structures

The stress ribbon follows a catenary curve similar to a cable subjected to an evenly distributed load. In the general situation of a simply supported cable, which has a high slenderness (sag/span ratio is small), bending is neglected. The horizontal force component of the cable can then be calculated with the following formula: $H = \frac{qL^2}{8f}$. here is H the resulting horizontal force, q the evenly distributed load, L the span length of the cable, and f the sag at midspan. It can be seen that a low sag-to-span increases the horizontal forces. An increased horizontal force will result in a stiffer cable structure. However, a high horizontal force requires a very stiff and strong supporting structure. The limit of the cable sag in a stadium roof structure is not determined by its serviceability requirements, but by the restrictions for free height above the pitch.

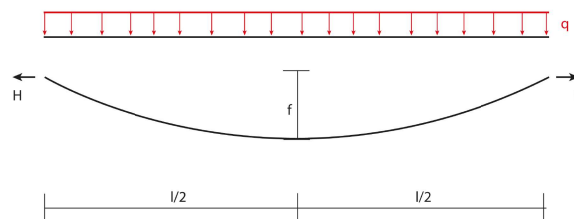


Figure 4.12: Cable subjected to uniformly distributed load

In order to analyse the structural behaviour of a stress ribbon structure the fundamental equations of a hanging cable are of interest. Figure 4.13 shows a cable suspended between two supports with a uniformly distributed load and its resulting external forces. The presented equilibrium equations are valid for the situation where both endpoints of the cable do not deflect. The horizontal component of the cable tension stays constant throughout the cable when no external horizontal force is applied. The cable tension is expressed as $N = \sqrt{H^2 + V^2} = H\sqrt{1 + z'^2}$, where H is the horizontal force component, V the vertical force component, and z' the slope of the cable rise for any point on the cable. The vertical force component differs at both endpoints in case of an inclined cable and is dependent on the angle between the normal force and the horizontal force component. The length of the cable L_0 , including the material strain that affects the length of the cable $\epsilon = \frac{N}{EA}$, can be expressed by $L_0 = \int_0^L \sqrt{1 + z'^2} (1 - \frac{H}{EA} \sqrt{1 + z'^2}) dx$. The fundamental properties of a simply supported hanging cable can be analysed with these equations.

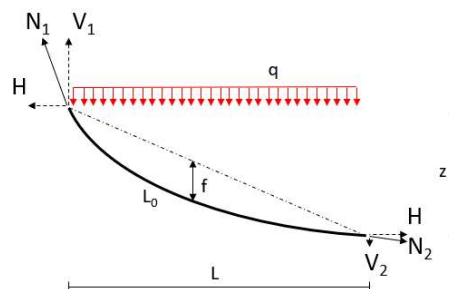


Figure 4.13: Inclined cable resulting in different vertical reactions

The stress ribbons in the roof structure for the New Feyenoord stadium are not simply supported at the central hub. The lack of supporting stiffness, especially in the straight sides of the rings, will result in large deflections. When the deflection is too high, the stress ribbons take up the transverse loading with beam action. A study done by Starsky, investigated the influence of the amount of bending stiffness on the force distribution and sag in a stress ribbon. Low inherent bending stiffness results in nearly the same deformation and force distribution as for a pure cable. Increased bending stiffness results in a reduction of the deformation and an increase in the bending moment. An optimum needs to be found between having enough bending stiffness to have a reasonable deflection while minimising the bending moment. The cross-sectional area of the stress ribbons will determine this balance.

Just as for the tension / compression ring, the stress ribbons find their equilibrium in the deflected state. The relation between load and deflections is thus also non-linear for the ribbons. The presented basic cable equations are valid for simply supported cables. In the case of the New Feyenoord stadium the stress ribbons are supported by hinges in a three-dimensional manner due to the radial configuration. Especially the hinge support provided by the inner ring allows large deformations to stabilise the structural system. This makes the created structural system mathematically very complex, and therefore it can not be calculated by hand. [34] [60] [49]

4.4 Finite element model

To find a solution for the structural verification of the feasibility of timber in a long-span football stadium roof structure, the system is modelled in a parametric environment coupled with FEM analysis. Namely, Grasshopper and Karamba3D which are both plug-ins of Rhinoceros, a 3D computer graphics and computer-aided design application software. Through the use of parametric programming in the Grasshopper environment, a computational model of components and structural systems is created. The plug-in Karamba3D makes it possible to do a structural analysis of the kinematic indeterminate system with many unknowns to determine the leading forces, moments, and deformations. These programs make it relatively easy to investigate the coherence of many different parameters on the structural performance of a complex structure. An iterative loop of finding the interaction between form, weight, stiffness, dimension, and internal forces is made very suitable by the computational environment of Grasshopper.

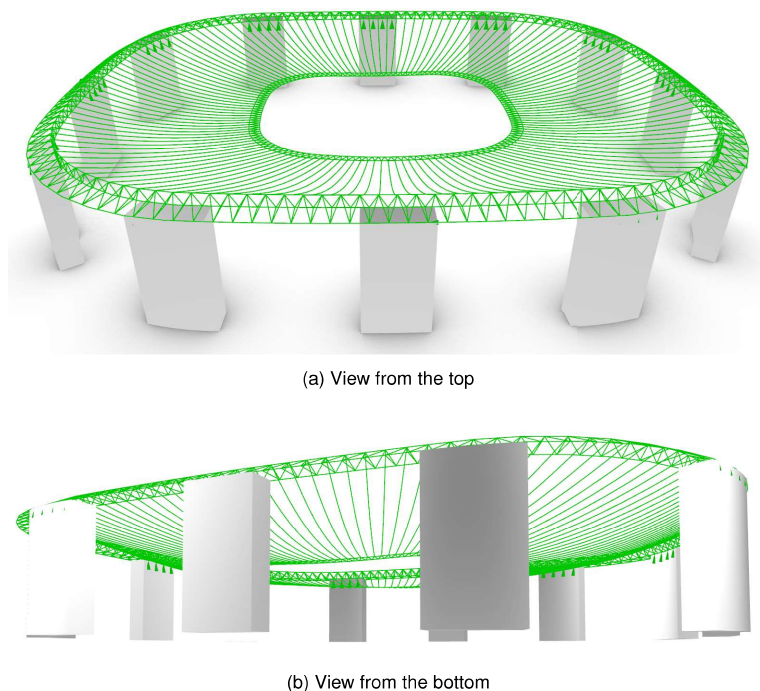


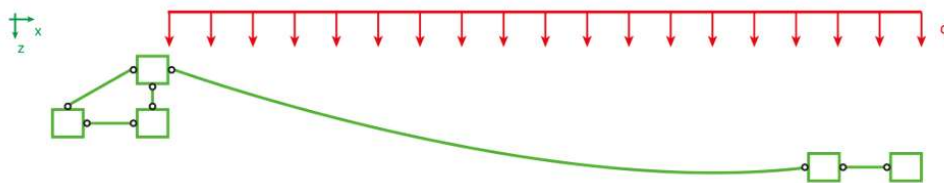
Figure 4.14: Overview of the finite element model

The dimensions taken from the global design analysis are used as initial dimensions in the finite element model. The secondary structural system of this final design will probably weigh less than that of the tension / compression ring in the global design. However, as the secondary system now consists of inclined cables with a high horizontal force, the large initial cross-sections will be a good estimation. The final design has a radial configuration of inclined ribbons instead of normal ribbons. This will reduce the sag of the cables which in turn result in higher internal forces and thus a larger cross-section of the elements. Furthermore, the inclusion of all design verifications instead of only snow loading will demand higher material characteristics, which will be provided by using the stronger laminated timber. It is checked if the initial dimensions of the elements can be reduced and if it will result in a feasible timber structure for the roof of the New Feyenoord stadium.

4.4.1 Model assembly

The model consists of different parametric definitions for all its building blocks. Almost everything that defines the structure can be made parametric. The benefit is that the structural behaviour can be investigated for different configurations, even after the model is constructed and analysed. The structural design calculation model is assembled where cross-sections, material, load cases, joints and supports are assigned.

Investigating the influence of every parameter in the system is to time consuming. Several checks are done to get a better force flow in the ribbons and rings. These checks involve changing the arch effect (up or down in long side), amount of ribbons, connections, sag of inner ring, outer truss diagonals configuration, support conditions, roof opening, and most of all the cross sectional areas of the elements. Several checks are presented in the next section and other are elaborated at the end of the chapter. First, the assembling of the model verification is presented.



(a) hinges between diagonals and at the end points of the ribbons

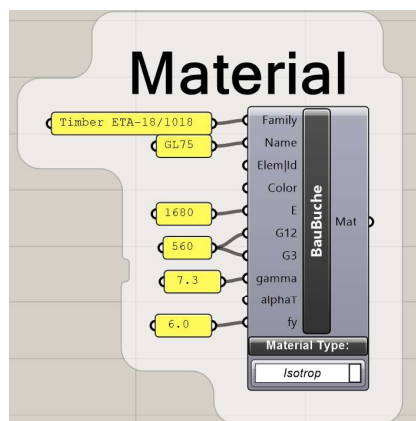


(b) Outer truss, inner truss and ribbon configuration

Figure 4.15: Schematisation of the modelled structural system

Material properties

The material characteristics of glued laminated timber made of Baubuche is used. These characteristics are presented in the Analysis chapter. A downside of Grasshopper is that its components are mainly developed for steel structures. It is thus difficult to assign the orthotropic behaviour of wood to the structural system. It is chosen to assign the tensile strength of the material to the system as the ribbons will mainly be subjected to tensile stresses. The orthotropic material characteristics will be used for the strength verifications of the different structural elements outside of the FEM program.



Load cases and boundary conditions

The loads from section 3.2 are implemented in their corresponding load cases into the finite element model. All loads are applied in the global projected coordinate system due to the expected large deformations of the structural system. The permanent and variable loads are applied on the ribbons and the elements of the inner ring as they make up the supporting system of the roof surface. The ribbons are loaded in plane and the inner ring is loaded out of plane due to its configuration as a flat truss. Furthermore, the boundary conditions are modelled in such a way that they do not obstruct the ring action to provide stiffness.

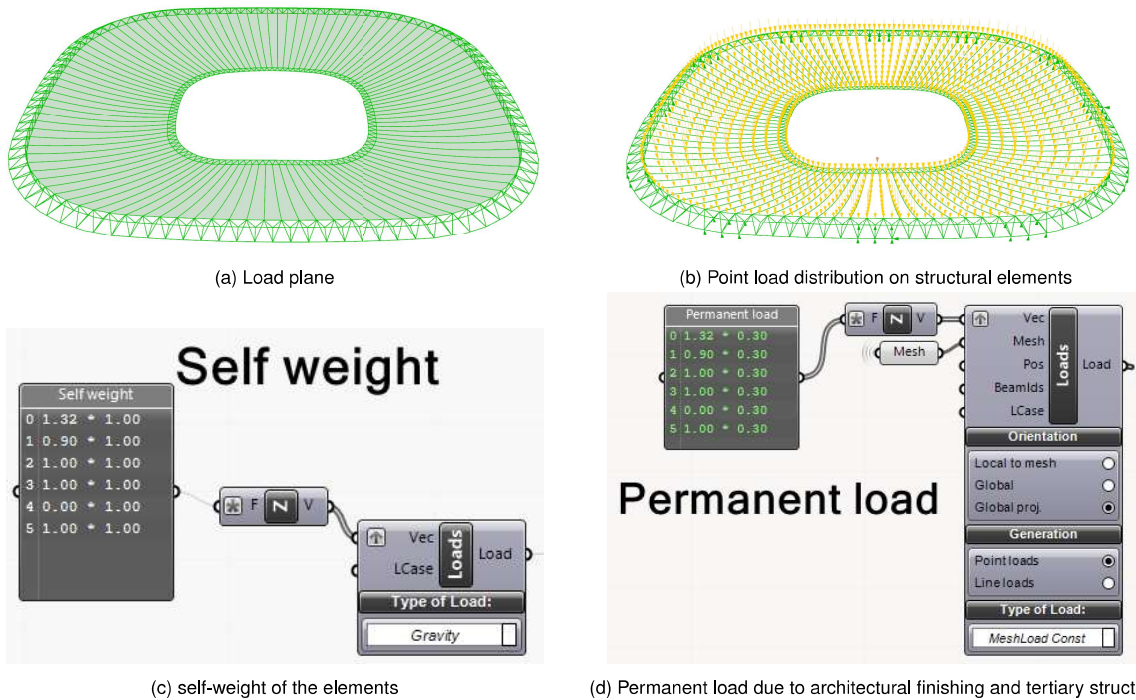


Figure 4.16: Permanent load modelling in grasshopper and Karamba3D

Permanent load

The Karamba3D plugin for Grasshopper has the option to apply a gravity load. This tool automatically applies the dead load in the global z-direction to all structural elements based on their cross-sectional area and density. The weight of the architectural finishing and tertiary members is applied by means of a surface load. The area load on the green marked area in figure 4.16a is converted to point loads on the ribbons and the elements of the inner ring, figure 4.16b.

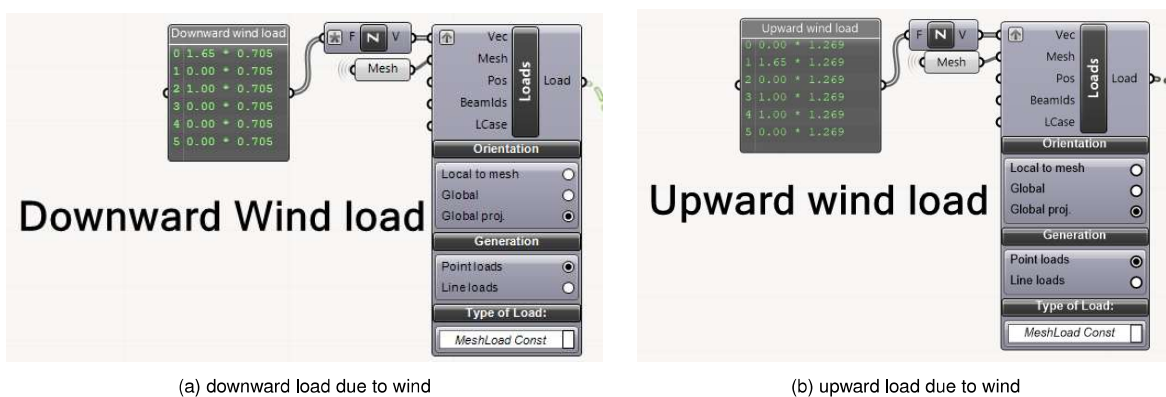


Figure 4.17: Variable load modelling in Grasshopper and Karamba3D

Variable load

It is concluded that the wind loading, instead of snow, results in a higher load on the roof surface. Only the variable loads due to wind are hence modelled. These loads are applied by means of a surface load similar as for the permanent load. For upward wind loading it is assumed that the self-weight has a favourable effect and thus the lower safety factor of 0.9 is used in these load cases. This can be seen in figure 4.16d. It is difficult to model a difference between the favourable and unfavourable effect that the self-weight has during asymmetrical loading. It is therefore chosen to apply the safety factor of 1.32 to the entire self-weight during asymmetrical loading. The asymmetrical load case is modelled as a separate load case and is thus not present in the list of numerical values (panels left in the sub-figures) as seen in all other loads. The asymmetrical loading configuration is seen in figure 4.18.

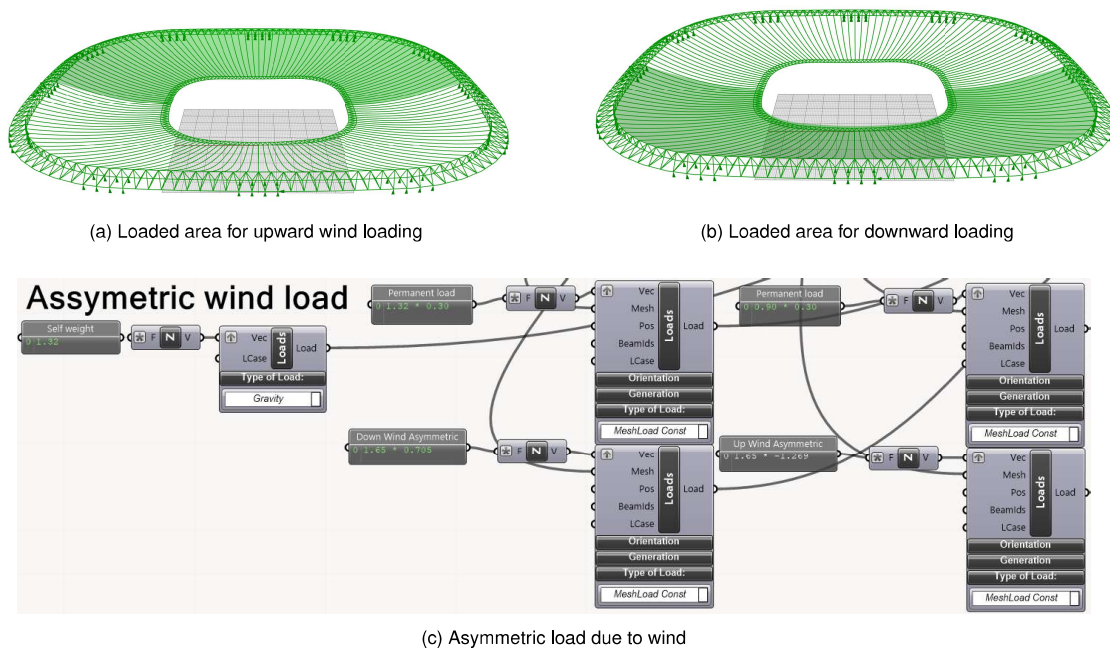


Figure 4.18: Asymmetric load modelling in Grasshopper and Karamba3D

Support conditions

The roof is supported on 12 cores, as mentioned in section 2.1, underneath the outer ring. Several supports are modelled at these locations, which only restrain movement in the global z-direction. Every core consists of four support points underneath the outer chord and four supports underneath the inner chord of the outer ring truss. However, one point in the model is also restraint in the global x-direction and another point also in the global y-direction. Otherwise the model would be able to "fly" away. These supports, in x-direction and y-direction, to create a working model take up no loading and are thus valid to be implemented. A overview of the roof with the cores is seen in figure 4.14a.

