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Alternative study conveyor belt bridge Harlingen

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BSc Thesis

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Summary
Frisia Zout B.V. is a company in Harlingen that extracts salt from the bottom of the sea and processes the salt into raw materials made for the industry. The salt is transported via conveyor belts that are located in bridges. The bridges are connected to buildings where the salt is processed. The bridges are heavily damaged by corrosion and negatively affected due to overdue maintenance.
Frisia has outsourced the engineering of the new bridges to Bilfinger Tebodin. Together with Tebodin, the best design is examined by studying three alternatives. This research is mainly focused on the best-supporting structure. Preliminary investigation indicates that the following support structures are interesting to analyse: a space frame, a castellated beam and a cable-stayed bridge. The designs of the alternatives are based on issues found at the existing bridges, like corrosion and support availability. In conveyor belt bridge 3 and 4, the moisture content of the salt is 2%. Although the moisture content is regulated, it still has a major influence on the degree of corrosion. The corrosion and support availability are incorporated in Frisia's requirements.

The research aim is to find the best design to replace the conveyor belt bridge 3 and 4. This is accomplished by taking into account Frisia's requirements in the design phase and alternative comparison phase. The dimensions of the different designs are determined by hand to get a rough view of the best alternative. The cable-stayed bridge alternative was dropped in an early stage of the research due to many flaws that had to be worked around. The advantage of the cable-stayed bridge, creating large spans by utilizing the construction height was due to horizontal actions not applicable and therefore dropped. The other two alternatives are elaborated and analysed in more detail. Both alternatives can technically comply with the design requirements of Frisia. However, with a multi-criteria analysis the space frame came up on top. This is due to slightly lower general construction costs. The dimensions of the space frame are in the final stage of this research optimized to create the most cost-efficient construction possible for Frisia.

1. Introduction

Every day people use salt in several different ways. It is surprising how many products contain salt, from our food to road sprinkling. Also, production processes use an extreme amount of salt. Salt can be produced in several ways. Close to Harlingen, there is a salt layer below the 'Waddengebied' where salt extraction takes place. This is done by pumping it up out of salt layers. The company exploiting these salt layers is 'Frisia Zout B.V.', in the report stated as Frisia. Frisia is a company located in Harlingen with a salt processing of 1.2 million tonnes per year with 120 employees. This makes Frisia the largest vacuum salt plant of the European salt company's parent company (Esco, 2022), abbreviated as Esco. Frisia was not always in the hands of Esco. Before the acquisition, Frisia was owned by the German company 'Kali und Salz'. Since 2006, Frisia extracts salt close to Tzummarum, a small town in the west of 'Friesland'. But due to reaching the maximum subsidence of 30 centimetres, they had to stop the salt extraction (Atsma, 2019). Luckily, they found a new salt layer three kilometres from the coast and two kilometres below the Waddenzee. Because the layer is located below the Waddenzee, the Ministry of Economic Affairs and Climate Policy comes into place. Their job is to protect the flora and fauna of the Waddenzee and ensure that it cannot suffer from salt extraction. Therefore, the ministry has set strict rules for the extraction.

Frisia has several conveyor belts. Some conveyor belts are used for the transport of a solution of water and salt, this is called brine. The brine is transported to the place where water is extracted from the solution to exclude the salt. The salt is processed from wet salt to dry salt. The other conveyor belts that Frisia uses, transport the dry salt to storage areas. The salt is used, inter alia to make blocks and tablets for water softening. The conveyor belts are in poor condition due to overdue maintenance and they are reaching the end of their lifetime. They have to be replaced. Frisia has approached Tebodin to come up with a design.

Frisia uses 4 different conveyor belts on the site, which can be seen in Figure 1. Conveyor belt bridge 1 and 2 (BB 1&2), located above each other, are already designed by Tebodin. This research is focused on conveyor belt bridge 4 (BB4) and mainly on conveyor belt bridge 3 (BB3). Using theory and literature together with the expertise of the Tebodin colleagues, three possible steel alternatives will be generated for conveyor belt bridge 3. There will be focussed on 3 alternative steel supporting structures: a space frame, a castellated beam and a cable-stayed bridge. The 3 different supporting structures are designed based on the available space for the supports. Later in this report, the alternatives will be discussed in more detail.

1.1 Problem context

Frisia is located in an industrial area in Harlingen. Figure 1 shows the map of Frisia's industrial area. They extract salt by pumping fresh water into the salt layer, where the salt will dissolve in the freshwater. The solution, called brine, is pumped up and transported via pipes to the 'Saline building' (Esco, 2017). In the 'Saline building', the brine is processed to salt and transported via four conveyor belt bridges to other buildings.

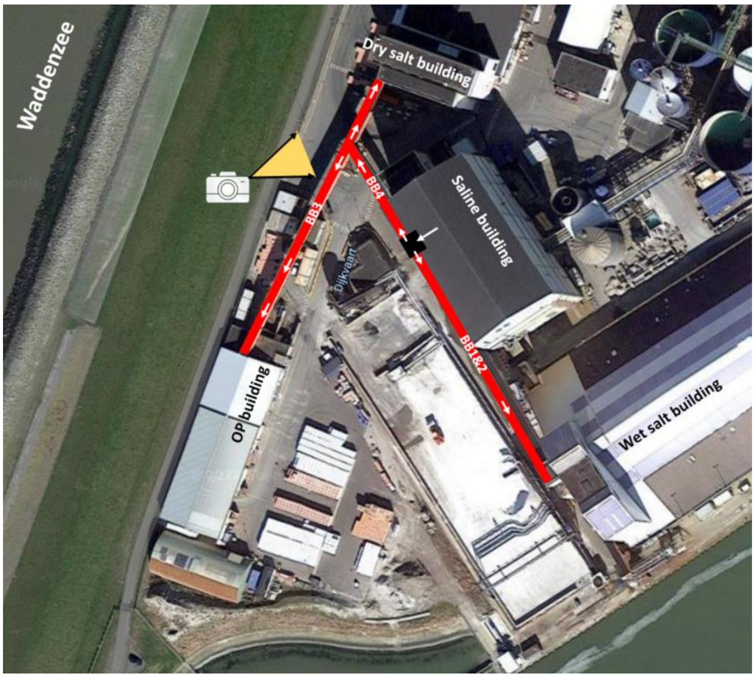


Figure 1 Map of Frisia Zout B.V. (Google Maps, 2022)

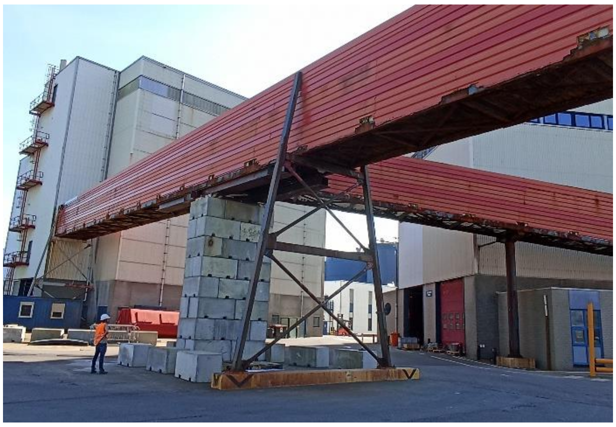


Figure 2 Existing T-junction conveyor belt bridge 3&4 with the extra concrete support

The condition of the conveyor belt bridges has not been addressed for years due to the fact that the salt vein in Harlingen seemed to run out, which results in the poor condition of the bridges. An extra concrete support has even been added to the existing bridge to prevent the bridge from collapsing. The bridges are damaged by the salty environment created by the salt that is transported over the conveyor belts, this can be seen in Figure 3 and Figure 4. Some of the structures are built almost 50 years ago (Hendriks, 2022).


Figure 3 Corrosion damage on the outside of bridge 3


Figure 4 Corrosion damage in the bridge 3 due to salt

Since Frisia discovered a new salt vein last year, the future production will exceed the capacity of the current conveyor belt bridges. Therefore the poorly maintained bridges will be replaced by new ones. The plan for the replacement of the first and second bridges is already made by Tebodin. Conveyor belt bridges 1 and 2 have been placed at the beginning of April (Hendriks, 2022). They transport salt with a moisture content of 9% from the saline building to the wet salt building. Conveyor belt bridge 3 and 4 transport salt with a moisture content of 2% from the saline building to the dry salt building and the OP building. More specific, bridge 4 has 2 conveyor belts that move salt to bridge 3. At the T-junction that can be seen in Figure 2, the salt is dropped to two different conveyor belts located in bridge 3. The conveyor belts move in different directions. Figure 1 shows the direction of the conveyor belts in the bridges. One conveyor belt goes to the OP building in the south and one to the dry salt building in the north. The existing bridge has a support in the middle of a road, which is illustrated in Appendix B**.** The support is sensitive to traffic collisions. The impact of a collision could make the support unstable. This has an influence on the bridge and endangers the bridge, because the unstable support cannot withstand the load anymore that it was designed for. For the new bridge, the available space for supports has to be taken into account to prevent vulnerable support locations of the bridge.

Stakeholders

Several parties are involved in the renewal of the conveyor belts. The stakes of the parties are shown below.

Frisia Zout B.V.

Frisia is the client of the project and owns the land at the industrial area in Harlingen. They have outsourced the design of the conveyor belts and the calculation that are involved to Tebodin. Frisia has stated requirements for the bridge, which must be translated by Tebodin to come up with a design. The requirements are based on the optimization of the process and the preferences of the employees. Frisia expects the plan to be finished around September 2022 for conveyor belt 3 and 4.

Tebodin

Tebodin is responsible for the design based on the requirement Frisia has drawn up. These requirements will be investigated in the research. Tebodin is the consultancy company for this project. They provide the design with the related justification. Tebodin is specialized in designing and engineering for the industrial market. The company was started in 1945 to focus mainly on the reconstruction of the industry after World War II.

Contractor

As of today, there is no contractor. Frisia will approach contractors, when they have received the design from Tebodin. Possible contractors are 'Jorritsma bouw' and 'Visser & Smit'. Visser & Smit was the contractor for conveyor belt bridge 1 and Jorritsma bouw for conveyor belt bridge 2. During the construction of conveyor belts, there were some problems with the contractors. The first conveyor belt bridge that was implemented was bridge 2. There was stiff communication between Frisia and 'Jorritsma bouw'. That was one of the reasons that 'Visser & Smit' was assigned for converyor belt bridge 1. The other reason was that the offer of Visser & Smit was much cheaper (Hendriks, 2022).

Municipality

The municipality has the responsibility to ensure the safety of its municipality. Before the design is put into realisation, the municipality has to accept the construction of the bridge. Then they also look at whether the bridge fits in with its surroundings by using the environmental code.

Local residents

The involvement of the local residents is next to nil. However, the plan can be viewed if the plan is complete to let the local resident in on the changes in their surroundings.

1.2 Research Questions

The aim of this research is to design a new steel conveyor belt bridge based on the requirements 'Frisia Zout B.V.' has stated by using hand calculations and the Finite element method (FEM).

The research focuses on answering the main questions. This can be done with the sub-questions. They guide the research and help to answer the main questions of the research.

Main questions

1. Which steel conveyor belt bridge alternative has to be implemented based on the requirements of 'Frisia Zout B.V.'?
2. What are the dimensions of the elements in the chosen steel conveyor belt bridge alternative?

Sub-questions

1. What are the requirements that 'Frisia Zout B.V.' has stated?

2. What are the forces acting on the bridge per alternative?
    a. What are the constant forces acting on the bridge per alternative?
    b. What are the variable forces acting on the bridge per alternative?

3. How is the salt environment taken into account?

4. Which position of the supports of the conveyor belt bridge is the best in relation to the available space below the bridge?

5. Which alternatives can be made by taking into account the requirements of 'Frisia Zout B.V.' based on the spaceframe, castellated beam and cable-stayed bridge alternatives?

6. What are the element dimensions of the different alternatives?
    a. What are the moments in the supports in the different alternatives?
    b. What are the support reactions in the different alternatives?
    c. What are the internal forces in the different alternatives?

7. How to decide which supporting structure is the best alternative?

8. Which supporting structure has the best value due to the chosen analysis for the conveyor belt bridge 3?

1.3 Outline of methodology

The research consists of a variant study. During this study, three supporting structures are examined based on the requirements of 'Frisia Zout B.V.'. The requirements will be collected during this research by means of interviewing employees of Frisia and Tebodin. They are familiar with the problem and the desires of the new design. Initially, the supporting structures will be designed and calculated by hand. This is done to get a quick rough indication of the forces in the bridge without computing the calculation of the calculated optimized dimensions of the bridge. To make it visual, one circle is made in the ending design loop in Figure 5. Far away from the allowable difference, just a quick indication. Subsequently, the designs are compared based on Frisia's requirements. This can be done by several decision support systems. The decision support system will be determined during the research. The chosen system will be used to find the alternative that comes best out of the decision support system. Finally, The design loop in Figure 5 is continued to end up in the 'Allowable difference' zone. The finite element method (FEM) is used to reach the zone. The method divides the structure into smaller parts that are called finite elements. The method is explained with a diagram in Appendix D. FEM is a very precise method, but time-consuming. Therefore computer software will be used to execute FEM and find acceptable dimensions of the bridge elements.

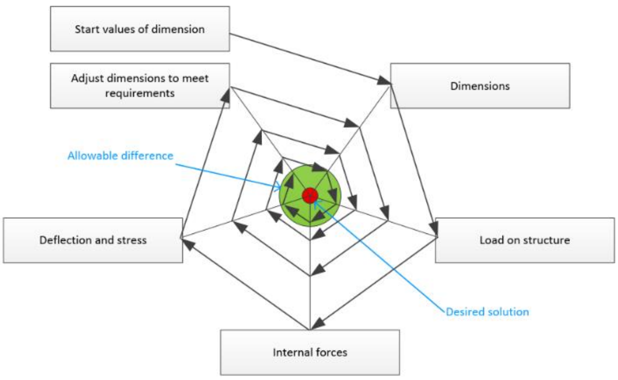


Figure 5 Ending design loop (G.H.Snellink, 2018)

1.4 Phasing

The research is done in different phases to get an answer to the main questions of '1.2 Research Questions'. The phases are divided into the six phases that are shown below:

1. Useful load determination
2. Design 3 alternatives
3. Rough constructive calculation
4. Alternatives comparison
5. Finite element method
6. Conclusion

All phases are dependent on each other. They will be executed one by one. The next phase needs the information found in the previous phase. So the phase cannot be done parallel. The phases are explained in more detail below.

Phase 1

Firstly, Phase 1 focuses on the requirements of Frisia. Secondly, the requirements are transformed to design requirements if necessary. Thirdly the forces acting on the structure are determined. This will be different per design due to different sizes and weights, therefore is looked at standard value for a specific area. These values can be determined by making assumptions based on available literature and specifically the Eurocode. This Phase can give answers to the sub-questions 1 and 2

Phase 2

In phase 2 is thought about solutions to prevent corrosion due to the salt environment. A solution against corrosion has to be found through literature research. Before designing the alternatives, the available space on the ground for possible supports in this specific situation has to be taken into account. When the available space has been mapped, the three steel alternatives of the conveyor belt bridge can be designed. During the design phase, the standard forces values found during 'Phase 1' are transformed into the designed alternatives. The established forces on the designed alternative can be used in 'Phase 3' to find the internal forces in the different structures. After Phase 2 is executed, sub-questions 3, 4 and 5 can be answered.

Phase 3

In phase 3, rough constructive calculations are executed. There will be looked at four aspects per alternative: The moments around the supports, the internal forces, the reaction forces and the stability. The internal forces in the structure and their supports (reaction forces) can be calculated with the force method. This method is explained in more detail in Appendix C. This will be done for all alternatives. With the internal forces, the element with the greatest dimensions can be determined. Elements will have the same dimensions as the same element with the highest internal force to simplify the hand calculation. This can be optimized in the computer software in 'Phase 5'. At the end of this phase, sub-question 6 and his related questions can be answered.

Phase 4

The 3 alternatives will be compared to find the best steel conveyor belt bridge. First, the best analysis to compare the different alternatives has to be chosen by literature research. The analysis will be based on the criteria derived in Phase 1. With the chosen decision support system the best alternative can be chosen by looking at which alternative comes out best in the analysis. This will give answers to sub-question 7 and 8.

Phase 5

In Phase 5, FEM will be used to optimize the dimensions of the best alternative his elements. During the optimization is tried to reduce the dimensions of the profiles and still meet the load criteria. FEM is ideally suited to solve physical problems in engineering analysis and design (Bathe, 2014). The process of finite element analysis is summarized in Appendix D. This process will be executed in a suited computer program in which the design can be modelled. The rough hand calculations can be used to validate the model.

Phase 6

The last phase gives the conclusion of the results found in the research. Main question 1 must be answered in this phase and that can be done with the information from the previous phases. After the conclusion is made, everything has to be documented in the report. The documentation of the research is also included in this phase.

2. Frisia's requirements

The requirements are given by Frisia after a meeting in April. The meeting was between two contact persons from Frisia and some of my colleagues within Tebodin. The intention of the meeting was to give the Tebodin collages a good picture of the expectations of Frisia in the project and to make their requirements clear. In this section, those requirements are elaborated to Frisia's interest and the context of the project.

Dimensions bridges

In conveyor belt bridge 3 and 4, Frisia wants to meet the standards of the FSSC 22.000 certification. This certification is for food safety management systems. Therefore Frisia has decided that they want the two conveyor belts next to each other in bridge 4 instead of above each other, because the upper conveyor belt spills salt on the lower conveyor belt. The width of the inside of conveyor belt bridge 4 has to be 4275mm. Also, the distance between the Saline building and bridge 4 has to be at least 800mm and the bridge must not protrude from the saline building.

For conveyor belt bridge 3 is also chosen for the conveyor belts next to each other. This results in a thicker section at the T-junction with conveyor belt bridge 4. Here the inside width must be at least 4275mm. In the direction of the dry salt building, the width must be 2725mm and to the OP building 2525mm according to Frisia. Figure 6 is given by Frisia.

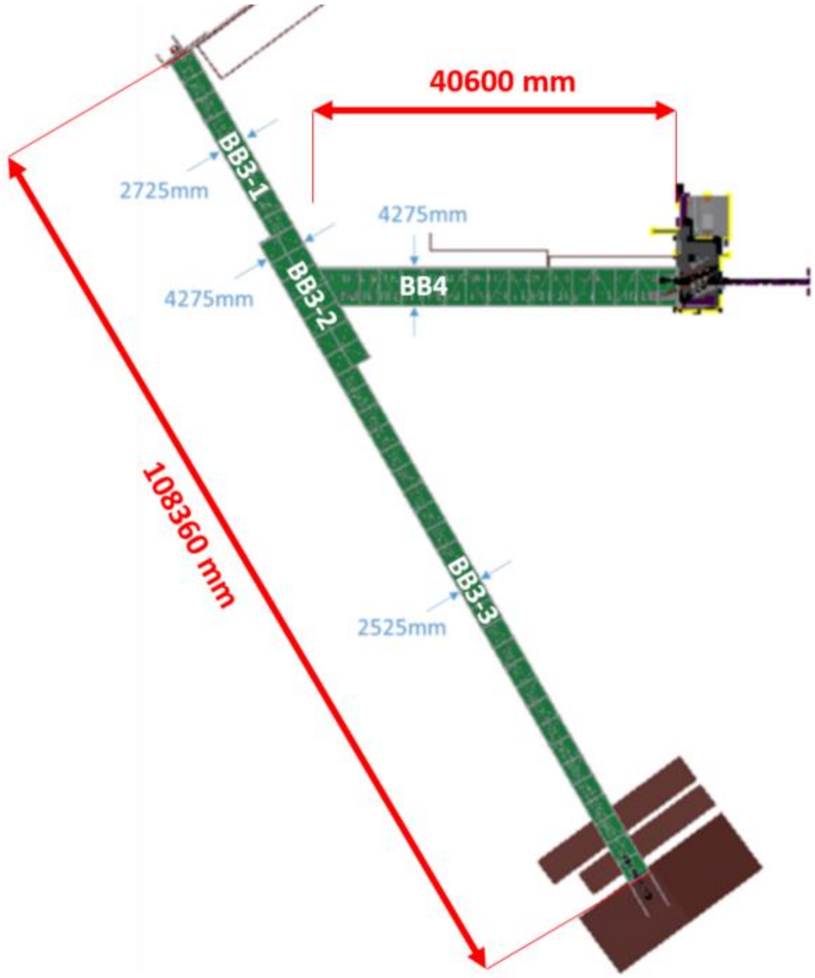


Figure 6 Required width inside conveyor belt bridge 3 and 4 (Frisia, 2022)

Support locations

Furthermore, the location of the supports of the bridges has to be reconsidered to improve the traffic on the road below the bridges. The traffic should not be hindered by the supports.

Cost-efficient

Frisia is a profit-oriented company and wants to reduce the cost as much as possible to maximize the profit of the salt extraction. Relevant to the bridge are the costs made by the realisation of the bridge. For example, the material costs and the processing costs of the bridge.

Speed of the installation

The speed of the installation of the conveyor belt bridge is relevant due to the fact that the salt extraction has to be halted during the installation of the bridge. This leads to no revenue during this period. That is why Frisia benefits from a short installation period.

Durability

The bridge must have a high durability. Durability can be interpreted in several ways. For this design, the durability is translated into two components, the bridge has to have a low maintenance cost and a long lifespan (Rabin, 2005).

