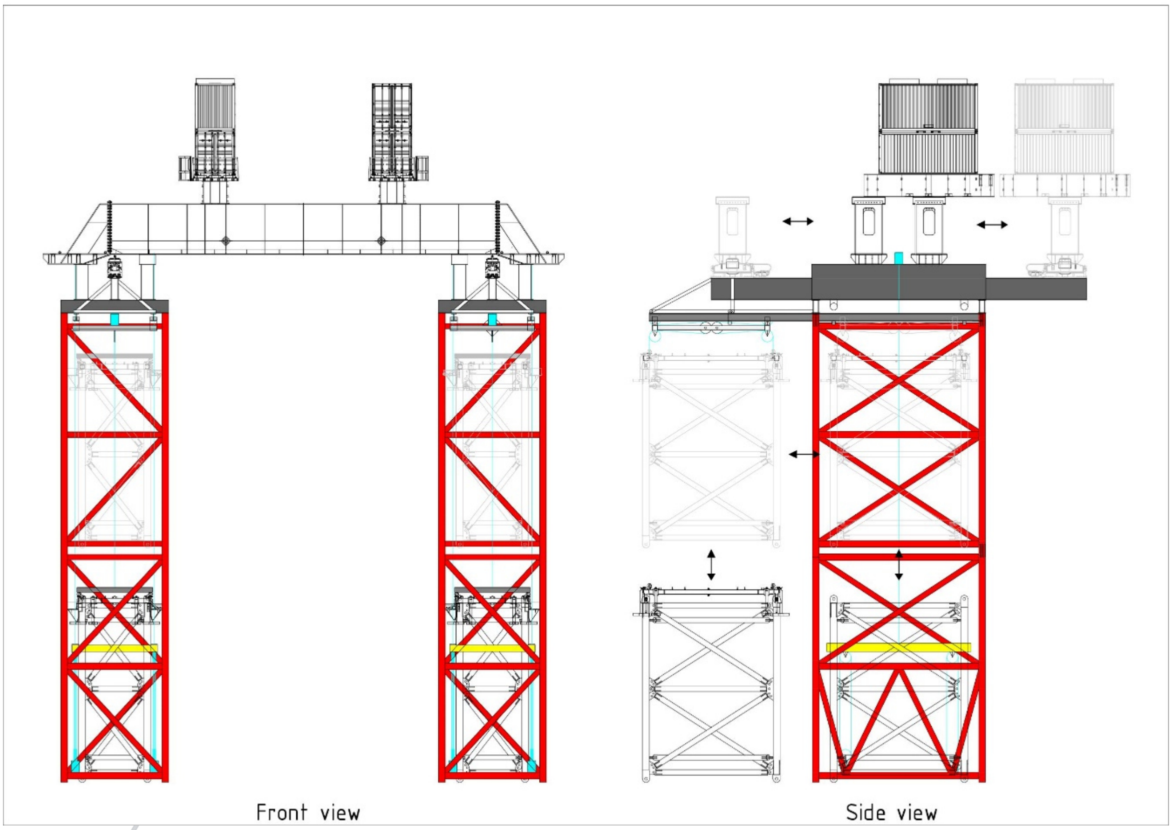
Self-erecting gantry

Msc. Thesis



Rens Warnar BSc

Self-erecting Gantry

A novel design in (dis)assembling a gantry, without the need for a large crane.

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In partial fulfilment of the requirement for the degree of
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Thesis committee

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ABSTRACT

In heavy lifting a paradoxical problem has arisen: A gantry is brought to life to omit the use of a large crane, because of spatial, durational, and monetary reasons. But, in order to assemble and disassemble the gantry, a large crane is needed. This thesis presents a solution to this paradoxical problem. The solution is a novel design that is able to (dis)assemble a gantry without a large crane.

The design is established via a set of requirements based on a case study, crane capacities, and ways of working in Mammoet. Best practice solutions were developed for the five stages from assembly to disassembly of a gantry. A selection of best practice solutions, based on the requirements, was put in a morphological chart. This chart generated seven concepts that were evaluated using a qualitative multi criteria analysis. The top three concepts had similar scores, therefore a second round of assessment was performed to decide on a final concept.

The final concept is a climbing frame, see Figure 1. This frame consists of two climbing cages (1.) that are connected via two gantry beams (2.). Each climbing frame is equipped with a hoisting system (3.) that is used to lift and roll the new MLS mast sections in place. The two towers that form the gantry will be erected simultaneously, with the total upper structure on top. Climbing will happen via the vertical moving frame (4.). This frame is equipped with retractable pins that grab on to jack-up blocks that are present on the mast sections. The vertical moving frame can translate in vertical direction by means of winches that are connected to the climbing frame. It was required that the disassembly procedure happened using the same system as the assembly procedure. Disassembly would be impossible if the upper structure is still over the vessel. Therefore, after the gantry lifted the vessel, the gantry beams must split. To split the gantry beams, two cantilevering guidance beams (5.) are present. The climbing frame is designed such, that when the gantry is in its final position, the case study gantry design is established. The only difference is that the self-weight of the climbing frame is acting on the gantry. Meaning that solely the original top frame beams (6.) and the added self-weight are decisive in the validity of the gantry's main purpose; lifting the vessel. This also means that the climbing frames are solely meant for climbing.

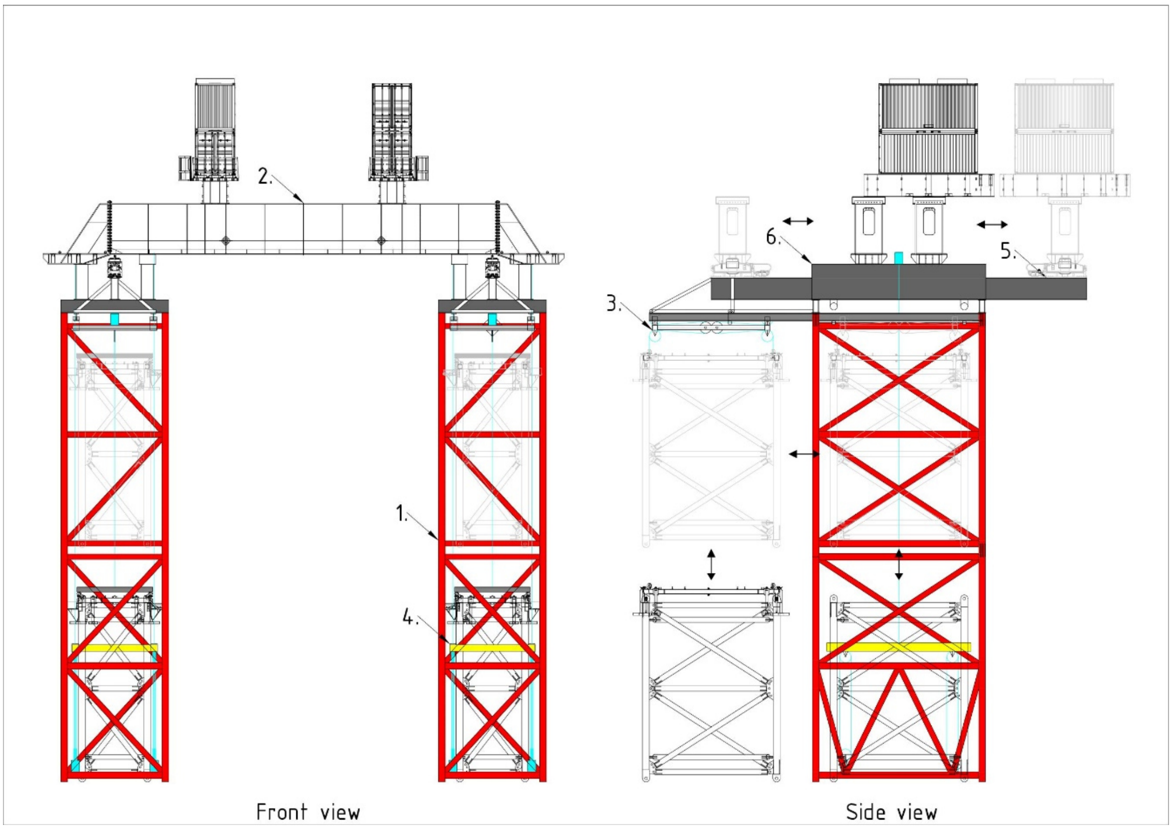

Figure 1 Final design climbing frame

The validation of the final design follows from hand calculations and a finite element model made in SCIA Engineer. Fundamental design features, such as the climbing system, the MLS hoisting system, and skidding of the upper structure are designed and validated too.

Debatable elements of this research are the scope limitations like other duties of the assist crane or being a tailored solution for a Mammoet gantry. However, this design offers freedom in projects and the route to arrive at the design can be applied more broadly. The subjective nature of the qualitative multi criteria analysis, the assumptions made, and the preliminary design stage are debatable too. Therefore, it is recommended to investigate other solutions to the problem. Promising solutions according to this thesis are; a climbing crane, a climbing frame that climbs from the bottom, and a skidding system that translates the whole gantry. Also, all assumptions need to be investigated to be able to fabricate and use this design.

Nevertheless, a proof of concept can be concluded from this research. This self-erecting gantry system is able to (dis)assemble a gantry. Compared to the crawler crane that was needed to perform the critical lift for the case study gantry, this design saves roughly 2100 m^2 of space. That is a reduction of 65%. Approximately €6,000,000.- is needed for the realization of this design. Meaning that during the ninth project the costs for the crawler crane are earned back. It is estimated that the erection of the gantry and the assembly of the climbing frame is comparable to the assembly of the crawler crane and the erection of the gantry. All in all, the design is a future proof solution that can conquer the never-ending need for cheaper and faster heavy lifting projects on dense locations.

ACKNOWLEGEMENTS

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I would like to thank my TU Delft graduation committee members Prof. Dr. M. Veljkovic, Ir. H. El Bamby, and Ir. W. van den Bos. Their expertise, motivation, and flexibility helped me to successfully complete this thesis. Despite being stressful moments, I enjoyed the discussions during the progress meetings.

I would like to thank Mammoet for providing me with a spot at their engineering department in Schiedam. Besides that, they gave me the opportunity to visit two of the largest and strongest land-based cranes in the world, the PTC210 and the SK6000. Seeing these machines in real life was very interesting and special. Special thanks to Ir. I. Harms who was always willing to answer my questions, discuss my ideas and boost my creativity. Also, my appreciation for the interviewees A. Bakker and L. van Oorschot, they provided a lot of useful and interesting information about their expertise. Lastly, I would like to thank my other colleagues at Mammoet. They made for a nice and fun working environment in which I could ask for help when needed but at the same time have a good laugh.

Rens Warnar

Delft, November 2024

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SYMBOLS AND ABBREVATIONS

Table 1 Symbols with their desciption

SYMBOL	UNIT	DESCRIPTION
#	-	Number of
A	mm^2	Area
a	mm	Weld throat thickness
b	mm	Width
$C_{m,i}$	-	The equivalent moment distribution factor around the local y-axis
D	mm	Depth
E	N/mm^2	Elastic modulus
e	mm	Extra length
F_{cr}	kN	Euler buckling force
F_H	kN	Horizontal force
f_u	N/mm2	Ultimate strength
F_V	kN	Vertical force
f_y	N/mm^2	Yield strength
G	kN	Self-weight
g	m/s^2	Gravitational acceleration
h	mm	Height
H	mm	Total height tower
I	mm^4	Moment of inertia
k	-	Interaction factor
L	mm	Length
l	mm	Physical weld length
L_{buck}	mm	Buckling length
l_{eff}	mm	Effective weld length
M_{Ed}	kNm	Acting design bending moment
M_{Ek}	kNm	Acting characteristic bending moment
M_{Rd}	kNm	Design bending moment resistance
M_{Rk}	kNm	Characteristic bending moment resistance
n	-	Minimum force amplifier to reach elastic critical buckling
n		Number of welds

N_{cr}	kN	Critical buckling force
N_{Ek}	kN	Acting characteristic compression
N_{Rd}	kN	Compressive resistance
N_{Rd}	kN	Design compressive resistance
N_{Rk}	kN	Characteristic compressive resistance
q	kN/m	Distributed load
Q	kN/m^2	Distributed areal load
Q_{cr}	kN	Critical buckling load along element
R	mm	Radius
R_i	kN	Reaction force
t	mm	Thickness
V_{Ed}	kN	Acting design shear force
V_{Rd}	kN	Design shear force resistance
W	mm^3	Section modulus
w	mm	Deflection
z	mm	Distance C.o.G. to datum
α	-	Imperfection factor
α_{cr}	-	Critical buckling factor
β	-	Correlation factor
γ	-	Safety factor
δ	mm	First order deflection
ζ	-	Ratio between the buckling load along the tower (Q_{cr}) and the buckling load at the top (F_{cr})
η	mm	Imperfections
λ	-	Relative slenderness
μ	-	Friction coefficient
σ	N/mm2	Stress
ΣT|X	kNm	Sum of moments around X
σ_{cr}	N/mm^2	Euler buckling stress
τ	N/mm2	Shear stress
ϕ	mm	Diameter

ϕ	-	Factor depending on imperfection and slenderness
χ	-	Buckling coefficient
ψ	-	Ratio between the bending moment at the bottom and the top of the tower

Table 2 Abbrevations with their description

ABBREVATION	DESCIPTION
CFB	Climbing frame bottom
CFT	Climbing frame top
c.t.c.	Center to center
C.o.G.	Center of gravity
FEM	Finite Element Model
MCA	Multi Criteria Analysis
MLS	Mammoet Lattice Sections
MSG	Mammoet Sliding Gantry
SL	Super lift
SLS	Serviceability Limit State
SPMT	Self-propelled modular transporter
U.C.	Unity Check
ULS	Ultimate Limit State

GENERAL TERMINOLOGY

For clarity, throughout the report the global X-direction is horizontal and parallel to the gantry beam. The global Y-direction is also horizontal, but perpendicular to the gantry beam. The global Z-direction is in the vertical direction. See Figure 2 for a visualization. The local x-, y-, and z-directions are respectively the beam axis, the strong axis, and the weak axis. Global directions will be addressed with uppercase letters (X, Y, and Z) and local directions in lowercase (x, y, and z).

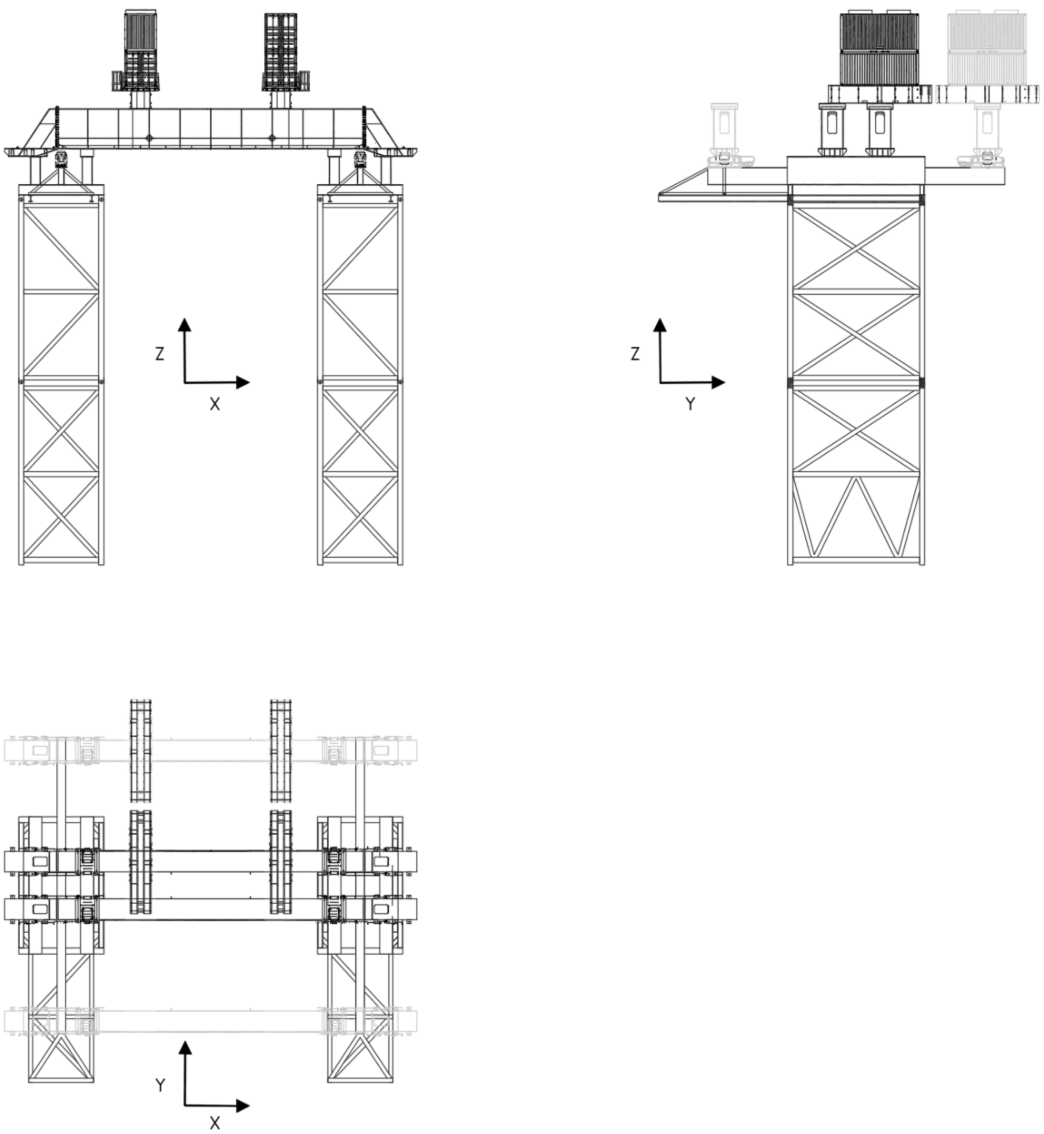


Figure 2 Global cooridinate system

A dot (.) will be used as decimal sign. A comma (,) will be used to separate thousands.