Connections

As mentioned in section 2.3.2, it is difficult and costly to create rigid connections in timber structures. It is even beneficial to apply hinged connections for the ribbons to allow them to stiffen by axial forces only. The large expected deformations are now possible without resulting in high bending moments near the rings. More forces are therefore taken by ring action instead of beam action, resulting in very slender ribbons. The diagonals in the trusses will also be hinge connected to transfer the forces evenly between all chords. The chords itself far exceed the allowed transportation limits and should be modelled with rotational springs to show its true structural behaviour. It is expected that this enables a better force distribution in the ring, which is beneficial for a more gradually distributed ring action than is expected now as a result of the non circular shaped rings. However, this will be accompanied with even greater deflections due to a loss of stiffness by means of these rotational springs in the rings. Due to modelling complexity and time limitation these rotational springs are left out of the scope. The supports (green arrows) and the hinges (yellow circles) in the model are shown in figure 4.19.

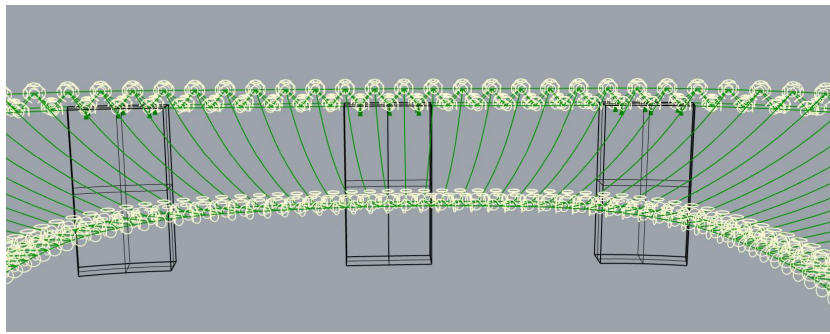


Figure 4.19: Supports at the location of the cores and the hinges between the elements

All elements are connected centric with each other, which is done to ease the modelling. This might be adapted in a later design phase in collaboration with the architects desires and depending on the cross sections of the elements. Some margin will be left in the unity check of the elements for this next design phase. Later design phases are left out of this thesis due to time restrictions.

Second order analysis

The problem of large deformations that occur in the stress ribbon and tension / compression ring roof can be handled by the second order theory. This theory takes into account the effect of the deflections of the structure in its calculation of the stresses and forces. The analysis is performed on a structural system where the ribbons and truss diagonals are hinge supported. The model with hinges has a few occurring problems that influence the reliability of the model, but can be ignored during the FEM model analysis. First, the second order analysis component doesn't work during upward wind loading because the system buckles. This is unrealistic due to the large buckling lengths of the ribbons, which is not the case in reality. Second, a slight twist of the inner ring is observed although only vertical loads are applied. This is the result of a small numerical error in the symmetry of the geometry. This error can be eliminated but will take a lot of man hours to do so. The twist is analysed and has a maximum of only 2 degrees and is thus ignored.

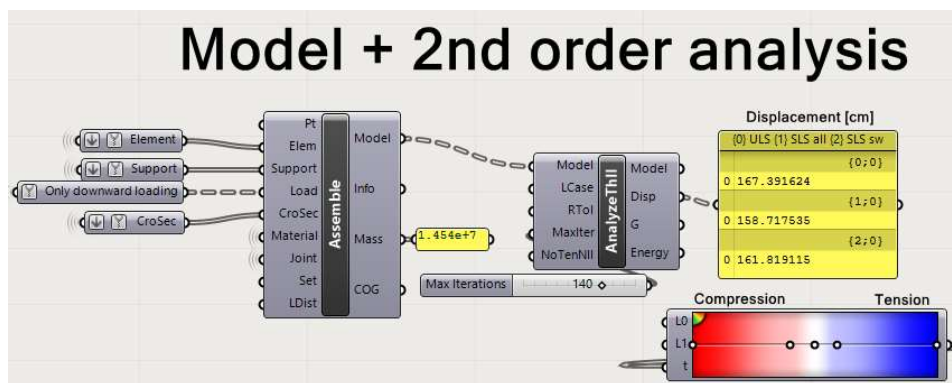
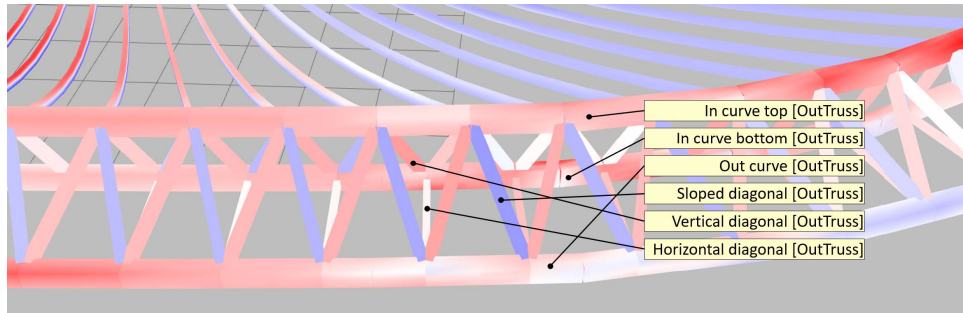


Figure 4.20: AnalyzeTH1 component for second order structural calculations

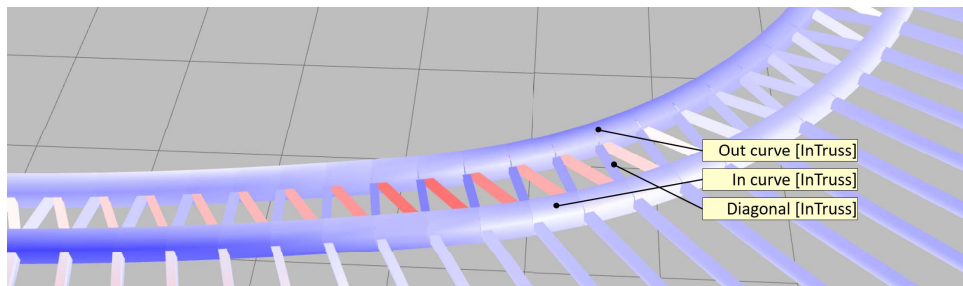
The second order algorithm takes the influence of axial forces on the structures stiffness into account. Compressive forces decrease a structure's stiffness, tensile forces increase it. When the system deflects the tensile forces in the ribbons and inner ring will stiffen the structure. The second order analysis component of Karamba3D can be seen in figure 4.20. In the same figure is the colour pallet shown that visualises the level of compression (red) and tension (blue) in a structural element. The darker the colour the higher the internal stresses. This colour scheme is normalized per result and will always take the maximum occurring stress as the darkest colour. This means that different types of modelling results can not be compared on colour alone. It will however provide a clear visual overview of the most stressed locations within a structural system. Other outputs of the model assemble component and the second order analysis component is the total weight of the structural elements and the maximum occurring deflections per load case.

4.4.2 Tension compression ring system

The way the structural systems are modelled is presented in the following sections. Insight is given in the way the cross-sections can be adapted due to the parametric environment of Grasshopper. Also the influence of certain parameters on the structural performance is presented. The visualisation of the structural performance is mainly shown as its axial stress distribution. Whereas mentioned in the previous section, the red colour resembles compressive stress and blue indicates tensile stress.



(a) Detail of the v shape configuration of the triangular outer truss



(b) Detail of the v shape configuration of the flat inner truss

Figure 4.21: Detail of the tension and compression ring systems

Outer ring

The outer ring consist of a triangular truss on which the stress ribbon is connected centric with the diagonals at the top part of the truss. The resulting horizontal force of the ribbon generates a tensile force in the sloped diagonals of the space truss in this way. The length of the sloped diagonals is quite long and a compression force would result in unfavourable buckling behaviour. The ribbons are connected under a angle to the outer truss, and this instigates a high vertical force as well, see figure 4.13. This force will be mainly carried by the vertical truss part of the triangular outer truss ring.

The chords of the outer ring are modelled as 288 straight elements in the FEM model. Karamba3D can only handle straight elements in its structural analysis. The amount of elements is double the amount of ribbons so that the diagonals can be modelled with a v-shape configuration. For every ribbon there are namely two diagonals. The amount of modelled elements is kept the same for all element groups in the ring trusses. These groups can be seen in figure 4.21.

The influence of the cross sections is mainly on the stiffness of the ring and thus the deflection of the ribbons and inner ring. A larger cross sectional area will provide more stiffness, this is especially valid for the width of the chords as they are in line with the forces generated from the ribbons on to the sloped diagonals. The height of the chords generates stiffness against the induced load by its self-weight between the cores. The outer ring spans around 60 metres between cores. So the sloped truss and vertical truss do most of the work of the outer ring and the horizontal diagonals strengthen the chords against lateral buckling. It is wanted to have similar dimensions for similar elements in the outer ring. This means that it is wanted to obtain a similar height and width of the chords and a similar height and width of the diagonals.

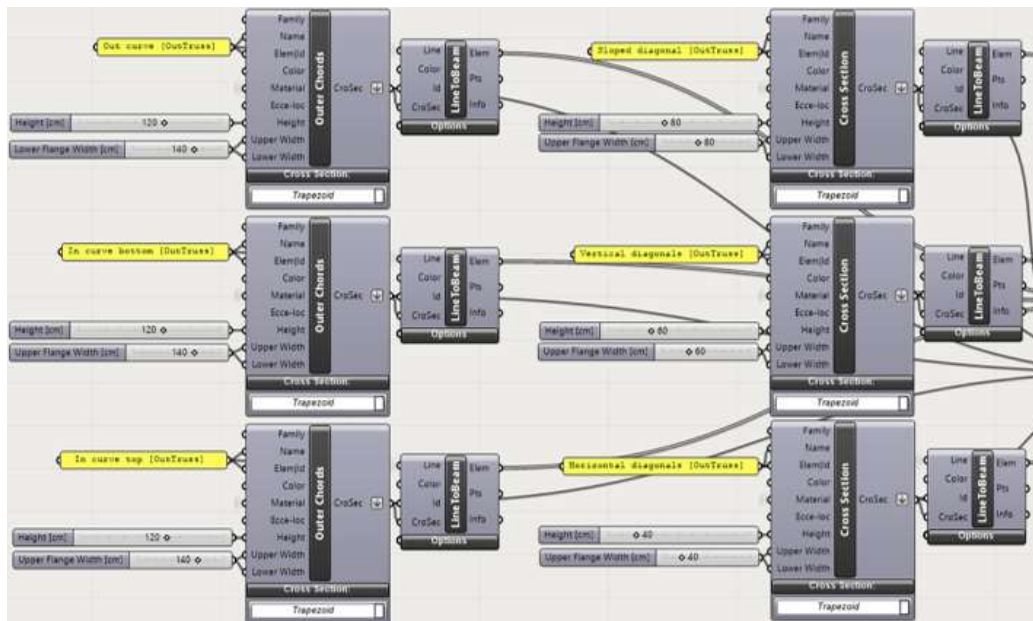


Figure 4.22: Parametric modelling of cross sections of the elements in the outer truss ring

First there was chosen to offset the v-shape configuration of the different trusses making up the outer truss ring (vertical, diagonal and horizontal truss which construct the triangular truss). This was done from the viewpoint that the connections would be less difficult when less elements meet in one node. Another reason is that the eccentricities will activate more neighbouring parts of the diagonals to activate more elements in circumferential direction for more ring action. However, this resulted in high shear forces between the nodes of the different diagonals. The diagonals now meet in the same node to avoid high shear forces due to eccentricities. This can be seen in figure 4.21a. The ribbons meet in the same node, as mentioned above, to transfer the forces without eccentricities.

Inner ring

The inner ring consists of a flat truss to reduce weight and still provide sufficient stiffness for the ring forces. The ribbons are connected centric with the diagonals and approach the inner ring under a very small angle. The resulting forces due to the ribbons will hence be mainly horizontal, see figure 4.13. Although the inner ring itself is transversely loaded it is expected that the ring forces will be much greater than these transverse forces.

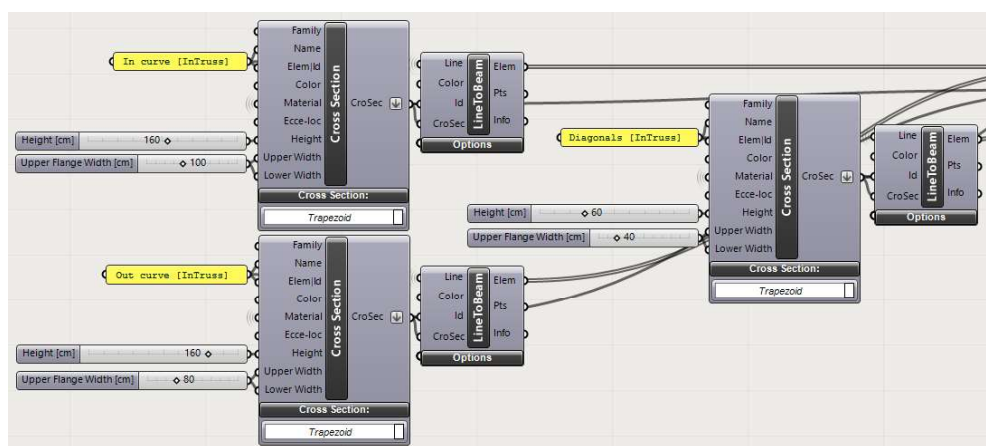


Figure 4.23: Parametric modelling of cross sections of the elements in the inner truss ring

the v-shaped configuration of the diagonals distribute the forces between both chords. A high tensile force is introduced from the ribbons, which would be beneficial to be dealt with by increasing the width of the chords as in the outer ring. However, as this inner ring is not supported and the difference in radius introduces beam action in the long and short side, it is more beneficial to increase the height of the chords. Also, loading is induced laterally on the flat truss from the roof panels so that increasing the height of the chords increases its stiffness against this load. The diagonals are kept square as they are being loaded by high tension and compression forces. This concept is seen in figure 4.21b.

A slight arch in circumferential direction is applied in the inner ring to reduce deflections at the sides with little curvature. This idea is taken from the Tokyo National Stadium as mentioned in 2.1.3. The ring arches downwards in the long and short straight sections of the stadium perimeter and arches upward in the corners. This is chosen due to their suspected deflection behaviour. The amount of arch is a parametric variable, which is chosen to generate a smooth transition within the ring but can be optimised when a more detailed design will be made. The influence of an arch effect upwards versus an arch effect downwards is shown in figure 4.24. It is clearly seen that for an arch effect upwards more beam action is present in the ring. This will reduce the deflection but will increase bending moments and compressive forces in the ring. As the design is oriented downwards and relies on internal tensile forces it is wanted to obtain only tensile forces in the inner ring. This is done by creating an arch effect downwards in the long and short side. The initial deflections will be higher but in this case the tensile forces will stiffen the whole structural system. Hence they decrease the overall deflection and maximum strength verifications of the inner ring. This in turn results in more bending moments in the ribbons at the straight sides. Which is not a problem as these ribbons have inherent bending stiffness. It fits the whole point of using timber ribbons instead of cables.

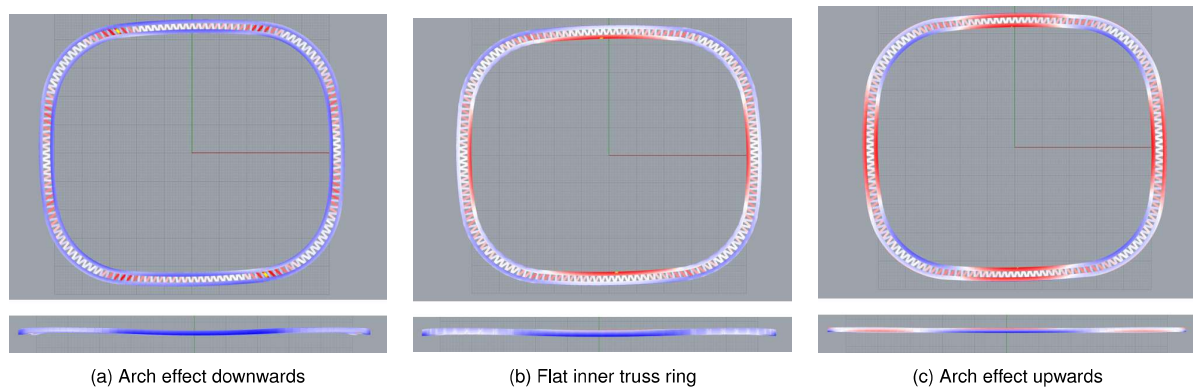


Figure 4.24: Visualisation of different arch effects consisting of a top-view and front-view

4.4.3 Stress ribbons

The ribbons follow an inclined catenary shape between the outer- and inner ring. The catenary shape is made by making use of the plug-in Kangaroo. Kangaroo is a Live Physics engine for interactive simulation, form-finding, optimisation and constraint solving. This engine allowed to make an equally curved catenary shape in the line of the ribbon elements. Another possibility of this engine is to find an optimal solution for the arch effect of the inner ring in combination with keeping the lengths of the ribbons as uniform as possible. This is time consuming, but gives a rewarding solution for a further design phase like easy manufacturing of the elements due to repetition of lengths. It is left out of the scope for now and thus have the ribbons different lengths due to the radial configuration in combination with the non-circular shape of the outer perimeter of the stadium.