Load criteria

The bridge must meet the load criteria. The criteria have been tightened since the old bridge was built. They went from the code NEN6700 to the Eurocode. In this research the focus lies on steel structures, so 'Eurocode 3: Design of steel structures' will be used intensively. The steel construction regulation K+S (C – 008 – DE) must also be complied with (Knie, 2017). This regulation is made by the Kali und Salz, the name of the company before the acquisition. And focuses mainly on the specific situation where salt is present. An example of the K+S regulation is that all construction parts in damp and outdoor areas must have a minimum profile thickness of 6mm.

3. Design alternatives

In this phase, the designs will be made for the different alternatives. Potential pitfalls are taken into account during design and efforts are made to avoid them. First Frisia's requirements are translated to design requirements. The support locations are an important requirement and are determined by looking at the support availability. Also, the dimensions of the bridge have to be enhanced. With all this information, the structures can be designed. By designing the structure conveyor belt bridge 3 is seen as a separate bridge from bridge 4. By contrast, the forces of bridge 4 acting on bridge 3 will be taken into account.

3.1 Design requirements

Support availability

In this section, there will be looked closer at the locations of the current supports and how to improve them. The planning was to go to Harlingen and have look at the support locations. However, this trip was delayed and made in a later stadium. Therefore no physical examination could be executed. The alternative was to look at a point cloud in the software program 'Autodesk Recap' and to look at google maps. There is made a point cloud of the existing situation by a drone. The drone determines exactly where in space a point is located by using inertial measurement and satellite positioning data (DJI Enterprise, 2021). After analysing the existing situation, there is made a 'no support zone'. This zone is shown in Figure 7 in black. The zone is validated during the visit of the plant. In Appendix A, the pointcloud is compared with the photos made during the visit.

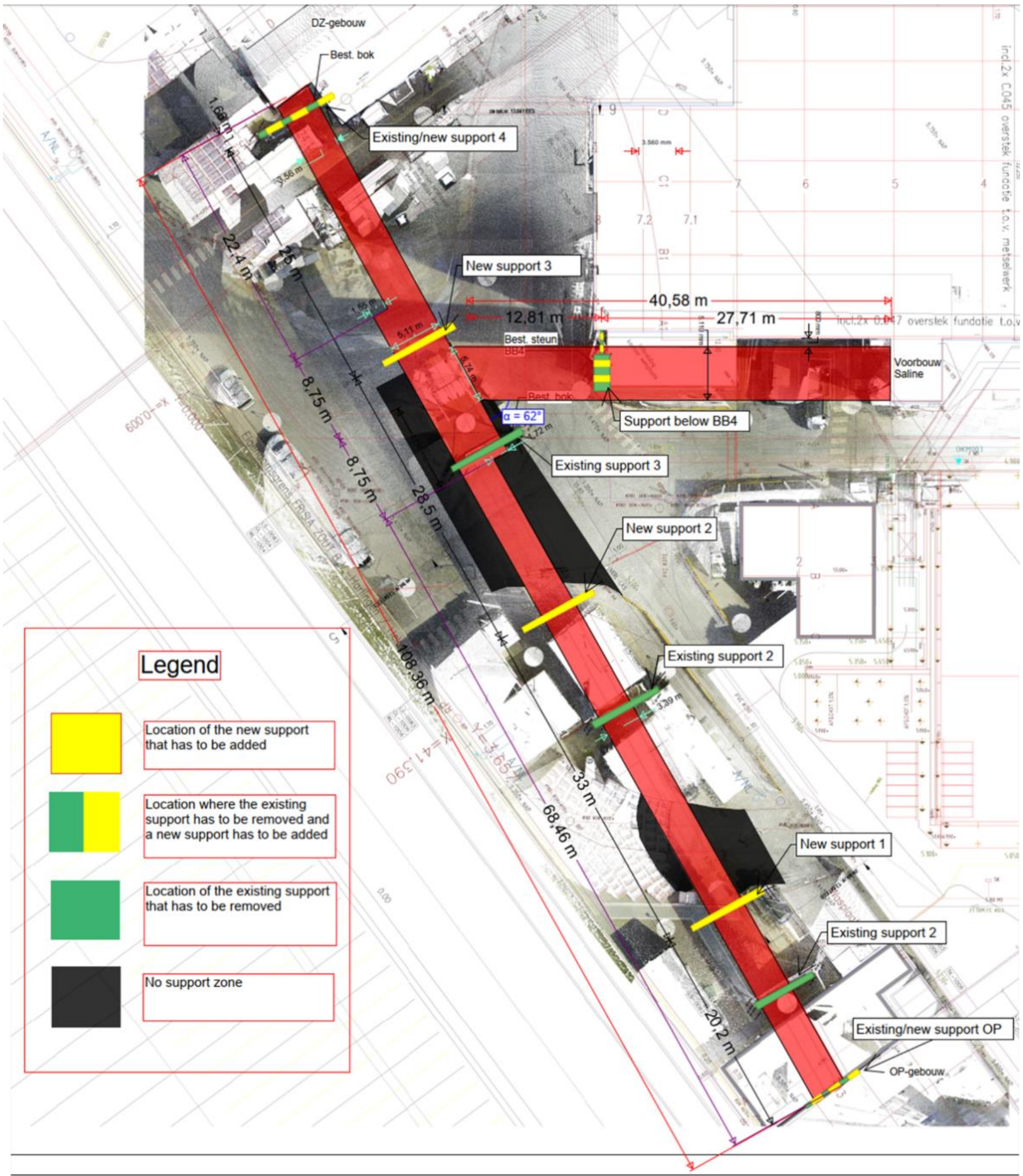


Figure 7 Conveyor belt bridges with no support zone and old and new supports

The current conveyor belt bridge 3 has 5 supports. This is a good starting point, for the alternatives of the space frame and the castellated beam. Later in the design phase, the number of supports can always be optimized. The locations of the new supports are based on the principle that the spans between the supports have around the same length and are shown in Figure 7.
For Support 1, 2 and 3 must build a new foundation. The other two supports are placed in the same position as the current supports. To determine if this foundation of these supports can be used again, further investigation is necessary. The supports will transfer mainly the vertical loads acting on the structure, but they must also transfer horizontal loads acting perpendicular to the sides of the bridge. The horizontal loads in the direction of the length of the bridge can be taken by the OP building on the south of the bridge. So the bridge cannot stand on its own. This is already the case in the existing situation, where the supports below the bridge look like Figure 8**.** In the designs of the space frame and castellated beam alternative, the supports will look similar to the existing ones. This report focuses not on the calculations of the supports. The assumption is that the supports can transfer all vertical loads and horizontal loads perpendicular to the bridge from both the top and the bottom. However, the supports have to be calculated before the realisation of the design.

Figure 8 Support Existing bridge in the front with extra concrete support in the back

Bridge dimensions

The required inside width of conveyor belt bridge 3 and 4 are defined in Figure 6. This inside width excludes the isolation and cover plates. To estimate the width of these layers, the design of conveyor belt bridge 1 and 2 are used. In these designs, the extra width on each side is 418mm. Because the bridges are fairly similar, the estimation for conveyor belt bridge 3 and 4 will also be 418mm extra width on each side.

For the height of the bridges, Frisia has not given an exact height. On the other hand, they want walkways in the bridge. So people have to be able to walk through the bridge. According to the Eurocode, the minimum required height for walkways is 2.40 meters. As on the sides, the top and bottom of the bridge need isolation and cover plates. Conservative is chosen for an outside height of 3.40 meters. The values are shown in the table below.

Table 1 Dimensions of conveyor belt bridge 3 & 4

	Width (inside) [m]	Width (outside) [m]	Height (outside) [m]
BB4	4.28	5.11	3.40
BB3-1	2.73	3.56	3.40
BB3-2	4.28	5.11	3.40
BB3-3	2.53	3.39	3.40

3.2 General specifications

Profile type

The bridges are located near the sea. So the expectation is that there will be a significant amount of wind acting on the bridges. Therefore the space frame will be made out of HEA profiles. This is done because they have a wider flange which can take more horizontal forces (Shane, 2020). This can be seen by comparing the moment of inertia values in the z-direction between a HEA and IPE profile with the same area.

Top and bottom of the structure

The horizontal forces are absorbed by the top and bottom of the structures. This structure will be the same for all alternatives, because the horizontal force is only about 28% of the vertical forces. Also, the bottom and the top are equal to each other because half of the horizontal force goes to the top and half goes to the bottom. These planes will only use diagonals under tension. The wind can act on the different sides of the bridge, therefore a cross is needed. To ensure the diagonals do not contribute to compression, the members are chosen based on a large relative slenderness ($\bar{\lambda}$). Because those members will bend elastically as soon as a normal force is applied to them and will not contribute by absorbing the horizontal force.

By only making crosses between horizontal beams, a bottleneck arises at the widening and narrowing of the bridge. Here 3 alternatives, shown in Figure 9**,** are compared to find the best solution to reduce this bottleneck.

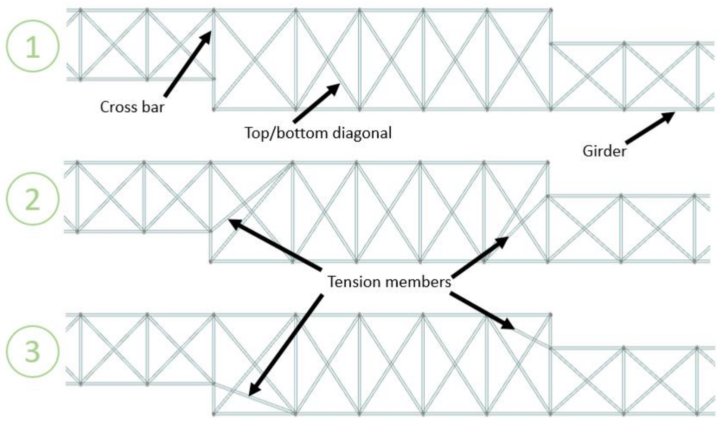


Figure 9 three alternatives top and bottom of the structure

The 3 alternatives are modelled in Scia and there is looked at the highest internal force per type member. Both west and east wind are taken into account. The members are sorted into 3 groups: the girders (members in direction of the length of the bridge), the members perpendicular to the girders and the diagonals. The results are shown below, where negative values are members under tension and positive values under compression.

Table 2 Scia internal forces of the top/bottom of the structure

	Max $N_{alternative1}$ [kN]		Max $N_{alternative2}$ [kN]		Max $N_{alternative3}$ [kN]	
Girders	-173.91	107.885	-101.3	59.67	-227.965	158.31
Cross bar	94.655		81.615		80.515	
Top/bottom diagonals	-141.55		-123.995		-118.25	

Based on the results, alternative 2 is chosen. It has, compared with alternative 3, around the same maximum internal force in the perpendicular beams and the diagonals. In contrast, the maximum internal force in the girders is less than 50% of the internal force in alternative 3. The final design of the top and the bottom is shown in Figure 10. Here the dimensions are not in proportion. They are determined in the last phase if the best design is optimized in the computer software.

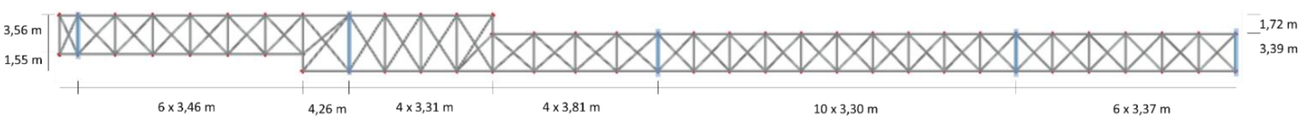


Figure 10 Topview Initial design space frame alternative

Salty environment
By designing a bridge that is made for transporting dry salt, the salt environment has to be taken into account. The salt environment has a great influence on the durability. Salt ensures that the corrosion process accelerates due to the presence of sodium and chloride ions. Saltwater accelerates corrosion 5 times faster than freshwater (Rodriquez, 2018). Therefore there is looked at literature to find a solution to prevent corrosion damage. Figure 3 and Figure 4 show corroded spots of the existing bridge that are damaged. There are several ways solution to avoid corrosion. This section looks at the corrosion prevention methods by literature research that will be related to the requirements of the bridge.

The corrosion prevention methods are based on different principles (Thyssenkrupp, 2021). All methods are shown in Table 3 with their working method.

Table 3 Corrosion prevention methods

	Working method	Underlying related prevention methods
Protective coating	Coating that act as a barrier between the steel and the environmental compounds like water, salt and oxygen	• Rubber paint • Polycoating • Smart coating
Metal plating	A metal layer is added to the steel to protect the steel from the environmental compounds	• Electroplating • Mechanical plating • Electroless • Hot dipping • Sacrificial plating
Corrosion inhibitor	Chemicals suppress the electrochemical processes that lead to corrosion	-
Environmental measures	This method tries to control the environment by reducing the compounds that causes corrosion	-
Modifying the design	The design can be optimized by avoiding cracks and pits where water and salt can be stored	-

Protective coating is easy to apply. It is a thin layer that can be sprayed on the steel. Especially smart coating is attractive, it has multi tasks such as sensing, protection and healing (Ahmed Abdel Nazeer, 2018). In contrast to the other protective coating methods, the protective layer heals itself. Therefore the coating has a high potential to be used more in the future with more developments. Looking at metal plating, it also protects the steel and adds an aesthetic finish. However, it is more difficult to apply than a coating. The other methods are focused on the environment. Corrosion inhibitor cannot avoid corrosion, only suppresses it. Environmental measures are difficult to realize due to the function of the bridge. It is made to transport salt, which is a compound that stimulates corrosion. This cannot be retrieved from the environment. Modifying the design will not be enough on its own to prevent corrosion, but it can be used in combination with other methods. The method focuses on the optimization of the design. However, the idea of avoiding cracks and pits can be taken into account during the initial design of the bridge. For example, choose profiles that have the least cracks and pits or use plates to ensure fewer cracks and pits.

For the design of the conveyor belt bridge 3 and 4 cracks and pits will be avoided and protective coating or metal plating will be used. There is no specific method determined, because it has no great influence on the design. The layers on the steel will be very thin in the order of magnitude of 10^{-6} m (Teknos, 2013) and will be neglected during the design phase.

3.3 Space frame

Space frame structures are an efficient way to span a long distance. They consist of bars and nodes in 3 dimensions. The advantage of a space truss is that there are mainly axial forces, both compression and tension occur. Axial forces are aligned with the extension of the structure (Cena, 2017). This ensures that the material is used optimally. It is a stiff, lightweight structure.

By designing the space frame, the challenge is to use as little as possible members under compression. The disadvantage of members under compression is that buckling can occur. This is not the case with members under tension. If the diagonals are put under tension, the columns will be under compression. Due to a smaller length and a smaller internal force in the columns, the columns are better resistant to buckling than the diagonals. The structure that is shown in Figure 11 has compressed verticals and falling diagonals under tension. This structure will be used on the sides of bridge 3.

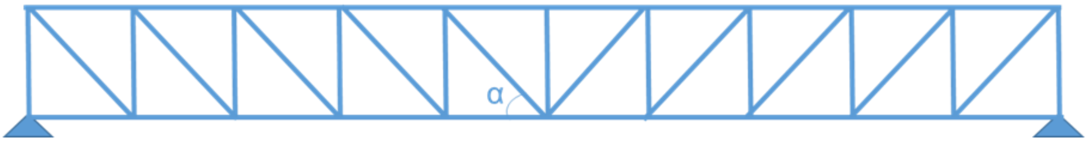


Figure 11 Spaceframe with verticals and falling diagonals

The widths of the squares in the space frame are based on the distance between the supports, because there must be a column above the supports to transfer the forces easily to the supports. Also, a column has to be placed at the locations where the bridge increases and decreases in width. The spans between the columns that are already determined are shown in Figure 14**.**

The smaller the width of a square the more vertical the diagonal is placed, the better the diagonals can withstand vertical forces. However, more diagonals and compressed columns are needed to transfer the forces to the supports, which results in more material, more connections and more costs. So a compromise has to be found. In this design is tried to get the diagonals at an angle of 45 degrees. That results in trying to get the width the same as the height of the square.

The initial design is shown in Figure 12**,** Figure 13 and Figure 14 The supports surround the bridge and are arched in blue in Figure 14. The side view shows the alternation between the directions of the diagonals. This alternation is based on the principle that all diagonals must be under tension. This can be checked in a later stadium in the computer model if the assumption is right.

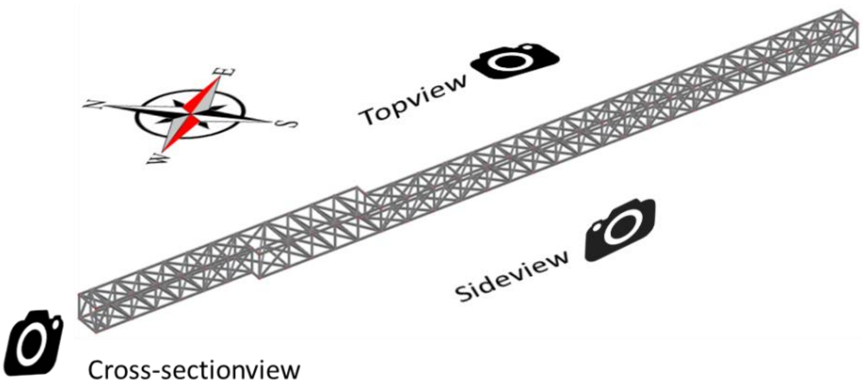


Figure 12 Initial spaceframe design

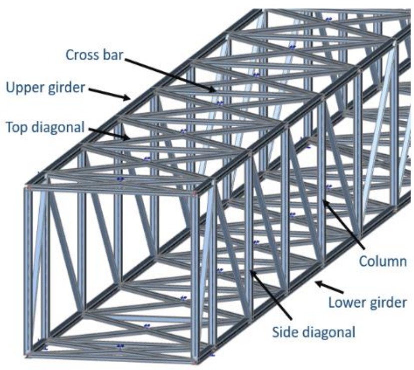


Figure 13 Cross-section view spaceframe alternative

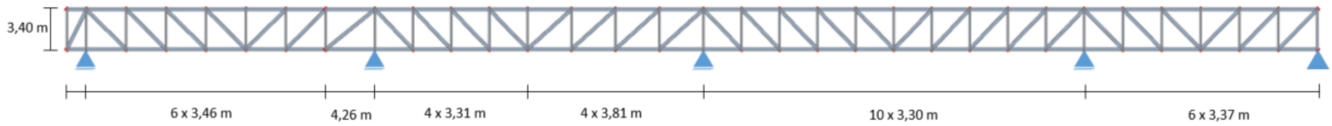


Figure 14 Sideview Initial design space frame alternative

3.4 Castellated beam

Plate girders are beams made of welded plates. The castellated beam is a variant of a plate girder (Timmermans, 1974). The construction of a castellated beam can be seen in Figure 15. An I or H-beam is cut in a pattern shown in the first picture. If both sides are separate from each other, one side is moved a little to make the beam higher without increasing its weight. Increasing the height of the beam results in stronger bending strength and stiffness due to an increase in the moment of inertia, also known as the second moment of area, and the section modules (T.P.Bradley, 2003). The beam can be made even higher by welding an intermediate piece between both cut sides. It increases the moment of inertia and the section modules even more. This results in an economical advantage over a normal H-beam. Also, a castellated beam has a higher maximal load that it can carry in the vertical direction. However, the web will buckle more easily due to a larger web.

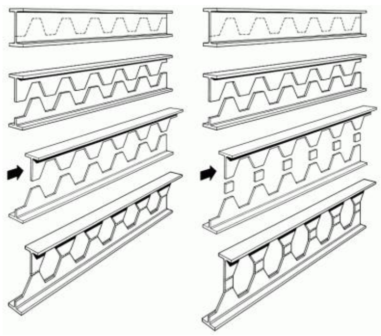


Figure 15 Castellated beam (Vree, n.d.)

In the design, the castellated beam cannot be placed below the bridge, because it results in a critical passage height due to trucks who need to go underneath the bridge. Therefore the beams are placed on the sides. The bottoms of the castellated beams are also the bottom of the bridge. With a height of 3.4 meters, the beams cannot cover the whole side of the bridge. So on top of the castellated beams, a structure of other beams has to be placed. The distances between the columns are based on the space frame design that is shown in Figure 14 to get the columns right above the supports.

All these members are only used to transfer the loads to the castellated beams. The castellated beams must withstand all vertical loads on the bridge on this own. So there are no diagonals needed on the sides. In total 6 castellated beams are needed due to the widening and narrowing of the bridge. At these places, the castellated beams are welded together. Bear in mind that the castellated beam has holes, this was not possible to include in the figure.