The gravitational acceleration 'g' will be 9.81 m/s^2.

The to-be-lifted load will be referred to as vessel.

All loads will be given in kilo Newtons (kN), all weights in tons (Te).

1 INTRODUCTION

1.1 PROBLEM STATEMENT

Cranes are needed to get heavy goods, like coke drums and vessels, at a certain height. Mammoet is a company that has expertise in heavy lifting. With cranes like the PTC and SK Mammoet is able to lift thousands of tons to their desired position [1]. However, with increasing demands at dense locations and the never-ending need for faster and less expensive operations, these cranes do not always suffice [2].

The PTC and SK cranes require an operational area with radii up to almost 56 meters [3]. Moreover, during assembly and disassembly of the crane the total main boom plus jib is lain down, this can be 246 meter long. This is a huge area and thus not always available. Additionally, it can take up to eight weeks to mobilize and demobilize these cranes. In order to still operate when less space and time is available, a gantry, see Figure 1-1 a), is brought to life. A gantry is a frame- or portal-like structure that can, in principle, perform a vertical lifting movement. Because of this limitation the structure has a smaller footprint and is faster to mobilize and demobilize. Making that a gantry is less expensive compared to the cranes mentioned before [4].

a) Mammoet gantry in Jubail Saudi Arabia [8]

b) Demag/Terex CC6800 Jubail [45]

Figure 1-1 Jubail gantry

The gantry itself needs to be erected on site. This is done with the help of an assist crane. Here the problem arises. Due to the increased weight and size of the to-be lifted goods, the gantry and the main lifting beam are getting too tall and heavy for a small crane to suffice. Especially because of the critical lift; the heaviest item, the gantry beam, needs to be lifted to the highest point [5]. Currently, the critical lift is performed by a crawler crane that has similar demands as the cranes mentioned before, see Figure 1-1 b). This creates the paradox of using a gantry to omit the use of a large crane because of spatial, durational, and monetary reasons. But, in order to assemble and disassemble the gantry, a large crane is needed.

1.2 RESEARCH OBJECTIVES AND RESEARCH QUESTIONS

The goal of this thesis is to break this paradoxical problem. Meaning that a new design for the (dis)assembly procedure without a large crane for a gantry is needed. This will be done by:

- Identifying boundary conditions that apply for the new design.
  The new design must satisfy certain requirements to comply with:
  - A strong, stable, and stiff gantry
  - The capabilities of small cranes
  - The demands from Mammoet
- Defining criteria and their importance to compare concepts for the new design.
  To compare the different concepts, criteria are needed for an evaluation. The criteria might not be equally important, therefore a weight factor must be established to account for this. This is done via a qualitative multi criteria analysis.
- Generating and evaluating concepts that can (dis)assemble a gantry.
  By splitting the problem, tailored best practice solutions can be developed. Combining these partial solutions, through a morphological chart, leads to concepts that can solve the paradoxical problem. With the qualitative multi criteria analysis the different concepts are evaluated. If concepts score equally good, a second round of assessment will determine the best concept.
- Verifying the best concept.
  The best concept needs to be verified on strength, stiffness, and stability.

The main question for this investigation is:

Which novel design is capable of (dis)assembling a gantry without the need for a large crane?

To answer the main question, it is divided into the following sub questions:

- What requirements apply to a gantry concept that can (dis)assemble without a large crane?
  - What boundary conditions follow from the case study?
  - What are the capacities of the cranes Mammoet uses?
  - What are the requirements from Mammoet?
- What criteria can be used to compare concepts with each other?
- What concepts of a gantry can (dis)assemble without a large crane?
- What is the structural performance of the new design that can (dis)assemble a gantry without a large crane?

1.3 RESEARCH METHODOLOGY

This mixed-methods research consists of several phases. First, all necessary information and boundaries need to be established. The next phase is about generating and evaluating concepts. In the third phase the best concept will be designed and verified.

Phase 1:

- A case study will be carried out on the largest self-standing gantry Mammoet has currently built. This is the gantry in Jubail, Saudi Arabia. This case study will gain necessary insight into the workings of a gantry, its components and the (dis)assembly procedure.
- The capabilities of the cranes within Mammoet will be investigated. Here the differences between a small and a large crane will be defined along with their capacities and demands. This will be done via a literature study, an interview with a crane engineer in Mammoet, and with the help of the program AutoCRANE.

- The standards within Mammoet will be obtained so that their demands and wishes are considered. This will be done by an interview with a technical engineering advisor in Mammoet.
- The requirements that follow from the crane capacities, the demands from Mammoet, and the case study will function as boundary conditions. The new concepts must comply with these.

Phase 2:

- Criteria for a qualitative multi criteria analysis are developed and evaluated. The difference between the requirements and the criteria is that requirements are strict guidelines for the new design, while the criteria function as a basis to compare different concepts. A weight factor will reflect the importance of each criterion. Although performing a qualitative multi criteria analysis is inherently subjective, through reasoning and a second opinion from a structural engineer within Mammoet this is done as objectively as possible. Moreover, the weight factors will be determined by the author along with several other engineers within Mammoet.
- The stages in the (dis)assembly procedure of a gantry will be unraveled. For each stage different best practice solutions will be developed. These best practice solutions come from current alternative (dis)assembly procedures, a structured motion analysis, and creativity. All original best practices will be drawn with Microsoft Visio.
- A first shift, based on the boundary conditions, will determine which best practice solutions will be considered. The selection of best practice solutions will be put in a morphological chart.
- Combining the different best practice solutions in the morphological chart will result in concepts. By means of the qualitative multi criteria analysis the concepts will be scored and evaluated. The three best concepts will follow.
- Via a second round of assessment, it will be determined which of the three best concepts will be designed. Here, hand calculations and procedures will be studied. Drawings that are needed will be made with AutoCAD.

Phase 3:

- A preliminary design will be made for the best concept. The whole gantry, fundamental components of the new design, and the connections will be verified. This will be done with a FEM analysis in SCIA Engineer and hand calculations. The validity will follow from them.

1.4 SCOPE

This thesis limits its scope through:

- Focusing on portal-like, self-standing gantries.
- The design will be based on the Jubail gantry in Saudi Arabia. That project will be used as a case study because it is the highest self-standing gantry Mammoet currently has used.
- Often the assist crane has other purposes on the building site. These other purposes will be excluded from this research. This makes the research solely into assembling a gantry without the need for a large crane, resulting in more freedom for performing the vessel-lifting job.
- The availability of cranes, concerning the location and the planning, is not considered in this thesis.

1.5 THESIS OUTLINE

Chapter 1 introduces the problem, defines the objectives, methodology, and limits the scope of the research.

Chapter 2 shows the state of art that is relevant for the thesis. After a brief introduction of a gantry, the case study of the Jubail gantry is presented. Followed by the crane capacities and the ways of working of Mammoet. These three form the basis for the list of requirements. The way to compare concepts, via a qualitative multi criteria analysis, is explained. Lastly, the best practice solutions will be presented.

Chapter 3 will, via the morphological chart, present the concepts. They will be scored through the qualitative multi criteria analysis. The conclusion contains the three best concepts.

Chapter 4 dives deeper into the three best concepts, resulting in a second round of assessment which will determine the best concept.

Chapter 5 presents the final design along with the validation. Fundamental details such as climbing, the hoisting system and the connections will be elaborated.

Chapter 6 concludes if the goal of (dis)assembling a gantry without a large crane is achieved.

Chapter 7 discusses the final design and the design route to it.

Chapter 8 will give recommendations for further research into this design and some concepts that were not designed.

2 STATE OF ART

First a basic understanding of the gantry itself is needed. The structure and its components will be touched upon. After that, a case study of the largest, self-standing gantry is carried out. Next, it will be determined what is meant by a small or large crane. Also, the capacities and demands of the intended crane that will be used for the (dis)assembly procedure are given. The Mammoet ways of working are elaborated. Together, so the case study, the crane capacities, and Mammoet, will form a set of requirements. Next to the requirements, criteria are developed to compare concepts through a qualitative multi criteria analysis. A morphological chart, containing a selection of best practice solutions, finalizes the state of art.

2.1 GANTRY

This Chapter will outline the principles of a gantry.

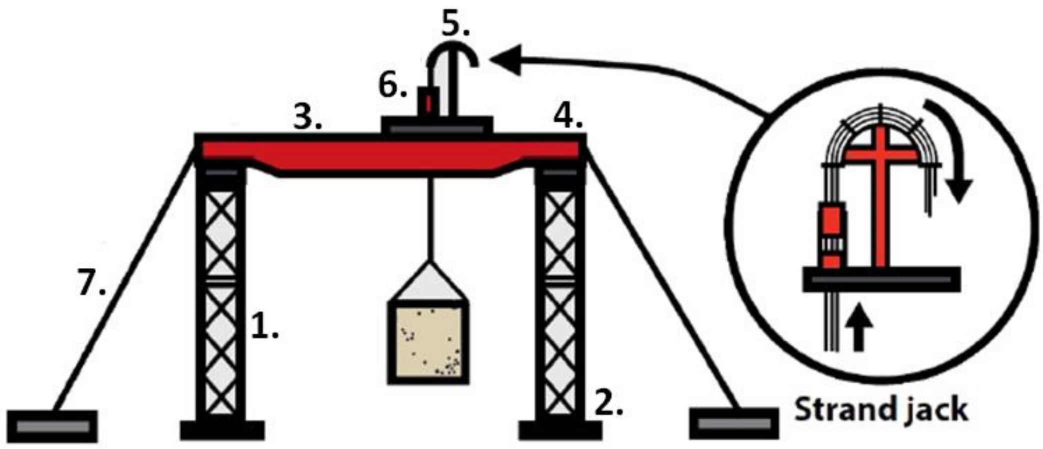


Figure 2-1 Gantry parts [26]

A gantry is schematized in Figure 2-1. Where the different parts correspond with the following:

1. Mast section
2. Base frame or moveable device
3. Gantry beam
4. Top frame
5. Lifting device
6. Skidding system
7. Guy line

The mast sections form the towers of the gantry. The towers are connected to the foundation via base frames, or they are placed on a moveable device. On top of the towers is a top frame, connecting the gantry beam to the towers. The gantry beam is the main lifting beam as the lifting devices are placed on this beam. Usually, the lifting devices are strand jacks. To enable horizontal movements the strand jacks could be placed on a skidding system. If needed the gantry can be stabilized by means of guy lines. A more in-depth description of these parts can be found in Annex A.1.

Gantries exist in numerous configurations. This is due to their modular design. Each part, see Figure 2-1 consists of standardized elements that can be put together. By doing so, a gantry is a tailored solution that is 'the most optimal and cost-effective for each job' [2]. Moreover, the structure is reusable and replaceable, to such an extent that e.g., mast sections can be used in a totally different crane. This saves fabrication time, costs, and environmental impact, as limited number of new elements are needed. Another smart design choice is that every element is containerized, which ensures easy transportation [6].

To distinguish the different configurations all types will be discussed. First, there are frame- and portal like gantries, see Figure 2-2. Where the first are considered as a three-dimensional structure and the second as two-dimensional.

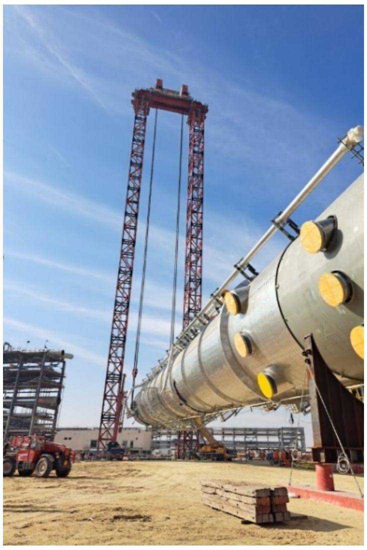
a) Frame gantry [7]

b) Portal gantry) [8]

Figure 2-2 Frame (a) and portal (b) gantry

Furthermore, gantries can be divided into five categories; self-standing, guyed, mobile, hydraulic and hammerhead, see Figure 2-3.

As its name implies, a self-standing gantry can stand on its own. It is inherently stable to stand and operate. To increase the stability, guy lines can be used. These lines are attached to the gantry, either on top or somewhere along the tower. The other ends of the lines are connected to counterweights or foundations on the ground. This has the disadvantage of requiring more space. Mobile gantries are, besides lifting, also able to move to the desired location. In principle this is a gantry placed on a moveable device, such as a skidding system or an SPMT (self-propelled modular transporter). Hydraulic or 4-point lift systems fall in the penultimate category gantries. Instead of a lifting device placed on the main lifting beam these cranes lift an object by extending their legs with hydraulic pressure [6]. The hammerhead gantry is different from the ones described above. Instead of a gantry beam over two (or more) towers, each tower is a separate gantry. It lifts the object through the side. The stability is ensured by means of a back tie, that is connected to the foundation. The advantage is that the gantry does not have to stand over the object, so taller objects can be lifted with smaller gantries. The downside is that double the load is introduced in the tower [4].



a) Self-standing gantry [45]

b) Guyed gantry [44]

c) Mobile gantry [47]

d) Hydraulic gantry [46]

e) Hammerhead gantry [49]

Figure 2-3 Gantry types

2.1.1 (Dis)assembly

First, all parts of the gantry need to be assembled. Some parts are pre-assembled before transportation, this saves assembly time on site. However, other parts are too large to be transported to the site. They need to be assembled close to the final location of the gantry.

The foundation/ base frame of the gantry needs to be leveled. This must be done precisely and in a controlled manner. A small deviation can lead to big second order effects in the gantry. Now, a crane is needed to stack the mast sections on top of the base frames. When the desired height is reached the top frames are placed on the mast sections. The gantry beam is lifted and placed on the top frames. Lastly, the lifting devices are lifted and put on top of the gantry beams.

2.2 CASE STUDY – JUBAIL GANTRY SAUDI ARABIA

To get a better understanding of what is currently possible, a case study is performed. The case study concerns the largest, self-standing gantry built by Mammoet. Late December 2022 Mammoet lifted a vessel in Jubail, Saudi Arabia. To perform this lift, a portal-like, self-standing gantry was used. The characteristics of this gantry are displayed in Table 2-1. A visual representation can be seen in Figure 1-1.

Table 2-1 Jubail gantry characteristics [9]

CHARACTERISTIC		UNIT
Load	1611	Te
Load incl. rigging and safety margin	1852	Te
Height of load	128.7	Meters
Width of load	10	Meters
Mast type	MLS 4000 x 8000 mm	-
# mast sections	12	Per tower
Gantry beam	PDV T38 PDV T44	-
# gantry beams	2	-
Lifting device	900 Te Strand jack	-
# lifting devices	4	-
Weight gantry	1490	Te
Total gantry height	145.0	Meters
Total gantry width c.t.c.	23.1	Meters
Skidding system	No	-
Mobile	No	-
Guy lines	No	-

2.2.1 Mast sections

The towers of the gantry are built from MLS mast sections. These are four meters wide, eight meters deep and eleven meters high. The total weight of one section is a little less than 40 tons. Figure 2-4 shows the MLS mast on its side. The capacities of all members are listed in Annex A.2.1.