The FEM plug-in Karamba3D can only handle straight beam elements instead of curved elements. Therefore, the resulting catenaries are divided into ten straight elements which follow the catenary shape. There is chosen for ten elements to be able to simulate a curved shape while minimising computational time. More elements will significantly increase computational time due to the size of the structure.

The stress ribbons in the straight sides will show more beam action because of the large occurring displacements at these places. The corners of the stadium roof are stiffer due to the ring action, which will result in mostly tensile axial stresses in the ribbons. Some places will hence experience bending moments independent from the low sag/span ratio.

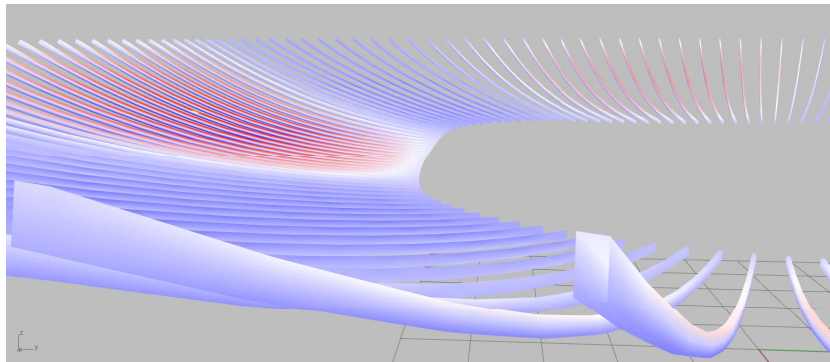
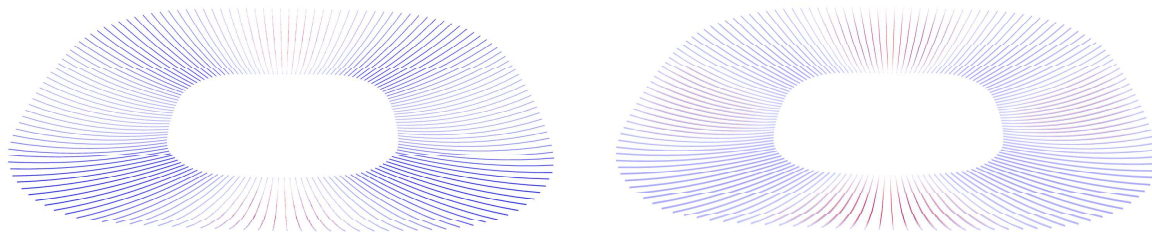


Figure 4.25: Detail view of the stress ribbons

An increased height of the ribbon results in more bending and smaller deformations as mentioned in section 4.3.2. Slender ribbons thus result in a better utilisation as they only carry forces by means of tensile stresses. This is seen in figure 4.26. The weight of the inner ring stiffens the ribbons and let them be more prone to solely tensile forces. Adding weight to the system by a heavier inner ring is hence beneficial, but will of course increase deflections. This is interdependent with the cross sectional area of the ribbons. A smaller cross section needs less weight of the inner ring to be stiffened. A downside is then that a very slender ribbon is more prone to buckling and instability during upward loading and asymmetrical loading. It is therefore wise to choose ribbons with some bending resistance and inherent stiffness.



(a) slender ribbons with more tension

(b) High ribbon with more bending

Figure 4.26: Axial stress utilisation of ribbons with different height/width ratio

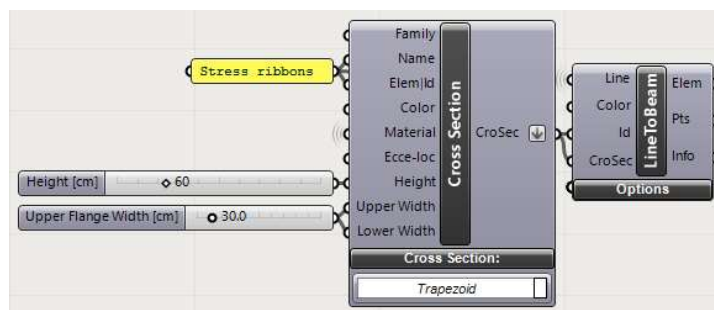


Figure 4.27: Parametric modelling of cross sections of the stress ribbons

The impact of the amount of ribbons is investigated. More ribbons will be better in distributing the loads between the trusses, result in less heavy loaded ribbons, and will minimise the peak forces in the ring elements. However, more ribbons also results in more diagonals in the trusses and thus more weight of the total structure. The spacing between the ribbons will be decreased when more ribbons are used, which influences the tertiary system. The estimation of the weight of this system is not changed and hence its load value stays the same. This influence should of course be altered in a next design phase. The outer perimeter of the New Feyenoord stadium consists of 36 straight elements creating the super ellipse shape. Due to the possibilities for parametric modelling, the amount of ribbons will be a multifold of this number. The effect of 72, 108 and 144 ribbons on the structural performance and cross sectional dimensions is investigated.

Increasing the amount of ribbons is beneficial for its structural performance due to less deflections and smaller internal axial forces. Which is also beneficial for the design of the internal connections of the ribbons. The individual ribbons can be dimensioned more slender in this case. Their is therefore chosen for a configuration with 144 ribbons as this will result in a better distribution between ring action and beam action. The different amount of ribbons is shown in figure 4.28, different dimensions of the ribbons are used to result in a similar total weight.

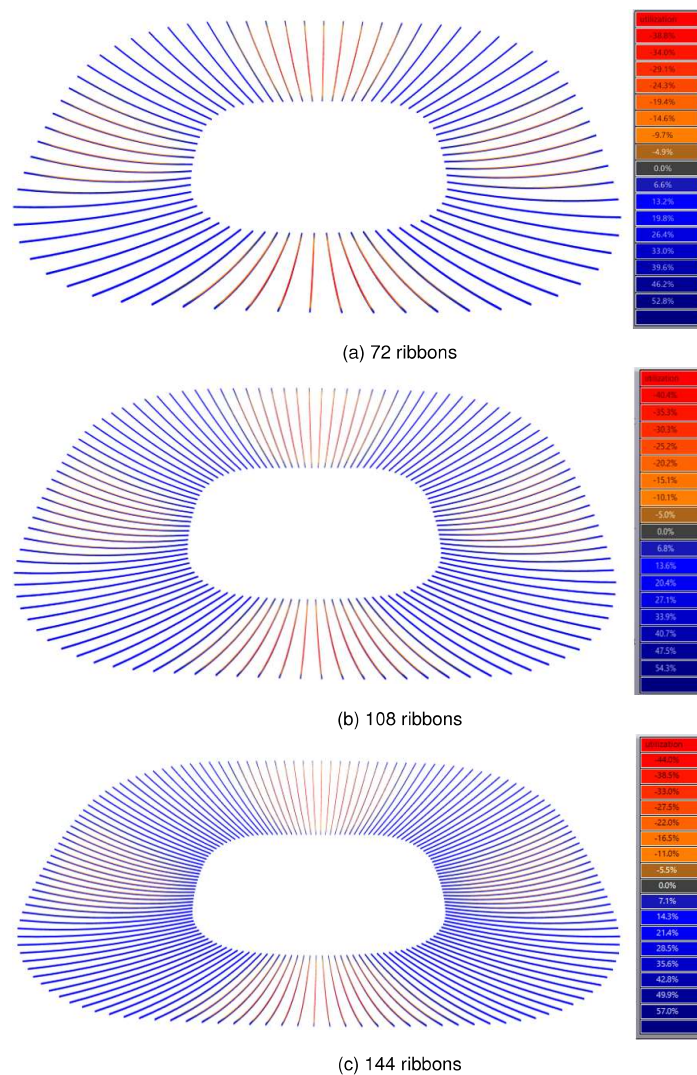


Figure 4.28: Amount of ribbons and their axial force utilisation

4.4.4 Model verification

As Karamba3D is not a certified FEM model it is of interest if the acquired results are reliable. The behaviour of a single ribbon which is loaded by a unity load is verified. A single ribbon is chosen in the Grasshopper model and a unity load of 10 kN/m is applied on only this ribbon. The same is done on a 2D schematisation of a single ribbon in the software program based on the finite element method SCIA Engineer.

A ribbon on the short side above a supporting point in a core is chosen for the quantification, see figure 4.29a. The ribbon is supported in z-direction at the location of the outer ring, which minimises the complexity of the system. The ribbon in SCIA is modelled with spring supports in all global directions at both endpoints. The material characteristics and non-linear analysis as well as the model discretization is kept the same as much as possible. A detailed explanation of the verification steps in both programs is explained in Annex B.

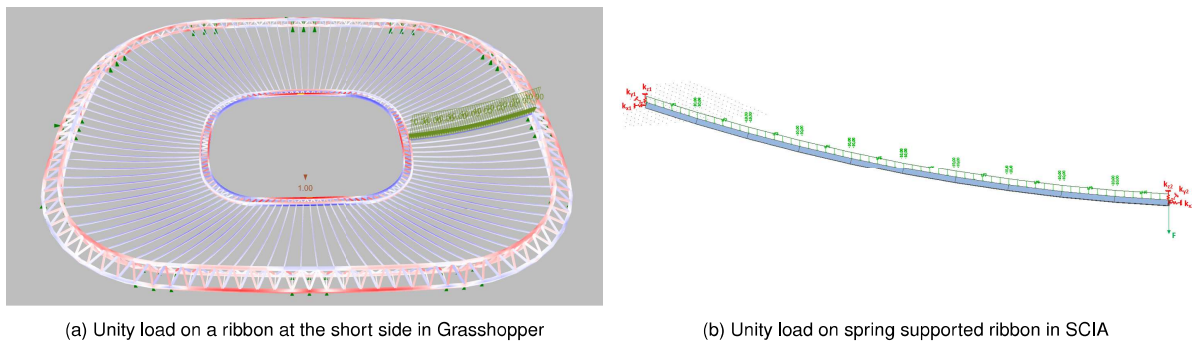


Figure 4.29: Model verification by means of a unity load on a single ribbon

The following values were compared: displacement in x-, y-, and z-direction, Normal force (N), Shear force (V), and moment (M). The ribbons are modelled as ten straight elements and therefore there are ten values for every structural indicator. The results from Grasshopper are presented in figure 4.30a and Those of SCIA Engineer in figure 4.30b.

Results Grasshopper						
Δu_x [mm]	Δu_y [mm]	Δu_z [mm]	ΔN [kN]	ΔV [kN]	ΔM [kNm]	
-48	-21	-1	1925	21	13.13	
32	-19	-223	1900	21	9.23	
96	-17	-419	1876	23	30.57	
142	-16	-577	1855	26	56.70	
171	-14	-689	1836	33	93.94	
184	-13	-753	1818	34	93.94	
186	-12	-773	1803	27	54.15	
177	-11	-742	1790	24	25.72	
161	-11	-658	1779	23	2.70	
141	-10	-528	1770	25	17.42	
121	-8	-364				

(a) Resulting values from unity load in Grasshopper

Results SCIA						
	u_x [mm]	u_y [mm]	u_z [mm]	N [kN]	V [kN]	M [kNm]
1	-45	-20	-1	1836	38	86
2	62	-13	-281	1811	31	138
3	148	-7	-524	1787	26	171
4	207	-3	-712	1766	22	187
5	241	-1	-831	1746	26	179
6	251	-1	-879	1729	27	181
7	244	0	-860	1714	22	195
8	222	2	-765	1702	28	185
9	189	2	-592	1692	30	154
10	151	4	-352	1684	44	104
	116	8	-62			

(b) Resulting values from unity load in SCIA Engineer

Figure 4.30: Resulting values of the model verification

It can be seen that both models show similar behaviour and that their values differ slightly. The difference lies in the amount of cable versus beam action that both ribbons use. The SCIA ribbon shows higher deflections with a higher internal moment, but a lower axial tensile force. It is assumed that the spring stiffness in z-direction in SCIA is modelled to stiff. Which is the reason that the ribbon deflects more at midspan resulting in more beam behaviour. In case of the Grasshopper model the ribbon can distribute more of its force through displacements near the inner ring and hence show more cable and ring action. This is probably due to the fact that the stiffness of the inner ring changes during the loading process in Grasshopper and therefore gradually stiffens. In contrast to the SCIA model were the final stiffness, which is the highest, is applied from the start. To really verify the results of the Karamba3D analysis, the entire structural system needs to be modelled into SCIA so that the stiffening influence of the tension and compression ring can be really incorporated. It is thus assumed that the Grasshopper model gives reliable results for now.

4.4.5 FEM Results

The structural system is designed to withstand large external forces, while minimising its total weight. Stiffness and stability can be found by assessing form, structure, material and analysis into one holistic approach. Simply increasing cross sectional dimensions of the elements to withstand the forces does not satisfy. This cross relatability is explored by analysing the impact of different cross sectional dimensions for the structural elements. The cross sectional analysis is therefore classed into the groups shown in figure 4.21 in the introduction of this section.

A stiffer outer- and inner ring reduce deflections and are therefore beneficial. However, they add weight to the system so their weight should be minimised, which can be done by increasing the cross sections of the ribbons. This results in larger horizontal forces which in turn increase the ring forces. The ring effect is mostly present in the chords of the trusses. These chords will show similar behaviour and have corresponding curvature and dimension, hence keeping them the same is beneficial for construction. The diagonals in the inner ring are tried to be kept the same as the vertical diagonals in the outer ring as they have similar length. The sloped diagonals of the outer ring take up most of the horizontal and vertical forces resulting from the ribbons and inner ring. Furthermore, they have the largest diagonal length and determine most of the stiffness of the outer ring. Increasing their cross section thus stiffens the whole structural system. The horizontal diagonals in the outer ring provide lateral stiffness for the vertical and sloped truss of the outer ring, but do not take up much of the applied loading.

The above stated effects are for the behaviour of the trusses as rings in the tension / compression ring system. Another aspect of the outer ring is its beam action between the cores. The outer ring spans distances of 60 metres between its supports and so needs to have enough rigidity to reduce its own vertical deflections due to self-weight. This means that increasing the cross-sectional dimensions of the outer ring to stiffen the structural system also has a negative influence on its own behaviour as a simply supported beam. This intricate process is analysed by alternating cross-sections to obtain better utilisation ratios of all elements as can be seen in Appendix C. The main criteria in this process is to keep the obtained utilisation ratio in the FEM program below 60%. This value is chosen because the performed checks in the FEM program are only valid for steel structures and because ultimately some margin is wanted in the unity checks done for timber. Furthermore, the elements in the loaded area gain weight by increasing the ribbon dimensions to obstruct uplift forces. The increased ribbon dimensions is also beneficial for the design of its connections. Almost all dimensions are similar, which is beneficial for manufacturing, transport and speed of construction. The cross sectional optimisation process resulted in the final cross sections of all elements shown in figure 4.31. These dimensions in turn determine the presented resulting strength verifications in this section.

Element	h [mm]	b [mm]
Out curve [OutTruss]	1000.0	1200.0
In curve bottom [OutTruss]	1200.0	1000.0
In curve top [OutTruss]	1200.0	1200.0
Sloped diagonal [OutTruss]	600.0	600.0
Vertical diagonals [OutTruss]	600.0	600.0
Horizontal diagonals [OutTruss]	400.0	400.0
Stress ribbons [144x]	600.0	400.0
In curve [InTruss]	1200.0	1200.0
Out curve [InTruss]	1200.0	1200.0
Diagonals [InTruss]	400.0	400.0

Figure 4.31: Resulting cross sections of the different elements

System checks

The load case with self-weight in combination with downward wind load results in the highest tensile forces in the ribbons and downward deflections. The highest compressive forces and largest upward deflection occur during favourable self-weight and upward wind load. The latter is however disregarded in the FEM model analysis as its influence is too severe to be controlled by the timber stress-ribbons alone. In figure 4.32 can be seen that the ribbons buckle during upward loading in the model. The ribbons work as arches when prone to upward wind loading and asymmetrical loads, resulting in a very large buckling length and thus instability. In reality the ribbons are propped every 2,5 metres by the tertiary structural system, which provides out-of-plane stability for the ribbons against buckling. Furthermore, the inner ring provides a stiff support for these arches. The effect of the tertiary system in combination with the inner ring against upward wind loading is unknown and due to time limitations left out of the scope.

If the presented system can not be stabilised by the tertiary system and Besista diagonals, more stabilising options are available. A small investigation on these solutions is provided in the next section. For now, it is assumed that the tertiary system in combination with the Besista rods stabilises the ribbons. It is stated that the resulting elements only provide stability for the symmetric downward loads applied on the roof structure. Therefore, the shown results are indicative for the answer to the research question and can not be seen as definitive. It is however of interest to see if timber can at least be feasible for the general case of downward loading as a structural material in long span stadium roof structures.