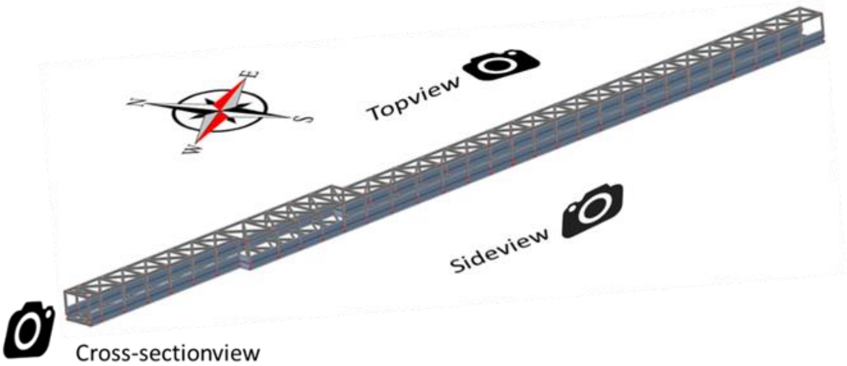


Figure 16 Initial design castellated beam alternative

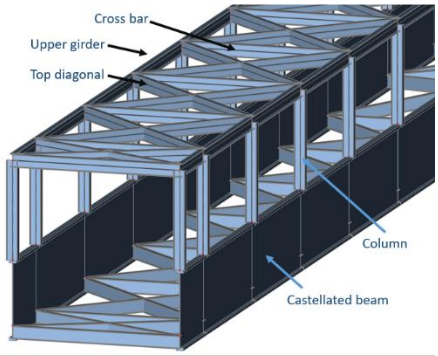


Figure 17 Cross-section view castellated beam alternative

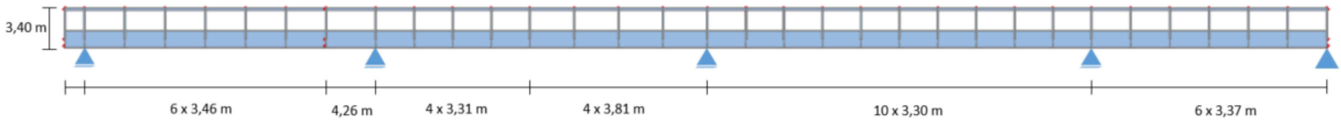


Figure 18 Sideview Initial design castellated beam alternative

3.5 Cable-stayed bridge

Hang constructions can span in general greater distances than plate girders, which means that fewer supports are needed. On the other hand, the bridge will be a lot higher due to the fact that the bridge transfers the loads through the cables above the bridge, while the other alternatives transfer the loads to the support beneath the bridge. There are a lot of different hang constructions. This research will focus on a cable-stayed bridge. Figure 19 shows how a cable-stayed bridge works. It consists a large support under compression, which has several cables above the bridge that lift the deck by tension in the cables. If the number of cables is increased, the frame of the bridge can be lighter. Viaduct de Millau, shown in Figure 20, is a good example of a cable-stayed bridge and is the highest bridge in the world.

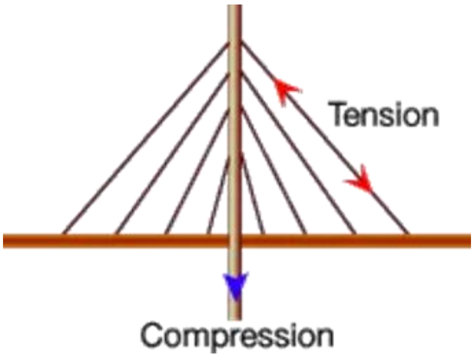


Figure 19 Cable-stayed bridge principle (Robert Lamb, 2021)

Figure 20 Cable-stayed bridge Millau du Viaduct (Wikipedia, 2022)

After some research to design a cable-stayed bridge, problems came up. By trying to solve them, new problems popped up. So with causal reasoning, it is concluded that a cable-stayed bridge is not a good alternative for this situation and therefore no calculation will be performed. These types of bridges are more applicable for larger spans where it is impossible to have a lot of supports. The problems are described in Appendix E.

4. Rough hand calculations

4.1 Loads

Before decisions can be made about the different designs, the forces acting on the bridge must be specialised in their design. Several loads are acting on the bridges. The permanent loads are the weight of the structure of the bridges and the weight of the conveyor belts inside the bridges and are calculated in Appendix F. These loads are determined by the information of the already replaced conveyor belt bridges 1 & 2. The variable loads acting on the bridges are the salt that is transported, people walking in the bridge, wind and snow. The calculations of these loads are shown in Appendix G. Only the loads of snow and wind are determined by the Eurocode 1, the other loads are given by Frisia.

The calculated characteristic loads on the space frame and castellated beam alternatives in the vertical direction are summarized in the table below with epsilon and their psi-values. The characteristic horizontal load due to the wind is 5.04 kN per meter bridge.

Table 4 Characteristic horizontal loads for the space frame and castellated beam alternatives

Vertical	$q_{EG,k}$ [kN/m^1]	$q_{,transport,k}$ [kN/m^1]	$q_{,salt,k}$ [kN/m^1]	$q_{,people,k}$ [kN/m^1]	$q_{,wind,k}$ [kN/m^1]	$q_{,snow,k}$ [kN/m^1]	TOTAL [kN/m^1]	TOTAL [kN]
	$\xi = 0.89$		$\psi_{0/1/2}$ = 1.00/ 1.00/ 1.00	$\psi_{0/1/2}$ = 0.25/ 0.00/ 0.00	$\psi_{0/1/2}$ = 0.00 / 0.20 / 0.00	$\psi_{0/1/2}$ = 0.00 / 0.20 / 0.00	-	
BB4	20.16	5	0.6	1.5	2.71	3.33	33.3	1351.35
BB3-1	14.91	2.5	0.6	1.5	1.5	2.46	23.47	525.73
BB3-2	20.16	5	0.6	1.5	2.03	3.33	32.62	570.85
BB3-3	14.34	2.5	0.6	1.5	1.45	2.37	22.76	1558.15

These characteristic loads (without safety factor) in Table 4 are used to calculate the design loads (with safety factor). The design loads are used to determine the dimensions of the elements of the structure. The magnitude of the safety factor depends on which load is dominant. The calculation of which load is dominant is executed in Excel and shown in Appendix H and gives that snow is the dominant factor for the vertical loads. By including the horizontal load of the wind, the wind is dominant. The first (hand) calculations were done with snow as the dominant factor. In a later stadium, the horizontal loads were included and gave that wind is the dominant factor. To undo the mistake, the distributed loads calculated with snow as the dominant factor must be multiply by $\frac{Total\ value\ wind}{Total\ value\ snow} = \frac{4244.88}{4443.48} = 0.955.$

The design loads can be determined with Table NB.4–A1.2(B) out of the national annex of Eurocode 0 and are executed in Excel. The calculations are shown in Appendix H and give 7.56 kN/m for the horizontal wind load and the results of the vertical loads are shown in Table 5.

Table 5 Dimensions of conveyor belt bridge 3 & 4

Vertical	$q_{EG,d}$ [kN/m^1]	$q_{,transport,d}$ [kN/m^1]	$q_{,salt,d}$ [kN/m^1]	$q_{,people,d}$ [kN/m^1]	$q_{,wind,d}$ [kN/m^1]	$q_{,snow,d}$ [kN/m^1]	TOTAL if snow is dominant [kN/m^1]	TOTAL if wind is dominant [kN/m^1]
BB4	24.22	6.01	0.90	0.56	0.00	5.00	36.69	35.05
BB5-1	17.91	3.00	0.90	0.56	0.00	3.69	26.07	24.91
BB5-2	24.22	6.01	0.90	0.56	0.00	5.00	36.69	35.05
BB5-3	17.23	3.00	0.90	0.56	0.00	3.56	25.25	24.12

4.2 Support reactions

The structures are presumed to be a line to simplify the mechanical model to calculate the support reactions. In Figure 21 conveyor belt bridge 4 is shown with a distributed load on top of the space frame and castellated beam alternative. The horizontal load acting perpendicular to bridge 4 is absorbed by the horizontal component of support 5 (H_5). This support is attached to the 'Saline building' and is shown in Appendix A with the photos made in Harlingen. The horizontal load linear to bridge 4 are taken by the saline support (H_{sal}).

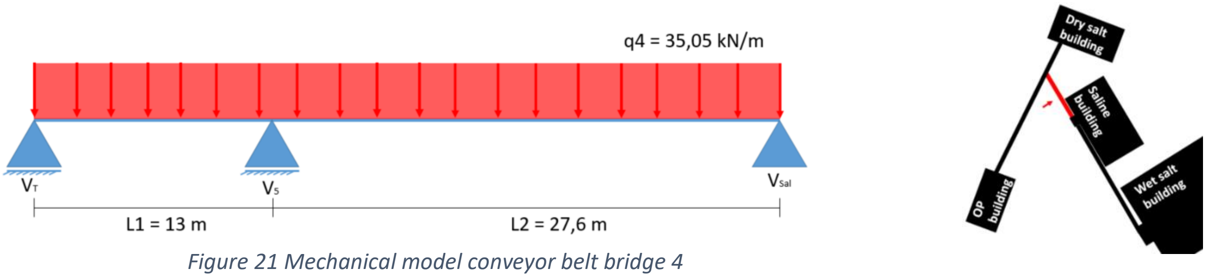


Figure 21 Mechanical model conveyor belt bridge 4

Conveyor belt bridge 3 and 4 are connected. Bridge 4 rests on bridge 3, which means that bridge 4 acts on bridge 3. The load is centred in the middle of the width of bridge 4 as a point load shown in Figure 22. Bridge 3 consists of three parts with different widths, therefore three different distributed loads are shown in Figure 22.

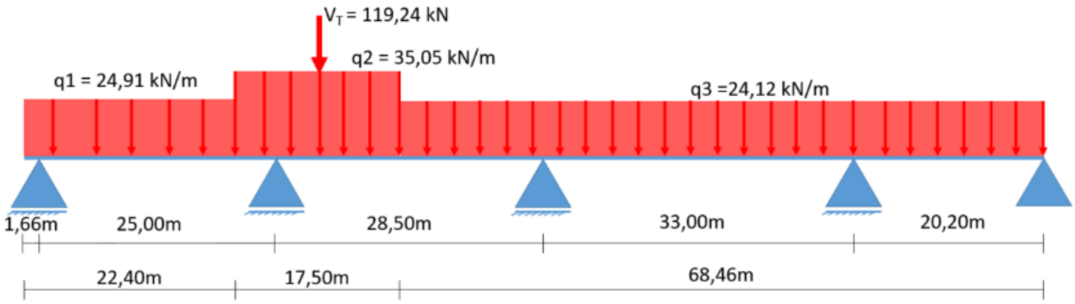
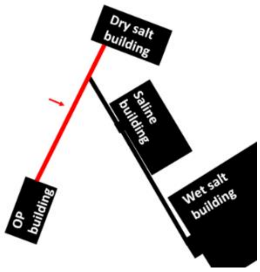


Figure 22 Mechanical model vertical loads conveyor belt bridge 3



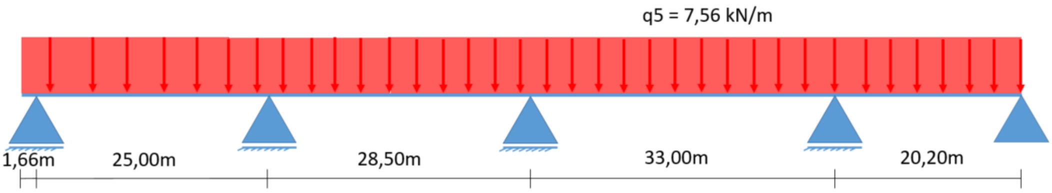


Figure 23 Mechanical model horizontal loads conveyor belt bridge 3

The reaction forces are determined with the Cross method. This method makes use of the force method which is explained in detail in Appendix C. These calculations are first done by hand and using Excel. The force acting on conveyor belt bridge 3 due to conveyor belt bridge 4 is calculated in Appendix I and gives a value of 36.7 kN. This calculation is revised due to the new insights. The calculation before gave a force of 119.24 kN. This value is used in all calculations and is not optimized, because the value has little influence on the bridge. The influence goes from 3.9% to 1.2% of the total load.
With the force of bridge 4 on bridge 3, all forces acting on the bridge are known. The reaction forces of bridge 3 are calculated in the same way as for bridge 3. The model of bridge 4 is more complex as can be seen in Figure 24. The calculation of the moments in the supports can be found in Appendix J.

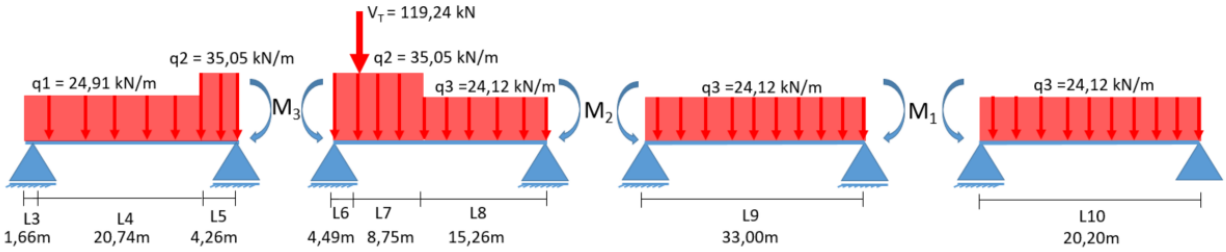


Figure 24 Conveyor belt bridge 3 schematic model

With the moments, the reaction forces are calculated in Excel and compared with values obtained from SCIA. The results are shown in Appendix K. Due to some simplifications in the hand calculations, the reaction forces are not exactly the same, but are in the same order of magnitude. Some distributed loads are taken into account by a point load in the middle of the distributed load. This gives a slightly different answer. The shear force diagram (V-diagram) of the conveyor belt bridge 3 was created in Excel. The difference between the extreme values at a support gives the reaction force of the support. The moment diagram (M-diagram) is the differential of the V-diagram. In Excel, it is difficult to differentiate a plotted line. Therefore a model is created in 'Scia' which can be checked by the shear force diagram of Excel. Finally, the V and M-diagram out of 'Scia' are used to find the internal forces in the structure. These diagrams are shown below.

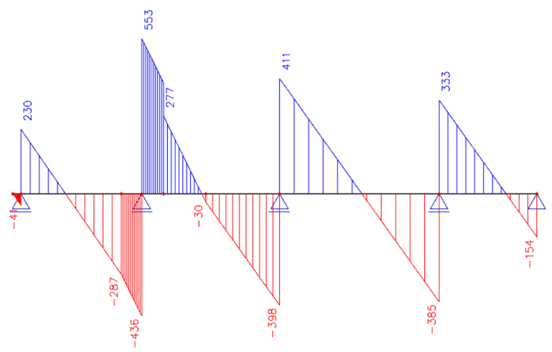


Figure 25 V-diagram conveyor belt bridge 3

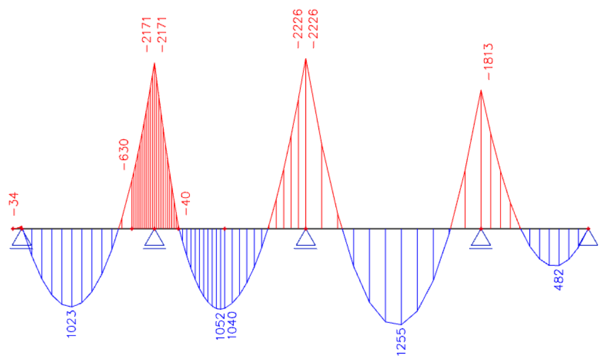


Figure 26 M-diagram conveyor belt bridge 3

4.3 Dimension determination

The dimensions are determined by testing the members on internal forces, deformation and stability. Both global and local. There is only looked at the member that needs the greatest dimensions. This dimension will be used for all other members. The initial calculation makes only use of the profile HEA to simplify the calculations. In a later stadium, the profile type will be changed if necessary. An overview of the tests is shown in Table 6 and Table 7 for the different alternatives. The tests are based on 'Eurocode 3: Design of steel structures' with his national annex. For the test of the castellated beam alternative, the ' HTI staalconstructies dictaat SC3' is used as a guideline with formulas out of Eurocode 3. The formulas used for the tests are derived in Excel to test the material with different dimensions.

Table 6 Tests for the space frame alternative both global and local

	Global	Local	
Space frame	Bending moment	Column	Axial compression
	Displacement		Buckling
	Global buckling	Upper girder	Axial tension/compression
		Lower girder	Axial tension/compression
			Buckling
		Diagonal	Axial tension

Table 7 Tests for the castellated beam alternative both global and local

	Global	Local
Castellated beam	Bending moment	Yield condition T-profile
	Displacement	Yield condition Web
		Buckling

All tests are executed with the ultimate limit state (ULS). According to the Eurocode, the displacement can be calculated with the serviceability limit state (SLS), this state has lower load factors and is for calculation of the displacements of the structure. The displacement was not dominant in the ULS, so it does not have to be calculated for the SLS. The other factors of the serviceability limit state do not apply to the design. The design experiences no vibrations, crack formation and creep due to the use of steel. Also, the steel is not pre-stressed. The used formulas of the tests with elaboration are shown in Appendix L, M and N and give the following result:

Table 8 Dimensions space frame alternative

Spaceframe alternative	Material
Columns	HEA 160A
Diagonals	HEA 100A
Upper girder	HEA 140A
Lower girder	HEA 140A

Table 9 Dimensions Top/bottom structure for both alternatives

Top/bottom alternatives	Material
Cross bars	HEA 100A
Diagonals	HEA 100A

Table 10 Dimensions castellated beam alternative

Castellated beam alternative	Material
Columns	HEA 100A
Upper girder	HEA 100A
Castellated beam	HEA 1000A

5. Alternatives comparison

The 3 alternatives are compared in this section to find the best-suited decision support system founded by means of literature research based on Frisia's requirements. Along the process, it became clear that the cable-stayed bridge has significantly more problems than other alternatives, which is explained in Appendix E. Therefore the cable-stayed bridge could be shot down in advance.

The other two alternatives can easily be compared by a multi-criteria analysis, shown in Table 11by looking at the design and their dimensions and comparing them with the satisfaction of the requirements. Both alternatives comply with the dimensions and load criteria. The support locations are the same. The durability and the speed of installation will not differ much from each other due to the same material and that both designs can be placed in prefabricated parts. The alternatives only differ in costs, processing costs and material costs.

Table 11 Multi-criteria analysis conveyor belt bridge alternatives

	Space frame alternative	Castellated beam alternative
Load criteria	✓	✓
Support locations	+ −	+ −
Durability	+ −	+ −
Speed of installation	+ −	+ −
Costs	+	−

A weak point of the castellated beam alternative is that the design needs a castellated beam with a HEA1000 profile. This profile is not fully exploited, but is needed to withstand the moments above the supports. Exploitation means how much force is taken by the material in comparison with its limit value. Even above support 2 and 3, the gaps have to be welded shut to withstand the moment. Additionally, the widening and narrowing of the bridge lead to difficult connections of the castellated beams. So a lot of cutting and welding is needed to realise the design of the castellated beam alternative.

The space frame alternative makes only use of hinged connections, which leads to easier connections. Also, the internal forces are dived over the upper and lower girder in comparison this the other design where only the castellated beam carries the force to the supports. The space frame alternative uses more members, by contrast the members have smaller dimensions.

The multi-criteria analysis shows that the space frame structure is the best-suited alternative for this situation. The great advantage of the space frame is that the connections are easier and that it can exploit their material better than the castellated beam alternative. The space frame will be modelled in the computer program to optimize the dimensions and exploit the members even more.

6. Model space frame alternative

There is looked at several programs to model the space frame. Eventually is chosen for the program Scia, because all programs have similar features and the external supervisor has a lot of information and experience in Scia. Additionally, Scia has already been used in the process to check values on the advice of the external supervisor.

To get realistic outputs out of the model, the model needs several inputs. The model starts with the design of the bridge created in ‘3. Design alternatives’ with the same support locations**.** The elements in the bridge will initial have the dimensions concluded in ‘4. Rough hand calculations’. The forces acting on the bridge are all placed in the nodes to avoid small moments in the bridge. The bridge will only be analysed globally. The forces are put in load groups and combinations to include all cases according to the equation 6.10a and 6.10b of NEN-EN-1990. In addition, the nodes have to be described and the top and bottom diagonals must be modelled not-linear to ensure they can only have tension in the members.

The output is the deformation per node and the internal force along the whole bridge and the unity check per element of the bridge. Based on the unity check per element, the profile of the element is changed to optimize the unity check value. The unity check must always be below 1. On the other hand, it is preferable to get the value as close to one as possible to save money and exploit the material. This is done for element groups and not done for each element on its own. Otherwise, the plating and other material cannot be attached easily to the members. The elements are divided into the same groups as in ‘3. Design alternatives’.

During the process, the directions of the side diagonals were estimated and could be validated with the Scia model. After modelling the space frame alternative 2 diagonals per side were under compression. After rotating the diagonals, all side diagonals experienced tension. This can be seen in Appendix O

The model is iterated many times to optimize the dimensions of the structure. The final dimensions of the elements in the structure are shown in the table below.