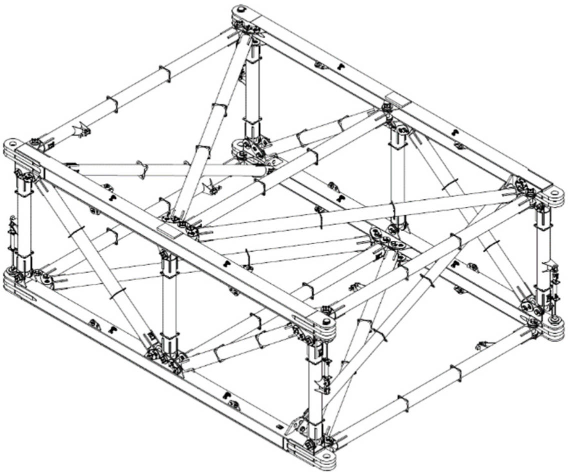


Figure 2-4 MLS mast section 4x8x11 [10]

Although never used for this purpose, Mammoet designed the MLS mast sections for self-erection [11]. The chords are equipped with a guidance edge and jack-up blocks, see Figure 2-5. These might come in handy for a final concept.

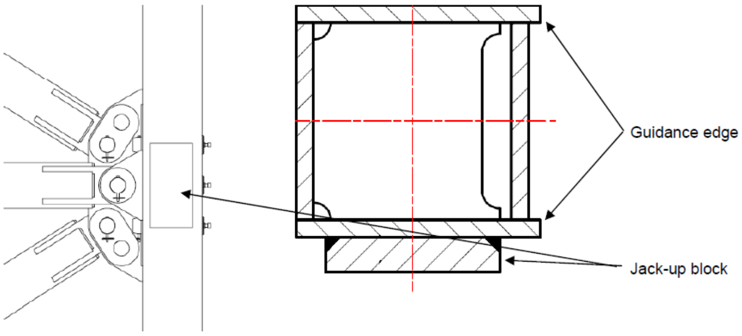


Figure 2-5 Provisions for self-erecting system [11]

2.2.2 Tower

Twelve MLS mast sections are used to form one tower of the gantry. This makes the total length of one tower 132 meters. The weight of one tower is roughly 470 tons. The towers are clamped to the foundation. The properties can be seen in Annex A.2.3.1. The resistances of the towers to the loads that might be applied to them are displayed in Annex A.2.3.8. These resistances are obtained by hand calculations done in Annex A.2.3.2 to A.2.3.7.

2.2.3 Upper structure

Figure 2-6 shows the upper structure of the gantry in detail. It shows all additional connections and protection that are important for a gantry. All items with their weights are listed in Annex A.2.2. The total weight of the upper structure is roughly 460 tons, see Annex A.2.2.

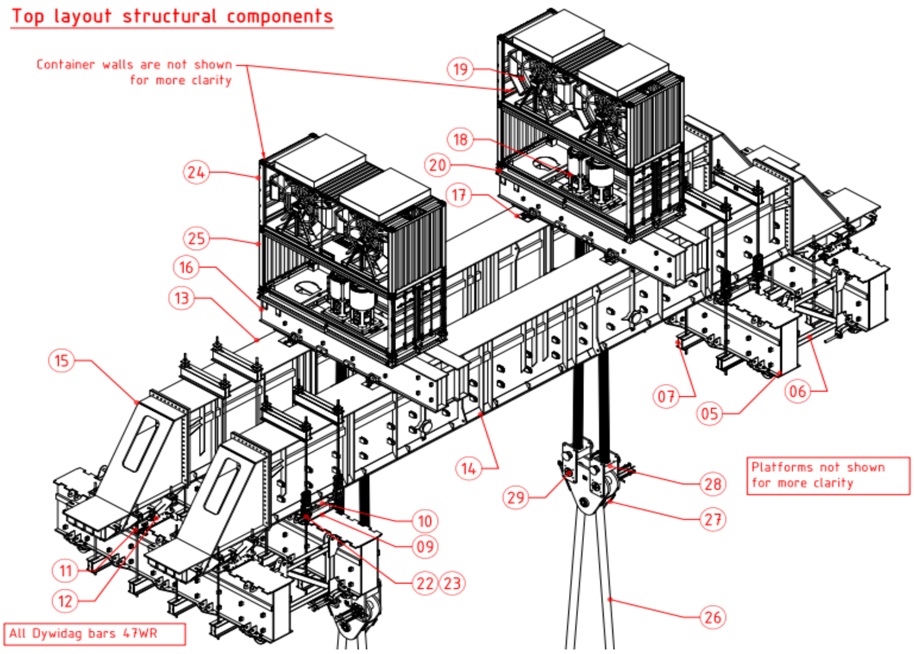


Figure 2-6 Upper structure Jubail gantry [51]

2.2.4 Gantry beam

From Figure 2-6 it can be seen that two gantry beams are used. These are the PDV T38 and the PDV T44, both with an extension. This part of the upper structure was the heaviest to lift, namely 100 tons per gantry beam, excluding rigging. The gantry beams are clamped to the top frames. The PDV T38 and PDV T44 differ in flange thickness, they are respectively 38 mm and 44 mm thick. The weakest beam, PDV T38, will be used as benchmark. The resistances of the PDV T38 gantry beam to the loads that might be applied to them are displayed in Annex A.2.4.

If the gantry beam is shorter than the desired width of the towers, it can be extended. The Jubail gantry used small PDV extensions. The gantry beam can also be extended by adding other parts. In total this can make the beam 47 meters long. This will function as a limit for the gantry beam length and thus the total gantry width.

Splitting one gantry beam is not possible, because that would locally decrease the strength and stiffness of the beam. The structural scheme of the gantry changes from project to project. Therefore, it might occur that the weakest point is at the location with the highest forces.

2.2.5 Loads

The loads on the gantry are grouped in load cases. The cases are; self-weight, secondary weight, lifting load, transverse load, longitudinal load, wind load, and global imperfections. These load cases form load combinations that the gantry must withstand. See Annex A.2.5 for a detailed overview.

2.2.6 Foundation

From the towers the loads are introduced in the base frames. Each tower has four foundation points, see Figure 2-7. Each point has a force in X, Y, and Z-direction.

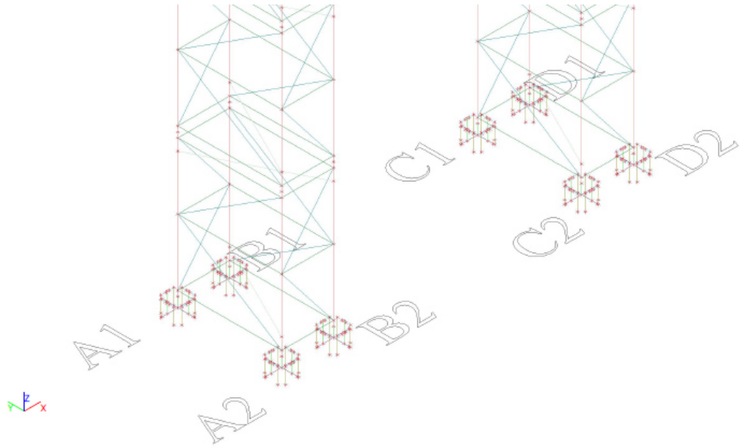


Figure 2-7 SCIA model gantry foundation [9]

The loads from the towers are transferred to the foundation via the foundation points (A1, A2, etc..). Annex A.2.6 shows that the extreme operational and survival loads that act on the foundation points are approximately 11,000 kN in compression, 7000 kN in tension, and 250 kN in shear. The loads are passed on to the client. They need to make sure their foundation is strong enough for these loads.

2.2.7 Assembly and disassembly

To assemble the gantry all elements were stacked upon each other using a crawler crane. First, smaller cranes were used. But the Demag/Terex CC6800, a large crawler crane, was used for the critical lift, see Figure 2-8. The crane was 172 meters high. It had a radius of 31.5 meters for the load and a 15-meter radius for the counterweight. This made for an operational area of 1430 m^2 and 1740 m^2 (dis)assembly area for the crane only, see Annex A.2.7 for the calculation. Moreover, a total of 490 tons counterweight was needed [12]. The assembly took around 30 days, disassembly around 27 [13].

Figure 2-8 Demag/Terex CC6800 Jubail [45]

2.2.8 Costs

The operational costs for the CC6800 crawler crane were [redacted] per month, using the 2021 currencies, this is [redacted]. Moreover, it needs to be mobilized and demobilized, costing [redacted] Saudi Arabian Riyals (SAR) ([redacted]). Also, a crawler crane operator is needed, this costed [redacted] per month [13]. In total the crane was rented for [redacted] months of which [redacted] months were used for (dis)assembly. Meaning that the total costs for this crane for this project were approximately €700,000.

2.3 CRANES

As mentioned, the goal is to (dis)assemble a gantry without a large crane. Detrimental is the definition of a large crane. Currently, mobile telescopic boom cranes, crawler cranes and mobile lattice boom cranes are being used for the (dis)assembly. Of the three the mobile telescopic cranes are the fastest to build and easiest to transport. Moreover, without a jib it is independent of other cranes. This all makes it less expensive to use [14]. They are considered small cranes and the other cranes, crawler cranes and mobile lattice boom cranes, are considered large cranes.

Mobile telescopic boom cranes are also known as hydraulic cranes, see Figure 2-9. This is because their main boom consists of tubes that can be extended/ retracted by means of a hydraulic mechanism. The slewing unit is placed on a truck, which makes it mobile. When on site, the crane extends its outriggers to ensure stability. These are placed on load spreaders [15]. To increase the lifting height a variety of jibs can be attached to the main boom. And to increase the capacity counterweights and a Y-guy can be used.

Figure 2-9 Mobile crane [54]

Without a jib, the crane is able to set up in a day without the need for an additional crane. Because of its mobility it can, independently of transportation trucks and permits, drive to the building site. With a jib it needs an extra crane to lay out the jib and it needs an extra truck to transport the jib to site [14]. It is not possible to move the crane when performing a lift or when the jib is connected because of the outriggers.

2.3.1 Capacities and demands

This section determines the capacities and demands of mobile telescopic cranes. Capacity in this sense is meant as the ability to lift a certain load at a certain height. The demands are what is needed to perform the lift, e.g. space, counterweight, etc. This is investigated with AutoCRANE. This is a tool, developed by Mammoet, that gives crane configurations that are the most suitable for the input parameters it gets.

The whole investigation can be found in Annex A.3. The data shows that hydraulic cranes can be used up to 90 meters, this is in line with the 100 meters indicated by crane engineer Arjon Bakker [14]. Table 2-2 shows the approximate weight that can be lifted to a certain height. Most configurations show that hydraulic cranes can lift to 30 meters without a jib. All this makes that a hydraulic crane will be the crane to use for the (dis)assembly of the gantry. Preferably without a jib so only one crane is needed.

Table 2-2 Capacities mobile telescopic cranes

WEIGHT [TE]	HEIGHT [M]
50	90
90	60
130	30
210 (max. weight)	30

The biggest and strongest mobile telescopic crane in the world, an LTM11200-9.1, is able to lift heavier and higher [16]. However, this crane is rare within Mammoet [14], so it cannot be considered a good alternative for crawler cranes.

2.4 MAMMOET

As mentioned before, Mammoet offers a tailored solution that is 'the most optimal and cost-effective for each job' [2]. Meaning that all elements they use for their gantries, cranes, and other structures must be multi-purpose. I.e. they will be used in taller, wider, or deeper configurations. Consequently, no plastic deformation is allowed [17]. Plastic deformation is permanent, irreversible deformation, meaning that connections and dimensions will differ. Thus, elements might not fit for future projects and new calculations should be made. Moreover, unlike for buildings or other 'regular' structures the ultimate limit state scenario will occur for a gantry: the to-be-lifted load must be lifted [17].

Since all elements must be multi-purpose, curved elements should be omitted too. The radius of a curved beam cannot change, resulting in an element that can only be used in one configuration. Moreover, the elements should be containerized or should fit in a container. Due to the multi-purposeness all equipment travels all over the world. To limit transportation costs Mammoet designs their equipment to function like a container [17]. This has a consequence that welded connections only apply for elements that are (within) the size of a container [17]. Other connections must be bolted or pinned. From these, bolted connections are not preferred either because when used properly, the bolts need to be pre tensioned. Pre tensioning causes plastic deformation in the bolts. So, the only time Mammoet wants to use bolted connections is for elements that stay connected for over a year. This is (most likely) not the case for a gantry. Therefore, pinned connections are to be used for elements, when assembled, that are bigger than a container [17].

The vessels will mainly be installed on refineries. Therefore, it must be considered that the space around the vessel is limited. It is certain that from one side lots of space is available as the vessel will be delivered horizontally. A good gantry design should be able to operate from one side only [17]. This means that the vessel demands space on both lateral sides of the gantry.

The most efficient design has the least number of systems that do the most amount of work. Therefore, assembly and disassembly should be done with the same system. It serves no purpose to design a separate system.

Also, it would be beneficial if the new design makes use of already-known elements/ equipment.

2.5 REQUIREMENTS

In order to select the most promising best practice solutions, certain requirements are needed. The requirements follow from the case study, crane capacities, and Mammoet.

2.5.1 Case study

Following from the case study are the requirements in Table 2-3.

Table 2-3 Requirements case study

#	REQUIREMENT	##	SUB REQUIREMENT
R1	The design load must be the vessel in Saudi Arabia	R1.1	The weight is 1611 Te.
		R1.2	The weight including safety margin and rigging is 1852 Te.
		R1.3	The height is 128.7 meter.
		R1.4	The width is 10 meters.
		R1.5	The C.o.G. is 56.5 meter high, from the bottom.
R2	Gantry must be self-standing		
R3	Gantry must use MLS 4000 x 8000 masts	R3.1	Crane must lift MLS mast including rigging and hook block of 50 Te.
		R3.2	Vertical forces on one tower may not exceed 16,000 kN.
		R3.3	Bending moments around y-axis on one tower may not exceed 380,880 kNm.
		R3.4	Bending moments around z-axis on one tower may not exceed 190,440 kNm.
		R3.5	The normal force and bending moment follow eq (23) and eq (24) in Annex A.2.3.7.
R4	The PDV T38 and the PDV T44 must be the gantry beams	R4.1	The bending moment around the y-axis in one gantry beam may not exceed 75,000 kNm.
		R4.2	Shear force in z-direction in one gantry beam may not exceed 20,000 kN.
		R4.3	Gantry beam may not be longer than 47 meters.
		R4.4	Gantry beam may not be split.
R5	Foundation points loads (forces)	R5.1	Max force Z-direction 11,000 kN.
		R5.2	Min force Z-direction -7000 kN.
		R5.3	Max shear force 250 kN.
R6	Foundation loads (bending moments)	R6.1	Max absolute bending moment around X-axis 61,000 kNm.
		R6.2	Max absolute bending moment around Y-axis 70,000 kNm.
		R6.3	Max absolute bending moment around Z-axis 200 kNm.

R7	The gantry should have the same dimensions as the Jubail gantry	R7.1	The height of the gantry is 145 meters.
		R7.2	The c.t.c. width of the gantry is 23.1 meters.
R8	The area of the new design must stay under 1430 m^2		

2.5.2 Crane capacities

The requirements regarding the assist crane are listed in Table 2-4.

Table 2-4 Requirements assist crane

#	REQUIREMENT	##	SUB REQUIREMENT
R9	Only a hydraulic crane must be used for the (dis)assembly.	R9.1	50-ton load is limited to a lift height of 90 meter.
		R9.2	90-ton load is limited to a lift height of 60 meter.
		R9.3	130-ton load is limited to a lift height of 30 meter.
		R9.4	Elements, including rigging and hook block, may not weigh more than 210 Te.
R10	The crane must not move while lifting.	R10.1	Pick-up and drop-off must be within the crane radius.
R11	The hydraulic crane should not use a jib.	R11.1	The lift height is limited to 30 meters.

2.5.3 Mammoet

Table 2-5 sums the requirements coming from Mammoet. It would be too extensive to describe all equipment Mammoet has, so requirement R15 will come into play during the evaluation of the concepts.

Table 2-5 Requirements Mammoet

#	REQUIREMENT	##	SUB REQUIREMENT
R12	The gantry elements must be multi-purpose.	R12.1	No plastic deformation.
		R12.2	No curved elements for elements that should have an adjustable length.
		R12.3	Elements should have pinned connections.
		R12.4	Elements should be containerized.
R13	The assembly and disassembly of the towers must be with the same system.		
R14	The new design should be able to operate from the lateral side.		
R15	The concept should make use of equipment known to Mammoet.		

2.6 QUALITATIVE MULTI CRITERIA ANALYSIS

To evaluate the concepts, they will be scored through a qualitative multi criteria analysis (MCA). The MCA is done according to the 'Integraal Ontwerp En Beheer' method by TU Delft [18]. The concepts will be scored based on six design criteria. Each concept will get a score from 1 to 5 for each criterion, see Table 2-6. After that, a weight factor is multiplied with the score to account for the importance of each criterion. By adding all factored scores, a total score per concept is established. The concept with the highest total score is the best and will be designed.