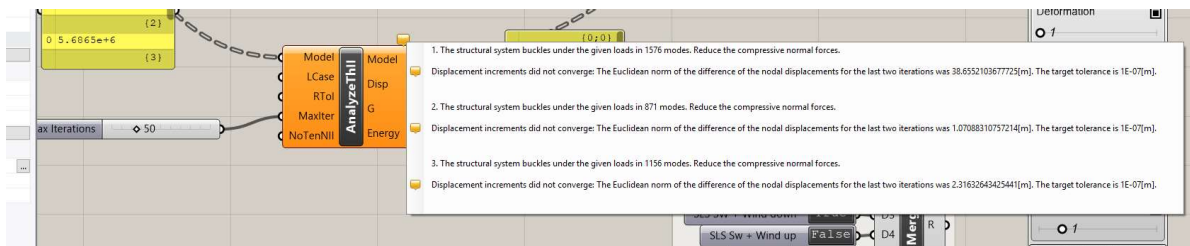


Figure 4.32: Resulting buckling modes for the analysis with upward wind loading

The maximum resulting values of the structural system for the load case with downward loading only are shown in figure 4.33. The supporting reactions are significantly lower than obtained in the global design phase. This is due to a better spread of the forces because of the ring action in the triangular truss, and a reduction of the weight of the system in the roof above the stands. The axial force utilisation of the entire roof system is shown in figure 4.34. The deflections are large, but it is expected that these can be allowed during construction by removing the substructure in a controlled manner. This is needed to activate the ring action of the inner and outer ring trusses. Besides, it will stiffen the ribbons by introducing internal tensile forces. For a next design step it is of interest to raise the height of the inner ring and thus reduce the inclination of the ribbons and see what a controlled release of the structure will do. The displacement of the entire roof system is shown in figure 4.35. Furthermore, the total weight of the structure is similar to the weight of the global design. This is an improvement as the structure now experiences heavier loading. The weight of the structural elements that make up the ribbons and the inner ring truss is also presented. It is purposely chosen to slightly increase this weight to counter the uplift forces with self-weight and minimise the occurring deflections.

		Structural system
Max. tension support	[kN]	1946 (short corner)
Max. compression support	[kN]	7090 (long corner)
Deflection at midspan	[m]	1.66
Total weight	[kg]	5.76E+06
Weight structure in loaded are	[kN]	31171

Figure 4.33: Maximum overall structural values

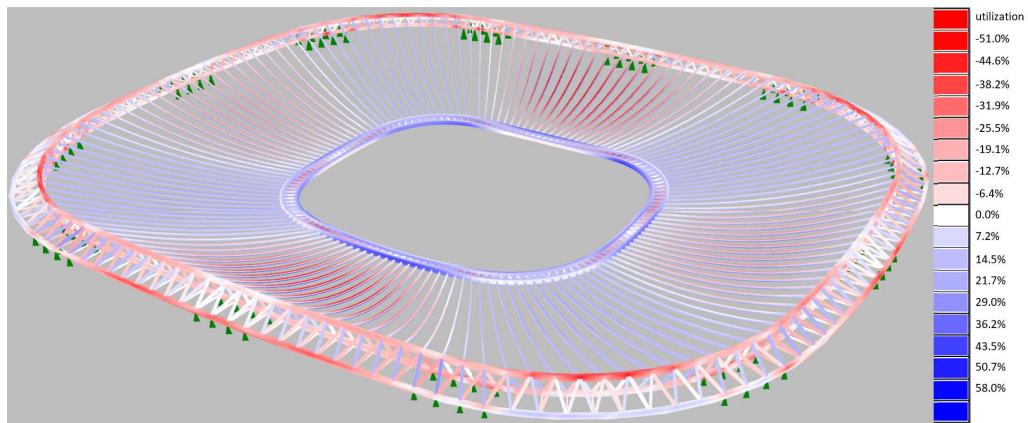


Figure 4.34: Axial stress utilisation of the total roof structure

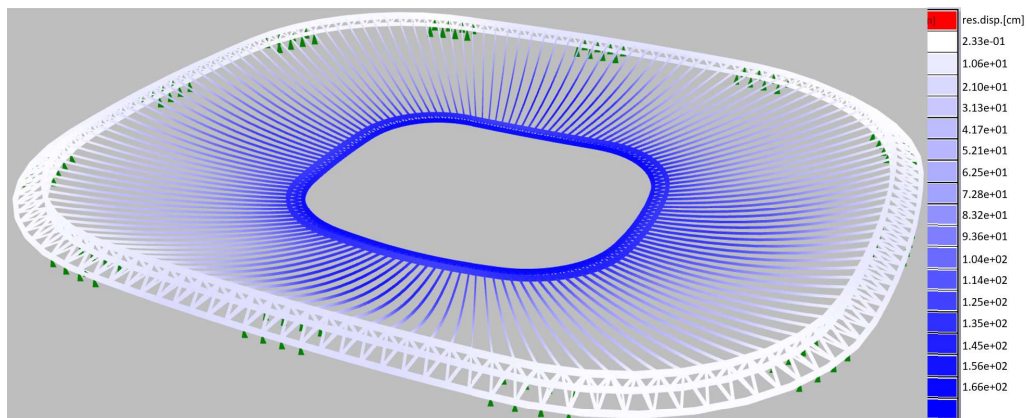


Figure 4.35: Displacement of the total roof structure

Tension-compression ring checks

The resulting strength verifications per element group for the load case with downward loading only are shown in figure 4.36, and 4.38. These strength verifications are done for the maximum internal force combination found in an element. Next to the unity checks, performed with timber strength verifications, are the average and maximum utilisation ratio provided by Grasshopper shown. These utilisation ratios are determined according to Eurocode 3: Steel structures. They are thus not accurate for this system but do provide insight in the utilisation of the elements.

Conclusions are drawn on these maximum unity checks in the structural elements. A unity check below 0.70 is wanted to incorporate the strength losses due to connections in a later stage. These maximum forces can occur because of localised effects as can be seen in the difference between average- and maximum utilisation in the elements. The 25% top values of the resulting internal forces are visually presented at their occurring locations with accompanying value in Appendix E. This is done per element group for normal force (N), shear force (V), and moment (M). Many values means that the calculated unity checks can be seen as normative for the entire element group. Few values means that the maximum unity check is only valid for localised effects. This insight will help answering the main research question and pinpoint the optimisation options for a next design phase.

Lastly, the range of occurring unity checks is arranged and combined with the percentage of occurring values that fall into each group. This is done for the unity checks determined by Grasshopper (Eurocode 3: Steel structures) due its possibility to process big data trees. This will over estimate the resistance of the structural elements against shear, but it will still provide insight in the utilisation of all elements. The range only goes to 60% as this is one of the demands for the cross section design, which is mentioned in the introduction of this section. It is optimal if all elements would find itself in the utilisation group of 0.4-0.5, however as will be shown, this is mostly not the case.

Outer ring

The resulting strength verifications of the element groups that make up the outer truss are presented in figure 4.36. The outer curve, the top inner curve, the sloped diagonals, the vertical diagonals, and the horizontal diagonals of the outer ring truss easily meet the strength requirements. This is because of the fact that their cross sectional dimensions are quite large to create a stiff compression ring, which in turn reduces deflections and internal forces in the other elements. These deflections are largest in the long side of the stadium and hence a lot of material is needed to provide transverse stiffness. It can be seen in figure 4.34 that the chords transfer high normal forces in a ring pattern. The top chord has a high axial stress in the corners, while the outer chord has a high axial stress in the long and short side. The diagonals undergo their highest axial stress utilisation near the transition from axial stress in the top chord on to the outer chord.

The bottom inner curve does not meet the strength requirements for the combination between torsion and shear forces. It can be seen in Appendix E that the maximum shear forces only occur around 8 places in the inner bottom curve. Thus can be considered as a local effect and hence can be strengthened by for instance screws. Another solution to this problem might be to use a v-shape configuration for the horizontal diagonals. This will result in a better distribution of the peak forces between the different outer truss elements. However, a lot of extra material will than be added for a problem that only occurs locally. The v-shape configuration should therefore only be applied at these 8 peak force locations. The local effect is the result of the different translational behaviour of the ring. The long- and short sides of the stadium want to move inwards while the corners want to move outwards. The maximum shear forces occur precisely at the locations of different translational behaviour (point of rotation).

The diagonals are mostly verified for their axial stresses. The occurring transverse forces and moments are near equal for all elements in these element groups. This is due to the fact that these internal forces largely come from their self-weight and not from the flow of forces in the structural system. The forces coming from the flow in the structural system are taken by means of axial stresses. This is what you would expect from the webs of a truss.

		Outer truss					
		Out curve		In curve bottom		In curve top	
Dimension	[h x b]	1000 x 1200 mm		1200 x 1000 mm		1200 x 1200 mm	
Bending + tension	[U.C.]	0.09	Average	0.07	Average	0.00	Average
Bending + compression	[U.C.]	0.33	[0.23]	0.32	[0.22]	0.41	[0.25]
Stability	[U.C.]	0.47	Max.	0.32	Max.	0.53	Max.
Torsion + biaxial shear	[U.C.]	0.39	[0.46]	1.09	[0.45]	0.16	[0.48]

		Sloped diagonals		Vertical diagonals		Horizontal diagonals	
Dimension	[h x b]	600 x 600 mm		600 x 600 mm		400 x 400 mm	
Bending + tension	[U.C.]	0.39	Average	0.27	Average	0.02	Average
Bending + compression	[U.C.]	0.16	[0.13]	0.18	[0.11]	0.08	[0.11]
Stability	[U.C.]	0.47	Max.	0.43	Max.	0.42	Max.
Torsion + biaxial shear	[U.C.]	0.00	[0.44]	0.00	[0.30]	0.00	[0.40]

Figure 4.36: Highest unity checks in outer truss due to the highest internal forces per element

The element groups in the outer ring truss are all made up out of 288 elements in the FEM model. A resulting utilisation ratio is found for every element. The percentage of the total elements within a group that fall within a certain utilisation range are presented in figure 4.37. The out curve and the in curve top are marginally utilised while the in curve bottom has a very bad utilisation, and therefore has the most potential to be optimised. The diagonals show even worse utilisation. However, the structural elements do provide stiffness with their increased cross-sections for the ribbons and inner ring. The utilisation ratios are the results of the load case with down-ward loading only. It is save to say that their utilisation will increase when the roof system undergoes the heavier upward loading.

Utilisation distribution [%]		Utilisation distribution [%]		Utilisation distribution [%]		Utilisation distribution [%]		Utilisation distribution [%]		Utilisation distribution [%]	
0 - 0.1	20.1 %	0 - 0.1	15.3 %	0 - 0.1	11.1 %	0 - 0.1	40.3 %	0 - 0.1	47.2 %	0 - 0.1	58.3 %
0.1 - 0.2	22.6 %	0.1 - 0.2	41.7 %	0.1 - 0.2	19.4 %	0.1 - 0.2	33.3 %	0.1 - 0.2	37.5 %	0.1 - 0.2	13.9 %
0.2 - 0.3	21.9 %	0.2 - 0.3	15.3 %	0.2 - 0.3	41.7 %	0.2 - 0.3	16.0 %	0.2 - 0.3	14.6 %	0.2 - 0.3	12.5 %
0.3 - 0.4	28.5 %	0.3 - 0.4	13.9 %	0.3 - 0.4	18.1 %	0.3 - 0.4	9.0 %	0.3 - 0.4	0.7 %	0.3 - 0.4	15.3 %
0.4 - 0.5	6.9 %	0.4 - 0.5	13.9 %	0.4 - 0.5	9.7 %	0.4 - 0.5	1.4 %	0.4 - 0.5	0 %	0.4 - 0.5	0 %
0.5 - 0.6	0 %	0.5 - 0.6	0 %	0.5 - 0.6	0 %	0.5 - 0.6	0 %	0.5 - 0.6	0 %	0.5 - 0.6	0 %

(a) Out curve elements (b) In curve bottom elements (c) In curve top elements (d) Sloped diagonals elements (e) Vertical diagonals elements (f) Horizontal diagonals elements

Figure 4.37: Utilisation distribution of the outer truss ring elements

Inner ring

The resulting strength verifications of the element groups that make up the inner truss are presented in figure 4.38. The outer curve, inner curve, and the diagonals meet the strength requirements. Their cross sectional dimensions are a bit larger than required. Increased weight of the inner ring stabilises the ribbons and makes them more prone to tensile forces only. Furthermore, increased weight will be beneficial when the design will include upward wind loading and asymmetrical wind loading. A heavier inner ring is hence beneficial for the overall structural behaviour, but too much weight will result in over dimensioned stress ribbons and outer truss.

Both chords experience high axial tensile stresses, which is desired as this means that they provide more ring action than beam action. The ribbons and the inner ring deflect in x- and y-direction to let the inner ring follow a more circular shape. This results in high normal forces, which is the most optimal force flow for the ring. The inner ring chords show the same behaviour as the outer truss ring. Namely the inner curve has its highest axial stress in the straight sides and the outer curve in the corners. In this way they form a more circular configuration to better transfer the forces. Both rings have most of their high bending moments in the straight sides. This is expected to be the case as these parts will deflect more and need inherent bending stiffness to reduce overall deflections. The arch effect downwards has a positive effect on this situation. It will introduce a catenary effect between the corners of the rings and result in more axial stresses than bending moments. Also the shear stresses are reduced by the better force flow because of the arch effect. The highest bending moments occur near the transition from the straight side to the corner. These points are the 'supporting restraints' for the catenary part of the inner ring, hence a large M_y occurs. Besides restraining the straight side of the inner ring they also experience large differences in normal force coming from the ribbons, which result in a moment M_z . Both these effects are large near the transition from the straight side to the corners.

The diagonals undergo their highest axial stress utilisation near the transition from axial stress in the outer chord on to the inner chord. Their most severe strength verification is for stability because of their slender cross sectional dimensions.

		Inner truss					
		In curve		Out curve		Diagonals	
Dimension	[h x b]	1200 x 1200 mm		1200 x 1200 mm		400 x 400 mm	
Bending + tension	[U.C.]	0.54	Average	0.56	Average	0.44	Average
Bending	[U.C.]	0.18	[0.28]	0.21	[0.31]	0.31	[0.18]
Stability	[U.C.]	0.02	Max.	0.03	Max.	0.59	Max.
Torsion + biaxial shear	[U.C.]	0.16	[0.43]	0.25	[0.45]	0.00	[0.47]

Figure 4.38: Highest unity checks in inner truss due to the highest internal forces per element

The element groups in the inner ring truss are also made up out of 288 elements in the FEM model. A resulting utilisation ratio is found for every element. The percentage of the total elements within a group that fall within a certain utilisation range are presented in figure 4.39. The in curve and the out curve are adequately utilised while the diagonals have a very bad utilisation, and therefore have the most potential to be optimised. The arch effect introduces mainly axial stresses and the truss configuration allows for more ring action resulting in a better utilisation distribution.

Utilisation distribution [%]		Utilisation distribution [%]		Utilisation distribution [%]	
0 - 0.1	0 %	0 - 0.1	0 %	0 - 0.1	29.2 %
0.1 - 0.2	25.0 %	0.1 - 0.2	0 %	0.1 - 0.2	27.8 %
0.2 - 0.3	27.8 %	0.2 - 0.3	54.2 %	0.2 - 0.3	26.4 %
0.3 - 0.4	37.5 %	0.3 - 0.4	37.5 %	0.3 - 0.4	13.2 %
0.4 - 0.5	9.7 %	0.4 - 0.5	8.3 %	0.4 - 0.5	3.5 %
0.5 - 0.6	0 %	0.5 - 0.6	0 %	0.5 - 0.6	0 %

(a) Inner curve elements (b) Outer curve elements (c) Diagonals elements

Figure 4.39: Utilisation distribution of the inner truss ring elements

Ribbon checks

The resulting strength verifications for the stress ribbons for the load case with downward loading only are shown in figure 4.40. The resulting unity checks are below 70%, which leaves design space for the connections, and stiffness for modelling errors that might be present. The resulting ribbons are slender which is what was expected after the global design analysis. Though they are still big enough to provide bending resistance to the system in the straight sides of the stadium. This is required due to the non circular configuration. However, their total weight in combination with the inner ring is not equivalent to the resulting force due to upward wind loading. If the ribbons should balance this force their cross sectional dimensions will be much bigger. An more in depth presentation on this situation is done in the next section.

The highest occurring moments are present in the long side. This is expected as they will deflect the most under the given load case. The arch effect downwards transfers more loading into these ribbons. An optimum can be found between the height of the arch and the bending utilisation of the ribbons. The ribbons are strong enough to provide stiffness against these bending moments. This shows the benefit of using cables with inherent bending stiffness. The highest axial stresses are present in the corners and these ribbons stabilises the structural system. Hence, the ribbons in the corner do more work as cables.

		Cables	
		Stress ribbons	
Dimension	[h x b]	600 x 400 mm	
Bending + tension	[U.C.]	0.65	Average
Bending	[U.C.]	0.60	[0.25]
Stability	[U.C.]	0.36	Max.
Torsion + biaxial shear	[U.C.]	0.07	[0.58]

Figure 4.40: Highest unity checks in stress ribbons due to the highest internal forces per element

The element group stress ribbons is made up out of 1440 elements in the FEM model. A resulting utilisation ratio is found for every element. The percentage of the total elements that fall within a certain utilisation range is presented in figure 4.41. The ribbons are marginally utilised due to the transition between cable action and beam action in circumferential direction, which requires less strength of those cables. However, the ribbons do provide stiffness to reduce the deflections of the inner ring. An optimum should be found between the allowed initial deflections, the additional deflections due to loading and its stiffness against upward wind loading and asymmetrical wind loading in combination with the available cable- and beam action.