Table 12 Optimized profiles space frame alternative

	Profiles out of the hand calculation	Profiles out of Scia
Column	HEA 160A	HEA100A
Side diagonal	HEA 100A	IPE100
Upper girder	HEA 140A	HEA120A
Lower girder	HEA 140A	HEA120A
Cross bar	HEA 100A	HEA100A
Top/bottom diagonal	HEA 100A	HFL_{eq}50x50x6
Tension member	HEA 100A	HEA100A

As can be seen in Table 12**,** no members of the bridge have greater profiles in the optimization in Scia than in the hand calculations. Most of the groups have smaller profiles. For the side diagonals and the top/bottom diagonals, the type of profile is changed due to the reason that the unity check for the smallest HEA profiles was far smaller than 1. The side diagonals experience more force in the z-direction of the profile than the top/bottom diagonals. Therefore IPE profiles are chosen on the sides and HFL_{eq} profiles on the top and bottom. Also because the top and bottom diagonals are only experiencing tension, so buckling cannot occur. The unity checks of the chosen profiles are shown in Appendix O. Only the lower girder has one element on both sides that does not have an unity check below 1. Nevertheless, there is chosen for HEA 120A profile with extra reinforcement at the element that does not comply. The other option was to choose a larger profile for the entire lower girder. This means that all unit checks are under 1, but the elements that were already sufficient for a HEA 120 A are exploited even less. In general, there can be seen that a lot of elements have a low unity check due to subdividing the elements into groups. So the structure can be cheaper and lighter. However, this gives all profiles other dimensions and therefore the plates are more difficult to attach. For the members that absorb the horizontal force at the widening and narrowing of the bridge, the tension members, are chosen for HEA 100 profiles despite their low unity check. HEA 100 is the smallest profile in the HEA group. By switching to the IPE group, the profile had to be IPE 180. This profile has a greater cross-section area which leads to more steel use. Also, the profile has a higher height which makes it more difficult to connect, because the contrast with the other profiles around the 4 members is less high.

7. Conclusion

The requirements of Frisia are translated into the designs of the alternatives. The forces acting on the bridges are due to wind, snow, own weight of the structure and the weight of the conveyor belts and the salt on top of them. Wind is the dominant factor and gives also a horizontal force. Therefore HEA profiles are used to withstand the horizontal force better, because it has a higher moment of inertia in the horizontal direction. The salt environment has to be taken into account, however it has no great influence on the design and is therefore not included during the design phase. Two No support zones are created where no support can be placed. The designs are calculated and compared with a multi-criteria analysis. The analysis gives that the space frame alternative is the most applicable to the situation of Frisia with their requirements.

The member dimensions of the space frame bridge are optimized in contrast to the dimensions calculated in the hand calculations. The dimensions have been reduced considerably in order to save costs.

8. Discussion

The research methodology is not followed exactly. Scia is used in an earlier stadium than planned to check both the model and the hand calculations. Also, the external supervisor from Tebodin gave small assignments in Scia to get me to understand the problem and the mechanism better. The difference between both values was small, so the Scia values were already used in the hand calculations. Also, the alternative comparison is not executed as planned. In the course of the research became apparent that some of the alternatives had too many problems to design an effective design.

In the hand calculations, some simplifications and assumptions are made to reduce the time of the calculations. For example, the snow load due to higher buildings in the surroundings is not included. Furthermore, all forces acting on the bridge seize to the upper girders. If it was done correctly, all member had their own weight which results in a lot of small moments.

In contrast, some parts of the hand calculations are done very precisely. The reaction forces of the bridge were calculated with the assumption that the supports behave like a fixed supports. Later on, the calculation is optimized by first calculating the moment in the supports. Since there are made more simplifications and assumptions, it is redundant to do this precise. The same applies to the displacement calculation. In this calculation is assumed that the displacement is always maximal in the middle between two supports. For an example is looked at a beam that has a clamp on one side and a hinge on the other side with a q-load on top. By comparing the displacements between the maximal displacements at 5/8 length versus at ½ length can be concluded that the difference is not significant. To be specific the difference is 1.7% between the displacement in the middle vs at 5/8 length.

The planning which was made in advance is shaken up. There is spent more time finding the forces acting on the bridges according to the Eurocode in the research. Also, more computer programs are used than planned to get the best view of the problem. Therefore more time was needed to master the skills in Autodesk and AutoCAD. Due to the use of Scia in an earlier stadium, there was less time needed for the optimization in Scia.

9. Recommendations

By designing a bridge, innumerable alternatives can be made. This research focuses on 3 different supporting structures. Within these supporting structures, different designs can be made. For example, There can be looked at the optimal alpha of the space frame alternative in Figure 11 or other space frame designs can be investigated.

The alternatives are compared with many factors fixed to compare the supporting structures correctly. In further research, the fixed factor can be made variable. For example, the steel type or the type profile. Concerning the type of profile, there can be looked at closed profiles with the advantage that water cannot be stored in cracks or pits. Concerning the steel type , this research only makes use of S235. There can even be made use of other construction materials like timber that is less sensitive to corrosion and can withstand compression better. Furthermore, the amount and locations of the supports can be changed.

Not all variables can be changed, because it is too time-consuming. Certain choices have to be made to be cost-efficient, otherwise the reduction in costs does not outweigh the extra time spent to find the optimal design. Experience is a great aspect that helps to reduce the time to find an acceptable design due to similar projects that have been executed in the past. Those projects give guidance and show the design direction.

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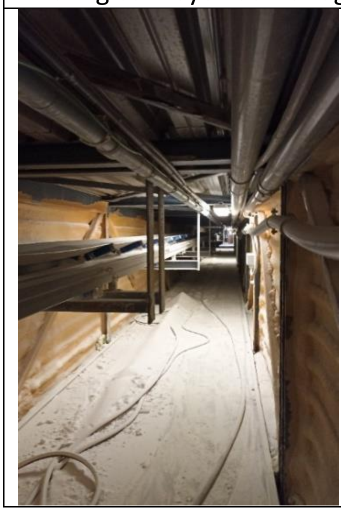
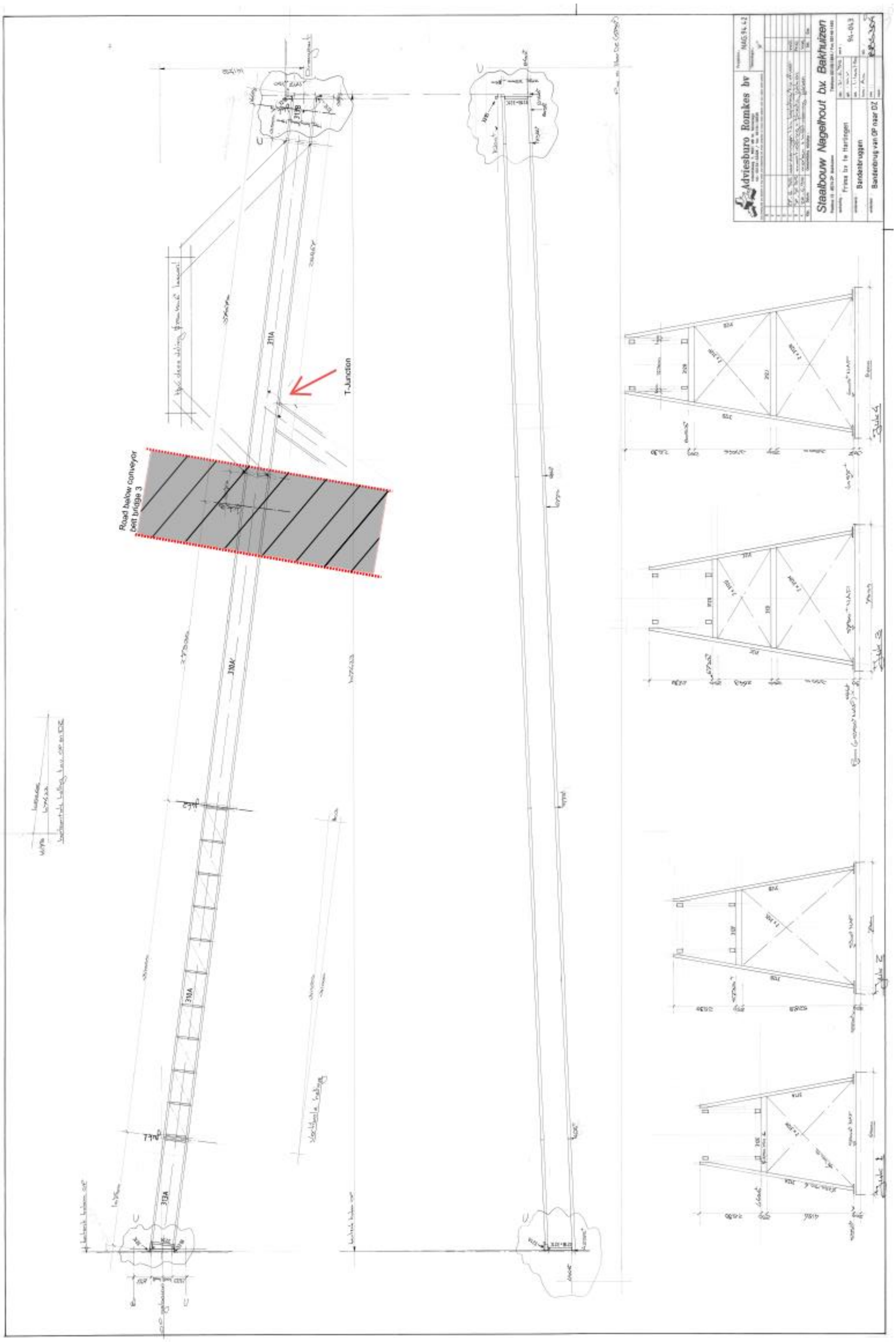
11. Appendix

Appendix A – Pictures of the existing situation

Overview Outside of the bridge Pointcloud vs site visit	
	
	
	

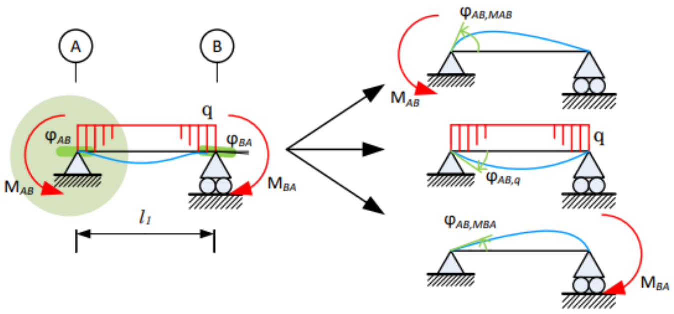
Existing conveyor belt bridge 3	New conveyor belt bridge 2
	

Appendix B - Construction drawing conveyor belt 3



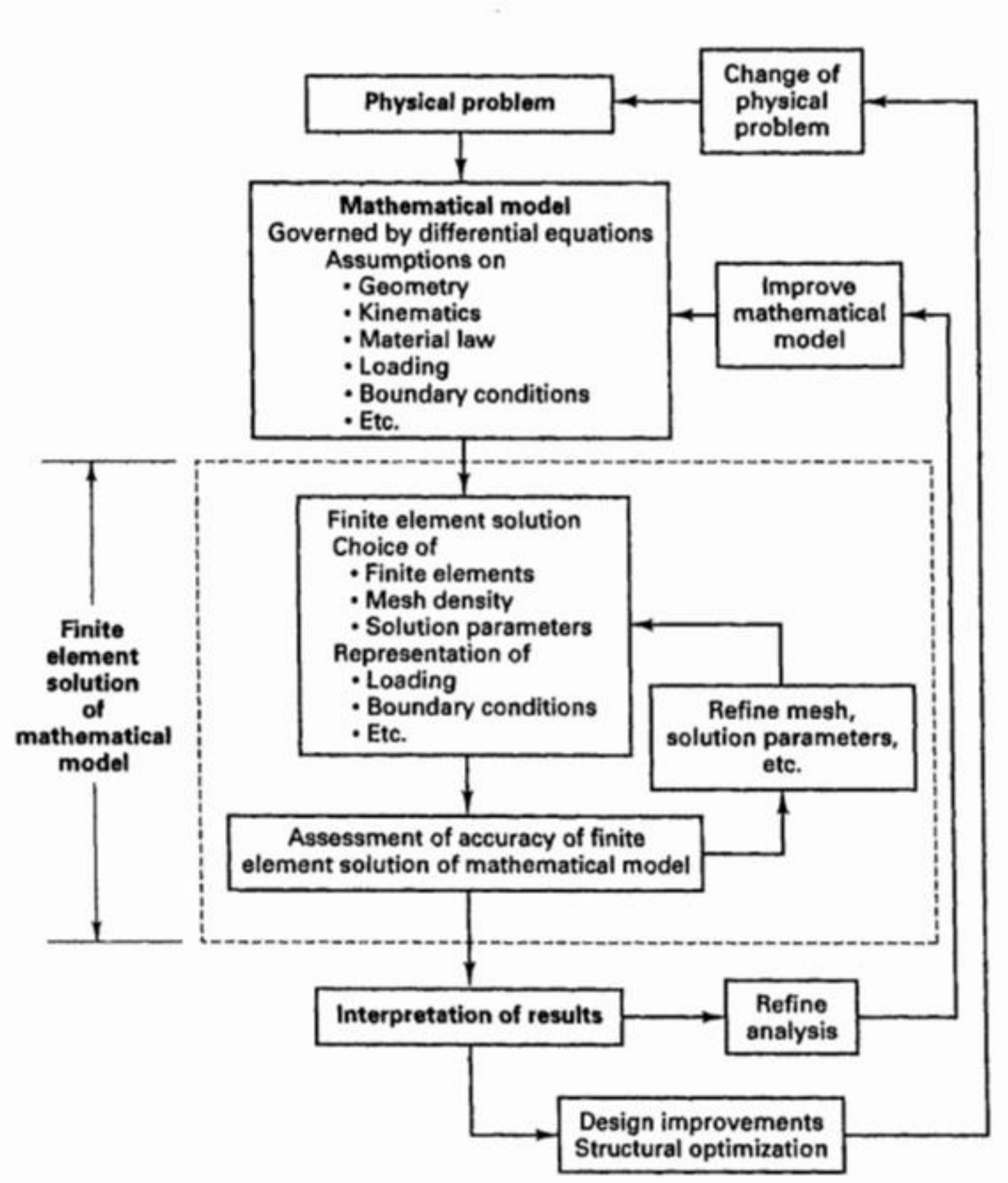
Appendix C - Force method

To evaluate the design of the conveyor belt bridge the force method will be used. This method is explained in detail in 'Reader Structural Mechanics Module 4' written by G.H.Snellink. For this method, a free body diagram (FBD) is needed. An FBD shows the internal forces acting on the structure with the supports. The structure can be divided by the supports in statically determined members, which results in more but simpler systems to analyse. For these small systems and the nodes between the systems, the equilibrium of forces can be used to calculate the reaction forces of the supports. Now the rotation on a node can be determined by splitting the smaller systems. This can be done by dividing a small system into different systems with only one type of force. This is shown in the figure below and reprinted from 'Reader Structural Mechanics Module 4'. The angles can be determined by standard formulas. The angles at node A of the different systems can be summed up to get the total rotation at node A.

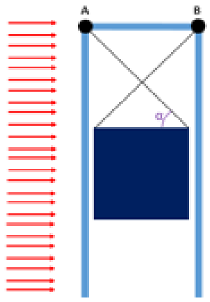
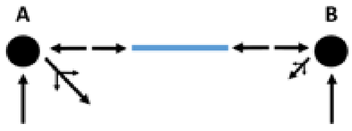
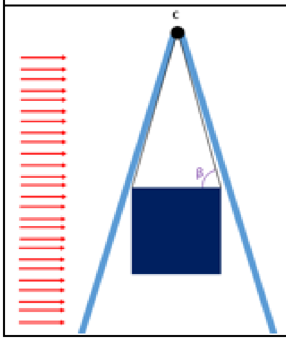
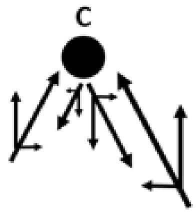


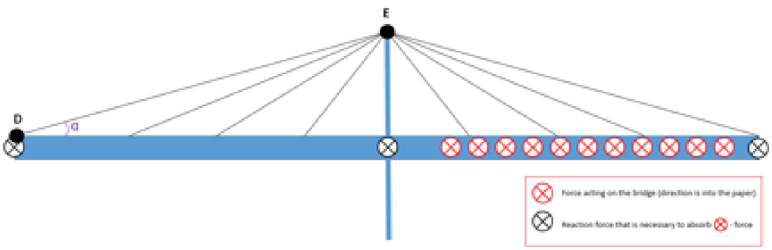
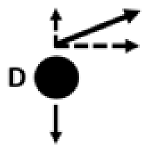
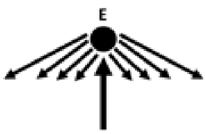
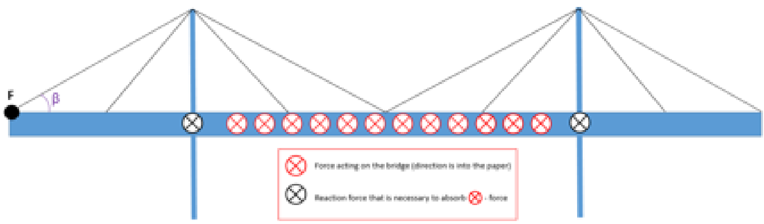
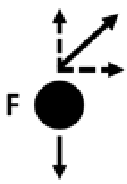
After this is done for one small system, the other systems can be analysed as well by taking into account that a clamp has zero rotation and the rotation of the left side of the node is the same as the rotation at the right side of the node ($\varphi_{clamp} = 0 \ \ \& \ \ \varphi_{BA} = \varphi_{AB} = \varphi_B$).

Appendix D - Finite element method (FEM)



Appendix E – Cable-stayed bridge

Horizontal force at the support		
Cross-section support	Problem of the design cross-section	Nodes with force directions
	• If there is a great horizontal force only from one side, only one side of the cables will transfer the loads. The horizontal force transferred through the cable must be transferred through the cross beam. This cross beam gets a compression force. The problem then is that the cross beam cannot transfer the force to the ground. By looking at Node B, the horizontal internal force in the diagonal has to be the same as the compression force in the cross beam. If this is not the case, there arises a resulting force in the same direction as the horizontal force due to the wind. The resulting force gives a great moment to the bottom of the column due to a large arm (length of the column).	
	• This design has columns which can with stand some horizontal forces. However, in comparison with the vertical force the column can withstand it is very little. The ratio between horizontal and vertical forces that it can withstand has to do with the angle of the column. This is the same for the cable. As can be seen in the pictures; $\alpha < \beta$. This means that the cables of this design can withstand horizontal forces less well. The columns can be placed more horizontal to change the ratio, but that results in a wider support and can be unpleasant on the ground. Or the dimensions have to be upgraded and results in more costs and the material will not be used optimal.	

Horizontal force on the side of the frame creating a moment		
Side view design options	Problems of the design	Nodes with force directions
	The problem with one support is that the horizontal force on both sides on the bridge has to be in the same order of magnitude. Otherwise the bridge wants to rotate around his only support and can be prevented by a force acting on the ends of the bridge. This is not ideal, because it is desired that the system of the bridge works on his own.	
	In point E, all horizontal forces together has to be zero ($\sum F_H = 0$). Otherwise the support gets a moment. This moment will be large due to the great length of the support which acts as the arm.	
	By adding a support. The rotation can be absorbed by only the supports and the angle of all cables are greater: $\alpha < \beta$. Which means that the cables can withstand more vertical loads. However a cable-stayed bridge is chosen for few supports. In comparison with the other alternatives, 3 supports less with only 1 support less far from the building does not outweighs the stability problems a cable-stayed bridge experiences.	

So a cable-stayed bridge is good is processing vertical forces, but less good is horizontal forces. Both perpendicular to the bridge as in the line of the bridge.

Appendix F – Permanent Loads

Own weight

The own weight of the bridge is assumed using the own weight of the conveyor belt bridge 2. This bridge is made with a space frame. One section of bridge 2 is taken and calculated how many elements it has with their weight. These weights are shown in Table 13**.**

Table 13 Dimensions conveyor belt bridge 3 & 4

	n [-]	L [m^1]	q_k [kN/m^1]	$q_{EG,k}$ [kN]
IPE330 - onderregel	2	3,09	0,49	3,03
HE160A - schoor	4	4,52	0,30	5,50
HE220A - bovenregel	2	3,09	0,51	3,12
HE220A - wandregel	2	3,09	0,51	3,12
IPE330 - vloerbalk	1	3,53	0,49	1,73
HE100B - vloerbalk	5	3,53	0,20	3,60
Bekleding gevel	2	11,23	0,50	11,23
Bekleding vloer + dak	2	12,19	0,50	12,19
Totaal	-	-	-	43,51

There is looked at a box with a length of 3.085 m. So the q-load due to this own weight is 43.51/3.085 = 14.10 kN/m. To transfer these values to BB3 and 4, the values will be adapted to the outside width of the bridges.

Table 14 Distributed load per different bridge section

	B (inside) [m]	B (outside) [m]	β [-]	$q_{EG,k}$ [kN/m^1]
BB1 / BB2	3.51	4.20	1.00	14.2
BB4	4.28	5.11	1.22	20.16
BB3-1	2.73	3.56	0.85	14.91
BB3-2	4.28	5.11	1.22	20.16
BB3-3	2.53	3.39	0.81	14.34

For the castellated beam alternative, the own weight is assumed to be the same per meter as the weight of the space frame bridge alternative after a discussion with colleagues within Tebodin.

Static loads

The static loads due to the conveyor belts are given by the client. According to Frisia is the load of one conveyor belt per meter 2.5 kN. Looking at the design of BB3-1 and BB3-3 who is designed for one conveyor belt, so the load for the conveyor belts in section 1 and 3 of BB3 is 2.5 kN/m. BB4 and BB3-2 have two conveyor belts, which results load of 5 kN/m.

Appendix G – Variable loads

Salt

The amount of salt per conveyor belt is derived from values Frisia gave for conveyor belt bridge 1 and 2. There were 3 conveyor belts in the bridges shown below with the loads.