Table 2-6 Scores with explanation

SCORE	EXPLANATION
1	Does not fulfill the criterion
2	Barely fulfills the criterion
3	Fulfills the criterion, but has disadvantages
4	Fulfills the criterion, but has some limitations
5	Fulfills the criterion

This Chapter will first explain all categories. After that, the weight factor will be established.

2.6.1 Motion controls

Each concept requires the structure, or parts of the structure, to move. These motions are performed by motion controls e.g. hydraulics, wheels, or gears. The concept which has the least motions, and thus motion controls, is preferred as this saves time, maintenance, and complexity, thus money. This will be scored according to the scheme in Table 2-7.

Table 2-7 Motion controls scoring scheme

SCORE	# MOTION CONTROLS
1	10 +
2	9 – 10
3	7 – 8
4	5 – 6
5	0 – 4

2.6.2 New technique

New technique refers to components, systems, or procedures that are not known to Mammoet. A concept would be preferable if it consists of components, systems, or procedures already known to Mammoet. Because this would save development time and costs. Moreover, the engineers within Mammoet already have knowledge of the components, systems, or procedures, so no new knowledge needs to be acquired.

2.6.3 New Material

All concepts require new material, new material costs money. The one that needs the least amount of new material is preferred.

2.6.4 Power

Power refers to the weight the motion controls need to relocate. More weight requires more power, which in turn costs more time and money. So, the concept that requires the least power is preferred.

2.6.5 (Not) working at height

Working at height is not preferred. With the current solution people have to fasten all elements at height, for example after the upper structure is lifted. If a concept can avoid working at height this is beneficial.

2.6.6 Foundation

The loadbearing capacity of the foundation has to be assured by the client. A simple and small foundation is preferred over a strong and large foundation.

2.6.7 Weight factor

Not all criteria mentioned above are equally important. To distinguish which criterion is more important, a table is created comparing each criterion against another. Each time the criterion in the row is compared to the criterion in the column. The one that is more important scores one point. If two criteria are deemed equally important, they both score one point [18]. Table 2-8 shows both the weight factor determined by the author and the mean weight factors determined by other employees at Mammoet.

Table 2-8 Weight factors

CRITERION	WEIGHT FACTOR AUTHOR	WEIGHT FACTOR MEAN
Motion controls	0.18	0.17
New technique	0.06	0.10
New material	0.12	0.21
Power	0.12	0.15
(Not) working at height	0.29	0.17
Foundation	0.24	0.21

2.7 BEST PRACTICE SOLUTIONS

The mobilization and demobilization of a gantry can be split into five stages. Coming Paragraphs contain different solutions for each of these stages. The first stage is that the tower needs to come to height. Second, getting the upper structure at height. After the gantry lifted the vessel, the vessel is in the way of the upper structure, so the upper structure needs to get out of the way. Then, the upper structure needs to be lowered. As well are the towers. The solutions are based on sources of inspiration, a motion analysis, and creativity. Annex A.6 checks the solutions with the set of requirements and eliminates the ones that do not fulfill them. The promising solutions are put in a morphological chart at the end of this Chapter.

2.7.1 Stage I Tower erection

First, the erection of the towers will be considered. There are three ways to do this: Using an additional crane, by the use of a climbing frame, or via a smart use of the structure. Figure 2-10 schematically shows all options to assemble the tower. All will be discussed and evaluated in Annex A.6.1.

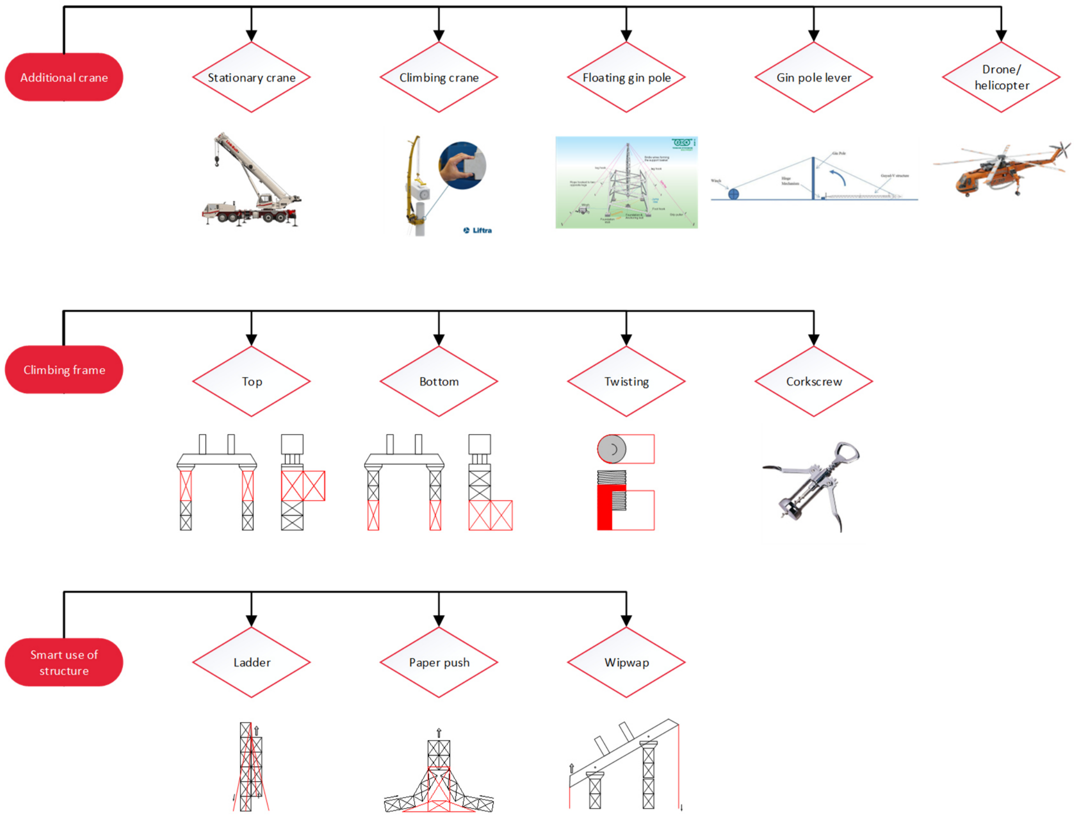


Figure 2-10 Stage I Tower assembly solutions

2.7.2 Stage II Raise upper structure

The upper structure is treated separately because this can either be placed on top of the towers after they are erect or while erecting. Figure 2-11 schematically shows all options to raise the upper structure. All will be discussed and evaluated in Annex A.6.2.

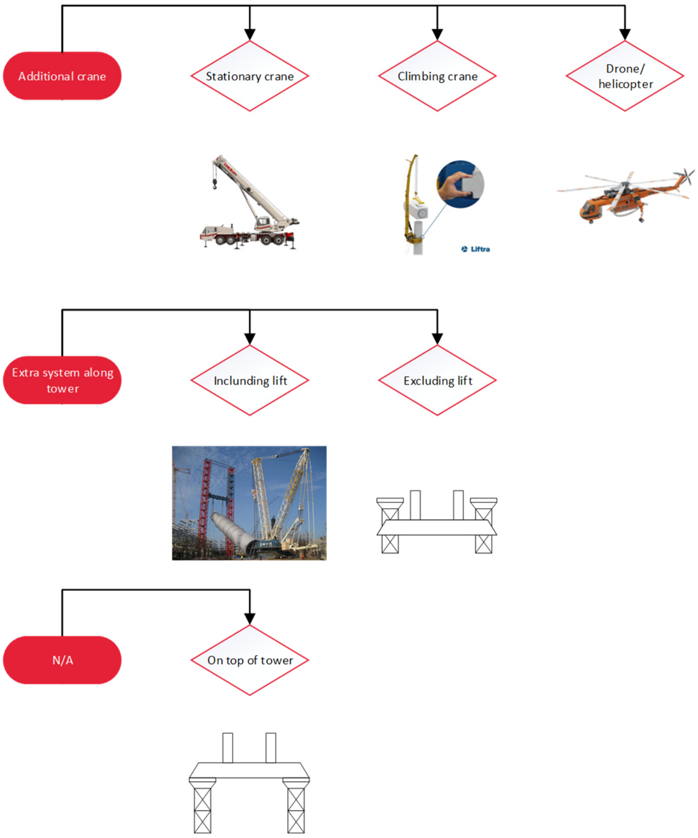
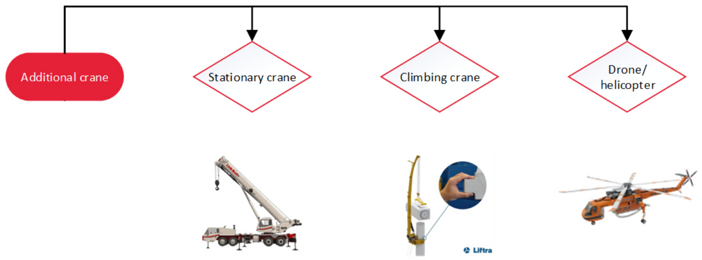
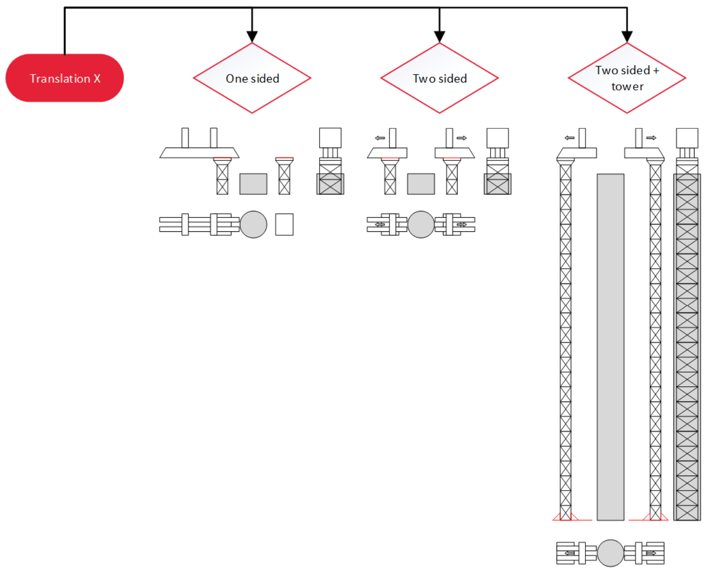


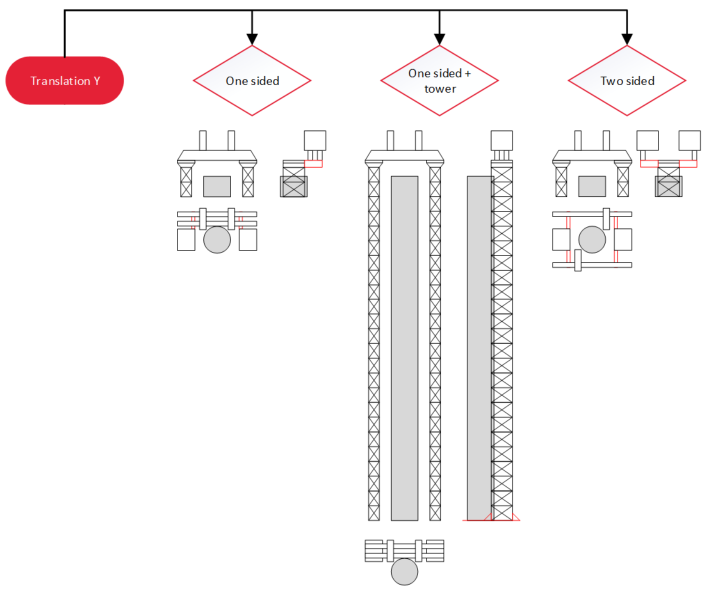
Figure 2-11 Stage II Raise upper structure

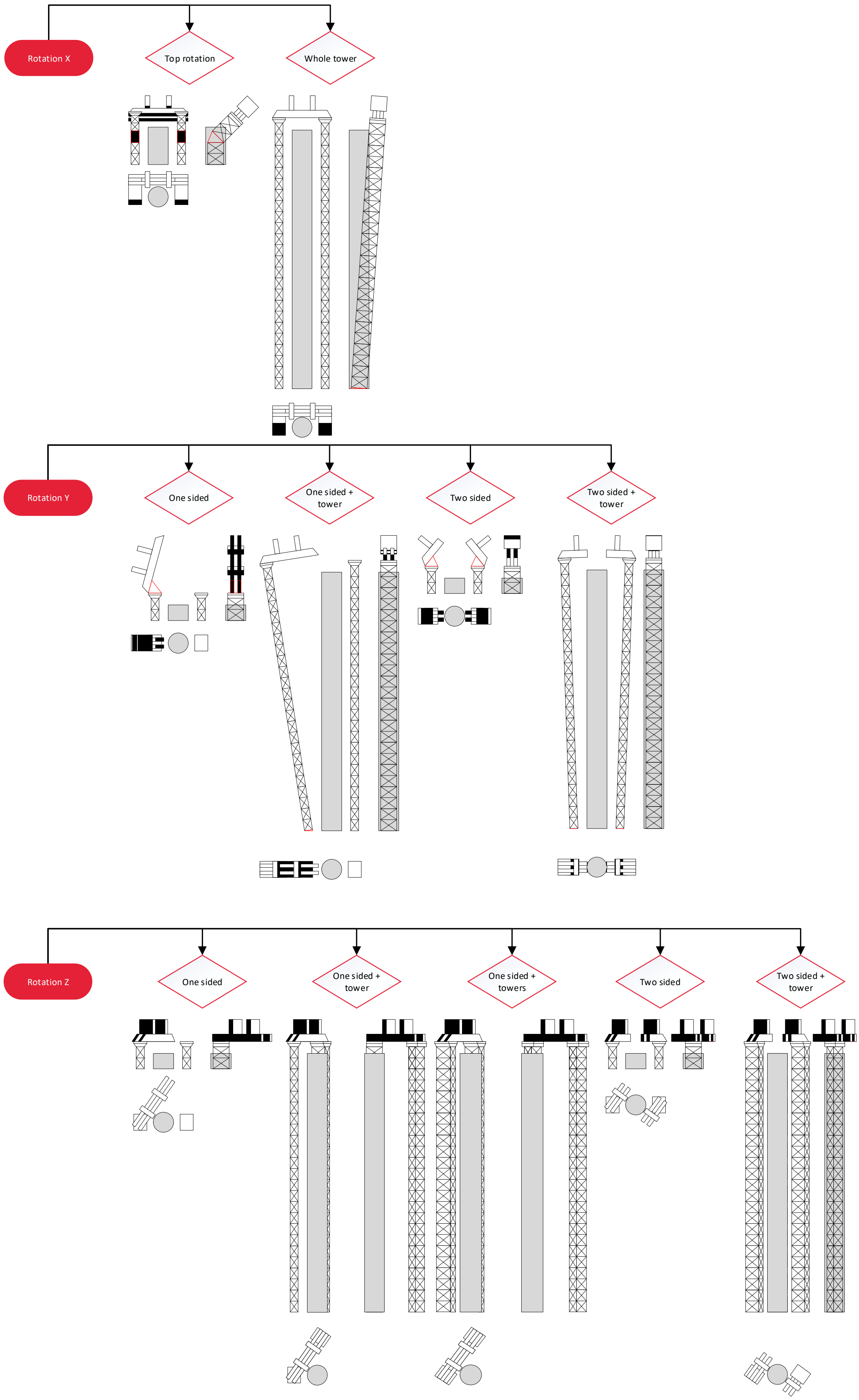
2.7.3 Stage III Upper structure out of the way

After the vessel is lifted, the gantry needs to be disassembled. Some of the concepts may require the upper structure to be on top for the erection of the towers. Then, the gantry cannot be disassembled similarly because the vessel is in the way of the upper structure. So, the upper structure needs to get out of the way of the vessel to ensure disassembly. In this Paragraph the ways to get the upper structure out of the way will be presented. This can be done via an additional crane. Also, by looking at all possible movements. Besides that, combinations of movements might improve the isolated movement variant. Lastly, the variants that do not have the upper structure in the way will be shown. See Figure 2-12 for all options. All will be discussed and evaluated in Annex A.6.3.