Utilisation distribution [%]	
0 - 0.1	0.3 %
0.1 - 0.2	37.2 %
0.2 - 0.3	33.9 %
0.3 - 0.4	16.3 %
0.4 - 0.5	6.7 %
0.5 - 0.6	5.7 %

Figure 4.41: Utilisation distribution of the stress ribbons

4.4.6 FEM discussion

Uplift wind loading and asymmetrical loading is disregarded in the FEM model to find an solution for the most simple case to come to a simplified conclusion. However, the forces generated on the roof structure during these load cases are of great importance for the feasibility of a stress ribbon roof configuration. The solutions for upward wind loading are addressed by suggesting three options to restrain the presented roof in the previous section. The influence of the asymmetrical loading is addressed by showing its behaviour in Grasshopper. Both situations are discussed in a generalised manner due to time limitation and are not incorporated in other aspects of the structural design.

upward wind loading

Upward wind loading can not be addressed in this stage in the FEM model, therefore a brief exploration on possible solutions is done in this section. The resulting force due to wind loading on the total area of the roof is: $A * q_{d,wind} = 35186 * 2.09 = 73674 kN$

The found optimum in the previous section has a design weight of the ribbons and inner ring of:

$$\gamma_g * G = 0.9 * 34635 = 31171 kN, \text{ see figure 4.33.}$$

This weight is around half of the resulting force. The weight of the outer ring is left out of this calculation as it does not contribute to resisting the uplift forces. The following design solutions for the chosen configuration of the tension / compression ring with slender ribbons are addressed:

1. Increase weight to balance uplift force.
2. Tensile bars in the long and short side of the stadium.
3. Tension cables at the bottom side of the ribbons in the long and short side of the stadium.

Solution 1: increase weight

The cross sectional dimensions of the elements are increased to a size that results in a similar weight of the ribbons and inner ring as the resulting uplift force. The cross sectional dimensions should fall within the maximum dimensions that can be manufactured. The maximum utilisation of the elements is again kept below 60% in the FEM model to fulfil the timber strength verifications and to have enough strength to incorporate connections. This results in the following dimensions presented in figure 4.42.

Element	h [mm]	b [mm]	Average	Maximum
Out curve [OutTruss]	1600.0	1200.0	25.7	48.3
In curve bottom [OutTruss]	1200.0	1600.0	24.8	51.1
In curve top [OutTruss]	1200.0	1600.0	29.1	59.9
Sloped diagonal [OutTruss]	1000.0	1000.0	4.7	15.9
Vertical diagonals [OutTruss]	1000.0	1000.0	7.4	20.2
Horizontal diagonals [OutTruss]	800.0	800.0	2.6	12.0
Stress ribbons [144x]	1200.0	800.0	16.7	34.4
In curve [InTruss]	1800.0	1200.0	33.0	47.1
Out curve [InTruss]	1800.0	1200.0	31.9	42.4
Diagonals [InTruss]	800.0	800.0	5.2	13.9
Weight [total] , [load area]	1.47e+07 kg		81464 kN	
Maximum displacement	1590 mm			

Figure 4.42: Resulting cross sections of the system with increased weight

Resisting the upward loading and asymmetrical loading by equalizing it with enough self-weight is not an economic solution. You will need a lot of weight and hence very big elements. Also the cross sectional areas of the outer ring will be greatly increased as the ring forces are greater with greater self-weight of the ribbons, which result in even more total weight. It is optimal to find a solution with a certain amount of weight to restrain itself against downward loading and other solutions to increase the stiffness of the system to restrain upward and asymmetric loading. The effect that the ribbons will work as slender arches when subjected to upward forces is left out of this analysis. This will be advantageous for their structural resistance against uplift wind loading. The principle is shown in figure 4.43. As mentioned earlier, secondary timber elements are placed between the ribbons every 2.5 metres to stabilise the ribbons against buckling during upward loading. The ring supports need to provide enough stiffness to restrain the resulting compressive force coming from the ribbons. The corners will provide much more stiffness than the straight sides of the stadium

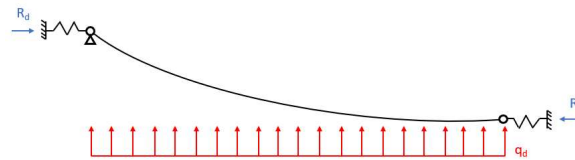


Figure 4.43: Stress ribbons work as slender arches when loading direction changes upwards

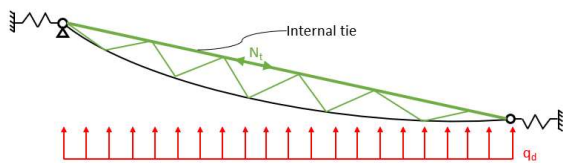
Solution 2: Internal ties

The resulting force from the uplift loading minus the self-weight of the timber elements can also be maintained by several internal ties placed above the ribbons. The material of these ties will be steel as they are placed in outside conditions, where the used timber material can not be placed according to the manufacturer. These ties will be situated at the long and short side of the stadium as it is assumed the least stiff part of the roof and will deform first. In this way the ties are immediately activated when the roof starts to deform during high upward wind loading. A schematisation of this system is shown in figure 4.44a. Diagonals are placed between the ribbon and the tie to stabilise them against buckling.

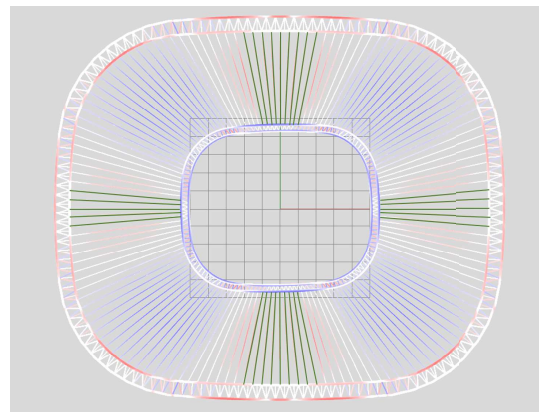
The resulting force is $F_d = 73674 - 31171 = 42503kN$. When this is translated to a distributed force over the length of a ribbon it gives $q_d = 42503/60m = 708.4kN/m$

The total tension component is $N_t = \frac{q \cdot l^2}{8f} = \frac{708.4 \cdot 60^2}{8 \cdot 2.5} = 127509kN$

A circular hollow section is chosen for the internal tie, CHS 273x16 in steel grade S355, with an area of $A_{CHS} = 12918mm^2$. This will give the required amount of ties to transfer the force. $X = \frac{N_t}{A_{CHS} \cdot f_y} = 28ties$. Therefore, 9 ties will be placed in the centre of each long side and 5 ties will be placed in each short side. This can be seen in figure 4.44b.



(a) Schematisation of the internal tie



(b) Placement of the internal ties

Figure 4.44: Internal ties to restrain the uplift forces

Solution 3: Tension cables

The same resulting force $F_d = 42503kN$ needs to be restrained. Tensile cables will be attached on the bottom side of the same 28 ribbons as in solution 2 under an angle of 30 degrees. When the ribbons start to move upwards due to wind suction these cables are activated and stiffened by tensile forces to obstruct the occurring movement of the ribbons. The resulting force from uplift wind results in the following tensile force per ribbon. $N_t = (42503/28) / \cos 30 = 1753kN$. To obstruct these forces 4 full locked coil strand cables with an diameter of 32mm from manufacturer Redaelli will be placed in the long and short side of the stadium to obstruct the movement. These cables have a design resistance of $R_d = 615kN$, which is more than a fourth of the resulting force per ribbon. The schematisation of this solution can be seen in figure 4.45.

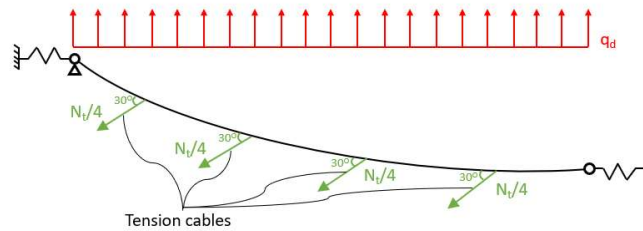


Figure 4.45: Schematisation of the tension cables to restrain the uplift forces

asymmetrical loading

The impact of the asymmetrical loading is explored by applying the load on the system with increased weight. Otherwise, the structural analysis can not be performed and the behaviour of the structure under this type of loading stays unknown. However, this system still misses the stabilising effect of the tertiary system and the Besista diagonals to transfer the forces to the stiffer corners. The asymmetrical loading is applied on the structural system as explained in section 4.4. The displacements in the structural system are shown in figure 4.46. The maximum deflection is very large and reaches a value of 5 metres. These deflections are very high at the long sides near the roof opening due to a lack of stiffness in transverse direction of the inner ring and a lack of ring action. Applying the same solutions as mentioned for the upward loading will result in a more stable structure.

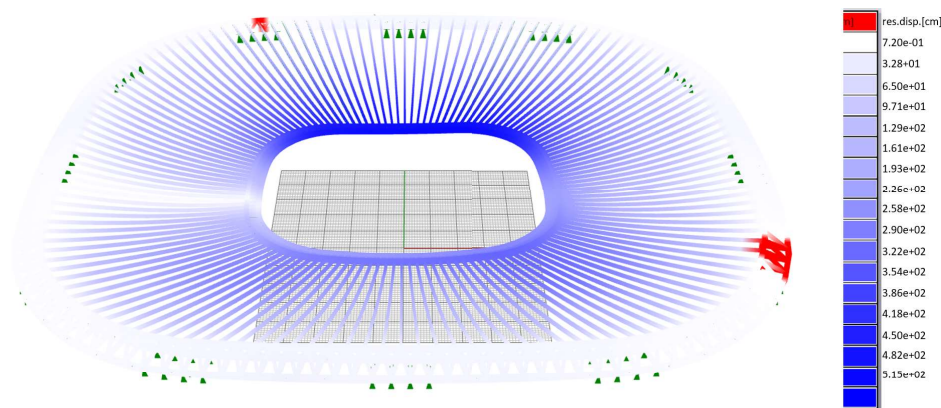


Figure 4.46: Deflections in the structural system due to asymmetrical loading

The force utilisation in the structure due to asymmetrical loading is shown in figure 4.47 from the perspective of the short side. This gives a good view on the high deflection that occurs in one long side in contrast to the other long side of the stadium. The ribbons in the corners do most of the work in the part that is subjected to upward loading as these are the stiffest. The ribbons in the long side deflect too much upwards to provide stiffness to the system for this load. For the downward loading it's actually the ribbons at the long side that are most heavily loaded. Large displacements downward will result in beam action in these ribbons. The mentioned solutions for upward loading will stiffen the system at the long and short side so that these large deflections will not occur. Increasing the stiffness of the rings in the straight sides is also a good idea to reduce these deflections and hence use more of the ribbons, which increases the usage of their beneficial strength properties. If these solutions are not enough to stiffen the roof than the number of steel elements mentioned in the solutions should be increased. More of these elements will direct more of the loading to the stiffer corners due to the radial configuration of the ribbons.

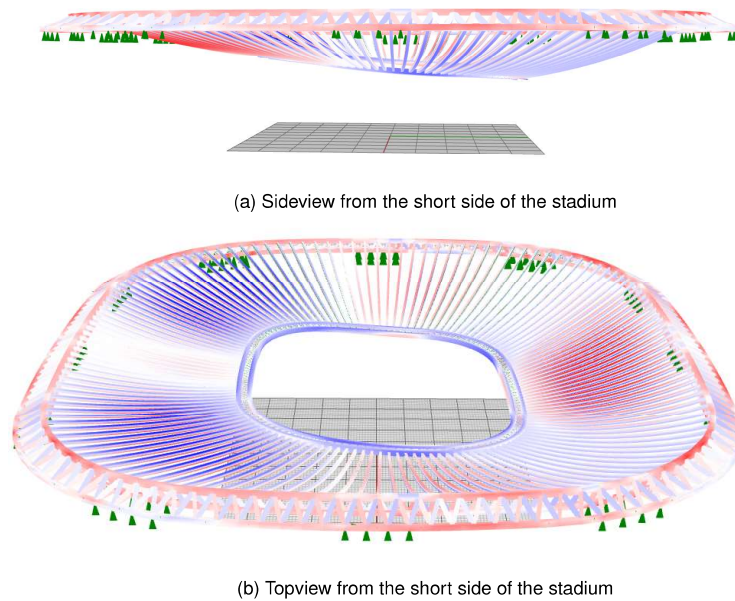


Figure 4.47: Axial stress utilisation in the structural system due to asymmetrical loading

4.5 Connections

The connections that are assembled on site will consist of mechanical steel fasteners as they allow for an easy assembly and are more reliable than on site glueing. The metallic joints should be less stiff than the timber material to ensure that the connections yield first as is described in the literature from the analysis phase. This ensures premature failure of the metallic joints and therefore a ductile failure behaviour of the entire system. The result is a safe structure that ensures safety for all spectators. The supplier of the timber material Baubuche recommends the use of new fasteners like self-drilling dowels with slotted in steel plates and glued in rods.[51] These new connections also minimise the inefficiency of the connections by making use of the timber to the utmost extend. However, the connections to create the long ring chords will use stronger dowels and steel fasteners in combination with steel plates. These chords are subjected to very high forces and would otherwise need many small fasteners.

There are seven types of connections between elements, where as much unity between the required tools and fasteners is kept. Not all connections are structurally verified due to complexity and time consumption to come to a good design solution. These connections are elaborated on their internal forces and design solutions are provided. This is valid for the connections of the tension and compression ring and their supports. The connections that are structurally verified are that of the ribbons, internal and external connections. The reasoning behind this is that truss structures have been made endless of times in the past and there are hence several proven solutions. This makes them less interesting than the connections of the ribbons, which are the main parts covering the spectators in the stadium. Another reason is the complexity of the internal nodes in the circular trusses, especially in the triangular truss of the outer ring, which need to be designed in collaboration with a timber supplier. The nodes need to be very stiff for excellent ring action, this requires high precision of the connection. Furthermore, an ideal repetitive connection is needed for manufacturability on site of the ring trusses. This is done in an intensive collaboration with the timber supplier to obtain the most feasible solution.

4.5.1 Truss-core

The outer truss will be supported on rolling bearing supports for stress distribution so that the ring action can be maximally utilised. There are six supports per core, which means three supports over the width of a core. The supports at the sides of the core do most of the work as they support the outer truss as beams between the supports. The top plate of the roller bearing support can be attached by glued in rods, like the GSA-AL mentioned in the analysis chapter.

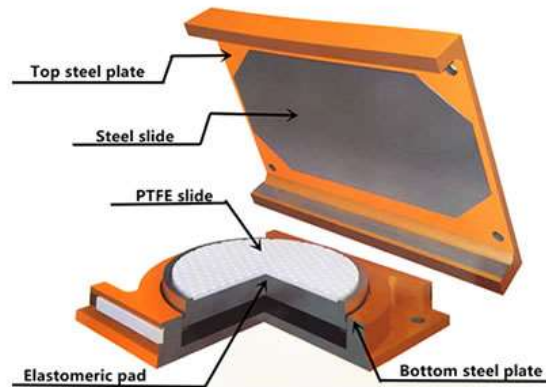


Figure 4.48: Roller bearing support for the outer truss - core connection

4.5.2 Outer triangular truss ring internal

This is a very difficult node due to the 3D configuration of the outer truss. It is chosen to create the vertical truss part with glued-in-rods as this falls within the allowed maximum dimensions for transport and can hence be prefabricated. The horizontal diagonals, sloped diagonals and outer curve will be connected by means of bolts in combination with dowels and slotted in steel plates. This connection type is also used in the Anaklia-Ganmukhuri bridge as shown in the analysis phase and in figure 4.49. The connection needs to be designed in close collaboration with the manufacturer. Many aspects come together in the design of this node, like high internal forces, transport limitations and many in situ nodes which ask for easy assembly with high repetition. An out-of-the-box solution might still be feasible due to the repetition of the node.



Figure 4.49: Complex node in the Anaklia-Ganmukhuri bridge for the internal connections of the outer truss ring

4.5.3 Outer ring - Outer ring

The splice joints of the outer truss ring will be made with steel plates with bolts and dowels like in the Anaklia-Ganmukhuri bridge. This bridge also consist of a triangular truss. However, the bridge also makes use of the HESS Limitless splice joint which is not recommended and cast steel parts to transfer the forces as shown in figure 4.50. It needs to be determined in close collaboration with the manufacturer what the best solution is for the outer truss ring. Parts of the chords of the outer truss ring are constructed with a curvature in contrast to the mentioned bridge.



Figure 4.50: Complex node in the Anaklia-Ganmukhuri bridge using cast steel parts for the splice joints in the outer truss ring

4.5.4 Inner flat truss ring internal

The inner ring also falls within the limitation due to transport and can therefore be prefabricated with glued in rods creating a very stiff ring. The curvature in the corners is the main challenge for the timber supplier. The goal is to distribute stresses over the internal elements and avoid accumulations of stresses. This can be done for instance by attaching the diagonals to a continuing beam part that has a length smaller than the maximum length for transport. The connections between the ring segments are than made at places where no diagonals meet.



Figure 4.51: Glued in rods connection in a truss for the internal joints of the inner ring

4.5.5 Inner ring - Inner ring

Rigid splice joints are wanted, just as in the splice joint connection in the outer ring. Slotted in plates with dowels and bolts or cast steel parts to transfer the high forces are a option. An intensive research is required to obtain a working solution in combination with upward wind loading and asymmetric wind loading for the high initial deflections. Furthermore, the connections need to fulfil the curvature due to the arch effect of the inner ring.