Table 15 Load due to salt in conveyor belt bridge 1 & 2

Naam	Locatie	$q_{zout,k}$ [kg/m^1]
TB1	BB1	35.0
TB7	BB1	27.0
TB12	BB2	117.5

For conveyor belt bridge 3 and 4 is an average of the values of BB1 and BB2 is used:
(35 + 27 + 117.5) / 3 = 60 kg/m^1 = 0.6 kN/m per conveyor belt.

People

The load of people on the walkways is 1.5 kN/m^2 according to Frisia. All walkways have a width of 1 meter, so the load of people on the (maintenance) walkways is 1.5 kN/m.

Snow

There are 3 different snow design situations:

- Permanent/temporary design situation
- Exceptional design situations where exceptional snow loading constitutes the exceptional loading
- Exceptional design situations where exceptional snow drift constitutes the exceptional load

Only the first situation has to be taken into account due to the fact that the last 2 situations do not occur in the Netherlands according to the National Annex. The permanent/ temporary design situation is calculated with the equation below. The referred equations, tables and paragraphs can be found in NEN-EN 1991-1-3 and his National Annex for the snow calculation.

$$s = \mu_i \ C_e \ C_t \ s_k \qquad \text{NEN-EN-1991-1-3 (5.1)}$$

The bridge has two types of snow: snow that falls directly on the bridge (μ_1) and snow that falls from higher buildings in the surrounding due to wind or the slope of the roof (μ_2). The indirect type will not be taken into account in the hand calculations to simplify the model.

Because the roof slope is less than 30°, $\mu_1 = 0.8$ according to Table 5.2.
C_e is the exposure coefficient which is always 1 for buildings in the Netherlands (§5.2 NB).
C_t is the heat coefficient and is also equal to 1 (§5.2).
s_k is the characteristic snow load and is determined for the Netherlands as 0.7 kN/m^2 (4.1 NB).

$$s = \mu_1 \ C_e \ C_t \ s_k = 0.8 * 1 * 1 * 0.7 = 0.56kN/m2 \qquad \text{NEN-EN-1991-1-3 (5.1)}$$

The snow that falls directly on the bridge is 0.56 kN/m^2. Based on the width of the bridge, the distributed load per linear meter bridge is known. The results are shown in the table below.

Table 16 Snow load on conveyor belt bridge 3&4

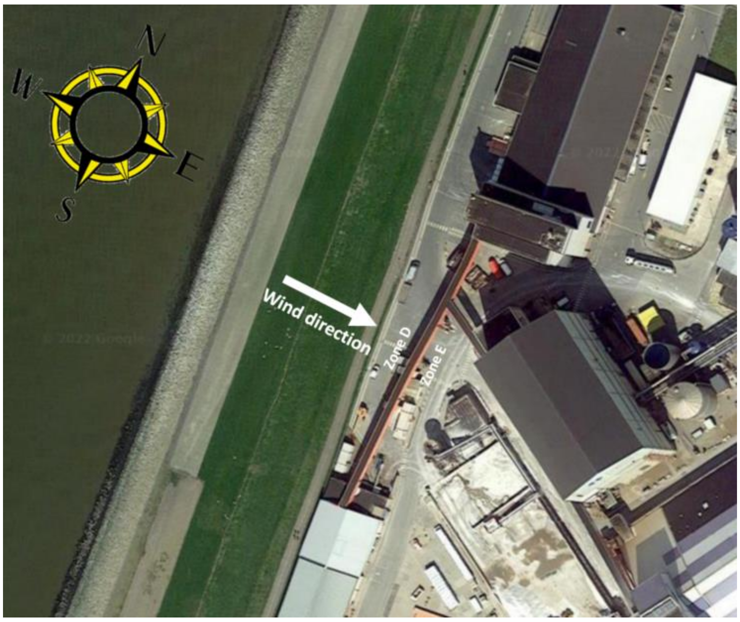
	B (outside) [m]	$q_{w,k,top/bottom}$ [kN/m]
BB4	5.95	3.33
BB3-1	4.40	2.46
BB3-2	5.95	3.33
BB3-3	4.23	2.37

Wind (horizontal)

The referred equations, tables and paragraphs can be found in NEN-EN 1991-1-4 and his National Annex for the wind calculation. Wind is calculated with the following equation:

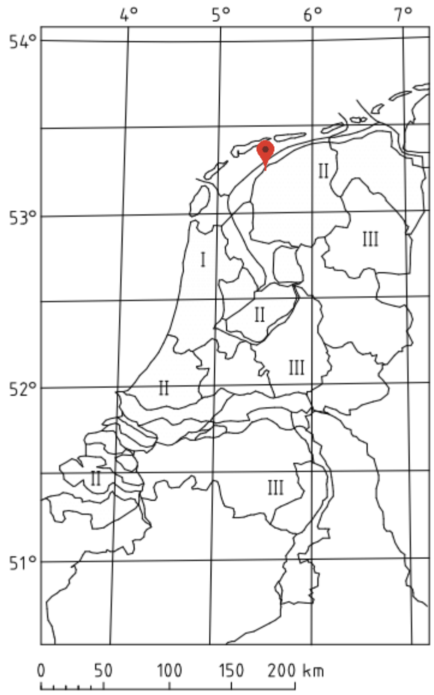
$$w_e = q_p(Z_e)\ c_{pe} \qquad \text{NEN-EN-1991-1-3 (5.1)}$$

Only wind perpendicular to the sides of bridge 4 is taken into account. Because they lead to the maximum force the wind can have on the structure. Wind in all other angles will glide partial along the sides and this gives less pressure on the bridge. The cardinal directions in the top left in the figure below are slightly customized to explain the calculation easier. The north to south line is parallel to conveyor belt bridge 3. Here the bridge can be seen with the wind direction and their zones.



The wind on the west side will be the greatest due to the fact that the west wind has as terrain category 'sea and coastal area', while the terrain category of east wind is 'uncultivated area' (NEN, 2011). For simplification of the hand calculation, The external force of the wind from the west will also be applied on the other sides of the bridges. In that case zone D and zone E of the figure above will flip around.

$q_p(Z_e)$ has standard values in the Netherlands. The values can be obtained from the table NB.5, where the Netherlands is divided into three areas. These areas are shown in the figure below with the location of the bridge. So the bridge is located in wind area II. The height used for the calculation is 5 meters. Now the table can be read and gives $q_p(Z_e) = 1.14\ kN/m^2$



The c_{pe} is the pressure coefficient for external wind and can be calculated using figure 7.5 and table 7.1.

$$h/d \approx 1 => Zone\ D: c_{pe,10} = +0.8 \wedge Zone\ E: c_{pe,10} = -0.5$$

$$w_{e,Zone\ D} = q_p(Z_e)\ c_{pe,Zone\ D} = 1.14 * 0.8 = 0.912$$ NEN-EN-1991-1-3 (5.1)

$$w_{e,Zone\ E} = q_p(Z_e)\ c_{pe,Zone\ E} = 1.14 * -0.5 = -0.57$$ NEN-EN-1991-1-4 (5.1)

To calculate the Wind force on the whole structure from the side the difference between Zone D and E will be used:

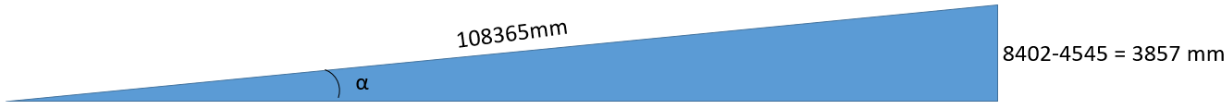
$$q_{w,k,side} = w_{e,Zone\ D} - w_{e,Zone\ E} = 0.912 - -0.57 = 1.482 kN/m^2$$ NEN-EN-1991-1-3 (5.1)

The outside height of the bridges is 3.40m to have an acceptable passage height in the bridge. This is the case for the whole conveyor belt bridge 3 and 4. So the distributed horizontal load is 5.04 kN per meter bridge. This is also applied on conveyor belt bridge 4.

Wind (vertical)

The calculation of the wind acting on the top and bottom of the bridge is made use of §7.3. It is made for open roofs like petrol stations. To calculate the wind pressure, the net pressure coefficient $c_{p,net}$ is used. It gives the maximal pressure difference between the top and bottom of the bridge. The net pressure coefficient is found in Table 7.6 with different roof pitches, zones and blocking. The blockings take into account the obstacles under the bridge. For the bridge, there is a great uncertainty about obstacles below the bridge. Therefore both the lowest and the highest value are taken into account. For the hand calculations, zone B and C are excluded, because the areas are significantly smaller than zone A and therefore less relevant.

The roof slope will be determined by looking at the values in the construction drawing of the exciting conveyor belt bridge. The bottom of the bridge is at the building on the north 8402mm above NAP and at the building at the south 4545mm above NAP. The length of the total bridge is 108365mm.



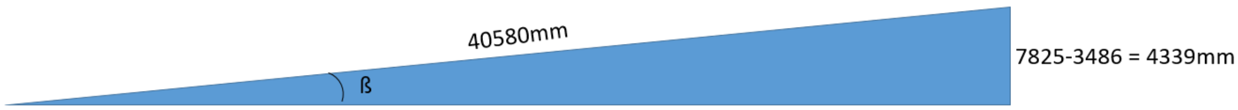
$$\alpha = \sin(3857/108365) = 2°$$

So with interpolating the values of Table 7.6:

Table 17 Dimensions conveyor belt bridge 3 & 4

Roof angle (α)	Overall Force Coefficients (c_f)	Net pressure-Coefficient A ($c_{p,net}$)	Coefficient top ($c_{p,boven}$)	Coefficient bottom ($c_{p,onder}$)
2.0^0	+0.3	+0.6	-0.3	+0.6
	-1.3	-1.5	-1.5	+0.2

$$q_{w,k,top/bottom} = q_p(Z_e)\ c_f = 1.14 * 0.3 = 0.342 kN/m2$$ NEN-EN-1991-1-4 (5.1)



$$\alpha = \sin(4339/40580) = 6°$$

Table 18 Dimensions conveyor belt bridge 3 & 4

Roof angle (α)	Overall Force Coefficients (c_f)	Net pressure-Coefficient A ($c_{p,net}$)	Coefficient top ($c_{p,boven}$)	Coefficient bottom ($c_{p,onder}$)
6.0^0	+0.4	+0.9	-0.5	+0.9
	-1.4	-1.6	-1.6	+0.2

$$q_{w,k,top/bottom} = q_p(Z_e)\ c_f = 1.14 * 0.4 = 0.456 kN/m2$$ NEN-EN-1991-1-3 (5.1)

Table 19 vertical wind loads per section

	L [m]	B [m]	$q_{w,k,top}$ [kN/m]
BB4	40.581	5.95	2.71
BB3-1	22.400	4.40	1.50
BB3-2	17.500	5.95	2.03
BB3-3	68.460	4.23	1.45

Appendix H – Design loads

Dominant load

All characteristic values will be translated into design values. These values will have a factor. This factor depends on which load is dominant. This can be calculated using the phi-values and the equations below.

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"} + \text{"} \gamma_P P \text{"} + \text{"} \gamma_Q \psi_{0,1} Q_{k,1} \text{"} + \text{"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$ NEN-EN-1990 (6.10a)

$$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} \text{"} + \text{"} \gamma_P P \text{"} + \text{"} \gamma_Q Q_{k,1} \text{"} + \text{"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$ NEN-EN-1990 (6.10b)

In these equations, epsilon and psi-values will be used. The ψ-factors are derived from Table NB.2-A1.1 of the national annex of Eurocode 0. There is usually a deviation from the ψ-value in the figure due to the fact that one value cannot represent the whole category. Also, the values are outdated and differ per country. Ψ-values are factors to combine different loads. By combining loads, not the whole load have to be taken into account based on the ψ-value. The ψ-value of the people and salt in the bridge are determined with logic reasoning. Equations 6.10a and 6.10b are elaborated in the tables below.

Table 20 Input values for equation 6.10a and 6.10b

Name	Value	Reference
γ_G	1.35	TA1.2(B) 1990 NB
γ_Q	1.5	TA1.2(B) 1990 NB
ε	0.89	TA1.2(B) 1990 NB
$\psi_{0,salt}$	1	Design Conveyor belt bridge 1 & 2
$\psi_{0,people}$	0.25	Design Conveyor belt bridge 1 & 2
$\psi_{0,wind}$	0	TA1.2.1 1990 NB
$\psi_{0,snow}$	0	TA1.2.1 1990 NB

Table 21 Answers equations 6.10a and 6.10b

Dominant factor	Formula	=	Answer [kN]
G	= y_g*Total_G + y_Q*Psi_0_salt *Total_Salt + y_Q*Psi_0_people*Total_people + y_Q*Psi_0_wind*Total_wind + y_Q*Psi_0_snow*Total_snow	=	4273.45
Salt	= y_g*ephalson*Total_G + y_Q*Total_Salt + y_Q*Psi_0_people*Total_people + y_Q*Psi_0_wind*Total_wind + y_Q*Psi_0_snow*Total_snow	=	3827.33
People	= y_g*ephalson*Total_G + y_Q*Total_people + y_Q*Psi_0_salt*Total_Salt + y_Q*Psi_0_wind*Total_wind + y_Q*Psi_0_snow*Total_snow	=	4078.67
Wind	= y_g*ephalson*Total_G + y_Q*Total_wind + y_Q*Psi_0_salt*Total_Salt + y_Q*Psi_0_people*Total_people+y_Q*Psi_0_snow*Total_snow	=	4244.88
Snow	= y_g*ephalson*Total_G + y_Q*Total_snow + y_Q*Psi_0_salt*Total_Salt + y_Q*Psi_0_people*Total_people + y_Q*Psi_0_wind*Total_wind	=	4443.48

The equation with the highest values gives the dominant load. Table 21 gives that snow is the dominant factor for the vertical loads.

Design loads

The design loads are calculated with equation 6.10b and Table NB.4-A1.2(B) out of Eurocode 0. Table 22 shows the vertical design loads if snow is the dominant load.

Table 22 Distributed vertical design loads when snow is dominant

vertical	$q_{EG,d}$ [kN/m^1]	$q_{,transport,d}$ [kN/m^1]	$q_{,zout,d}$ [kN/m^1]	$q_{,mesnen,d}$ [kN/m^1]	$q_{,wind,d}$ [kN/m^1]	$q_{,sneeuw,d}$ [kN/m^1]	TOTAL [kN/m^1]
BB4	24.22	6.01	0.9	0.56	0	5	36.69
BB5-1	17.91	3	0.9	0.56	0	3.69	26.07
BB5-2	24.22	6.01	0.9	0.56	0	5	36.69
BB5-3	17.23	3	0.9	0.56	0	3.56	25.25

The first (hand) calculations were done with snow as the dominant factor. In a later stadium, the horizontal loads were included and gave that wind is the dominant factor. To undo the mistake, the distributed loads calculated with snow as the dominant factor must be multiply by $\frac{Total\ value\ wind}{Total\ value\ snow} = \frac{4244.88}{4443.48} = 0.955.$

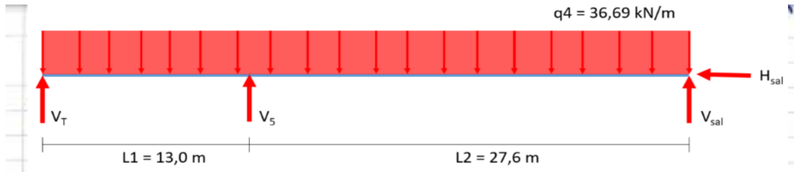
Table 23 Distributed vertical design loads when wind is dominant

Vertical	Total loads when snow is dominant [kN/m1]	Total loads when wind is dominant [kN/m1]
BB4	36.69	35.05
BB5-1	26.07	24.91
BB5-2	36.69	35.05
BB5-3	25.25	24.12

Table 24 Distributed horizontal design loads when wind is dominant

horizontal	$q_{EG,d}$ [kN/m^1]	$q_{,transport,d}$ [kN/m^1]	$q_{,zout,d}$ [kN/m^1]	$q_{,mesnen,d}$ [kN/m^1]	$q_{,wind,d}$ [kN/m^1]	$q_{,sneeuw,d}$ [kN/m^1]	TOTAL [kN/m^1]
BB4	0	0	0	0	7.56	0	7.56
BB5-1	0	0	0	0	7.56	0	7.56
BB5-2	0	0	0	0	7.56	0	7.56
BB5-3	0	0	0	0	7.56	0	7.56

Appendix I – Reaction forces conveyor belt bridge 4



$$-\varphi_{S,L,M} = \frac{-M L_1}{3EI} = -\left(4\tfrac{1}{3} M\right)\frac{1}{EI}$$

$$\varphi_{S,L,q_4} = \frac{q L_1^3}{24EI} = \frac{1}{EI}(3358{,}66)$$

$$\varphi_{S,L} = \frac{1}{EI}\left(3358{,}66 - 4\tfrac{1}{3} M\right)$$

$$-\varphi_{S,R,q_4} = \frac{q L_2^3}{24EI} = \frac{-1}{EI}(32141{,}32)$$

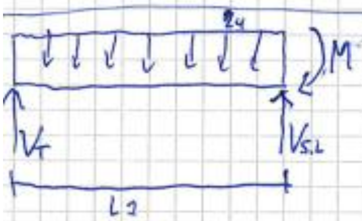
$$\varphi_{S,R,M} = \frac{ML}{3EI} = \frac{1}{EI}(9{,}2M)$$

$$\varphi_{S,R} = \frac{1}{EI}(-32141{,}32 + 9{,}2M)$$

$$\varphi_{S,L} = \varphi_{S,R} \Rightarrow \frac{1}{EI}\left(3358{,}66 - 4\tfrac{1}{3}M\right) = \frac{1}{EI}(-32141{,}32 + 9{,}2M)$$

$$13.533\,M = 35499{,}98$$

$$M = 2623 \text{ kNm}$$



$$\Sigma V_{S,L} = \tfrac{1}{2}\cdot q_4 \cdot L_1^2 - M - L_1 \cdot V_T = 0$$

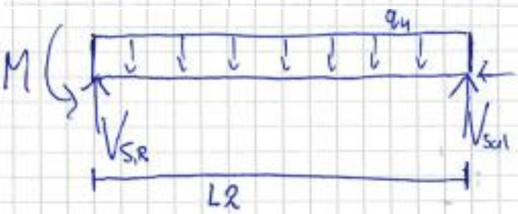
$$3100{,}31 - 2623 - 13V_T = 0$$

$$13V_T = 477{,}15$$

$$V_T = 36{,}7 \text{ kN}$$

$$\Sigma V = q_4 \cdot L_1 - V_T - V_{S,L} = 0 \Rightarrow V_{S,L} = q_4 \cdot L_1 - V_T = 476{,}97 - 36{,}7$$

$$V_{S,L} = 440{,}3 \text{ kN}$$



$$\Sigma M_{V_{S,R}} = L_2 V_{sal} + M - \tfrac{1}{2} q_4 L_2^2 = 0$$

$$27{,}6\, V_{sal} = \tfrac{1}{2}\cdot 36{,}69 \cdot 27{,}6^2 - 2623$$

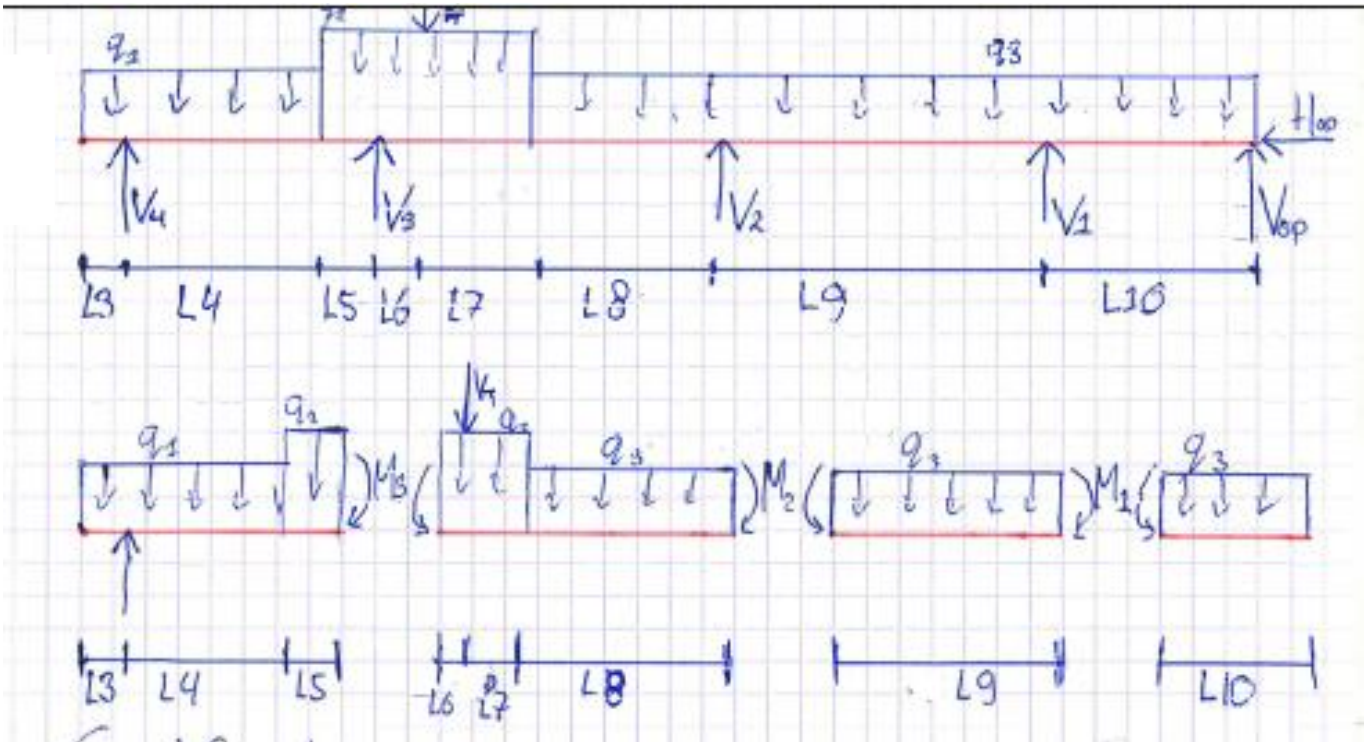
$$27{,}6\, V_{sal} = 11351{,}5$$

$$V_{sal} = 411{,}3 \text{ kN}$$

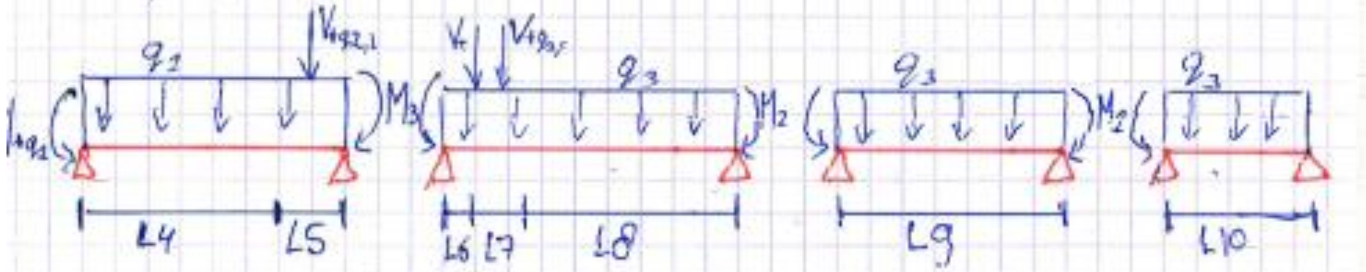
$$\Sigma V = V_{S,R} + V_{sal} - L_2 \cdot q_4 = 0 \Rightarrow V_{S,R} = 27{,}6 \cdot 36{,}69 - 411{,}3$$

$$V_{S,R} = 601{,}4 \text{ kN}$$

$$V_S = V_{S,L} + V_{S,R} = 440{,}3 + 601{,}4 = 1041{,}6 \text{ kN}$$



Simplification:



Known values

$L4 = 20{,}74\,m$

$L5 = 4{,}26\,m$

$L6 = 4{,}49\,m$

$L7 = 8.75\,m$

$L8 = 15.26\,m$

$L9 = 33.00\,m$

$L10 = 20.20\,m$

$q_1 = 26.07\,kN/m$

$q_3 = 25.25\,kN/m$

$M_{+q_1} = \frac{1}{2} \cdot q_1 \cdot L_3^2 = 35{,}92\,kNm$

$V_F = 119{,}24\,kN$

$V_{+q_{2,L}} = (q_2 - q_1) \cdot L_5 = 45{,}24\,kN$

$V_{+q_{2,R}} = (q_2 - q_3) \cdot (L_6 + L_7) = 151.47\,kN$

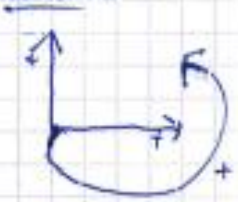
Unknown values

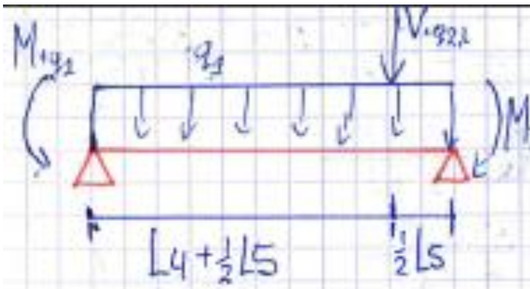
M_1, M_2 and M_3

Methods Used

Joint method, forget me nots

Axis





$$\varphi_{3L,M_{-q_1}} = -\frac{M\cdot l}{6EI} = -\frac{35{,}92\cdot(20{,}74+4{,}26)}{6EI} = \frac{-898}{6EI}$$

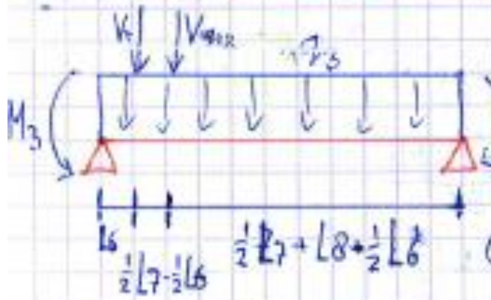
$$\varphi_{3L,q_1} = \frac{ql^3}{24EI} = +\frac{26{,}07\times(20{,}74+4{,}26)^3}{24EI} = \frac{407343{,}75}{24EI}$$

$$\varphi_{3L,V_{-q_2L}} = \frac{Fab(l+b)}{6lEI} = +\frac{45{,}24\cdot(2{,}13\times 22{,}87)(25+22{,}87)}{6\cdot 25\ EI} = \frac{703{,}30}{EI}$$

$$\varphi_{3L,M_3} = -\frac{ML}{3EI} = -M_3\frac{25}{3EI}$$

$$\varphi_{3L} = \varphi_{3L,M_{-q_1}} + \varphi_{3L,q_1} + \varphi_{3L,V_{-q_2L}} + \varphi_{3L,M_3} = -\frac{898}{6EI} + \frac{407343{,}75}{24EI} + \frac{703{,}30}{EI} - M_3\frac{25}{3EI} =$$

$$= \frac{1}{EI}\left(17526{,}29 - \frac{25}{3}M_3\right)$$



$$\varphi_{3R,q_3} = -\frac{ql^3}{24EI} = -\frac{25{,}25\cdot 28{,}5^3}{24\ EI} = -\frac{24354{,}81}{EI}$$

$$\varphi_{3R,V_1} = \frac{Fab(l+b)}{6lEI} = -\frac{119{,}24\times 4{,}49\times 24{,}01(28{,}5+24{,}01)}{6\times 28{,}5\ EI} = -\frac{3947{,}36}{EI}$$

$$\varphi_{3R,V_{q_2R}} = \frac{Fab(l+b)}{6lEI} = -\frac{151{,}47\times 6{,}62\times 21{,}88(28{,}5+21{,}88)}{6\times 28{,}5\ EI} = -\frac{6463{,}89}{EI}$$

$$\varphi_{3R,M_3} = +\frac{ML}{3EI} = \frac{28{,}5M_3}{3EI} \quad \wedge \quad \varphi_{3R,M_2} = \frac{ML}{6EI} = \frac{28{,}5M_2}{6EI}$$

$$\varphi_{3R} = \varphi_{3R,q_3} + \varphi_{3R,V_1} + \varphi_{3R,V_{q_2R}} + \varphi_{3R,M_3} + \varphi_{3R,M_2} = -\frac{24354{,}81}{EI} - \frac{3947{,}36}{EI} - \frac{6463{,}89}{EI} + \frac{28{,}5M_3}{3EI} +$$

$$\frac{28{,}5M_2}{6EI} = \frac{1}{EI}\left(-34766{,}06 + 9\tfrac{1}{2}M_3 + 4\tfrac{3}{4}M_2\right)$$

$$\varphi_{2L,q_3} = +\frac{ql^3}{24EI} = -\varphi_{3R,q_3} = \frac{24354{,}81}{EI}$$

$$\varphi_{2L,V_1} = \frac{Fab(l+a)}{6lEI} = +\frac{119{,}24\times 4{,}49\times 24{,}01(28{,}5+4{,}49)}{6\times 28{,}5\ EI} = +\frac{2479{,}97}{EI}$$

$$\varphi_{2L,V_{q_2R}} = \frac{Fab(l+a)}{6lEI} = +\frac{151{,}47\cdot 6{,}62\cdot 21{,}88(28{,}5+6{,}62)}{6\times 28{,}5\ EI} = +\frac{4505{,}99}{EI}$$

$$\varphi_{2L,M_3} = \frac{ML}{6EI} = -\frac{28{,}5M_3}{6EI} \quad \wedge \quad \varphi_{2L,M_2} = \frac{ML}{3EI} = -\frac{28{,}5M_2}{3EI}$$

$$\varphi_{2L} = \varphi_{2L,q_3} + \varphi_{2L,V_1} + \varphi_{2L,V_{q_2R}} + \varphi_{2L,M_3} + \varphi_{2L,M_2} = \frac{24354{,}81}{EI} + \frac{2479{,}97}{EI} + \frac{4505{,}99}{EI} - \frac{28{,}5M_3}{6EI} - \frac{28{,}5M_2}{3EI}$$

$$= \frac{1}{EI}\left(31340{,}77 - 4\tfrac{3}{4}M_3 - 9\tfrac{1}{2}M_2\right)$$

$$\varphi_{2R,q_3} = \varphi_{1L,q_3} = \frac{q l^3}{24EI} = \frac{25{,}25 \cdot 33{,}00^3}{24\ EI} = \frac{37808{,}72}{EI}$$

$$\varphi_{2R,M_2} = \frac{ML}{3EI} = \frac{33{,}00\ M_2}{3\ EI}$$

$$\varphi_{2R,M_1} = \frac{ML}{6EI} = \frac{33{,}00\ M_1}{6\ EI}$$

$$\varphi_{2R} = -\varphi_{2R,q_3} + \varphi_{2R,M_2} + \varphi_{2R,M_1} = -\frac{37808{,}72}{EI} + \frac{33\ M_2}{3\ EI} + \frac{33\ M_1}{6\ EI}$$

$$\varphi_{1L,M_2} = \frac{ML}{6EI} = \frac{33\ M_2}{6\ EI} \ \wedge \ \varphi_{1L,M_1} = \frac{ML}{3EI} = \frac{33\ M_1}{3\ EI}$$

$$\varphi_{1L} = \varphi_{1L,q_3} - \varphi_{1L,M_2} - \varphi_{1L,M_1} = \frac{1}{EI}\left(37808{,}72 - 5\tfrac{1}{2} M_2 - 11 M_1\right)$$

$$\varphi_{1R,q_3} = \frac{q l^3}{24EI} = \frac{25{,}25 \cdot 20{,}20^3}{24\ EI} = \frac{8671{,}70}{EI}$$

$$\varphi_{1R,M_1} = \frac{ML}{3EI} = \frac{20{,}20\ M_1}{3EI}$$

$$\varphi_{1R} = -\varphi_{1R,q_3} + \varphi_{1R,M_1} = \frac{1}{EI}\left(-8671{,}70 + \frac{20{,}20}{3} M_1\right)$$

$$\varphi_{3L} = \varphi_{3R}$$

$$\frac{1}{EI}\left(17526{,}29 - \frac{25}{3} M_3\right) = \frac{1}{EI}\left(-34766{,}06 + 9\tfrac{1}{2} M_3 + 4\tfrac{3}{4} M_2\right)$$

$$\frac{1}{EI}\left(-17{,}83 \cdot M_3 - 4{,}75 M_2\right) = \frac{1}{EI} \cdot -52292{,}35$$

$$\varphi_{2L} = \varphi_{2R}$$

$$\frac{1}{EI}\left(31340{,}77 - 4\tfrac{3}{4} M_3 - 9\tfrac{1}{2} M_2\right) = \frac{1}{EI}\left(-37808{,}72 + 11 M_2 + 5{,}5 M_1\right)$$

$$\frac{1}{EI}\left(4{,}75 M_3 + 20{,}5 M_2 + 5{,}5 M_1\right) = \frac{1}{EI}\left(69149{,}49\right)$$

$$\varphi_{1L} = \varphi_{1R}$$

$$\frac{1}{EI}\left(37808{,}72 - 5\tfrac{1}{2} M_2 - 11 M_1\right) = \frac{1}{EI}\left(-8671{,}70 + \frac{20{,}20}{3} M_1\right)$$

$$\frac{1}{EI}\left(5{,}5 M_2 + 17{,}73 M_1\right) = \frac{1}{EI}\left(46480{,}42\right)$$

$$\frac{1}{EI}\left(4{,}75\,M_3 + 20{,}5\,M_2 + 5{,}5\,M_1\right) = \frac{1}{EI}\,69149{,}49 \qquad \text{Eq 1}$$

$$\frac{1}{EI}\left(17{,}83\,M_3 + 4{,}75\,M_2\right) = \frac{1}{EI}\,52292{,}97 \qquad \text{Eq 2}$$

$$\frac{1}{EI}\left(5{,}5\,M_2 + 17{,}73\,M_1\right) = \frac{1}{EI}\,46480{,}42 \qquad \text{Eq 3}$$

$* \; 17{,}73 \cdot Eq1 - 5{,}5\,Eq3 \Rightarrow$

$$\frac{1}{EI}\left(84{,}22\,M_3 + 333{,}22\,M_2\right) = \frac{1}{EI}\,970\,378{,}15 \qquad \text{Eq 4}$$

$* \; 70{,}15\,Eq2 - Eq4 \Rightarrow$

$$\frac{1}{EI}\left(1250{,}77\,M_3\right) = \frac{1}{EI}\,2697973{,}70$$

$$M_3 = 2157{,}05 \text{ kNm}$$

$$Eq2: \left(17{,}83 \cdot 2157{,}05 + 4{,}75\,M_2\right)\frac{1}{EI} = \frac{1}{EI}\,52292{,}97$$

$$\left(38460{,}21 + 4{,}75\,M_2\right)\frac{1}{EI} = \frac{1}{EI}\,52292{,}97$$

$$4{,}75\,M_2 = 13832{,}76$$

$$M_2 = 2912{,}16 \text{ kNm}$$

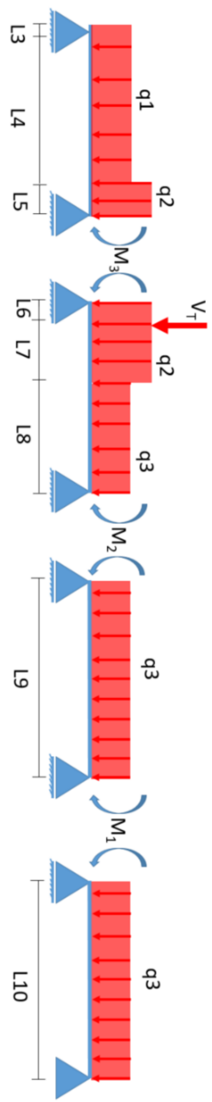
$$Eq3: \frac{1}{EI}\left(5{,}5 \cdot 2912{,}16 + 17{,}73\,M_1\right) = \frac{1}{EI}\,46480{,}42$$

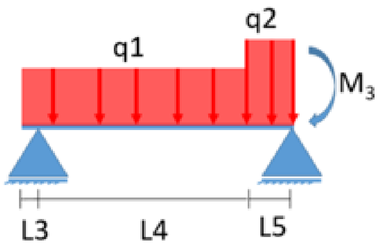
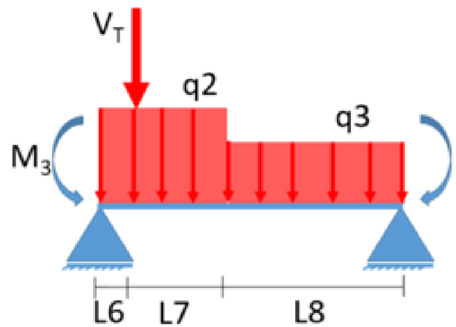
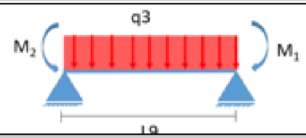
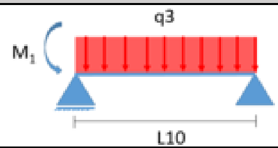
$$\frac{1}{EI}\left(16016{,}88 + 17{,}73\,M_1\right) = \frac{1}{EI}\,46480{,}42$$

$$17{,}73\,M_1 = 30463{,}54$$

$$M_1 = 1718{,}19 \text{ kNm}$$

Appendix K – Reaction forces conveyor belt bridge 3



Name	Formula		Value	Unit	Figure
V4,q1l	= q1 * L3	=	43.28	kN	
V4,q1r	=((q1_*L4_) * (((L4_+L5_) - 0.5*(L4_))^2) * (1.5*(L4_) + (L4_+L5_ - 0.5*(L4_)))) / (L4_+L5_)^3	=	338.78	kN	
V3,q1	=((q1_*(L4_)) * (((0.5*L4_)^2 * (0.5*(L4_) + 3*(0.5*L4_+L5_)))) / (L4_+L5_)^3)	=	201.91	kN	
V4,q2	=((q2_*(L5_)) * (((0.5*L5_)^2 * (0.5*(L5_) + 3*(0.5*L5_+L4_)))) / (L4_+L5_)^3)	=	3.21	kN	
V3,q2l	=((q2_*L5_) * (L4_+0.5*L5_)^2 * (1.5*L5_ + (L4_+0.5*L5_))) / (L4_+L5_)^3	=	153.09	kN	
V3,q2r	=(q2_*(L6_+L7_) * ((L8_+0.5*(L6_+L7_))^2) * (1.5*(L6_+L7_) + L8_+0.5*(L6_+L7_))) / (L6_+L7_+L8_)^3	=	414.97	kN	
V2,q2	=((q2_*(L6_+L7_)) * (((0.5*(L6_+L7_))^2 * (0.5*(L6_+L7_) + 3*(L8_+0.5*(L6_+L7_))))) / (L6_+L7_+L8_)^3)	=	70.81	kN	
V3,vt	=(V_T*((L8_+L7_)^2) * (3*L6_+L7_+L8_)) / (L6_+L7_+L8_)^3	=	110.76	kN	
V2,vt	=(V_T*((L6_)^2) * (L6_+3*(L7_+L8_))) / (L6_+L7_+L8_)^3	=	8.49	kN	
V3,q3	=((q3_*(L8_)) * (((0.5*(L8_))^2 * (0.5*(L8_) + 3*(L6_+L7_+0.5*L8_)))) / (L6_+L7_+L8_)^3)	=	60.22	kN	
V2,q3l	= (q3_*(L8_)*((L6_+L7_+0.5*(L8_))^2) * (1.5*(L8_)+L6_+L7_+0.5*(L8_))) / (L6_+L7_+L8_)^3	=	300.35	kN	
V2,q3r	= 0.5 * q3_*L9	=	429.00	kN	
V1,q3l	= 0.5 * q3_*L9_	=	429.00	kN	
V1,q3r	= 0.5*q3_*L10_	=	255.03	kN	
Vop	= 0.5*q3_*L10_	=	255.03	kN	
Handcalulation values (with the loads if snow is dominant)					SCIA Values (with the loads if snow is dominant)
Vop	$\sum V_{OP}$	=	255.03	kN	251 kN
V1	$\sum V_{1,x}$	=	684.02	kN	689 kN
V2	$\sum V_{2,x}$	=	808.64	kN	809 kN
V3	$\sum V_{3,x}$	=	940.95	kN	952 kN
V4	$\sum V_{4,x}$	=	385.26	kN	373 kN

Appendix L – Space frame dimensions

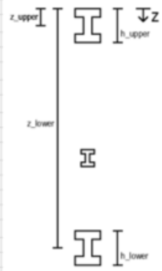
Legend

(orange)	Input value
(green)	Conclusion

Name	Formula	Value	Unit	Comment	References
l3	=	1.66	m		
l4	=	20.74	m		
l5	=	4.26	m		
l6	=	4.49	m		
l7	=	8.75	m		
l8	=	15.26	m		
l9	=	33.00	m		
l10	=	20.20	m		
q1	=	12.46	kN/m		
q2	=	17.53	kN/m		
q3	=	12.06	kN/m		
M1	=	907	kNm		
M2	=	1112	kNm	Calculated on paper and shown in the appendix	
M3	=	1081	kNm		
M_greatest	=	1111.62	kNm	This moment can be derived from the moment diagram. The moment acts on support 2	
V_T	=	113.8742	kN		
Height Spaceframe	h =	3.4	m		
Yield strength steel Yield strength steel	f_y =	235000	kN/m^2		
Elastic modulus	E =	210000000	kN/m^2		
	=				
Height Spaceframe (♥<->♥)	= h-0.5*(h_upper/1000)-0.5*(h_lower/1000) =	3.27	m	From the heart of the upper girder to the heart of the lower girder	
Greatest shear force	V_greatest =	494.50	kN	The greatest shear force is found at support 3 and is divided by 2, because this force is absorbed by both sides of the bridge	
Greatest moment	M_greatest =	1112	kNm	Greatest moment is around support 2, therefore the tension is in the upper girder and compression in the lower girder	
Compression lower girder due to vertical forces	= M_greatest / h_heart =	340	kN		
Compression lower girder due to horizontal forces	= ABS(Max_com_girder) =	101	kN	This value is calculated by Scia	
Compression lower girder	=N_lower_vert + N_lower_horizontal =	442	kN	Both horizontal and vertical loads act on the girders	
Minimal area lower girder	= N_lower / (0.7 * f_y) =	0.0026842	m^2	Assume max 70% of capacity can be used to taken into account possible buckling.	
Minimal area lower girder	= A_min_lower * 10^6 =	2684.24	mm^2	So HEA 140A will be used (with A = 3142mm^2)	
Tension upper girder due to vertical forces	= M_greatest / h_heart =	340	kN		
Tension upper girder due to horizontal forces	= ABS(Max_ten_girder)	60	kN	This value is calculated by Scia	
Tension upper girder	=N_upper_vert + N_upper_horizontal =	400	kN	Both horizontal and vertical loads act on the girders	
Minimal area upper girder	= N_upper / f_y =	0.0017018	m^2		
Minimal area upper girder	= A_min_upper * 10^6 =	1701.82	mm^2	So HEA 140A will be used (with A = 3142mm^2)	