Additional crane
Stationary crane
Climbing crane
Drone/
helicopter


Translation X
One sided
Two sided
Two sided +
tower


Translation Y
One sided
One sided +
tower
Two sided


Rotation X
Top rotation
Whole tower
Rotation Y
One sided
One sided + tower
Two sided
Two sided + tower
Rotation Z
One sided
One sided + tower
One sided + towers
Two sided
Two sided + tower

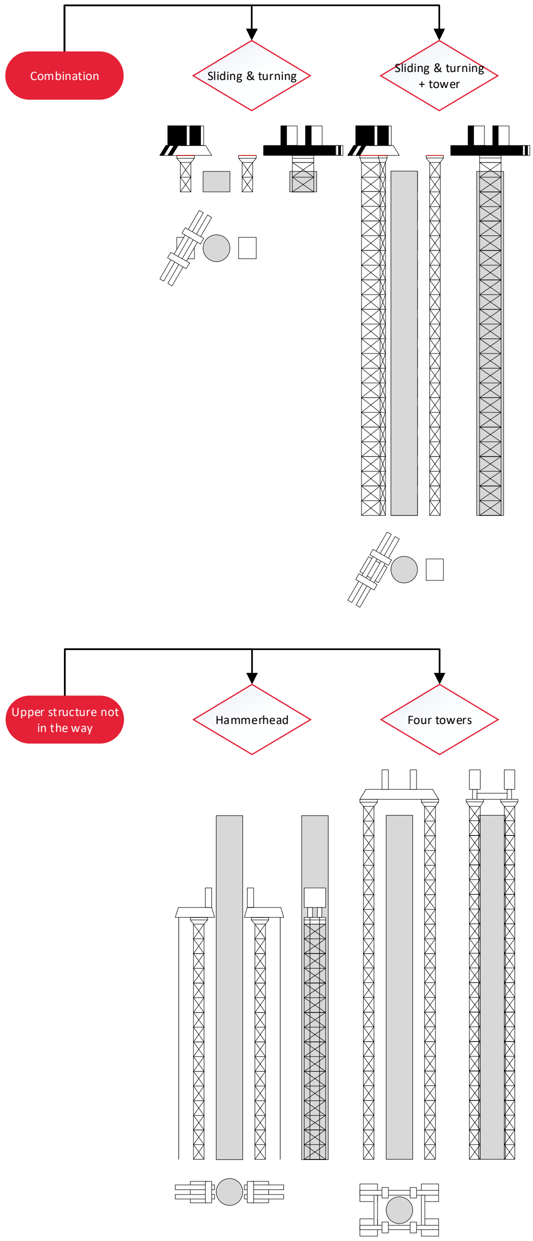


Figure 2-12 Stage III Upper structure out of the way

2.7.4 Stage IV Lower upper structure

According to requirement R13 the way the upper structure is lowered to the ground is the same, but opposite, to the way it is being raised.

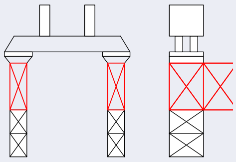
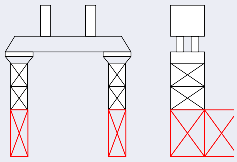
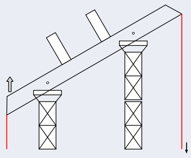
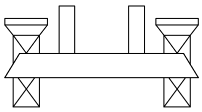
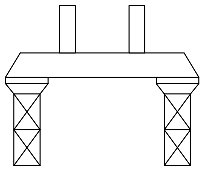
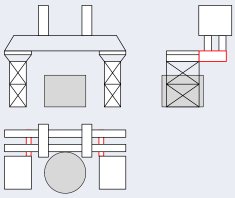
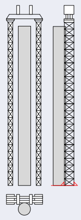
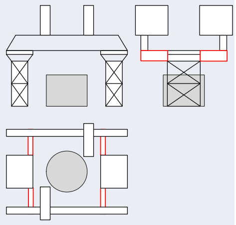
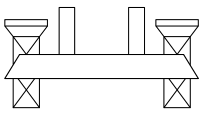
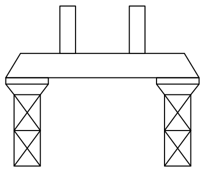
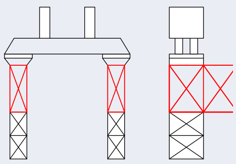
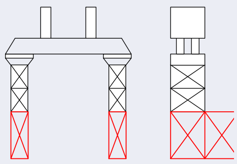
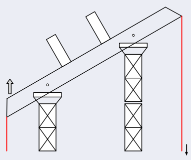
2.7.5 Stage V Tower de-erection

Also, the disassembly is done via the same mechanism as the assembly according to requirement R13.

2.7.6 Morphological chart

Based on the requirements, a first shift is made of all solutions mentioned above. The solutions that are left are put in a morphological chart, See Table 2-9. The morphological chart will be used to develop concepts. This will be done by selecting an option for each stage.

Table 2-9 Morphological chart

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

3 CONCEPTS

This Chapter will discuss and score seven concepts developed by the morphological chart in Table 2-9. Annex B shows each route through the morphological chart and describes the rationale behind the scoring.

3.1 CONCEPT I CLIMBING CRANE

Concept I is similar to the Liftra LT1500, see Annex A.6.1.1; a crane that fulfills all tasks needing to be done. The crane needs to be able to clamp to the MLS mast. From there it should lift the next section on top. Next, it needs to climb to this next section. When both towers are assembled it must lift the upper structure in place. It would be better to lift the individual elements of the upper structure like it is done in the case study. When the upper structure is brought down, the crane climbs down to de-erect the tower. It would be beneficial to use one crane that can erect both towers and lift the parts of the upper structure. However, it might be better to use two smaller cranes as the radius of one crane should be at least 23 meters.

3.2 CONCEPT II CFT EXTRA SYSTEM

When using a climbing frame at the top, the system to raise the mast sections can be used to lift the upper structure. Two climbing frames erect the towers after which they lift the upper structure. When the lift is performed the upper structure can go out the same way it came in. Then it is lowered and after that the climbing frames can disassemble the towers.

3.3 CONCEPT III CFT SKID

When using a climbing frame, the upper structure can already be on top while the towers are erected. This has the advantage that no extra system is needed to perform this task. The downside is that more weight needs to be pushed. The difference with the former concept is the way the upper structure gets out of the way. Here the whole gantry will be skidded to in Y direction to come free from the vessel.

3.4 CONCEPT IV CFT SPLIT

This concept is making use of a climbing frame at the top as well. Similar to the former concept is that the upper structure is already in place during the assembly of the towers. However, the way the upper structure gets out of the way differs. Here the upper structure will be split and translated in opposite Y direction to come free from the vessel.

3.5 CONCEPT V CFB SKID

Like the FOCUS crane, see Annex A.6.1.3, this concept makes use of a climbing frame that pushes the whole gantry from the bottom. The upper structure is in place during the assembly of the towers. In this concept the upper structure gets out of the way by translating the whole gantry in Y direction. After that, the whole structure can be disassembled by removing mast sections from the bottom via the climbing frame.

3.6 CONCEPT VI CFB SPLIT

This concept is similar to concept V, the difference being that the upper structure is split, and each part translates in opposite Y direction to come free from the vessel.

3.7 CONCEPT VII WIPWAP

This is the most out of the box concept. It is added to see how realistic it can be. The upper structure needs to be in place as this is the lever arm. By pulling on one side the other side, including tower, elevates. A new section is put underneath, and the process is repeated on the opposite side. After the structure is at height the upper structure should be fastened to secure a clamped connection. Now the vessel can be lifted. To get out of the way the only option is to skid the whole gantry away. When free from the vessel the wipwapping of the structure is done to de-erect the gantry. The wipwapping process can be seen in Annex A.6.1.4

3.8 CONCEPTS EVALUATION

All scores from the concepts can be seen in Table 3-1. As said in Chapter 2.6, each concept will get a score from 1 to 5, see Table 2-6, on all criteria. After that, the weight factor is multiplied with the score to account for the importance of each criterion. By adding all factored scores, a total score per concept is established. It can be seen that concept VI, CFB split, scores best. However, concepts IV, CFT split, and V, CFB skid, fall within 10 percent of the score. Combining that with the subjective nature of a qualitative multi criteria analysis, the three best concepts should be worked out in detail to come to a sound conclusion. Therefore, a second round of assessment is needed.

Table 3-1 Scores all concepts

	I	II	III	IV	V	VI	VII
Motion controls	3	4	3	2	4	3	4
New technique	1	4	4	4	5	5	2
New material	2	2	4	3	3	2	2
Power	5	3	2	4	1	2	1
(Not) working at height	1	2	3	2	5	3	3
Foundation	4	3	1	4	1	4.5	0
WF author	**2.65**	**2.82**	**2.59**	**2.94**	**3.18**	**3.24**	**2.06**
WF mean	**2.76**	**2.89**	**2.74**	**3.12**	**2.99**	**3.16**	**1.94**

4 SECOND ROUND OF ASSESSMENT

The three concepts will be compared and evaluated based on their differences. The differences between the concepts lay in the climbing frame and the skidding/ splitting system. Concept IV uses a climbing frame at the top. Whereas concepts V and VI use a climbing frame at the bottom. Concepts IV and VI use a splitting system at the top while concept V skids at the bottom. Therefore, the second round of assessment will compare these two differences. So, a climbing frame at the top against a climbing frame at the bottom and a skidding system at the bottom against a splitting system at the top. Each comparison will determine the best. Combining both results will therefore lead to the best design.

4.1 CLIMBING FRAME BOTTOM VS. CLIMBING FRAME TOP

First, some general features of the climbing frame will be discussed. These will apply to both the climbing frame at the bottom and the climbing frame at the top. After the general information each frame will be evaluated separately.

4.1.1 General climbing frame

The general features of the climbing frames consist of the placement and the elements of the FOCUS crane on which the climbing process is inspired on.

4.1.1.1 Placement

For both the top climbing frame and the bottom climbing frame two options are possible for the placement of the 'sliding frame'. It can either be placed in the longitudinal direction of the gantry or in the lateral direction of the gantry, see Figure 4-1. The crane must stay put in one location, according to requirement R10. If the frame is placed in lateral direction, one crane is able to provide all new sections to both towers. Otherwise, two cranes are needed. Moreover, one cannot guarantee that there is space next to the gantry. Therefore, the climbing frame should be in the lateral direction.

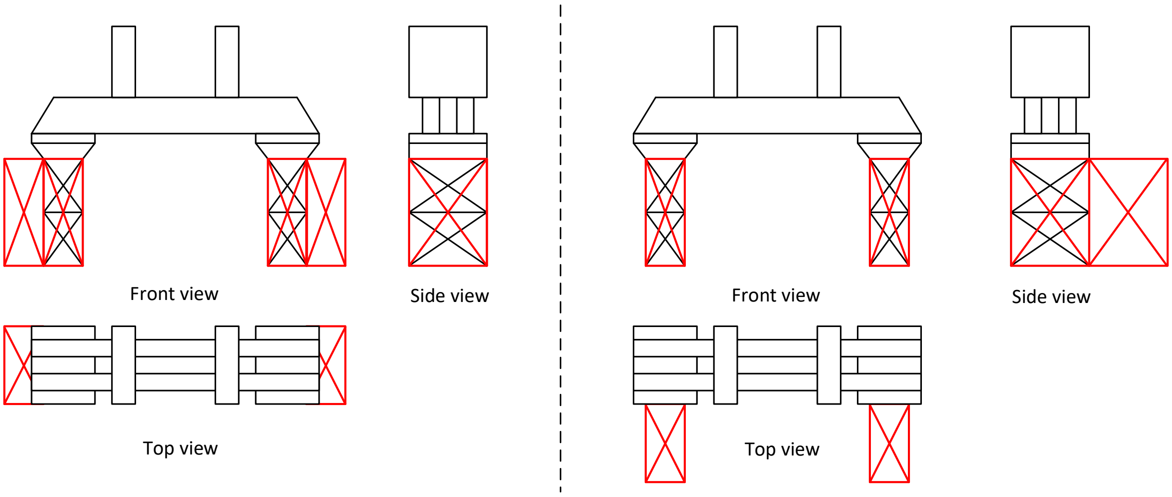


Figure 4-1 Climbing frame, in logitudinal direciton (left) and in lateral direction (right)

4.1.1.2 Climbing features

From the case study it was seen that the MLS chords are equipped with jack-up blocks. They should be used for either climbing frame. Figure 4-2 shows the jack-up block. The capacity of one block is 680 Te, see Annex C.1.1.1. Meaning that 2720 tons can rest on each tower. Each column has one jack-up block halfway to the height. This has the consequence that either climbing system has an unusable height of half the mast section's height.

The guidance edge, also visible in Figure 4-2, should be used to guide the mast straight.

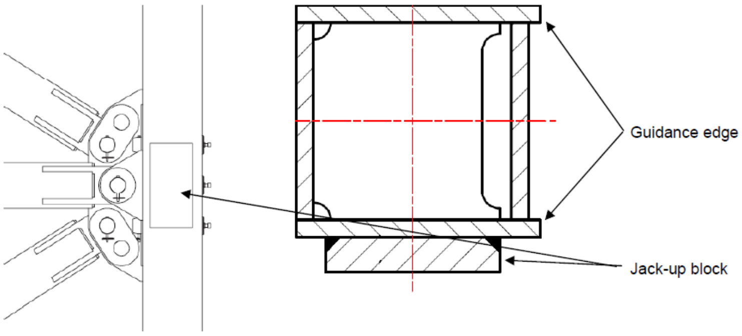


Figure 4-2 Provisions for self-erecting system [11]

4.1.1.3 FOCUS crane Mammoet

The FOCUS crane, see Figure 4-3, is used as a reference for this both frames.

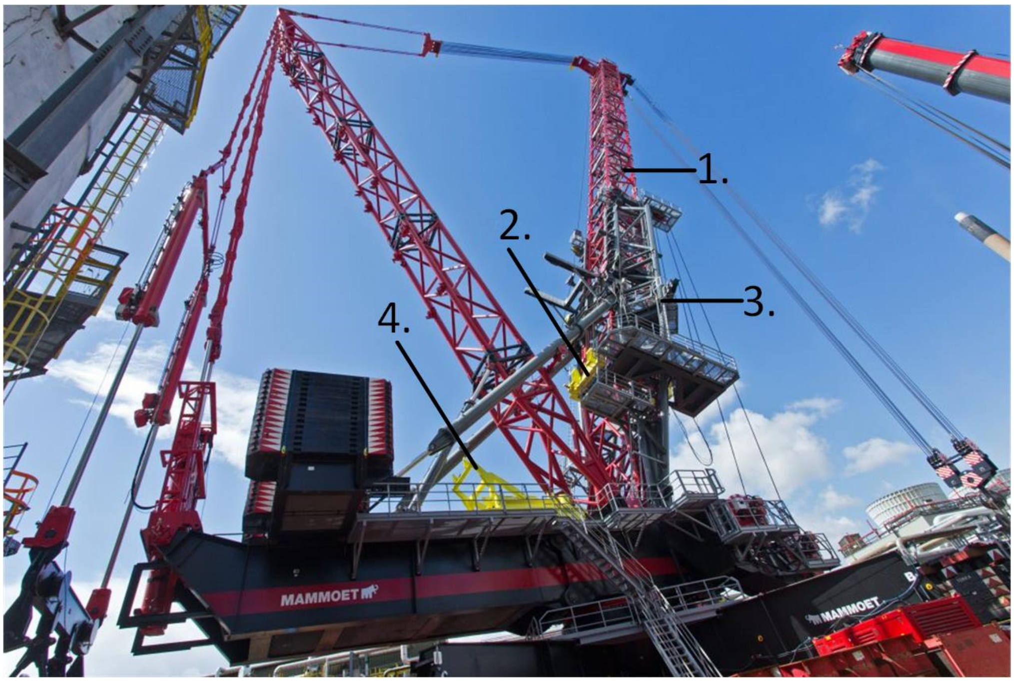


Figure 4-3 FOCUS crane by Mammoet [19]

The mast sections (1.) of the FOCUS have jack-up blocks. Pins embedded in a vertical moving frame (2.) can be extended and retracted, see Figure 4-4. When extended, they grab underneath the jack-up blocks. The vertical moving frame in the FOCUS crane has a capacity of 400 Te [20]. The mast sections that need to be lifted are 11,700 mm long, 3400 mm wide and 3400 mm deep.