Figure 4.52: Complex node in the Anaklia-Ganmukhuri bridge using slotted in steel plates and the HESS Limitless joint for the splice joint of the inner ring

4.5.6 Stress ribbon internal

The splice joint to connect the segments of the individual ribbons, which is required due to transportation restrictions, is made with slotted in steel plates and self-drilling dowels reinforced with screws. This will result in visually continuing elements, which is architecturally appealing. These connections will be very visible as they are situated right above the stands. The reinforcement with screws diminish the group effect of the dowels for transverse splitting. Therefore reduces the required fasteners and improves the joint behaviour. It is best for fast assembly and a high contact area between timber and dowel if the connection can be made by self-drilling dowels. Their maximum penetration depth is 200 mm and can thus be applied from both sides of the ribbon to double the allowed internal steel plates and further strengthen the force transfer of the joint. The joint consists of 6 slotted in steel plates S235 of 5mm and 2*112 WS-T-7 dowels S235 of 7mm with 2*24 reinforcing screws. The schematisation of the joint can be seen in figure 4.53, all dimensions are in mm. The calculation of the splice joint is presented in Appendix F.

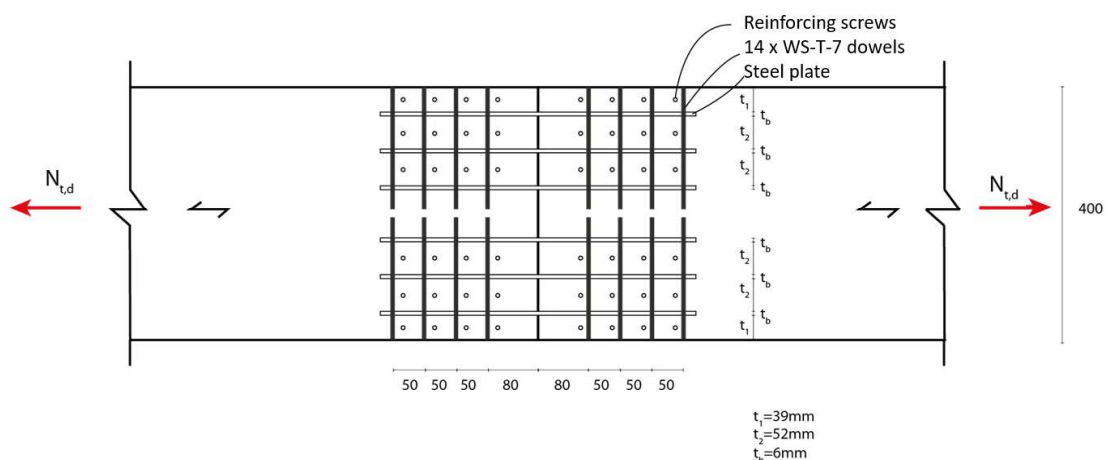


Figure 4.53: Splice joint in the ribbons using slotted in steel plates with self-drilling dowels from two sides

4.5.7 Outer ring - Stress ribbon

The connection between the outer truss ring and the stress ribbons consist of glued in rods connected to a steel end plate connection in the ribbons and a steel plate connected with screws on to the outer ring. The glued in rods can transfer high tensile forces and the hinge is obtained by a dowel connection between steel elements. The connection to the outer truss will be done with screws in a staggered pattern as they have a high resistance when applied in transverse direction in the timber. The staggered pattern diminishes the group effect of the screws and hence the full capacity of the group can be used. Both connection types result in an easy assembly on site, whereas the glued-in rods connection is prefabricated. The joint consist of 34 glued-in rods S355 of 16mm and 56 WR-T-13 screws of 13 mm. The dimensions of the steel parts is to be determined. All steel connectors follow the direction of the forces as can be seen in figure 4.54, all dimensions are in mm. The calculation of the joint is presented in Appendix F.

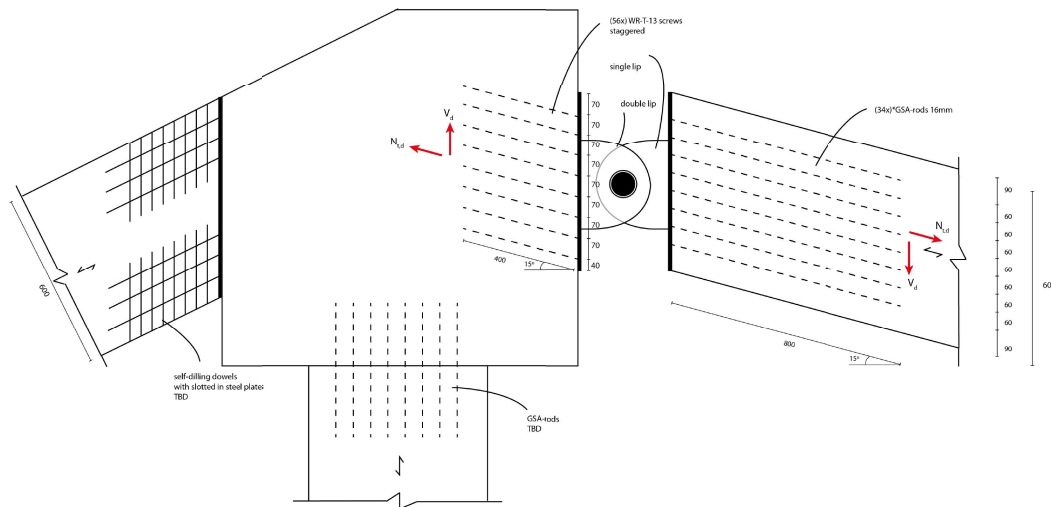


Figure 4.54: Connection between the ribbon and the outer truss ring using glued-in rods, screws and cast steel parts

4.5.8 Stress ribbon - Inner ring

The connection between the stress ribbon and the inner truss ring is the same as the connection for the outer truss ring and the stress ribbon. The joint consists of 34 glued-in rods S355 of 16mm and 56 WR-T-13 screws of 13mm. The dimensions of the steel parts is to be determined. All steel connectors follow the direction of the forces as can be seen in figure 4.55, all dimensions are in mm. The calculation of the joint is presented in Appendix F.

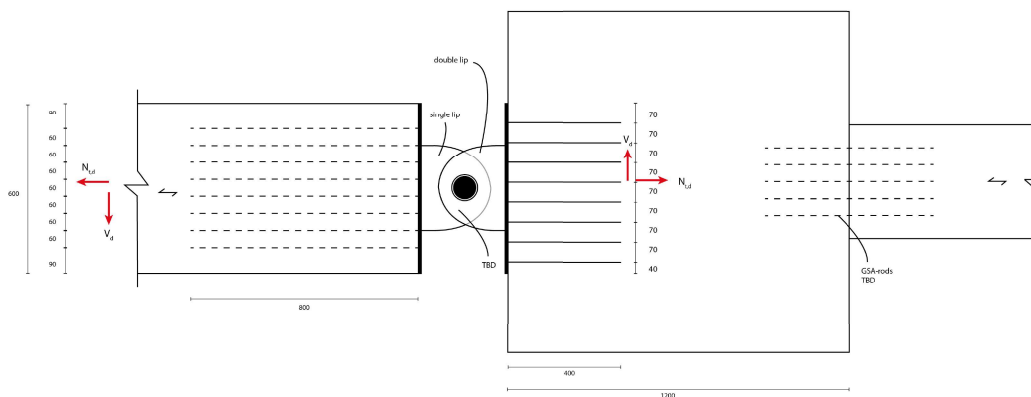


Figure 4.55: Connection between the ribbon and the inner truss ring using glued-in rods, screws and cast steel parts

4.6 Conclusion of structural design

The presented results only take downward loading into account, but does make a suggestion to withstand the resulting forces of the different load cases. It shows the possibility of a tension / compression ring system with stress ribbons as radial cables constructed with engineered timber made of European hardwood. However, it only shows its potential for the general case. An more in depth analysis of its stability against uplift forces and asymmetrical loading is needed to verify the proposed solutions.

Results of FEM

The presented structural design has a big difference between its maximum and lowest utilisation ratios. This comes mainly from the non-circumferential configuration of the tension and compression ring. The elements in the outer ring are required to provide a lot of the stiffness to reduce the occurring deflections. A large amount of weight can be diminished from the system by trying to find optimums between cross sections in the different element groups instead of using one cross section per element group. In this case the system can be designed for providing stiffness where needed. The straight sides of the stadium are a place where a lot of stiffness of the outer ring is needed. The corners, however, can be much more slender as they provide sufficient stiffness through ring action. The transition point between these areas where beam action goes over into ring action in the rings are most heavily loaded in an unfavourable manner. Different cross sections within an element group might reduce the occurring peak stresses and thus the maximum strength verifications. Otherwise, local reinforcement of these transition points can strengthen the elements against the peak stresses. It was also seen that the ring action divides itself over the different chords in both rings to follow a more circular pattern. This means that the inner curve top of the outer truss needs a large cross section for the axial stresses in the corners, while the outer curve needs less material as they take up less of the loading. Finding an optimal cross section of the elements within an element group is a good follow up for the next design phase to increase the feasibility of the system.

The same goes for the cross sections of the ribbons, which are now kept equal throughout. The ribbons provide a lot of stiffness in the corners with pure cable action, while the ribbons in the long and short side make use of their inherent bending stiffness to stabilise the system. It is shown that a slimmer ribbon will make more use of cable action, but also allow for larger deflections. Furthermore, the system will be weaker during upward loading or asymmetrical loading. For downward loading it is hence beneficial to have slender ribbons in the straight sides, while allowing the large initial deflection in a controlled manner to stiffen the system. That in turn reduces the induced forces on the weaker parts of the ring trusses and thus minimise the peak stresses. For upward loading it is wanted to have heavier and stronger ribbons in the long and short side of the stadium. These will encounter most of the unfavourable deflections upward owing to the weaker stiffness of the rings in these areas. An alternative to reduce the occurring stresses in the long and short sides is by decreasing the cross section of the ribbons at these locations and apply tensile ties or tension cables to restrain the upward loading.

There is still uncertainty with the structural system, which can be addressed and rectified over time by a collaboration between experts and people with the required skills. These rectifications can for instance be done by exploring pre-tensioning or post-tensioning of the ribbons like normally is done in cable roof structures. Searching for optimised solutions for the height and sag of the different elements relevant to each other. Fixing the arch effect of the inner ring in such a way that all the ribbons will have the same length or at least as much as possible. Tweaking the curvature of the inner and outer ring to obtain a better force flow in the ring action, the amount of ribbons and their configuration, and the configuration of the ring trusses. However, to obtain a view on the required elements and connections so that a conclusion can be made on the feasibility of timber all these aspects are chosen with a specific value. Thus the resulting model is not ideal nor totally realistic, but at least it provides a view on structural timber in a configuration and span width never done before. Different aspects that determine the entire structural design in this chapter are elaborated in the following subsections.

Finite element model

The second order analysis provides expected results, which are considered realistic when compared to the behaviour presented in the analytical model section. A simplified verification of the behaviour of the ribbons is done with the program SCIA Engineer. This showed similar behaviour to the FEM calculation of Karamba3D, but the results differ slightly. It is expected that this is the result of an unrealistic high value for the stiffness of the inner ring support in SCIA. To verify the results found by Karamba3D it is recommended to model the entire structure in SCIA Engineer.

Also, a small geometrical error is found in the Grasshopper model, namely twisting of the inner truss ring although only vertical loading is applied. This twisting is very low and thus its influence is assumed negligible. The geometrical error can be rectified, only this is a very time-consuming task. In this case, it is assumed that the results from the second order analysis are realistic for the load case of downward loading. For the case of upward loading and asymmetric loading, the second order analysis did not converge due to buckling of the system. Buckling should be prevented by implementing the tertiary system in the model, and by incorporating one of the possible solutions to help the ribbons cope with the upward loads.

Furthermore, the used FEM program has seen most of its development for steel structures. This includes a component that automatically calculates the strength verification of the elements according to Eurocode 3: Steel structures. Another component that exists is a cross-section optimizer that analyses the available steel profiles for their applicability in the structural system by doing automatic iterative calculations between the new found forces and the strength of the cross-section. The two tools make it a very applicable program for finding structural solutions for complex steel structures in a very short period of time. If these components would be developed for timber structures, the applicability of structural timber could be investigated more easily. Which will result in more confidence to implement timber structural solutions in the initial design phase. Especially, when the real behaviour of the connections can be automatically calculated and implemented.

The entire finite element model is constructed parametric, which allows to analyse the impact of many components. However, not all components are investigated as it is still time-consuming to do these analyses. The biggest advantage of the parametric model is that unforeseen problems can be easily altered to obtain new results. Furthermore, adaptations in next design phases are quickly made and a more detailed design is conveniently implemented in the original preliminary design.

Design variables

The impact of the design variables that can be altered are elaborated on their influence on the force flow in the structural system.

Height outer ring: when increased it will create longer sloped and vertical diagonals. The influence on the force distribution of the ring action is insufficient as almost the entire ring is in axial compression. However, the internal force flow in the outer truss ring due to the beam action can be reduced. The increased diagonals will be more prone to instability, and weaken the stiffness of the ring as their axial deformations will increase. Increased height of the outer ring will increase the wind force as well due to increased height of the entire structure.

Sag of the roof: increasing the sag of the roof is beneficial for cable action in the ribbons. Also, the initial deflections are lower when the sag is increased as the cables reach their state of equilibrium sooner. However, too much sag will increase the angle with which the ribbons join the outer truss ring, increasing its vertical force component and decreasing its horizontal component. The latter is guided through ring action, while the vertical component will be guided through the vertical diagonals into the supports.

Location of inner ring: determines the roof area that covers the spectators and thus the length of the ribbons. Placing the inner ring closer to the outer ring severely reduces forces in the system as the resulting forces depend quadratically on the length of the ribbons.

Arch effect of the inner ring: is most beneficial in this structural system when positioned downwards in the straight sides. Resulting in mainly tensile forces in the inner ring, which is what this structural system is designed for. The internal tensile forces also stiffen the system during the initial deflections. Increasing the sag of the arch will result in more tensile forces in the ring, but will introduce more bending moments in the ribbons in the straight sides. Furthermore, lowering the arch effect can obstruct sight lines of the spectators in the straight sides.

Amount of ribbons: more ribbons will result in more weight but smaller cross sections of the timber ribbon elements. The internal forces in the ribbons will be lower due to the fact that they carry less of the applied loading per ribbon. An optimum can be found between the cross sectional dimensions and total weight, also taking in consideration the effect of a smaller end-to-end distance on the tertiary system.

Configuration outer ring: the chosen triangular configuration with v-shaped diagonals is assumed to be the most efficient for this system with radial ribbons. The triangular configuration provides stiffness in the straight sides of the stadium. The horizontal diagonals should also follow a v-shaped configuration to distribute the peak stresses in the inner bottom chord of the truss. Furthermore, a requisite is that the radial ribbons are connected centric with the top part of the sloped- and vertical diagonals. The horizontal and vertical ribbon force can then be distributed in circumferential direction. The utilisation of all elements would be much more efficient if the outer truss would be able to follow a more circular perimeter.

Configuration of inner ring: follows the same curvature as the outer ring to minimize the eccentricity effects in the ring action. However, the internal axial stresses try to follow a more circular pattern so that it is assumed beneficial to let the inner ring follow a more circular configuration. This will diminish the necessity of the arch effect.

A triangular configuration of the inner ring will provide more internal stiffness, but will be heavier as well. The latter can have a positive influence on the internal stresses of the ribbons, as they will hence make more use of cable action. Increased weight of the inner ring is also beneficial for upward- and asymmetrical loading. However, a triangular configuration needs to be stabilized in the perpendicular direction against torsion or warping. Also, the amount of connections will increase and their complexity.

The height of the designed flat truss is a parametric variable that is kept constant at the same height as the vertical part of the outer ring. This is beneficial for construction, manufacturing, and transport. In contrast, enlarging this height will reduce the internal forces up to a certain level. It might therefore be beneficial to make it the same size as the horizontal or sloped part of the outer truss ring. Special attention should be given to the fact that parts of the inner ring truss can then not be prefabricated any more due to transport limitations. Hence, the connections will cost more effort and will have a negative influence on the design.

Configuration radial ribbons: to ease modelling complexity there is chosen for single ribbons at a regular spacing. The amount of ribbons is checked as mentioned earlier, however it is not checked if for instance coupled ribbons at a regular spacing are more beneficial. This might be true due to a reduction in weight, and increase of stiffness of a coupled element like in the Aquatics Height Centre. It will be mostly beneficial for uplifting forces as the 'arches' now are constructed with a stiffer configuration.

The initial curvature of the ribbons have impact on the occurring stresses. The curvature is created by means of a uniform line load creating a uniform curvature with the use of Kangaroo. An increased curvature will result in more cable action, but also higher vertical reactions in the outer truss. Timber elements that are prefabricated with a curvature have some residual stresses, their influence on the final internal stresses should be investigated.

The ribbons should be of even length as much as possible for ease of construction. Which can be modelled within the Grasshopper model as a restriction determining the arch effect of the inner ring. This is a difficult and time consuming restriction to implement and was therefore left out of the thesis. It would be very beneficial for the final design as repetition reduces the possibility for errors during construction.

Material characteristics: the strongest timber product is chosen and therefore provides the most slender cross-sections. Regular glued laminated timber would result in even bigger elements. Furthermore, the hardwood glulam has higher transverse resistance, which is beneficial for the connections with steel fasteners. The embedment strength of the timber is higher and hence the load carrying capacity per dowel. Adequate detailing is required to keep the material out of direct contact with the elements to satisfy the requirements for service class 2.