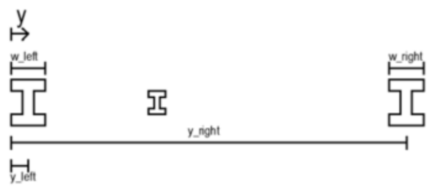
Moment of inertia space frame y-direction

Name	Formula	Value	Unit	Comment
	=			
				The reader Structural Mechanics Mod 4 is used
Height upper girder	h_upper =	133	mm	
Moment of iniertia upper girder	Iy_upper =	10330000	mm^4	
Area upper girder	A_upper =	3142	mm^2	
position upper girder in axis system	z_upper =	66.5	mm	
Sz_upper	= A_upper * z_upper =	208943	mm^3	
d0_upper	= z_upper - d0 =	-1633.5	mm	
	= d0_upper^2 *A_upper =	8.38E+09	mm^4	
Height lower girder	h_lower =	133	mm	
Moment of iniertia lower girder	Iy_lower =	10330000	mm^4	
Area lower girder	A_lower =	3142	mm^2	
position lower girder in axis system	z_lower =	3333.5	mm	
Sz_lower	= A_lower * z_lower =	10473857	mm3	
d0_lower	= z_lower - d0 =	1633.5	mm	
	= d0_lower^2 * A_lower =	8.38E+09	mm^4	
	=			
d0	= (Sz_lower + Sz_upper) / (A_lower + A_upper) =	1700	mm	
Moment of inertia space frame without diagonal	= (Iy_upper + Iy_lower) + (d0_upper^2 *A_upper + d0_lower^2 * A_lower) =	1.68E+10	mm^4	
Moment of inertia space frame	= Iy_nodia / 1.25 =	1.34E+10	mm^4	δv is taken into account by deviding the Iy,tot by 1.25 (this factor is an assumption)
Moment of inertia space frame	= Iy_tot / 10^12 =	0.013431	m^4	This is done to change the unit from mm^4 to m^4
EI	= E * I_spaceframe =	2820451	kNm^2	



Moment of inertia space frame z-direction

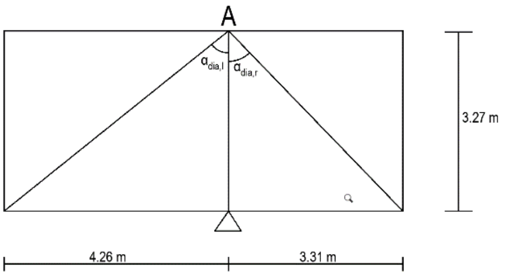
Name	Formula	Value	Unit	Comment
				The reader Structural Mechanics Mod 4 is used.
				Iz is based on the upper girders with the distance between them of the width of the
Width space frame	w_spaceframe =	3.39	m	
Width space frame (♥<->♥)	=	3.25	m	From the heart of the left upper girder to the heart of the right upper girder
Width left upper girder	w_left =	140	mm	
Moment of iniertia left upper girder	Iz_left =	3893000	mm^4	
Area left upper girder	A_left =	3142	mm^2	
position left upper girder in axis system (y_left)	=0.5*w_left =	70	mm	
Sz_left	= A_left * y_left =	219940	mm^3	
d0_left	= y_left - d0_z =	-1625	mm	
	= d0_left^2 *A_left =	8.3E+09	mm^4	
Width right upper girder	w_right =	140	mm	
Moment of iniertia right upper girder	Iz_right =	3893000	mm^4	
Area right upper girder	A_right =	3142	mm^2	
position right upper girder in axis system (y_right)	= y_left + w_heart*1000 =	3320	mm	
Sz_right	= A_right * y_right =	10431440	mm^3	
d0_right	= y_right - d0_z =	1625	mm	
	= d0_lower^2 * A_lower =	8.38E+09	mm^4	
	=			
d0_z	= (Sz_left + Sz_right) / (A_left + A_right) =	1695	mm	
Moment of inertia space frame without diagonal	= (Iz_left + Iz_right) + (d0_left^2 *A_left + d0_right^2 * A_right) =	1.66E+10	mm^4	δv is taken into account by deviding the Iz,tot by 1.25 (this factor is an assumption)
Moment of inertia space frame	= Iz / 1.25 =	1.33E+10	mm^4	

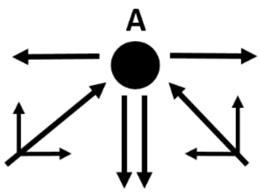


Global buckling

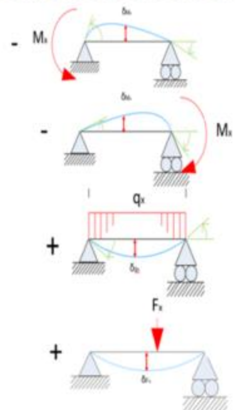
				The bridge can buckle in z-direction more easily than y-direction due to a lower moment of inertia	
Area upper girders	= A_left + A_right =	6284	mm^2		
Area upper girders	= A_uppers / 10^6 =	0.006	m^2		
Moment of inertia space frame	I_z =	0.013	m^4		
EI_z	= E * I_z =	2789047.5	kNm^2		
$F_{critical}$	=(PI()^2 * EI) / (L6_ + L7_ + L8_)^2 =	34271	kNm^2		
λ	= SQRT(f_y * A_uppers_m / F_critical) =	0.21	-	λ << λlocaal, So global buckling is not dominant	(6.49)

		Buckling column	2097.0581			
Buckling column	Internal force column	N_column =	492.80865	kN	The column with the greatest internal force is the column above the support with the highest reactionforce. For BB3 is that support 3 with a force of 1032.06kN. Due to a column on both sides of the bridge is the higest internal force in a column 516.03kN	
	Height column	h_column =	3172	mm	The column has a HEA 160A profile	
	Area column	A_column =	3877	mm²		
	Moment of inertia column in weakest direction	Iz_column =	6156000	mm^4	By comparing Iz and Iy, Iz was the lowest. This gives that the column will buckle in z-direction	
	Fcritical	=(PI()^2*E*(Iz_column/10^12))/(h_column/1000)^2 =	1268	kN		
	λ_column	= SQRT(f_y * (A_column/10^6) / F_critical_column) =	0.85	-		(6.49)
	Imperfection factor	α_imperfection =	0.49	-	This factor can be found by table 6.1 and 6.2 of NEN 1991-1-1. Because the S235 steel profile is rolled with h/b <1.2 and it buckles around the z-axis	
	Φ_column	= 0.5 * (1 + Alpha_imperfection * (Lambda_column - 0.2) + Lambda_column^2) =	1.02	-		(6.49)
	X_column	= 1 / (Phi_column + SQRT(Phi_column^2 - Lambda_column^2)) =	0.63	-		(6.49)
	Specific partial factor	γ_M1 =	1	-		(Chapter 6.1)
	Calculation value of the buckling resistance	= X_column * f_y * (A_column/10^6) / y_M1 =	576.08818			(6.47)
	Buckling stability factor column	N_Ed / N_b,Rd =	0.8554396	< 1	So with column with the profile HEA 160A they are resistant to buckling	(6.46)
		Bending moment				
Bending moment	Section modulus (Wy_tot)	= Iy_nodia / (h*1000 - d0) =	9875527.7	mm^3	The section modulus can be calculated by the moment of inertia divided by the distance of the utmost fibre to the neutral line.	
	Specific partial factor	γ_M0 =	1			(Chapter 6.1)
	Critical moment	= (Wy_tot / 10^9) * f_y / y_M0 =	2320.75	kNm²	This formula can be used because the spaceframe can be classified in class 1	(6.13)
	Bending moment factor	= M_greatest / M_c_Rd =	0.48	< 1	So the bridge is resistant to the bending moment	(6.12)
		Buckling lower girder			The greatest moment is at support 2. This moment causes compression in the lower girder. If the girder can withstand this moment at support 2, it can withstand in the whole bridge	
Buckling lower girder	Fcritical_lower	=(PI()^2 * E * (Iy_lower/10^12)) / ((L6_ + L7_ + L8_) / 8)^2 =	1686.98	kN		
	λ_lower	= SQRT(f_y * (A_lower / 10^6) / Fcritical_lower) =	0.66	-		(6.49)
	Imperfection_factor_lower	α_imperfection =	0.49	-		
	Φ_lower	= 0.5 * (1 + Imperfection_factor_lower * (lambda_lower - 0.2) + lambda_lower^2) =	0.83	-		
	X_lower	= 1 / (Phi_lower + SQRT(Phi_lower^2 - lambda_lower^2)) =	0.7483166	-		
	Specific partial factor	γ_M1 =	1	-		
	Calculation value of the buckling resistance	= X_lower * f_y * (A_lower / 10^6) / y_M1 =	552.5345			
	Buckling stability factor lower	N_Ed / N_b,Rd =	0.7991485	< 1	So the lower girder with the profile HEA 120A is resistant to buckling	





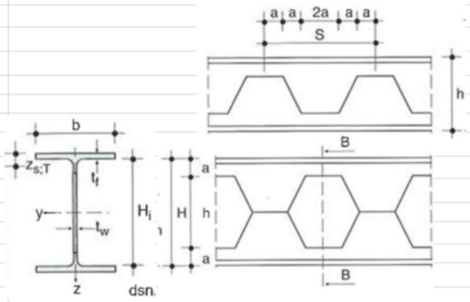
		Axial tension diagonals			There is looked at two diagonals and the diagonal with the highest internal force will be used for the determation of the profiles
Axial tension diagonals	Height Spaceframe (♥<->♥)	= h-0.5*(h_upper/1000)-0.5*(h_lower/1000) =	3.27	m	From the heart of the upper girder to the heart of the lower girder
	length space frame_r	=(L6_ + L7_) / 4 =	3.31	m	This is the horizontal length between 2 columns right of support 3
	N_Ed_column_r	= 574 / 2 =	287	kN	The internal force is equally divided over the columns on both sides of the bridge. The 574 kN is optained from the v-chard.
	α_diagonal_r	=ATAN(length_space_frame_r / h_heart) * 180 / PI() =	45.37	°	
	N_Ed_diagonal_r	= N_Ed_column__r / (COS(alpha_diagonal_r * PI() / 180)) =	408.56	kN	
	length space frame_l	= L5_ =	4.26	m	This is the horizontal length between 2 columns left of support 3
	N_Ed_column_l	= 456 / 2 =	228	kN	The internal force is equally divided over the columns on both sides of the bridge. The 456 kN is optained from the v-chard.
	α_diagonal_l	=ATAN(length_space_frame_l / h_heart) * 180 / PI() =	52.52	°	
	N_Ed_diagonal_l	= N_Ed_column_l / (COS(alpha_diagonal_l * PI() / 180)) =	374.66	kN	
	N_Ed_diagonal_max	= MAX(N_Ed_column_r; N_Ed_diagonal_l)	408.56	kN	
	A_diagonal	= (N_Ed_diagonal_max / f_y) * 10^6	1739	mm	So HEA 100A will be used (with A = 2124mm²)
Support		**Global displacement**			
3 to 4	δ_{q1}	= 1000 * (5 / 384) * q_een * (L4_ + L5_)^4 / EI =	22.46	mm	All displacements are determined in the middle between the supports and are derived from the forget me nots. All values are multiplied by 1000 to get the answer in mm.
	$\delta_{q2,pointload}$	= 1000* ((L5_ * (q2_ - q_een) * (0.5 * L5_) * (3 * (L4_ + L5_)^2 - 4 * (0.5 * L5_)^2)) / (48 * EI)) =	0.63	mm	
	δ_{M3}	= 1000 * ((M_drie * (L4_ + L5_)^2) / (16 * EI)) =	14.97	mm	
	$\delta_{Total\ 3\text{-}4}$	$\delta_{q1} + \delta_{q2,pointload} - \delta_{M3}$ =	8.13	mm	This means the space frame is moved downwards between support 3 and 4
2 to 3	δ_{q3}	= 1000 * (5 / 384) * q3_ * (L5_ + L6_ + L7_)^4 / EI =	5.22	mm	
	$\delta_{q2,pointload}$	= 1000 * ((L6_ + L7_) * (q2_ - q3_) * (0.5 * (L6_ + L7_)) * (3 * (L6_ + L7_ + L8_)^2 - 4 * (0.5 * (L6_ + L7_)^2))) / (48 * EI) =	7.38	mm	
	$\delta_{Vt,pointload}$	= 1000 * ((L6_) * (V_T) * (L6_) * (3 * (L6_ + L7_ + L8_)^2 - 4 * L6_^2)) / (48 * EI) =	39.95	mm	
	δ_{M3}	= 1000 * (M_drie * (L6_ + L7_ + L8_)^2) / (16 * EI) =	19.45	mm	
	δ_{M2}	= 1000 * (M_twee * (L6_ + L7_ + L8_)^2) / (16 * EI) =	20.01	mm	
	$\delta_{Total\ 2\text{-}3}$	δq3 + δq2,pointload +δV_T,pointload - δM3 - - δM2 =	13.10	mm	
1 to 2	δ_{q3}	= 1000 * (5 / 384) * q3_ * (L9_)^4 / EI =	66.0	mm	
	δ_{M2}	= 1000 * (M_twee * (L9_)^2) / (16 * EI) =	26.8	mm	
	δ_{M1}	= 1000 * (M_een * (L9_)^2) / (16 * EI) =	21.9	mm	
	$\delta_{Total\ 1\text{-}2}$	δq3 - δM2 - - δM1 =	17.3	mm	
OP to 1	δ_{q3}	= 1000 * (5 / 384) * q3_ * (L10_)^4 / EI =	9.3	mm	
	δ_{M1}	= 1000 * (M_een * (L10_)^2) / (16 * EI) =	8.2	mm	
	$\delta_{Total\ OP\text{-}1}$	δq3 - δM2 - - δM1 =	1.1	mm	
	δ_{Max}	= MAX(Dis_34;Dis_23;Dis_12;Dis_OP1) =	17.32	mm	So the greatest displacement is between support 1 and 2
	Displacement per meter	= Dis_12 / (L9_ * 1000) =	0.000525	mm/mm	The displacement per meter is very low, so the spaceframe is resistant to displacement



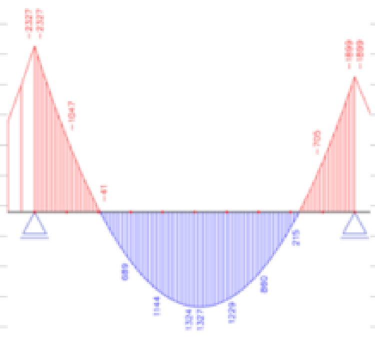
Appendix M – Top/bottom structure

Name	Formula	Value	Unit	Comment	References
Buckling cross bar					
Internal force cross bar	N_cross bar =	124	kN	The crossbar with the greatest internal force is the crossbar above the support with the highest reactionforce. For BB3 is that support 3 with a force of 124kN. Due to a crossbar on both sides of the bridge is the higest internal force in a crossbar 516.03kN	
Length crossbar	L_crossbar =	5110	mm		
Area crossbar	A_crossbar =	2124	mm²	The crossbar has a HEA 100A profile	
Moment of inertia crossbar in weakest direction	Iz_crossbar =	1338000	mm4	By comparing Iz and Iy, Iz was the lowest. This gives that the crossbar will buckle in z-direction	
Fcritical	=(PI()^2*E*(Iz_crossbar/10^12))/(l_crossbar/1000)^2 =	106.20	kN		
λ_crossbar	= SQRT(f_y * (A_column/10^6) / F_critical_column) =	2.17	-		(6.49)
Imperfection factor	α_imperfection =	0.49	-	This factor can be found by table 6.1 and 6.2 of NEN 1991-1-1. Because the S235 steel profile is rolled with h/b <1.2 and it buckles around the z-axis	
Φ_crossbar	= 0.5 * (1 + Alpha_imperfection * (lamba_crossbar - 0.2) + lamba_crossbar^2) =	3.33	-		(6.49)
X_crossbar	= 1 / (Psi_crossbar + SQRT(Psi_crossbar^2 - lamba_crossbar^2)) =	0.17	-		(6.49)
Specific partial factor	γ_M1 =	1	-		(Chapter 6.1)
Calculation value of the buckling resistance	= X_column * f_y * (A_column/10^6) / y_M1 =	85.14			(6.47)
Buckling stability factor crossbar	N_Ed / N_b,Rd =	0.69	< 1	So with crossbar with the profile HEA 100A they are resistant to buckling	(6.46)
Axial Tension diagonal top/bottom					
Maximal tension in top/bottom diagonal	N_ED_diagonal_top_max =	81.615	kN		
Area top/bottom diagonal	= (N_ED_diagonal_top_max / f_y) *10^6 =	347.30	mm²	So HEA 100A will be used (with A = 2124mm²)	
Moment of inertia (z-direction)	Iz_top_diagonal =	1338000	mm⁴	HEA 100A properties	
L_top_diagonal	=SQRT(4260^2 + 5110^2) =	6653	mm		
Area diagonal of the top	A_top_diagonal =	2124	mm²		
Fcritical	=(PI()^2*E*(Iz_top_diagonal/10^12))/(L_top_diagonal/1000)^2 =	63	kN		
λ_column	= SQRT(f_y * (A_top_diagonal/10^6) / F_critical_top_diagonal) =	2.82	-	Lambda is greater than 2, it has a large relative slenderness	(6.49)

Appendix N – Castellated beam alternative dimensions

	Name	Formula	Value	Unit	Comment	Eurocode references
	Height Structure	h =	3.4	m		
	Yield strength steel Yield strength steel	f_y =	235000	kN/m²		
	Elastic modulus	E =	210000000	kN/m²		
	Shear modulus	G =	793000000	kg/mm		
		HEA100A propperties				
	Height	h_HEA1000A =	0.99	m		
	Width	b =	0.3	m		
	Web thickness	t_w =	0.0165	m		
	Flange thickness	t_f =	0.031	m		
	Moment of inertia (z-direction)	I_z =	0.00014004	m^4		
	Moment of inertia t	I_t =	0.000008374	m^4		
	Area	A =	0.03468	m²		
	Connection flange / web	r =	0.03	m		
		Castellated beam HEA100A propperties				
	Moment of inertia at gap	$I_{y,min}$ =	0.01284378	m^4		
	Area T-profile	A_T =	0.01326	m²		
	Section Modulus T-profile	$W_{y,T,el}$ =	0.0002742	m^3		
	height web in T-profile	a (ht) =	0.2475	m		
	Thickness flange middle	$Z_{s;T}$ (e_T) =	0.0495	m		
	length repeating gaps	S (m) =	1.485	m		
	Moment of inertia T-Profile (z-direction)	$I_{z,T}$ =	0.00006993	m^4		
	Section Modulus castellated beam	$W_{y,min}$ =	0.017295	m^3		
	EI	E * (I_castellatedbeam * 1.15) =	3101772.87	kNm²	The displacement of the castellated beam is based on Iy,min and has to be multiplied by 15% if the castellated beam has no plate inbetween the T-	
Support		Global displacement			Forget me nots are used to determine the global displacement	
3 to 4	δ_{q1}	= 1000 * (5 / 384) * q_een * (L4_ + L5_)^4 / EI_castellatedbeam =	20.42	mm	All displacements are determined in the middle between the supports and are derived from the forget me nots. All values are multiplied by 1000 to get the answer in mm.	
	$\delta_{q2,pointload}$	= 1000* ((L5_ * (q2_ - q_een) * (0.5 * L5_) * (3 * (L4_ + L5_)^2 - 4 * (0.5 * L5_)^2)) / (48 * EI_castellatedbeam)) =	0.57	mm		
	δ_{M3}	= 1000 * ((M_drie * (L4_ + L5_)^2) / (16 * EI_castellatedbeam)) =	13.61	mm		
	$\delta_{Total\ 3\text{-}4}$	$\delta_{q1} + \delta_{q2,pointload} - \delta_{M3}$ =	7.39	mm	This means the space frame is tilted up between support 3 and 4	
2 to 3	δ_{q3}	= 1000 * (5 / 384) * q3_ * (L5_ + L6_ + L7_)^4 / EI_castellatedbeam =	4.75	mm		
	$\delta_{q2,pointload}$	= 1000 * ((L6_ + L7_) * (q2_ - q3_) * (0.5 * (L6_ + L7_)) * (3 * (L6_ + L7_ + L8_)^2 - 4 * (0.5 * (L6_ + L7_)^2))) / (48 * EI_castellatedbeam) =	6.71	mm		
	$\delta_{Vt,pointload}$	= 1000 * ((L6_) * (V_T) * (L6_) * (3 * (L6_ + L7_ + L8_)^2 - 4 * L6_^2)) / (48 * EI_castellatedbeam) =	36.33	mm		
	δ_{M3}	= 1000 * (M_drie * (L6_ + L7_ + L8_)^2) / (16 * EI_castellatedbeam) =	17.69	mm		
	δ_{M2}	= 1000 * (M_twee * (L6_ + L7_ + L8_)^2) / (16 * EI_castellatedbeam) =	18.19	mm		
	$\delta_{Total\ 2\text{-}3}$	δq3 + δq2,pointload +δV_T,pointload - δM3 - δM2 =	11.91	mm		
1 to 2	δ_{q3}	= 1000 * (5 / 384) * q3_ * (L9_)^4 / EI_castellatedbeam =	60.0	mm		
	δ_{M2}	= 1000 * (M_twee * (L9_)^2) / (16 * EI_castellatedbeam) =	24.4	mm		
	δ_{M1}	= 1000 * (M_een * (L9_)^2) / (16 * EI_castellatedbeam) =	19.9	mm		
	$\delta_{Total\ 1\text{-}2}$	δq3 - δM2 - δM1 =	15.7	mm		
OP to 1	δ_{q3}	= 1000 * (5 / 384) * q3_ * (L10_)^4 / EI_castellatedbeam =	8.4	mm		
	δ_{M1}	= 1000 * (M_een * (L10_)^2) / (16 * EI_castellatedbeam) =	7.5	mm		
	$\delta_{Total\ OP\text{-}1}$	δq3 - δM1 =	1.0	mm		
	Maximal discplacement along beam	δ_{Max} =	15.75	mm	So the greatest displacement is between support 1 and 2	
	Displacement per meter	= Dis_max_castellatedbeam / (L9_ * 1000) =	0.0005	mm/mm	The displacement per meter is very low, so the castellated beam is	
Bending moment		Bending moment				
	Height HEA1000A	h_HEA1000A =	0.99	m		(Chapter 6.1)
	Height castellated beam	= 1.5 * h_HEA1000A =	1.485	m		
	Specific partial factor	γ_M0 =	1			
	Critical moment	= (W_ymin) * f_y / γ_M0 =	4064.33	kNm²	This formula can be used because the spaceframe can be classified in	(6.13)
	Bending moment factor	= M_greatest / Critical_moment =	0.27	< 1	So the castellated beam is resistant to the bending moment	(6.12)