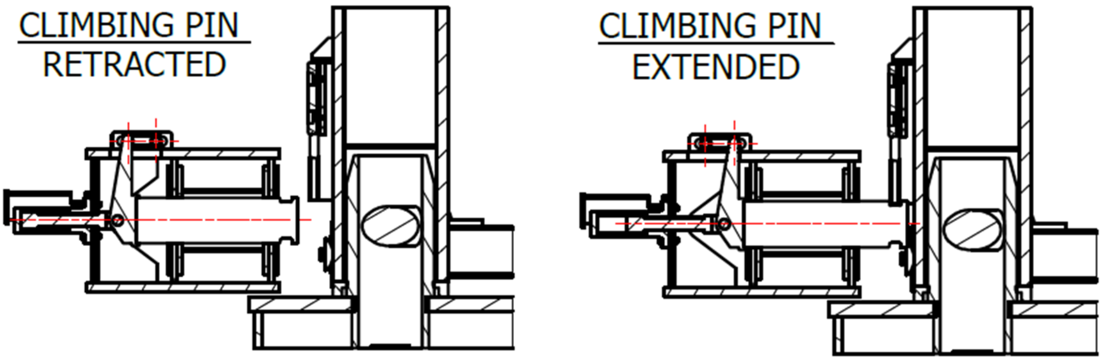


Figure 4-4 Climbing frame pin FOCUS crane [21]

The frame can be seen in Figure 4-5. This frame can move in vertical direction by means of two winches. They are such that solely a vertical force will be exerted on the masts. All horizontal forces and bending moments will be taken by the erection frame (3.). The new mast sections are slid in place via a horizontal moving frame (4.).

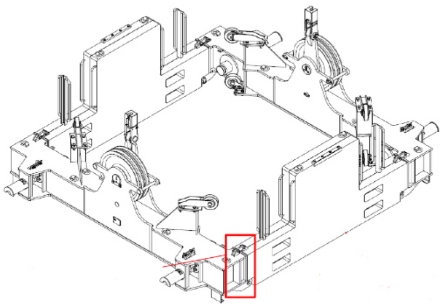


Figure 4-5 FOCUS climbing frame [20]

Both climbing frames will use a frame, like Figure 4-5, that travels up and down. The frame will be equipped with retractable pins and will move by means of hoists. The horizontal loads and bending moments will be taken by the erection frame.

4.1.2 Climbing frame bottom

The climbing frame needs to be able to lift the whole gantry and slide the new mast sections in place at ground level. Figure 4-6 shows the configuration of the structure before the last mast section needs to be inserted. The climbing frame is drawn in red.

Like the FOCUS crane the retractable pins grab underneath the jack-up blocks in order to lift the gantry. The lifting capacity of one frame will be determined by the height of the gantry, since every stroke adds more weight. For this design the capacity should be at least 650 tons per frame.

4.1.2.1 Stability

All horizontal forces and bending moments will be taken by the climbing frame. These are considerable at the final stroke as the 'arm' increases at every stroke. The higher the frame, the better these forces can be countered. Adding to that, the unusable height, mentioned in Chapter 4.1.1.2, makes that a strong, tall, and wide frame is needed. Therefore, it is chosen to make the base out of MSG mast sections. The disadvantage is that the available space for the vessel decreases if the new sections need to be inserted via the lateral direction.

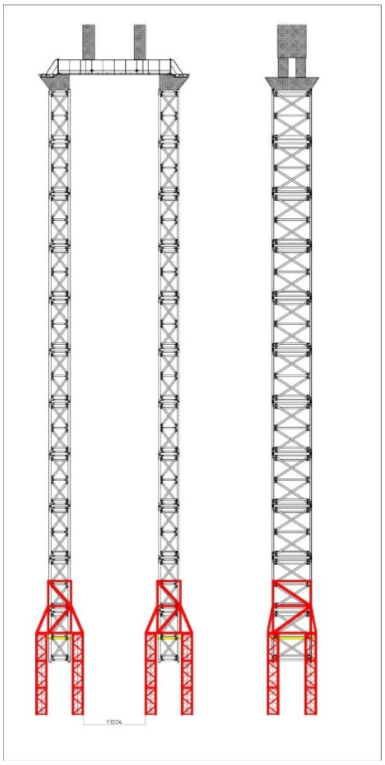


Figure 4-6 Climbing frame bottom final stroke

4.1.3 Climbing frame top

When climbing from the top the climbing frame needs to be able to lift itself and the upper structure of the gantry. By doing so, an opening will be formed. Via a hoisting system and a sliding system, a new mast section can be inserted. Figure 4-7 shows the configuration of the structure before the last mast section needs to be inserted. The climbing frame is drawn in red.

The retractable pins grab on the jack-up blocks in order to lift the climbing frame and the upper structure. The winches are attached to the bottom of the climbing frame.

4.1.3.1 Stability

By lifting a new section an eccentric load is introduced on the climbing frame. Annex C.1.3.1 calculates that no tension will occur on the retractable pin connection.

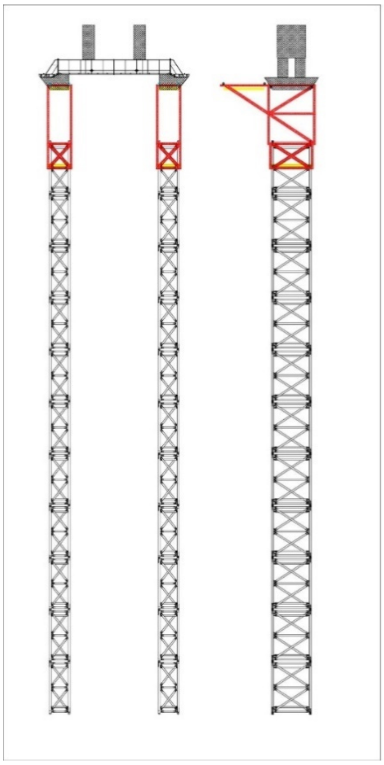


Figure 4-7 Climbing frame top final stroke

4.1.4 Conclusion

The pros and cons of both options can be seen in Table 4-1 and Table 4-2. The pros and cons evaluated in the design criteria from Chapter 3 are also included. A climbing frame at the top is chosen for the final design. Its pros are more advantageous than the pros of the climbing frame bottom. Besides that, the cons are better to solve than the cons of the climbing frame bottom.

Table 4-1 Pros and cons climbing frame bottom

CLIMBING FRAME BOTTOM	
Pros	**Cons**
Working on the ground	Much material
Less steps	Stability challenges
	Less available space for lateral placement
	Tall frame
	Limited capacity

Table 4-2 Pros and cons climbing frame top

CLIMBING FRAME TOP	
Pros	**Cons**
Less material	Working at height
Stable solution	Extra hoist system
Future proof	
Lateral placement possible	

4.2 SKIDDING SYSTEM BOTTOM VS. SPLITTING SYSTEM TOP

Both systems will be evaluated separately.

4.2.1 Skidding system bottom

This Paragraph will discuss the stability and the required skidding systems.

4.2.1.1 Stability

To translate the whole gantry both a skidding system and SPTMs can be used. It must be checked whether the gantry will not fall over because of the side loads. Different side loads apply to a skidding system or SPMTs according to the Engineering Handbook Special Devices 1.02 [6]. The horizontal loads that apply can be found in Table 4-3, for the calculation refer to Annex C.2.1.1.

Table 4-3 Horizontal loads on gantry

CASE	SIDE LOAD [TE]
Stationary gantry	75
Acceleration during skidding	120
Acceleration during SPMT	150

This horizontal force acts in the C.o.G. of the gantry, this is roughly calculated at 90 meters. The bending moment generated by both translations will cause tensile forces in the foundation, see Annex C.2.1.1. Both skidding systems and SPMTs cannot counter tensile forces, meaning that the gantry will tip over.

This can be solved by widening the base. Skidding would have the preference over using SPMTs as their acceleration force is smaller. Annex C.2.1.1 has calculated that, for no tensile forces, the base needs to be at least 14.5 meters. A smooth force transfer is needed. Meaning that diagonal braces are needed.

4.2.1.2 Skidding systems

Each tower needs at least two skid tracks to be stable enough to skid. The four foundation points per tower should get an individual skid beam or shoe. As Mammoet does not have skid beams that are 10 meters long [6], eight skid beams or shoes are needed, each having a vertical load capacity of 380 Te and a push/ pull capacity of 14 Te, see Annex C.2.1.2 for the calculation. These capacities can be found in the fleet of skidding systems from Mammoet [6].

4.2.2 Splitting system top

Figure 4-8 shows the split upper structure. To split the upper structure a skidding system will be used. When applying a skidding system at the top, guidance beams (1.) are required, see Figure 4-8. The gantry beams will be uncoupled from the top frame and will skid over the guidance beams, so the upper structure comes free from the vessel.

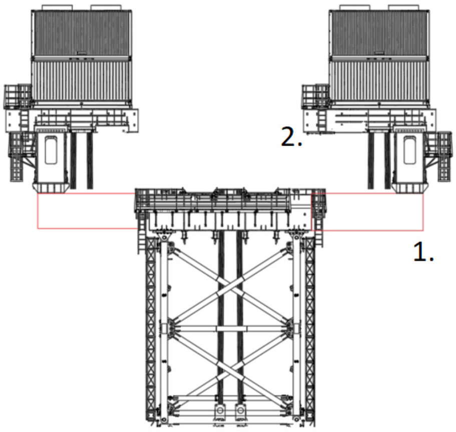


Figure 4-8 Split upper structure

This Paragraph will discuss global stability, the skidding systems, the guidance beam, and the strand jack beam, indicated with 2. in Figure 4-8.

4.2.2.1 Stability

During the lift the gantry beams and the towers have a clamped connection. A clamped connection cannot be formed by a skidding system. Therefore, the gantry beams cannot be placed directly on the skidding systems from the start. After the lift is performed, the skidding system needs to jack-up so that the gantry beams solely rest on the skid shoes or beams. After that, the skidding process can begin.

The clamped connection between the gantry beams and the towers that is present before skidding must be loosened. By loosening the clamped connection, the structural scheme of the total gantry changes. Annex A.2.3.5 calculated the buckling resistance of the tower without an upper structure. The load from the current upper structure plus the auxiliary weight of a guidance beam and the skidding systems will not cause global instability.

Skidding will generate horizontal forces on the gantry [6]. However, because each beam skids in the opposite direction, globally the horizontal forces cancel each other out.

4.2.2.2 Skidding systems

Two gantry beams need to skid in opposite direction. The gantry beam will be skidded at their ends, so four skidding systems are needed. This requires each skidding system to have a vertical force capacity of 120 Te and a push/ pull capacity of 5 Te, see Annex C.2.2.1 for the calculation.

4.2.2.3 Guidance beam

Annex C.2.2.2 calculated the required section modulus of the guidance beam to be in the order of magnitude of $3.0 * 10^7$ mm^3. This is lower than the gantry beam, meaning that such beams exist.

Figure 4-8 shows that the strand jack beam is cantilevering instead of being simply supported. Annex C.2.2.2 verified that the beam still suffices. Cantilevering this beam will introduce torque in the gantry beam. This needs to be checked when this alternative is chosen.

Also, when disassembled, one gantry beam is behind the vessel, this might be difficult to reach. This needs to be solved too.

4.2.3 Conclusion

The pros and cons of both options can be seen in Table 4-4 and Table 4-5 The pros and cons evaluated in the design criteria from Chapter 3 are also included. Splitting at the top is chosen as the better solution. The cons of this solution are less challenging to solve than the cons of skidding bottom. Moreover, the pros are more advantageous than the pros of skidding bottom.

Table 4-4 Pros and cons skidding system bottom

SKIDDING BOTTOM	
Pros	**Cons**
Working on the ground	Stability challenges
	Large foundation needed

Table 4-5 Pros and cons splitting system top

SPLITTING TOP	
Pros	**Cons**
Future proof	Access to rear gantry beam
Small foundation needed	Working at height
Stable solution	New guidance beams needed
	Torque in gantry beam

4.3 CONCLUSION

The final design will climb by means of a climbing frame at the top and the upper structure will get out of the way via splitting the gantry beams.

5 FINAL DESIGN

This Chapter will describe and validate the design. First, the general design will be discussed. After that, some fundamental features will be elaborated and validated. These features are; the MLS hoisting system, climbing, and skidding of the upper structure. The FEM validation of the total model is discussed in the next Paragraph. Lastly, the connections, the assembly and disassembly sequence, and the demands for this design will be discussed.

5.1 GENERAL DESIGN

Figure 5-1 shows the final design. It consists of two climbing cages (1.) that are connected via the gantry beams (2.). Each climbing frame is equipped with a hoisting system (3.) that is used to lift and roll the new MLS mast sections in place. Climbing will happen via the vertical moving frame (4.), which is based on the FOCUS crane, see Chapter 4.1.1.3. To split the gantry beams, two cantilevering guidance beams (5.) are present. The final design is designed such, that when the gantry is in its final position, the original Jubail gantry design is established. The only difference is that the self-weight of the climbing frame is acting on the gantry. Meaning that solely the original MLS top frame beams (6.) and the added self-weight are decisive in the validity of the gantry's main purpose; lifting the vessel. This also means that the climbing frames are solely meant for climbing.

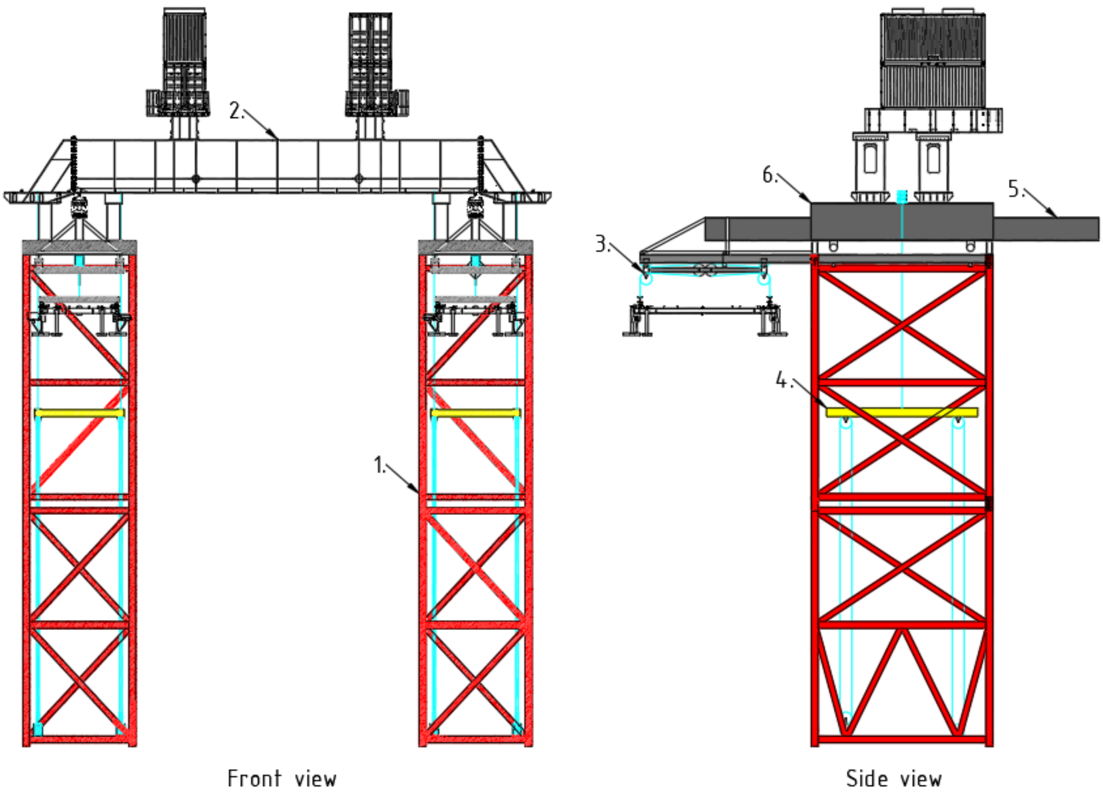


Figure 5-1 Final design climbing frame, front view and side view

Figure 5-2 shows a 3D render from the SCIA model to give a spatial visualization of the design. Both configurations are shown, before lifting (right) and after lifting (left).