Loading: the loading is applied as uniform mesh loads. They are modelled as separate load cases that create separate models. This is a clever way of modelling, however works mainly for uniform loading. Asymmetric load cases were the self-weight works favourable and unfavourable in the same load case is difficult to model in Grasshopper and Karamba3D. The problem is in the application of different safety factors to an incorporated component for self-weight.

The structural system is verified for wind loading as this results in the highest loads on the structural elements. However, the value of the load is a conservative estimation as the real values are very difficult to determine by generalised factors for such a special structure. The realistic load factors should be determined by a wind tunnel analysis and a computational fluid dynamics analysis. In this way the influence of the adjacent buildings can be incorporated, as well as the non-circumferential configuration with a downward sloped roof that has an opening in the middle. Also, the influence of the outer truss that initially directs the wind loading upward by means of its chosen triangular configuration can than be determined.

Type of analysis: a non-linear analysis is required due to large deflections, which in turn influences the internal force flow of the system. Furthermore, it is a very complex and large parametric model and hence prone to small errors that lead to a deviation from reliable results. More confidence in the found results will be obtained when the structural system is verified by a different FEM program in its totality. It can hence be concluded that the chosen structural system is very complex and might not be a realistic and mostly a affordable solution. However, this is required when a structure is made of twice the size ever constructed to be feasible. Expertise in the field of stress ribbons and tensile compression ring structures can provide a view on the matter, and conclude its potential on the scale of The New Feyenoord stadium. The models might also struggle with the lack of stiffness and strength of the timber in combination with the large deformations. The found results acquired with a second order analysis can hence be verified with a similar analysis in a different FEM program, but the real answer will come from experience with similar structures.

Cross section of the elements: a problem with the iterative cross section study of the elements is that their impact needs to be verified by hand within the FEM model. Karamba3D has a cross section optimizer for steel cross sections that finds the optimums by itself. A similar optimizer would be very beneficial in the case of timber structures. This requires the incorporation of a component which determines the utilisation of the elements based on Eurocode 5: Timber structures and a component that searches for potential cross sectional dimensions that can be made. Now, a lot of time is spent on finding an optimum by individually adjusting sliders and than rerun the structural analysis. A structural design made of timber becomes very time consuming for such an innovative structure as the New Feyenoord stadium.

Supports: the supports are restrained in z-direction in the outer truss ring. This allows the outer truss ring to carry the forces with ring action. They show very low supporting reactions due to the light weight of the timber. However, when the required structural solution for asymmetrical loading and upward loading caused by the wind is implemented these reactions will be different. nonetheless, when the more light weight solutions are implemented the difference is minimal.

FEM discussion

It would be best if the ribbons could resist the uplift wind forces as slender arches, which are strutted every 2.5 metres. Then only timber material is required with no extra complex nodes and elements. However, as the uplift forces are twice the weight of the ribbons this might be insufficient.

The solution of adding more weight to the system by increasing the cross sections of the elements to huge proportions is unwanted. This will never be excepted from an architectural point of view. Furthermore, the increased weight is in contrast to the mindset behind the choice for this structural system. Which is to create an elegant lightweight structure so that it can be a good alternative for the now build steel roof structures.

The solution with tensile ties is interesting as these elements do not obstruct the bottom side of the timber structure. Not many ties are required and they have a moderate cross section for such a span. However, their impact during downward loading should be investigated and might result in unfavourable buckling behaviour despite being stabilised by diagonals.

The solution with tensile cables can be a very promising one as the cables only work when the roof is subjected to uplift forces and hence no complex additional investigation is required for downward loading. However, these cables need to be attached somewhere, and in the presented solution this is in the stands. Which is of course not wanted, because they clash with the demand of having column free tribunes.

If all solutions are not sufficient than the question arises what an appropriate size is for the football stadium roof to be made out of slender ribbons with additional solutions for uplift forces. A combination of steel and timber is already used in the Nice and Tokyo stadium to work for the different load cases as a smart unity.

Connections

The connections that need to construct the ring trusses are very complex and should be designed in close collaboration with the manufacturer. This should be done while bearing in mind not only strength, but also stiffness, ductility, durability, and ease of construction. The highest performance can be reached when as much connections as possible are prefabricated.

The designed joints for the connections between the ring trusses and the ribbons fulfil the strength verifications. This resulted in a connection with 56 WR-T-13 screws that attach a steel plate to the truss ring, a to be determined steel pin connection, and a steel plate attached to 34 glued-in rods S355 of 16mm in the ribbon. The screws follow a staggered pattern to diminish its group effect and are screwed in transverse direction to the grain in the timber element. The glued-in rods are glued in longitudinal direction to the grain in the end points of the ribbon, where block tearing is diminished by the low yield strength of the individual rods.

The internal connection to join the different segments of the ribbons together to reach the required lengths fulfils the strength verifications. This resulted in a connection with 6 slotted in steel plates S235 of 5mm and 224 WS-T-7 dowels S235 of 7mm with 48 reinforcing screws. The self-drilling dowels are shot into the timber from two sides. This is assumed to be applicable, but should be verified by the supplier. The reinforcing screws diminish the group effect of the dowels and hence maximise the efficiency of the joint. This type of connection requires many steel fasteners to obtain a ductile failure behaviour. The amount of required fasteners can be reduced when higher steel grades are used and larger diameters for the dowels or even bolts. An increased capacity per fastener is the result, but also a more brittle failure behaviour. The redundancy of the structural system is than much more determining for the overall safety of the structure.

The impact of the different connections must be incorporated in the FEM model as these will influence the results. The connections will result in a spring stiffness of the joint, and cannot be a rigid connection. Which means that the chords of the trusses will have spring stiffness every 20 meters due to transportation limits. This will probably increase deflections, but will redistribute the peak forces in a

more favourable manner. The ribbons will weaken as well and also result in higher deflections of the system. This influence can be incorporated in the FEM model in Grasshopper due to the parametric framework in a next design phase.

Construction

Much unity is kept between the structural elements and as many aspects as possible should be pre-fabricated. This will result in a fast assembly of the structure, due to a easy build up. Furthermore, the transportation of the elements can be very efficient because of the similar dimensions.

It is of interest if the ribbons can be attached on the ground to the inner ring and than jacked up to the outer truss on the cores. Otherwise, the inner ring should be supported in the centre and the ribbons are attached at height between the two rings.

The deflections are high, but there might be a chance that these can be allowed. The temporary supports of the inner ring should be slowly lowered during construction. The structure is than able to settle and stiffen itself until the expected deflection due to self weight is reached. This process needs to start above the expected height of the inner ring with precisely the height of the expected deflection. Sort of like a camber of a beam, but in this case for the entire roof. 'De Kuip' made use of the same principle to allow the tension ring to strengthen itself during the lowering of the temporary supports.

The required connections should be assembled on the ground at the building site, creating the possibility for the joints to be assembled with more ease and by more heavy manufacturing tools in production tents.

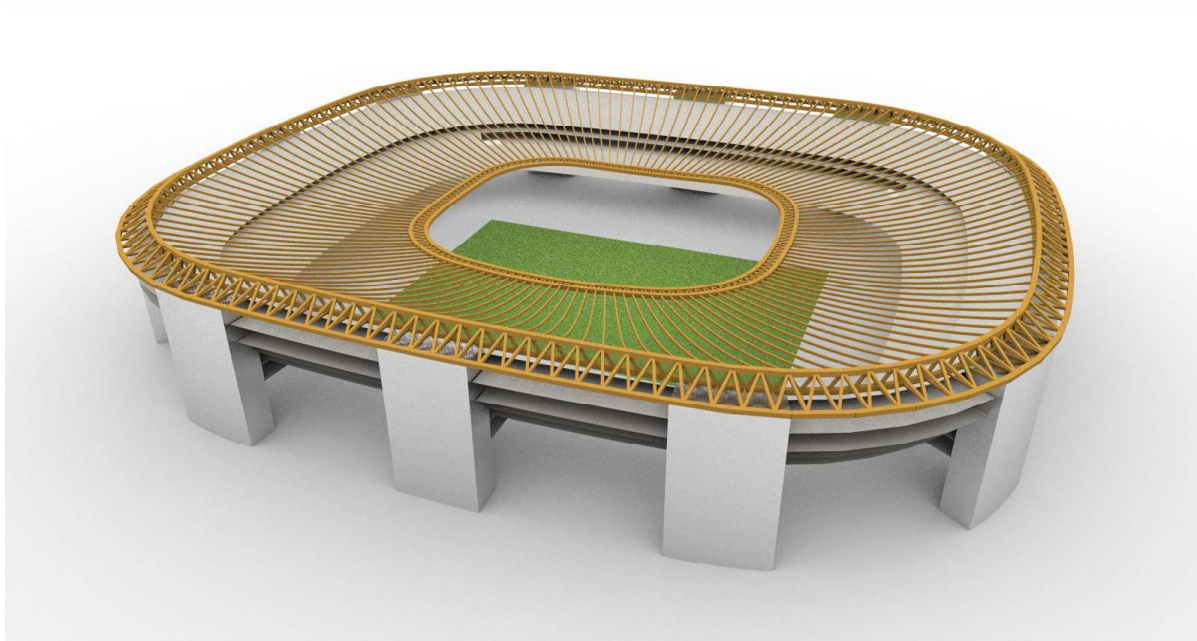


Figure 4.56: Final long-span timber roof structure for The New Feyenoord stadium

5

Conclusion

An answer to the main research question is given and shows the potential of the system. The structural design is elaborated on its feasibility. The force distribution and the strength verifications in the system are discussed. Finally, the main conclusions of this thesis are drawn.

The conclusion of this thesis is based on the structural performance of a tension / compression ring system with stress ribbons as secondary structure. It is estimated after research, based on the input gathered from literature and rough calculations, that this will be the most efficient configuration for a long-span football stadium roof for the New Feyenoord stadium.

Feasibility of a timber long-span stadium roof structure for the New Feyenoord stadium

The structural design shows a feasible solution for the load case with downward wind loading, which is the loading that results in the highest downward load. However, the impact of upward loading will be very severe for this type of structure. Several potential solutions to withstand this problem are shown, like increasing the weight of the structural elements in the loaded area, tensile ties in the long and short side of the stadium, or tensile cables attached to the bottom side of the ribbons in the long and short side. These solutions show the possibility to restrain the upward loading and will make the design more feasible.

The force distribution is non-linear in the New Feyenoord stadium due to the non-circumferential perimeter of the stadium. This results in peak forces, which in turn result in peak stresses in the elements. The only strength verification that is not satisfied during downward loading is the shear stiffness of the bottom chord at the inside perimeter of the outer truss ring. It is, however, a very localised effect and hence strengthening the cross section with screws against shear at these spots can be seen as a feasible solution.

All other element groups of the ring trusses fulfil the strength verifications for downward loading with enough margin to incorporate the complex connections. These strength verifications are done on the maximum occurring force combinations within the element group. These element groups show a undesirable utilisation distribution. So a lot of the elements have a lower utilisation, making them even more feasible for the situation of downward loading. The large cross sections are required to provide stiffness to the entire structural system due to the non-circumferential perimeter of The New Feyenoord stadium.

The ribbons are very slender and show an elegant internal force flow consisting of almost only tensile forces in the corners. The ribbons in the long and short side of the stadium make use of their inherent bending stiffness to carry the loading with beam action. The question remains if this will be the case during upward wind loading as well, but then for internal compression forces. Furthermore, the resulting horizontal forces of the ribbons are nicely taken by the rings and thus minimise the forces on the supports. At last, the designed connections which are attached to the timber fulfil the strength verifications. They consist of many small steel fasteners to increase the efficiency factor of the joint.

It is thus concluded that a timber long-span stadium roof structure consisting of the chosen structural system shows potential to be a feasible solution for the New Feyenoord stadium. It will be a grand architectural statement that makes a stadium iconic, being the only timber tension / compression ring stress ribbon roof structure spanning with an exceptional distance.

6

Discussion

This chapter discusses aspects that might influence the results and conclusions in this thesis. The structural system to bridge the long-span in timber. The geometry of the structural system. The impact of the connections. Design aspects as the natural durability of the Glulam and the design value of wind loading. The error sensitivity of the modelling and calculation methods. The solutions to provide structural stability.

Structural system

The combination of a tension / compression ring with stress ribbons to create the timber roof structure was chosen for this project. This choice is made after a relative basic analysis on applicable systems. However, little is known on such a system as it is never built before. To determine the feasibility of a structural system the decision should be based on rigorous cost/benefit analyses wherein functional, technical, economical and aesthetic criteria are met. Other configurations might be possible between a tension / compression ring and stress ribbon configuration as well. It was chosen to implement a radial rib configuration and keep an circular opening above the field.

Geometry

The geometry has been taken from RHDHV and due to time limitation a basic overall system is created. The opening above the roof is visually scaled to the edges of the playing field. Its location is not precisely determined so that it protects every spectator. Also, the radial configuration of the ribbons does not perfectly start and end symmetric to each other due to the way the system is modelled. It will have a negligible influence on the structural performance, but will influence manufacturing and determining the lengths of the individual ribbons. The ribbons should have a similar curvature and length as much as possible to improve the prefabrication of the timber elements. Otherwise, the feasibility of the structural design will be in jeopardy in a next design phase. At last, the truss configurations are chosen to provide the best possible solution for the chosen structural system. However, other configurations might be better, especially if upward wind loading is included, that will mostly influence the structural performance of the inner ring.

Connections

The connections will have impact on the distribution of the internal stresses and deformations of the system. Their impact needs to be investigated for further development of the design. As timber connections are never entirely rigid they will allow stress distribution in the elements. Their time dependent behaviour is also not included in the model.

Design aspects

The natural durability of the timber needs to be insured for service class 2 by the manufacturer. The working life is 50 years but this depends on many factors and can therefore be shorter. Also, the loads are determined in the initial stage of the design but depend on the final shape and configuration of all elements itself. Especially, the design value of upward wind loading is a rough over estimation. No definite conclusion on the actual feasibility of the system can be given without a wind tunnel test or computational fluid dynamics analysis.

Modelling and calculation methods

The error sensitivity within the grasshopper model components are unknown and their influence on the structural outcome should be verified in the more detailed design phase with other FEM programs. The total structural design is mathematically a very complex system of which there is almost no literature available. More certainty can be reached when multiple models will give similar results. Opposed to that, the inaccuracies are minimised by verifying the structural behaviour of a individual ribbon with SCIA Engineer.

Structural stability

The effect of uplift forces due to wind and asymmetrical loading is roughly explored and the solutions provided should be investigated in more depth to be assured feasible. The Grandview Aquatics Height Centre and the Braga National stadium found a working optimum by using hanging cables. After an intensive process of analysing all factors in combination with the predicted wind behaviour a solution was found for these slender elements spanning great distances. The dimensions of the New Feyenoord stadium are however far greater than these and will thus complicate the solution.

7

Recommendation

This chapter shows improvements to the design and work-flow that could have a positive contribution to the feasibility of long-span roof structures made of timber. Improvements for the design of the New Feyenoord stadium. Recommendations and possibilities for the chosen structural system. Improvements and advice on the feasibility of special timber structures.

Long-span timber roof structures for football stadiums is a relative new and broad subject. There are still numerous areas to improve and new aspects to be discovered.

New Feyenoord stadium

The timber design can be improved to be a more feasible structural solution for The New Feyenoord stadium. This includes design steps to improve the structural performance of the system for all load cases, while still complying with the design vision of OMA.

Stiffer tension compression rings

All connections in the trusses are modelled as hinges which allow for more deformation. In reality it is possible to create semi-rigid connections. This will result in a stiffer ring configuration, but also unfavourable shear forces and moments in the elements. The stiffer connections between the chords and diagonals are a possibility as their unity checks have enough margin to manage the extra unfavourable structural behaviour. Improving the stiffness is especially beneficial for the structural performance of the system in the straight sides of the rings. The connections in the corners can remain hinged due to the beneficial ring action that is provided by the curvature. However, stiffer connections are costly and more difficult to construct. It is of interest if an optimum can be found between the effort of creating semi-rigid connections and the structural performance.

The stiffness can also be improved by providing diaphragm action in the straight sides of the outer perimeter. This is done for the Grandview Aquatics Heights Centre: LVL panels are screwed onto the ribbons to provide weight and stiffness. LVL panels can be easily screwed to the timber elements. Doing so for the ring trusses creates a box girder configuration which provides more stiffness against the horizontal forces coming from the ribbons. The amount of improvement on the overall stability of the system is of interest.

Stiffer structural system

More research is needed to determine if the presented structural system is stable enough to withstand the uplift forces. A combination between timber ribbons and post-tensioned cables have the possibility to improve the overall stiffness of the structure. Post-tensioned cables are regularly used in tension / compression ring football stadium roof structures. These systems normally come with high post tensioning forces that need to be retained by the ring system. However, these forces might be reduced when combined with stress ribbons due to their inherent bending stiffness and self-weight. It might also allow for the distribution of peak forces to the more stiff corners of the rings by applying variations in post tensioning forces. [17] [26] An idea to implement post-tensioning cables in combination with stress ribbons while also increasing the in-plane cross section of the inner ring is shown in figure 7.1.