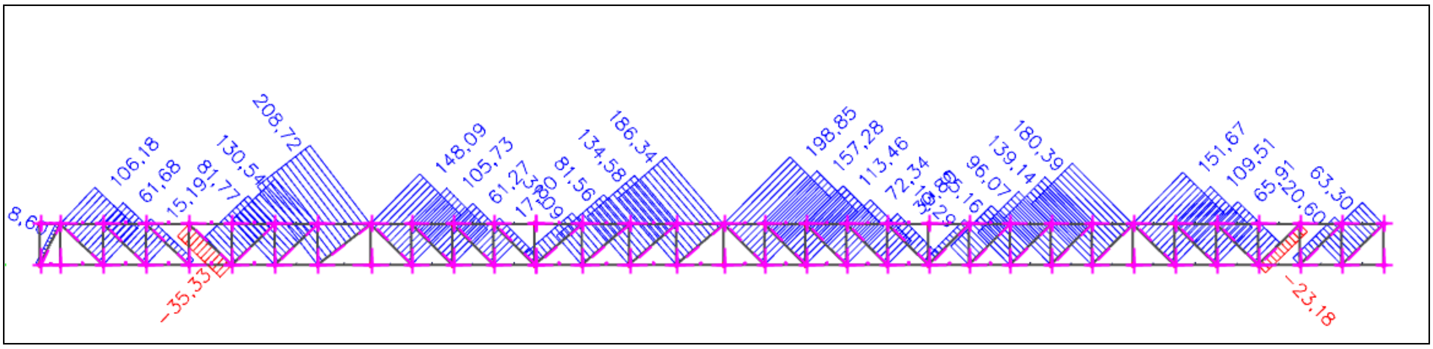
	$M_{y,Ed}$		β	L_{kip}	B^*
0	-1163.5	0-1	0.4	3.4	-0.24
1	-523.5	1 to 2	0.0	4.7	0.01
2	20.5	2 to 3	0.1	4.5	0.17
3	344.5	3 to 4	0.6	3.0	0.26
4	572	4 to 5	0.9	2.3	0.29
5	662	5 to 6	0.9	2.2	0.27
6	614.5	6 to 7	0.7	2.8	0.21
7	430	7 to 8	0.3	4.0	0.06
8	107.5	8 to 9	-0.3	5.4	-0.18
9	-352.5	9 to 10	0.4	3.6	-0.37
10	-949.5				

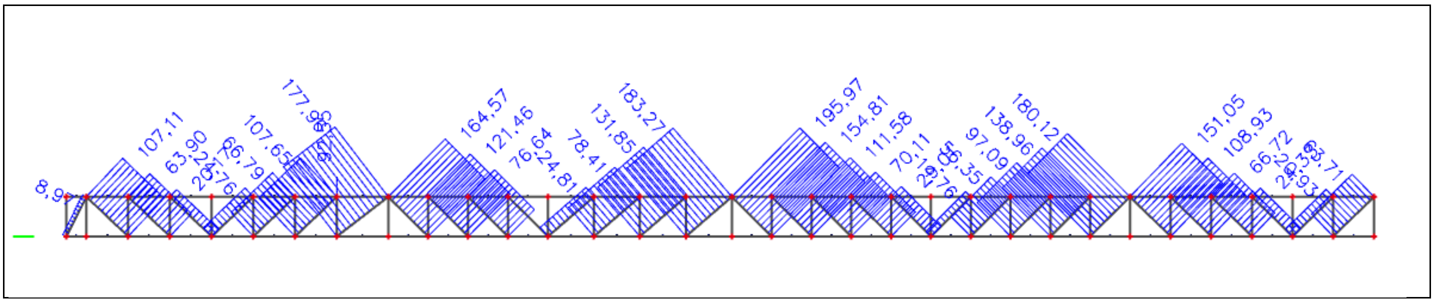


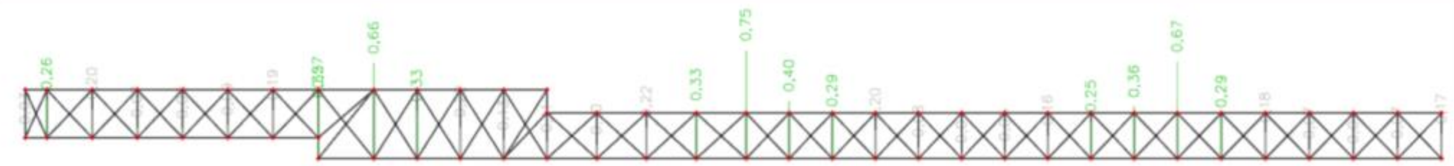
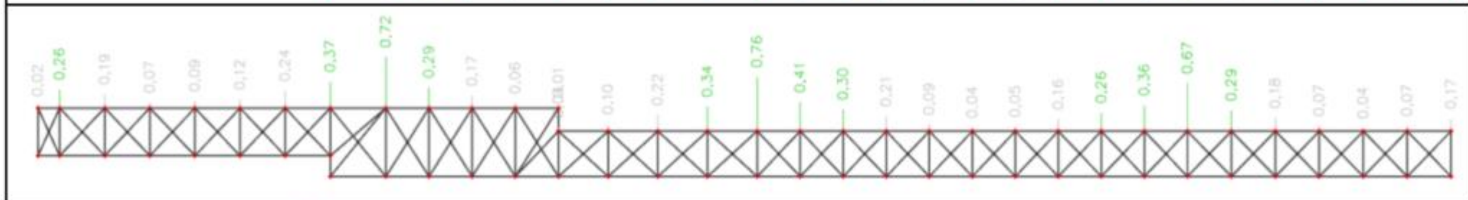
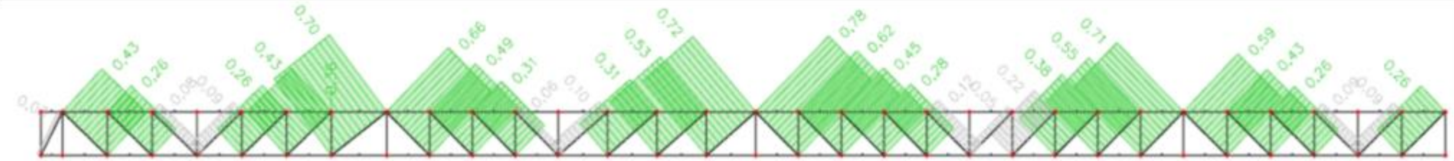
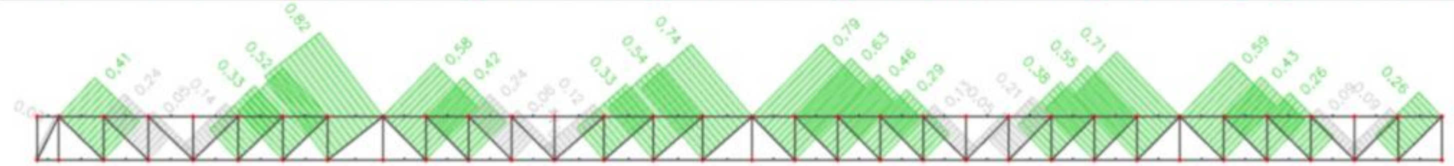
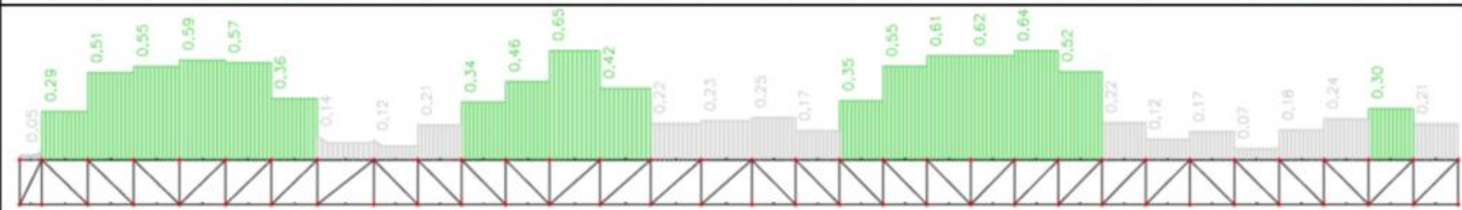
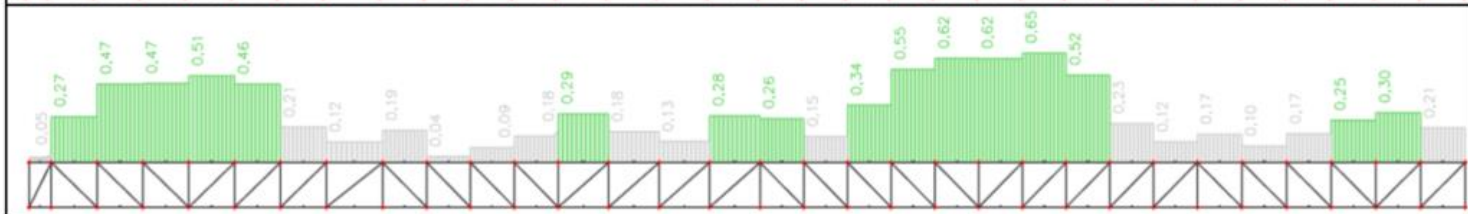
Yield condition T-profile		**Yield condition T-profile**			Module SC3050 of 'HTI staalconstructies dictaat SC3' is used	
	N_{tx}	= M_greatest / (H_castellatedbeam - 2*e_T) =	802	kN		
	σ_{T1}	=N_tx / A_T =	60485	kN/m²		
	M_{Tx}	= V_greatest * S / 12 =	61	kNm²		
	σ_{T2}	= M_tx / W_y_T =	223174	kN/m²		
	σ_{Tx}	= Sigma_T1 + Sigma_T2 =	283660	kN/m²		
	τ_{Tx}	= V_greatest / (2 *(h_t - t_f) * t_w) =	69214	kN/m²		
	Yield condition T-profile	= (Sigma_Tx / f_y)^2 + 3*(Tau_Tx / f_y)^2 =	1.72	< 1	Only directly above support 2 & 3. This is concluded by changing the V_greatest and look if the factor is below 1	
Yield condition web		**Yield condition web**			Module SC3050 of 'HTI staalconstructies dictaat SC3' is used	
	F	= q3_ * S =	17.91	kN		
	A_w	= (4 * h_t)* t_w =	0.0163	m²	h_t is equal to a. The with of a gap is equal to 4a	
	W_w	= (4 * h_t)^2 * t_w / 6 =	0.0027	m³	needed to calculate sigma_w2	
	N_w	= 0.5 * F_web =	8.95	kN	be multiplied by 0.5	
	σ_{w1}	= N_w / A_w =	548.18	kN/m²		
	F_H	=(S*(2*V_greatest-F_web))/(2*(H_castellatedbeam-2*e_T)) =	520.23	kN		
	τ_{wx}	= F_H / A_w =	31847.40	kN/m²		
	M_w	= F_H *(0.5*h_HEA1000A) =	257.51	kNm		
	σ_{w2}	= M_w / W_w =	95542.20	kN/m²		
	σ_w	= Sigma_w1 + Sigma_w2 =	96090.38	kN/m²		
	Yield condition T-profile	= (Sigma_w / f_y)^2 + 3*(Tau_wx / f_y)^2 =	0.22	< 1	This factor is with the greatest shear force still below 1	
		Lateral torsional buckling			Lateral torsion will excist first where the highest moment is, that is the case	
	Distance between rigid supports	L_st =	3.3	m	A plate is needed to rigidly support the castellated beam at the top. These plates are attached by all columns. A cross beam could also have been a	
	Constant 1	C1 =	1.1	-	Optained from Figure NB33/NB34 with bèta is 0.9 and B* is 0.26. This	
	Constant 2	C2 =	0.35	-	values are optained by dividing the length between support 1 and 2 in 10	
	t_w^*	=t_w *((2*h_t) / S)^(1/3) =	0.011	m		(NB.156)
	H/tw*	=H_castellatedbeam / E83 =	129.80	> 75	be used to determine K_{red} depending on alpha	(NB.153)
	α	= (H_castellatedbeam * t_f) / (t_w^3 * b_HEA1000A * L9_^2) =	31.37	< 575	So Figure NB.29 out of NEN 1993-1-1 NB can be used to determine Kred	(NB.155)
	Reduction factor	K_red =	1.00	-		(Figure NB.29)
	Imperfectionfactor lateral torsion	α_{LT} =	0.34		is 0.34, because the profile is rolled and has a height over width ratio higher than 2	(Table 6.3 and 6.4)
	C	=(PI() * C_1 * L9_ / L_LT_middle)*(SQRT(1+(PI()^2 *S^2 *(C_2^2+1)/L_LT_middle^2))+(PI()*C_2 * S / L_LT_middle)) =	170.36			(NB.157)
	Mcr	= K_red *((Coëfficient_C / L9_) * SQRT(E * I_z_T * G_ * I_t_HEA1000A)) =	50979.82			(NB.150 and NB.152)
	λ_{LT}	=SQRT(W_ymin * f_y / M_cr_cb) =	0.28		Wy,min is used because the calculation has to be based on the worst case scenario. Therefore it is also used in the Mb,Rd formula	(6.56)
	ϕ	=0.5*(1+alpha_LT*(labda_LT_cb-0.2)+labda_LT_cb^2) =	0.55			(6.56)
	X_{LT}	= 1 / (phi_LT_cb + SQRT(phi_LT_cb^2 - labda_LT_cb^2)) =	0.97			(6.56)
	$M_{b,Rd}$	=X_LT_cb * W_ymin * f_y / y_M1 =	3944.61			(6.55)
	Existing bending moment	M_{Ed} =	1315			
	Unity check	=M_Ed_cb / M_b_Rd_cb =	0.33	< 1	The HEA1000A castellated beam is not sensitive to lateral torsion with the maximal excisting bending moment	(6.54)

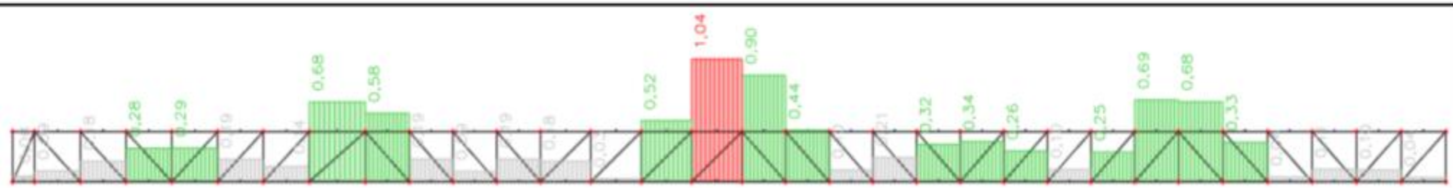
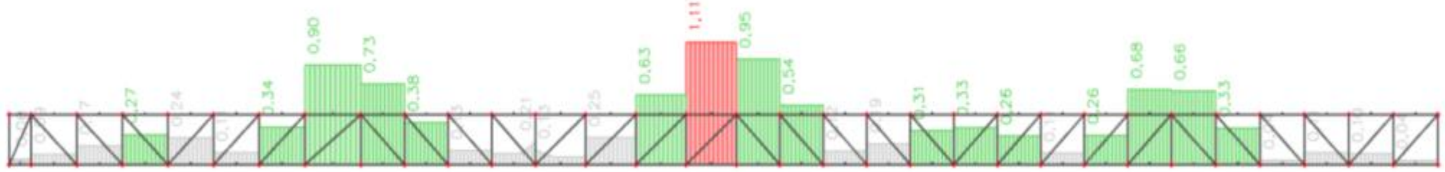
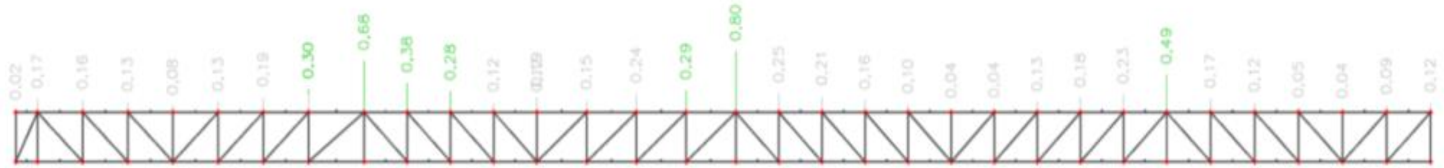
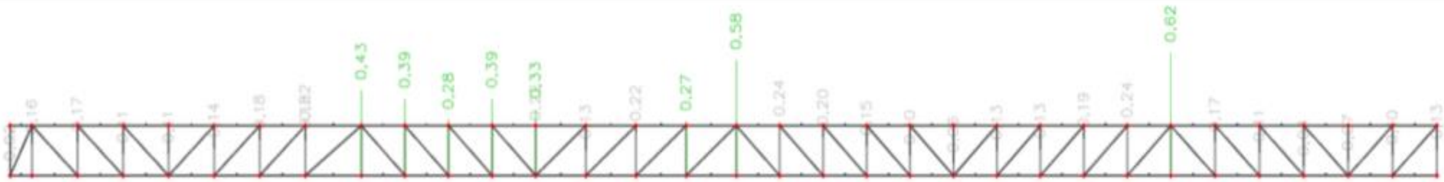
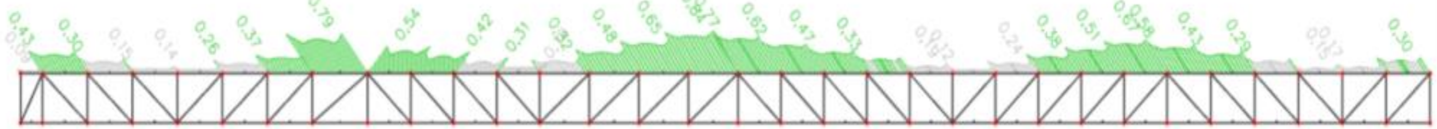
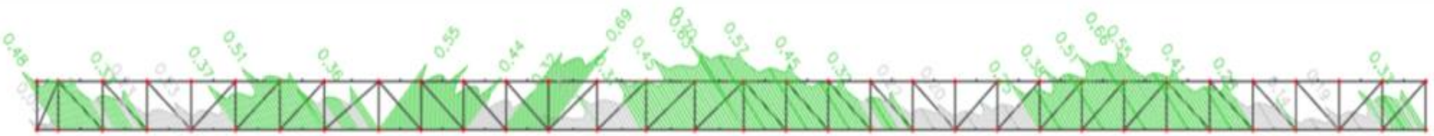
The calculation for the castellated beam on the previous pages is also done for a HEA 500. The conclusion gave a problem at all 5 supports, where the sum (σ_(τ_xy)/f_y) + 3 (τ_x/f_y) greater was than one. This can be solved by welding the holes shut. This method cost a lot more time and is inefficient in comparison with taking a greater profile. For the profile HEA 1000, only at support 2 and 3 holes have to be welded shut, because at those places the moments are the highest.

Appendix O – Scia values





	Profiles	Unity check	Side
Side column	HEA100A		West
			East
Side diagonal	IPE100		West
			East
Upper girder	HEA120A		West
			East

Lower girder	HEA120A		West
			East
Cross bar	HEA100A		Top
			Bottom
Top/bottom diagonal	HFL_{eq} 50x50x6		Top
			Bottom
Tension member	HEA100A	0.53 & 0.20	Top
		0.18 & 0.10	Bottom