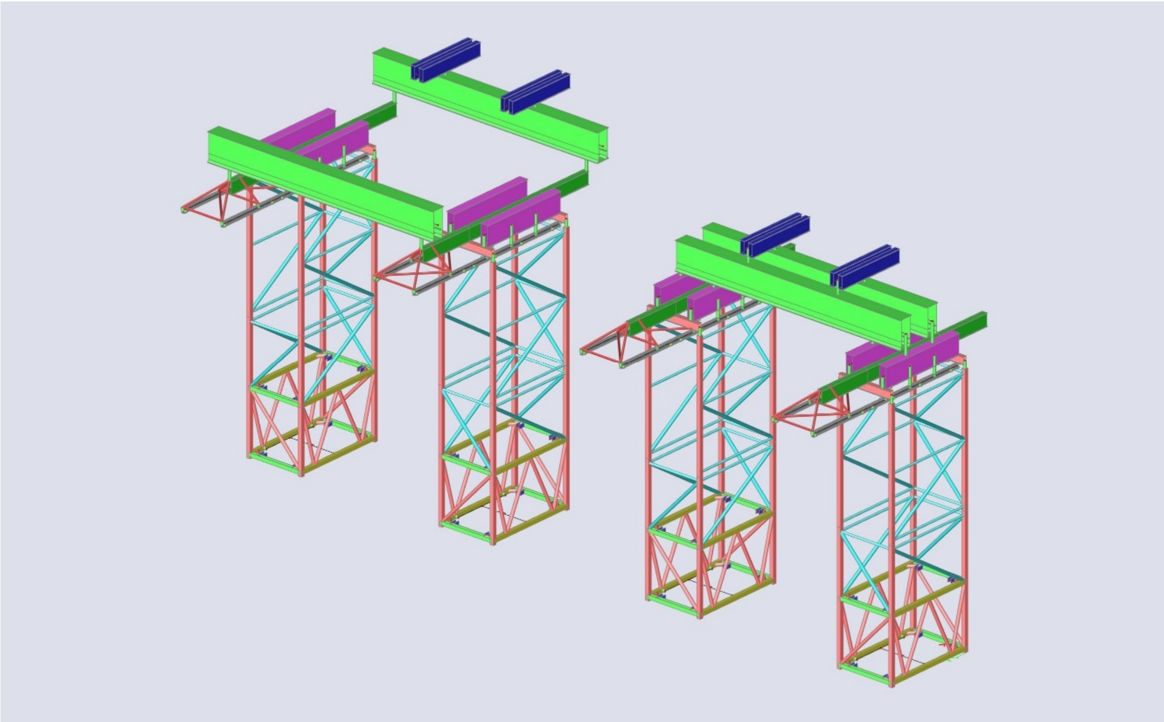


Figure 5-2 3D render final design, made by SCIA

5.1.1 Dimensions

Figure 5-3 shows the dimensions of the climbing frame in both configurations. Compared to the original Jubail gantry, the width is unchanged, as this is still determined by the width of the gantry beam. The depth is increased from 8.6 to 29.7 meters, due to the splitting of the upper structure and the MLS hoisting system. The height, when the whole gantry is assembled, is 0.8 meters higher, because of an extra top frame beam in Y direction.

The self-weight of the frame (excluding the gantry beam, and the strand jack beams) is approximately 600 Te, according to SCIA. This includes secondary steel and additional equipment like winches, ropes, skid shoes, skid tracks, powerpacks, etc..

A list of all members with their names can be found in Annex D.1.1

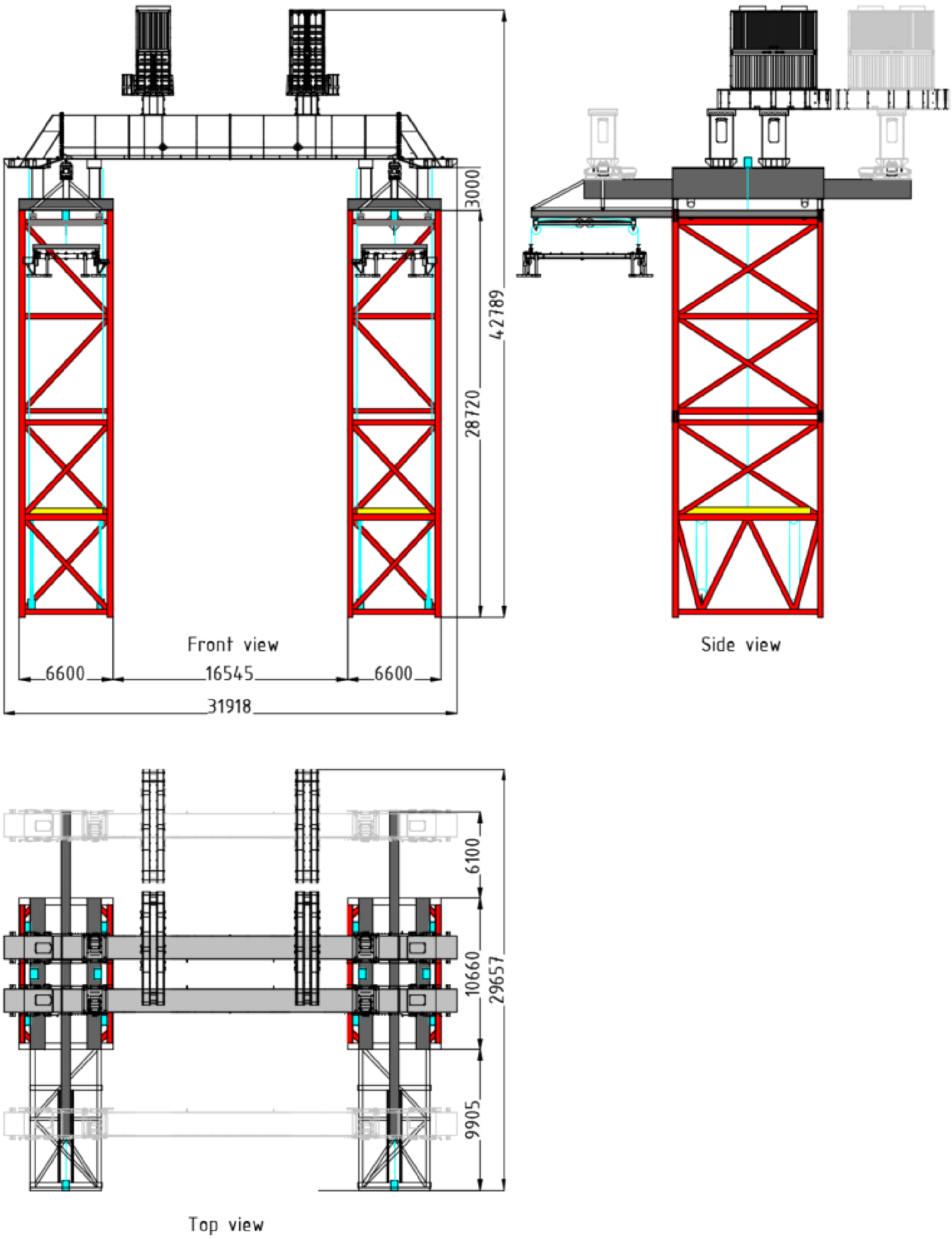


Figure 5-3 Dimensions final design

5.1.2 MLS hoisting system

The first step in assembling the gantry is lifting and inserting the MLS tower sections. This will happen via the MLS hoisting systems. As mentioned, both climbing cages are equipped with an MLS hoisting system. These systems will operate simultaneously. Lifting will be done via winches. Each system will lift an MLS section until the opening of the climbing cage is reached. Then, the section is rolled in. The hoisting systems are equipped with beam trolleys so they can roll over the rails. Once the MLS section is inside the climbing cage, it is lowered on top of either the foundation or the previous section. See Figure 5-4 for the parts and the possible movements.

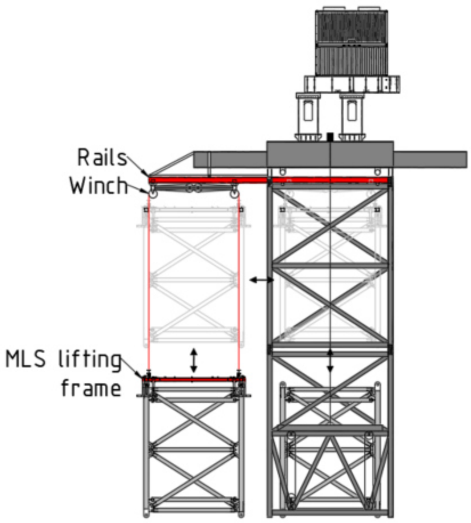


Figure 5-4 Possible translations MLS hoisting system

5.1.3 Climbing

The climbing process will start after two MLS sections are inserted and put on to each other. In order to climb, two hold points are needed. The structure rests on one hold point so that the other hold point can move towards the new level, and vice versa. For this frame the two hold points are the MLS top frames and the vertical moving frame. The vertical moving frame has climbing pins that can extend and retract.

During step 1, see Figure 5-5, the climbing frame is connected to the towers via the MLS top frame, the first hold point. The climbing pins of the vertical moving frame are retracted. Now, the vertical moving frame is pulled up via the winch that is located on the MLS top frame. By doing so, step 2 is reached. The climbing pins extend so that they grab on the jack-up blocks of the MLS chord. This is the second hold point. The winches that are located at the bottom of the climbing cage will wound up. This will pull up the whole climbing frame, see step 3. After fourteen meters of climbing, the opening is large enough. A new MLS mast section can be lifted and inserted. When the new section is in place and connected, the climbing frame is lowered so that the tower can be connected to the MLS top frame, step 4. This finalizes one climbing sequence. This process needs to be repeated until the desired height is reached. Applying this frame to the Jubail gantry will look like Figure 5-6.

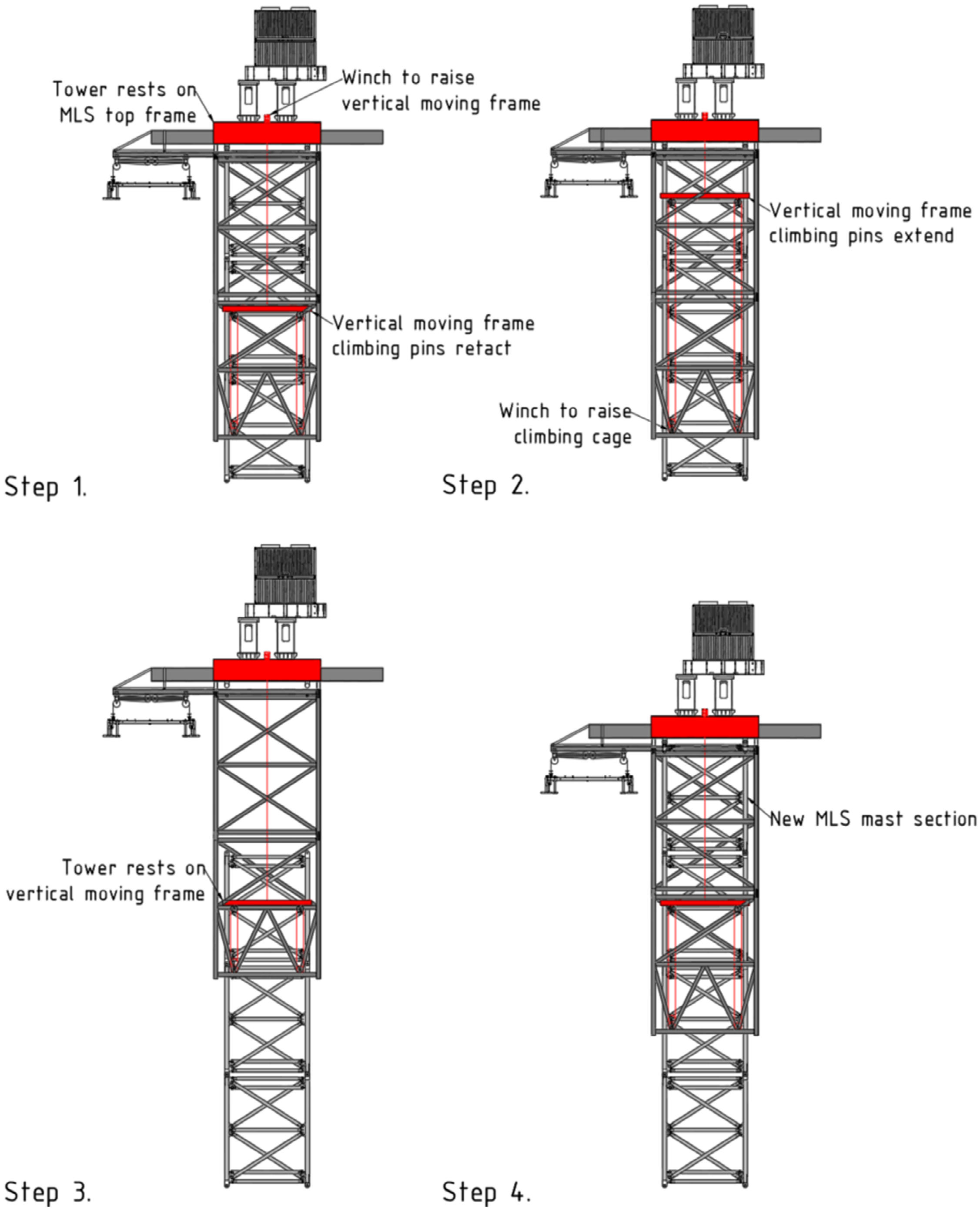


Figure 5-5 Climbing sequence

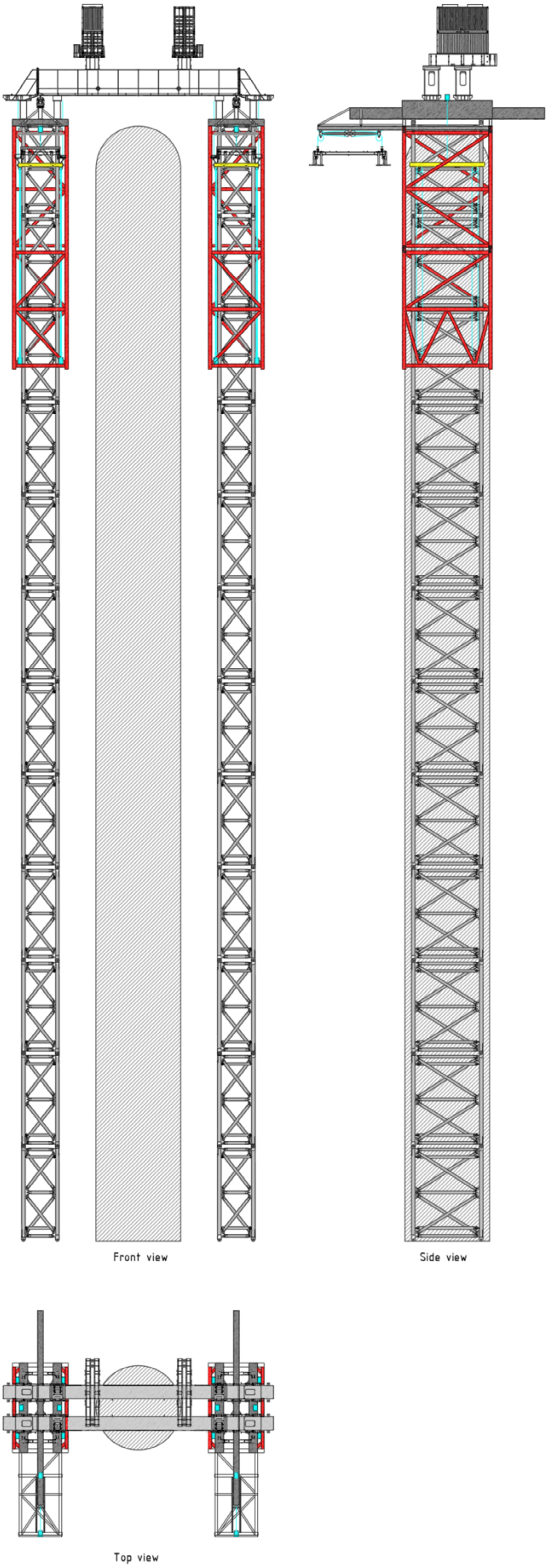


Figure 5-6 Jubail gantry with climbing frame

5.1.4 Upper structure skid

When the gantry is at the desired height, it is ready to perform its job; lifting the vessel. The gantry beams rest on the MLS top frames. After the vessel has been lifted, the gantry beams need to skid in opposite Y direction. The splitting of the gantry beams will happen by means of a skid shoe. The skid shoes are located on the guidance beam, underneath the gantry beams. The gantry beams are loosened from the MLS top frames. Then, the skid shoes will jack-up the gantry beam so that it comes free from the MLS top frames. Once they are free, the skidding process can begin. This creates the opening that is needed for disassembly. Figure 5-7 shows the parts and the possible movements, it also shows the maximum diameter the vessel can have. This is 15.4 meters without safety margins.

The strand jack beams that are needed for the lifting of the vessel are placed on the gantry beams such that after the split both strand jack beams are balanced on the rear gantry beam. This has the benefit that no torque will occur in the gantry beams. Moreover, more weight is put in the rear of the climbing frame. This makes the climbing frame better balanced while lifting a new MLS section.

Four skid shoes having a capacity of 110 Te each are needed. They are placed on the guidance beam. The height between the guidance beam and the gantry beams is 1.4 meters. This is the height (minus the height of the skid shoe itself) that needs to be jacked. The push/pull capacity of the shoe needs to be 5 Te. The maximum length of the skid shoe is 3.6 meters. For the calculation see Annex D.1.2.