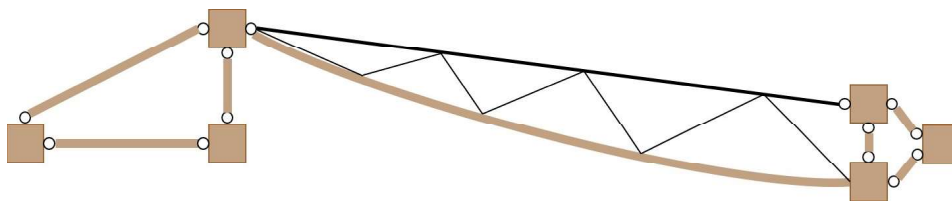


Figure 7.1: Tension / compression ring with stress ribbons and post tensioning cables

These cables can be stiffened regularly, by applying more stress on the cable year after year to cope with the time dependent behaviour of timber. More research is needed to determine if this is a long-term improvement of the structural system.

It is not investigated what the influence is of an extra truss ring half way of the ribbons. This will stiffen the entire structure by applying increased weight at the location that is now most subjected to deformations when uplift forces are applied. Furthermore, it might increase the ring behaviour of the system and reduce peak stresses in the outer and inner truss ring. The Tokyo National stadium also makes use of an extra truss ring half way the span of the steel / timber cantilevers.

In depth analysis of the wind behaviour

The used values for wind loading are a conservative estimation. The realistic load factors should be determined by a wind tunnel analysis and a computational fluid dynamics analysis. In this way the influence of the adjacent buildings can be incorporated, as well as the interaction between the load and the geometry. The geometry of the non-circumferential configuration with a downward sloped roof that has a opening in the middle, and the influence of the outer truss that initially directs the wind loading upward by means of its chosen triangular configuration. This should be determined to find an answer to the feasibility of the timber roof structure. The discussed solutions to resist the uplift forces and asymmetrical loading depend on these values and hence are required for a more detailed design for The New Feyenoord stadium.

Cross section optimisation

Optimisation of the cross sectional dimensions for the elements within an element group is not considered in this research. There are big differences between the minimum and maximum utilisation ratios of the elements. This is mainly due to the non-circular shape of the stadium. Finding an optimum for individual elements showing similar structural behaviour is beneficial. This means that the chords and diagonals of the outer- and inner truss ring as well as the ribbons should also be divided into groups with regard to their position along the circumferential perimeter. Namely, the long side, short side, and the corners. The goal is to increase the average utilisation of the elements while maintaining enough stiffness.

Connections

The chords of the truss rings are modelled as continuing elements, however splice joints need to be applied every 20 metres due to transportation restrictions. These joints should be modelled in the FEM program with their corresponding rotational stiffness to determine the internal force flow. This will probably increase deflections, but will redistribute the peak forces in a more favourable manner. Also, suitable connections should be developed for the complex nodes in the truss rings in close collaboration with a timber supplier like HESS or Neueholzbau.

Timber tension / compression ring with stress ribbons

The chosen structural system can become more feasible when several adjustments are made. These include improvements on structural configuration, changes in the initial design constraints, and its applicability for other cases.

Configuration of structural system

Making the rings follow a more circular shape will increase its ring action. Their stiffness will increase due to the geometric configuration and hence less material is needed to provide stiffness. In this way the ring trusses are able to take up the horizontal force component coming from the individual ribbons more efficiently.

The demand of unobstructed sightlines makes it difficult to design an optimal elegant structure. The span is enormous and results in more structural material needed to stabilise the structural system against uplift forces and asymmetrical loading. However, by allowing some structural material, obstructing sightlines, the roof can be more easily retained against uplift forces by for instance cables. Another aspect is the possibility to support the inner ring at locations where high deflections occur and allow the ribbons to make more use of cable action instead of beam action.

A great quantity of material is needed to provide stiffness against beam action in the outer ring in the design for The New Feyenoord stadium. Supporting the outer ring at more points along its perimeter will allow a more efficient design of this ring to accommodate the horizontal forces coming from the ribbons. The chords of the outer truss ring can then be designed with more width than height to carry the resulting shear stresses occurring due to radial ribbons following the non-circumferential perimeter of the stadium. This will increase its stiffness in the straight sides of the stadium. More supports also result in lower vertical support reactions per support, and hence allow for more weight of the loaded area.

If the structural system is used on a smaller stadium the uplift wind forces can be more easily countered by self-weight of the ribbons. For instance, a reduction to two tiers will reduce the span of the individual ribbons by approximately 20 metres and the height by 10 metres. The resulting horizontal forces from the ribbons are dependent on the length squared. Shorter ribbons hence allow for slimmer ring trusses. A lower total height will result in a lower design value for wind loading as the wind speed is reduced. [3]

Type of roof structure

The most ideal design for a football stadium roof is non-circular with unobstructed sight lines. However, there are examples of football stadium roof structures that will be less demanding on these aspects. Otherwise it is of interest to implement the structural system on other types of buildings requiring a long-span roof. A museum for instance would like an architectural statement as the chosen structural system in this thesis, and can allow tensile cables to be attached to the bottom side of the ribbons and attached to the floor. These cables can then restrain the uplift forces in a very elegant manner. The weight of the roof system can be severely diminished and hence result in a better option for the chosen structural system in this thesis.

Feasibility of special timber structures

Timber can become a competitive material to the more traditional materials for special structures. The complexity that the material brings into initial design can be reduced by considering the following aspects.

Better digital workflow

The components in the parametric environment of Grasshopper and Karamba3D are specialised for steel structures. Therefore, the analysis of timber structures takes more time before workable values are determined. It is recommended to incorporate a material analysis component according to Eurocode 5: Timber structures. As well as making a cross-section optimizer with the help of suppliers. The future of timber engineering for these special structures goes hand-in-hand with automating the difficult iterative design steps you need to take when designing a timber structure. Think about the orthogonality of timber, the three dimensional flow of forces (difficult to apprehend by 2D calculations), and the loss in strength due to connections. Steel does not have these strength losses as timber does, which makes it preferable by engineers to use steel for complex structures. Automation will make it easier to assess timber as a material in the initial design phase. Which in turn will show its positive aspects and possibilities.

Construction

Timber structures have proven to result in a fast assembly on site and hence reduce the costs of this phase in the development of a structure. Timber structures are largely prefabricated and permit an easier site assembly. The prefabrication creates a high standard in terms of accuracy and speed of production. Therefore, it is interesting to investigate if a long-span timber structure would result in a more economic solution than a steel long-span structure when the construction phase is included.

Connections

To incorporate the proposed new connections, that result in a high efficiency of the joint, many steel fasteners are needed. This is seen as a downside, but many of these connections are prefabricated and are easily put together on-site. Whereas, the steel connections in steel structures consist of intensive welding and many tight fitting bolts. It would be a very interesting analysis to investigate the differences of the aspects concerned with connections in these long-span structures.

Design

Timber structures require smart design to make use of its beneficial aspects as much as possible. When architects allow the structural engineers for more design freedom a timber structure will be more easily considered a feasible option. The recommendations for the structural system stated above are an example of this. Other specialisations should be included in an early design stage as well. Then you can incorporate their prerequisites in design and you have the ability to make a feasible design in timber economically as well. Extra time spent on a good design can be regained by the fast production and construction. The previously mentioned automation possibilities will speed up design and the incorporation of different disciplines.

Bibliography

- [1] The Timber Industry Manifesto: Developed by Wood for Good.
- [2] Eurocode: Basis of structural design. The Netherlands Standardization Institute, Delft, 2011.
- [3] Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions. The Netherlands Standardization Institute, Delft, 2011.
- [4] Eurocode 5 : Design of timber structures - Part 1-1: General - Common rules and rules for buildings. The Netherlands Standardization Institute, Delft, 2011.
- [5] Structural Timber Engineering Bulletin: Timber as a structural material - an introduction. pages 1–6, 2014.
- [6] Wooden wonder: World's first all-seater wooden stadium set for construction, 2017.
URL <https://www.worldbuild365.com/news/2z96dtuhd/building-architecture/wooden-wonder-worlds-first-all-seater-wooden-stadium-set-for-construction>.
- [7] Neue Holzbau AG. Gsa-technology. URL https://neueholzbau.ch/en/produkte/gsa-technologie/?noredirect=en_US.
- [8] Simon Aicher and Gordian Stapf. BLOCK GLUED GLULAM – BRIDGES , BEAMS AND ARCHES. In World Conference on Timber engineering 2014, number August, Quebec, 2014.
- [9] Simon Aicher, Jürgen Hezel, and Gordian Stapf. Mechanical and glued joints in glulam of ultra high efficiency. In World Conference on Timber Engineering 2012,, number January, Auckland, 2012.
- [10] Janko Arts, H.G. Kraaijenbrink, A. Schram, G. Joosen, F.S. de Zwart, and F. van Gool. Feyenoord new stadium structural design ontwerp-rapportage vo. Technical report, Royal Haskoning DHV, Rotterdam, 2018.
- [11] F. Asdrubali, B. Ferracuti, L. Lombardi, C. Guattari, L. Evangelisti, and G. Grazieschi. A review of structural, thermo-physical, acoustical, and environmental properties of wooden materials for building applications. Building and Environment, 114:307–332, 2017.
- [12] Amanda Baillieu. Large Span Timber Structures.
- [13] H.J. Blaß and C. Sandhaas. Timber Engineering: Principles for design. Karlsruher Institut für Technologie Scientific Publishing, Karlsruhe, 2017.
- [14] Hans Joachim Blaß and Patrick Schädle. Ductility aspects of reinforced and non-reinforced timber joints. Engineering Structures, 33:3018–3026, 2011.
- [15] Hans Joachim Blaß and Johannes Streib. BauBuche Beech laminated veneer lumber Design assistance for drafting and calculation in accordance with Eurocode 5. Technical report, PollMeier, Karlsruhe, 2017.
- [16] H.J. Blass, P. Aune, B.S. Choo, R. Görlacher, D.R. Griffiths, B.O. Hilson, P. Racher, and G. Steck. Timber Engineering STEP 1: Basis of design, material properties, structural components and joints. Centrum Hout, Almere, 1995.
- [17] Ivar Boom. Tensile-compression ring - a study for football stadia roof structures. PhD thesis, TU Delft, 2012.

- [18] Martin Cepelka and Kjell Arne Malo. Review on on-site splice joints in timber engineering. In Steffen Franke, Bettina Franke, and Widmann. Robert, editors, COST Timber Bridge Conference 2014, number September, Biel, 2014. Bernd University of Applied Sciences, Switzerland: Institute for Timber Construction and Architecture.
- [19] Roberto Crocetti. Large-Span Timber Structures. In Proceedings of the World congress on Civil, Structural, and Environmental Engineering (CSEE'16), pages 1–23, Prague, Czech Republic, 2016.
- [20] FIFA. Football Stadiums. Technical recommendations and requirements. Technical report, FIFA, 2011.
- [21] Deutsches Institut für Bautechnik. Self-tapping screws for use in timber constructions [ETA-11 / 0190]. Technical report, 2016.
- [22] Deutsches Institut für Bautechnik. BESISTA rod systems [ETA-08/0038]. pages 1–5, 2013.
- [23] Rui Furtado, Carlos Quinaz, and Renato Bastos. The Engineering of Braga Municipal Stadium. Technical report, afaconsult, 2003.
- [24] Ernst Gehri. High performing jointing technique using glued-in rods. In 11th World Conference on Timber Engineering WCTE, number June, Riva del Garda, Italy, 2010.
- [25] SHERPA Connection Systems GmbH. Design guide: SHERPA connection systems. 2019.
- [26] V Goremikins, K Rocens, and D Serdjuks. Rational Structure of Cable Truss. World Academy of Science, Engineering and Technology. Special Journal Issues., 5(0052):571–578, 2011.
- [27] Hasslacher Group. Glued Laminated Timber (HNT0202_EN_201810), 2018.
- [28] Hasslacher Group. Baubuche (HNT0106_EN_201904), 2019.
- [29] Richard Harris, Michael Dickson, Oliver Kelly, and Jonathan Roynon. The Use of Timber Gridshells for Long Span Structures. Proceedings of the 8th world conference on timber engineering, pages 1001–1006, 2004.
- [30] Günther Hartmann. Sustainable Building Resource Transition, 2017.
- [31] C Hartsuijker and H Welleman. Toegepaste mechanica part 1, 2 and 3. 2003.
- [32] HESS-Timber. Anaklia-ganmukhuri pedestrian bridge. URL <https://www.hess-timber.com/en/references/detail/anaklia-ganmukhuri-pedestrian-bridge/>.
- [33] HESS TIMBER GmbH & Co. Brettschichtholz mit dem Keilstoß-System HESS Limitless, 2015.
- [34] Samuel Hofverberg. Long-span tensile timber roof structures Development of design proposals adopting the Stress Ribbon concept. PhD thesis, Chalmers University of Technology, 2016.
- [35] SFS intec. Timber Work: Innovative fastening systems and applications. Technical report, 2015.
- [36] Simone Jeska and Khaled Saleh Pascha. Emergent Timber Technologies: Materials, Structures, Engineering Projects. Birkhäuser Verlag GmbH, 2014.
- [37] Simone Jeska, Khaled Saleh Pascha, and Rainer Hascher. Emergent Timber Technologies Materials, Structures, Engineering, Projects. Birkhäuser, 2014.
- [38] Geraint John, Rod Sheard, and Ben Vickery. Stadia a design and development guide 4th edition. Elsevier Ltd, 2007.
- [39] I. Robert Kliger, Reza Haghani, Maurice Brunner, Annette M. Harte, and Kay Uwe Schober. Wood-based beams strengthened with FRP laminates: improved performance with pre-stressed systems. European Journal of Wood and Wood Products, 74(3):319–330, 2016.
- [40] Knapp. Connection Systems MEGANT Statics [ETA-15/0667]. pages 1–48, 2017.

- [41] Marc Wilhelm Lennartz and Susanne Jacob-Freitag. *New Architecture in Wood : Forms and Structures*. Walter de Gruyter GmbH, 2015.
- [42] Isabelle Lomholt. Wanda metropolitano stadium in madrid. URL <https://www.e-architect.co.uk/spain/wanda-metropolitano-stadium-in-madrid>.
- [43] Atletico Madrid. An impressive roof to make the wanda metropolitano. URL <https://en.atleticodemadrid.com/noticias/an-impressive-roof-to-make-the-wanda-metropolitano-a-pressure-cooker>.
- [44] MakMax. Completion of wanda metropolitano. URL <https://www.makmax.com/news/2017/12/06/38>.
- [45] Nuno Martins, Elsa Caetano, Sandro Diord, Filipe Magalhães, and Álvaro Cunha. Dynamic monitoring of a stadium suspension roof: Wind and temperature influence on modal parameters and structural response. *Engineering Structures*, 59:80–94, 2014.
- [46] Nicola Moggio. *Glued laminated timber space truss systems*. PhD thesis, Lunds Universitet, 2012.
- [47] OMA. FEYENOORD CITY concept masterplan fase 2. Technical report, OMA, 2016.
- [48] D. Otero Chans, J. Estévez Cimadevila, and E. Martín Gutiérrez. Withdrawal strength of threaded steel rods glued with epoxy in wood. *International Journal of Adhesion and Adhesives*, 44:115–121, 2013.
- [49] Kristoffer Persson and Kristoffer Persson. *Analysis of Catenary Shaped Timber Structures*. PhD thesis, Lund University, 2017.
- [50] Pollmeier. Product profile 01, 2019.
- [51] Pollmeier. Fasteners and connections 05, 2019.
- [52] Pollmeier. BauBuche Trusses 08, 2019.
- [53] Michael H Ramage, Henry Burrigge, Marta Busse-wicher, George Fereday, Thomas Reynolds, Darshil U Shah, Guanglu Wu, Li Yu, Patrick Fleming, Danielle Densley-tingley, Julian Allwood, Paul Dupree, P F Linden, and Oren Scherman. The wood from the trees : The use of timber in construction. *Renewable and Sustainable Energy reviews*, 68(2017):333–359, 2017.
- [54] RDW. *Overzicht maten en gewichten in Nederland*, 2012.
- [55] Katherina Santoso. *Wide-Span Cable Structures*. PhD thesis, Massachusettes Institute of Technology, 2004.
- [56] Benjamin Elisha Sawe. The most popular sports in the world, 2018. URL <https://www.worldatlas.com/articles/what-are-the-most-popular-sports-in-the-world.html>.
- [57] Katherine Smale. World's first wooden stadium, 2017. URL <http://www.newcivilengineer.com/latest/the-gallery-worlds-first-wooden-stadium/10018897.article>.
- [58] Angelo Spampinato. Stadium history. URL http://www.worldstadiums.com/stadium_menu/architecture/historic_stadiums.shtml.
- [59] Österreichisches Institut für Bautechnik. HASSLACHER BauBuche [ETA-18/1018]. Technical report, 2018.
- [60] Jiří Stráský. *Stress ribbon and cable supported pedestrian bridges*. PhD thesis, Akademie věd České republiky, 2006.
- [61] SwedishWood. *Swedishwood: About glulam*. URL https://www.swedishwood.com/about_wood/about-glulam/.

-
- [62] Matthias Theiler, Andrea Frangi, and René Steiger. Strain-based calculation model for centrally and eccentrically loaded timber columns. *Engineering Structures*, 56:1103–1116, 2013.
- [63] HASSLACHER NORICA TIMBER. Hasslacher norica timber: Glued laminated timber. URL <https://www.hasslacher.com/glue-laminated-timber>.
- [64] UEFA. Uefa guide to quality stadiums. Technical report, 2011.
- [65] UEFA. The european club footballing landscape - club licensing benchmarking report financial year 2016. Technical report, 2016.
- [66] UEFA. Uefa stadium infrastructure regulations. Technical report, UEFA, 2018.