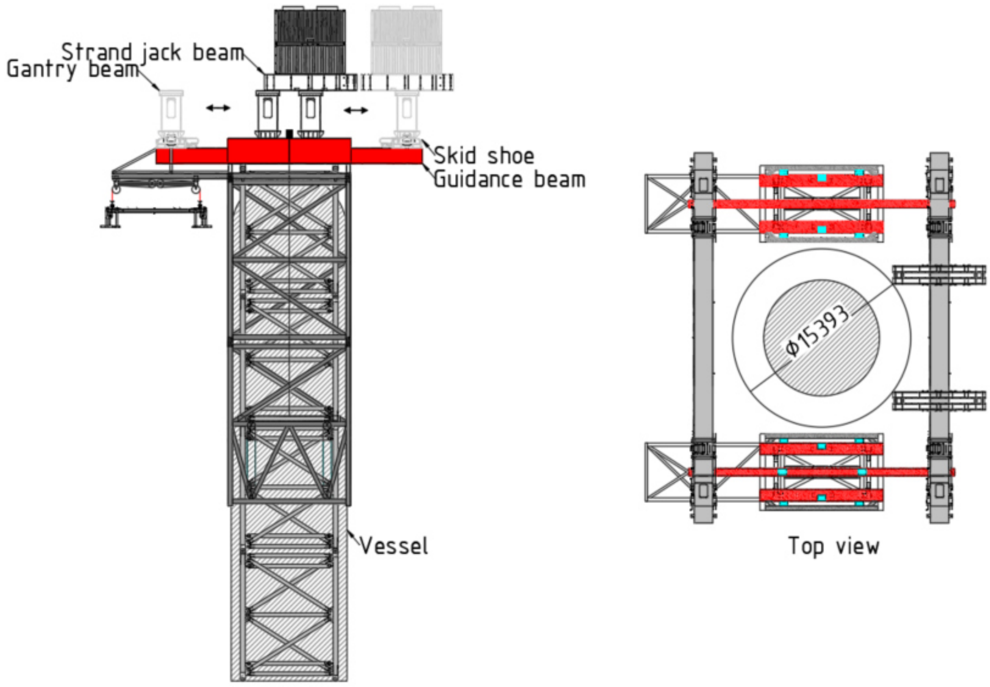


Figure 5-7 Upper structure skid

5.1.5 Force transfer

5.1.5.1 Vertical load

During climbing, all loads will be transferred via the bottom of the climbing cage, see Figure 5-8, to the tower of the gantry. The vertical load transfer happens via the bottom horizontal Y braces. They are equipped with winches at the locations where the diagonal braces are attached. The winches are connected to the vertical moving frame. To assure that the vertical moving frame will transfer one vertical force, in Y direction one rope is used. This rope starts at the winch and anchors, after reeving, at the anchoring points. To guarantee no difference in X direction two machine-controlled winches will be used. A computer will maintain equal loads in both winches.

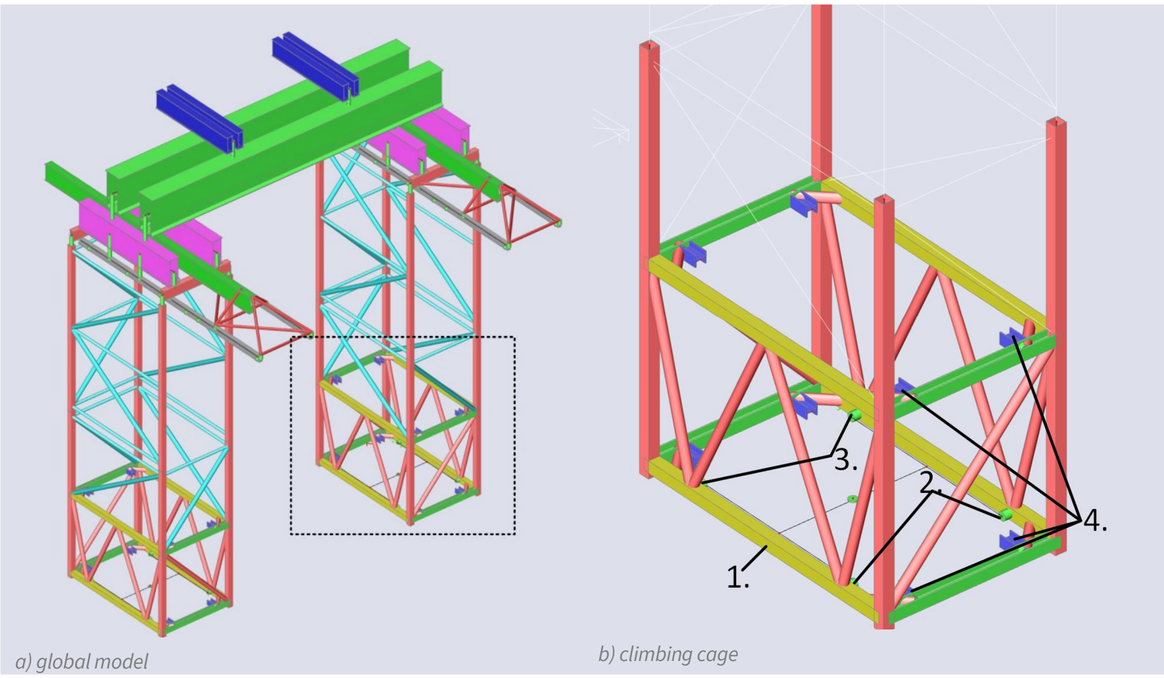


Figure 5-8 3D render climbing cage, made by SCIA

1. Bottom horizontal Y brace
2. Winch locations
3. Anchoring points
4. Stability beams

5.1.5.2 Horizontal loads and bending moments

All horizontal loads and bending moments will be transferred via pressure only connections between the stability beams to the towers. Each stability beam presses to the MLS chords between and on the guidance edges. Transferring loads in both X directions, because of the guidance edges, and in one Y direction (pressure only). Opposite stability beams are needed to transfer the load in the opposite Y direction. Four stability beams per level are needed to concur bending moments around the Z axis. Two levels of stability beams are needed to concur bending moments around the X and Y axes.

5.2 VALIDATION

For the validation of the design, first a calculation is done to the global change in structural scheme. After that a local check on all members is done via SCIA engineer. SCIA is also used to check the deformations of the structure.

5.2.1 Global stability total gantry

By splitting the gantry beams the clamped connection between the towers and the gantry beam can no longer be secured. This makes the structural scheme change to a portal with two hinges at the top. According to Annex D.2.7 the critical reaction forces coming from one climbing cage to one tower are:

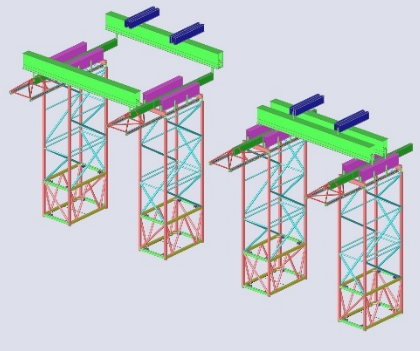
N_{Ed} = 6813 kN
$V_{y,Ed}$ = 342 kN
$V_{z,Ed}$ = 367 kN
$M_{y,Ed}$ = 15,518 kNm
$M_{z,Ed}$ = 7562 kNm

It is calculated in Annex D.2.1 that exerting these loads on the tower will not result in global instability of the gantry. The calculation is based on the combined compressive and bending moment stability check in Eurocode 1993-1-1. The Unity Checks are 0.63 and 0.94.

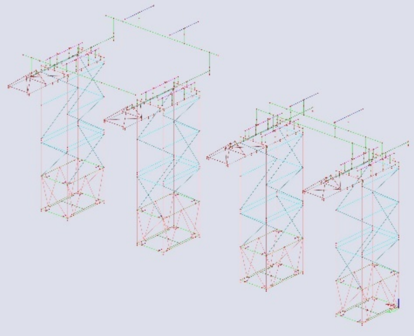
5.2.2 Finite Element Model

Two configurations can be seen in Figure 5-9 a). Configuration 1 (right) is for the assembly of the gantry, configuration 2 (left) is for the disassembly. Figure 5-9 b) shows the 1d model of the two configurations.

The supports are located in the bottom of the climbing frame. In vertical direction only one support per frame is modelled. This is to guarantee one vertical force per frame and that all bending moments will be taken by the stability beams. The stability beams are supported by non-linear pressure only supports in Y direction and linear supports in X direction. All supports are hinged supports.



a) rendered model



b) 1d model

Figure 5-9 FEM model in SCIA Engineer

5.2.3 Loads

5.2.3.1 Load factors

To comply with NEN-EN 1993-1 the loads must be multiplied with load factors to obtain the SLS and ULS load combinations. The factors from Table 5-1 are used:

Table 5-1 Load factors

LOAD	FACTOR
Self-weight	1.35
Lifting load	1.50
Dynamic factor	1.15

5.2.3.2 Loads and load cases

The following loads were applied on the model:

- Self-weight
- Side load
- Lifting load

The self-weight of the components is determined by SCIA. However, SCIA calculates the weight based on the gross cross section. To account for extra fabrication weight to the members, e.g. stiffeners or lifting points and extra weight coming from powerpacks, winches, and skid shoes, a conservative factor of 2.0 is applied over the self-weight calculated by SCIA. The SCIA model of the Jubail gantry used factors ranging between 1.66 and 1.86 over the self-weight, meaning that a factor of 2.0 is roughly 14% on the conservative side.

According to 'Engineering Handbook Special Devices' a conservative assumption for the side load is 5% of the self-weight or lifting load [6]. This factor includes; 1% standard, 1% wind, 1% settlement, 0.5% misalignment and 1.5% unforeseen. This factor is applied over all loads, in both X and Y directions.

MLS mast sections need to be lifted. Their self-weight and auxiliary rigging/ lifting frames form the lifting load. The load is the result of the analysis of the reaction forces coming from the MLS lifting system. Three positions of the MLS mast are checked, these are:

- Lifting the MLS mast from the ground
- Inserting the MLS mast (halfway rolling)
- Installing the MLS mast on foundation or tower

5.2.3.3 Load combinations

Four load combination categories will be acting on the climbing frame, these are; self-weight, operational, survival, and stability combinations. All will be discussed.

Self-weight combinations

When climbing, only the self-weight of the structure and the side loads are acting on the climbing frame. This results in two combinations, see Table 5-2 for the load factors.

Table 5-2 SLS and ULS load factors self-weight

LOAD	SLS FACTOR	ULS FACTOR
Self-weight	1.00	1.35
Side load X direction	1.00	1.35
Side load Y direction	1.00	1.35

Operational combinations

Operational in this sense is lifting the new MLS section. Three positions are considered, all have the same factor. Therefore, it is referred to as position 'i', for 'i' being 1, 2, or 3. This results in six combinations, see Table 5-3 for the load factors.

Table 5-3 SLS and ULS load factors operational

LOAD	SLS FACTOR	ULS FACTOR
Self-weight	1.00	1.35
Side load X direction	1.00	1.35
Side load Y direction	1.00	1.35
Lifting MLS mast position i	1.15	1.73 (1.5*1.15)
Lifting MLS mast side load X position i	1.15	1.73 (1.5*1.15)
Lifting MLS mast side load Y position i	1.15	1.73 (1.5*1.15)

Survival combination

No survival combinations are considered as no climbing will be performed during survival circumstances. This means that the climbing frame can connect to the towers, like it would for the lifting of the vessel. As this is (almost) the original Jubail configuration, it is assumed that it will be able to withstand this load case.

Stability combinations

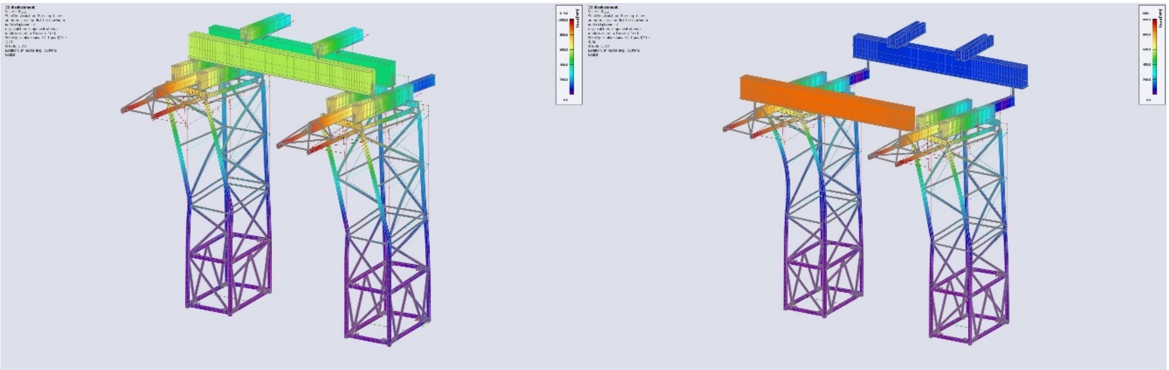
The stability of the climbing frame is calculated and checked by SCIA. The operational ULS combinations are used for the calculations.

5.2.4 Stability climbing frame

Table 5-4 shows the α_{cr} factor for global buckling of the climbing frame for both configurations. Figure 5-10 shows the global buckling for both configurations. For both configurations the stability combination based on the ULS combination for lifting MLS mast position 1 gave the critical α_{cr} factor.

Table 5-4 α_{cr} value for the three operational load cases

CONFIGURATION	ACR
1. Gantry beams not split	7.33
2. Gantry beams split	4.86



a) configuration 1 *b) configuration 2*

Figure 5-10 Global buckling shape climbing frame

According to NEN-EN1993-1-1, for α_{cr} smaller than 10, the climbing frame should be considered a sway frame. Since α_{cr} is bigger than 3 it is chosen to use the amplified sway method for the calculations. So, performing a first-order analysis where the sway imperfections are accounted for.

The imposed imperfection is the global buckling shape from Figure 5-10. The magnitude of the imperfection is based on the sum of the tolerance in the stability beams to MLS mast connections, the imperfection of the climbing frame, and the imperfection of the towers. This is roughly 1/185, see Annex D.2.2 for the calculation. 1/185 multiplied with the height of the frame holds for a scaling magnitude of 120 mm. This is the height from the upper support to the top of the climbing cage.

5.2.5 Unity Checks

The Unity Checks tell if the member suffices to the imposed loads. All members and their SCIA calculation report can be seen in Annex D.2. The colours refer to how optimized the member is, see Table 5-5.

Table 5-5 Unity Check degree of optimization

COLOUR & U.C. RANGE	RATIONALE
0.00 – 0.30	Not optimized
0.30 – 0.60	Not optimized, ULS wise
0.60 – 0.80	Safely optimized
0.80 – 0.99	Well optimized
1.00 – inf	Not acceptable

Both configurations have different strengths and weaknesses. Therefore, Table 5-6 to Table 5-10 contain the Unity Checks for both configurations. The members in orange are not optimized, however, these come into play during lifting of the vessel. Most of the yellow members can be optimized more ULS wise, however these add stability and stiffness, so they decrease deformations.

The Unity Checks are based on both the section and the stability checks from Eurocode EN1993-1-1. The governing Unity Checks per member are given, see Annex D.2 for all checks.

Table 5-6 Climbing cage bottom Unity Checks

CLIMBING CAGE BOTTOM	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Bottom chords	0.62	0.60	0.62
Bottom horizontal X brace	0.98	0.97	0.98
Bottom horizontal Y brace	0.37	0.37	0.33
Bottom diagonal brace	0.67	0.66	0.67
Stability beams	0.85	0.85	0.80

Table 5-7 Climbing cage top Unity Checks

CLIMBING CAGE TOP	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Top chords	0.81	0.73	0.81
Top horizontal brace	0.39	0.34	0.39
Top diagonal brace	0.58	0.43	0.58

Table 5-8 Top frame Unity Checks

TOP FRAME	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
MLS top frame beam	0.06	0.06	0.02
Guidance beam	0.73	0.46	0.73
Top frame Y	0.91	0.81	0.91

Table 5-9 MLS lift Unity Checks

MLS LIFT	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Rails	0.97	0.68	0.97
Rails diagonal	0.33	0.23	0.33

Table 5-10 Non-check Unity Checks

NON-CHECK	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Gantry beam	0.09	0.09	0.09
Tuban beam	0.01	0.01	0.00

5.2.6 Deformations

The maximum deformations of both configurations can be seen in Figure 5-11 and Figure 5-12. Table 5-11 quantifies the deformations. The maximum deformations occurred during lifting the MLS section position 1 for both configurations. For both configurations the end of the rail deformed the most. This element is 28.5 meters from the support. So, a deformation ratio of 257.2 / 28,458 = 0.009. This is the deformation of the climbing frame only. The deformation of the MLS towers is not included.

Table 5-11 Deformations climbing frame

DIRECTION	CONFIGURATION 1 DEFORMATION [MM]	CONFIGURATION 2 DEFORMATION [MM]
Total	186.9	257.2
X direction	129.8	212.9
Y direction	102.6	102.6

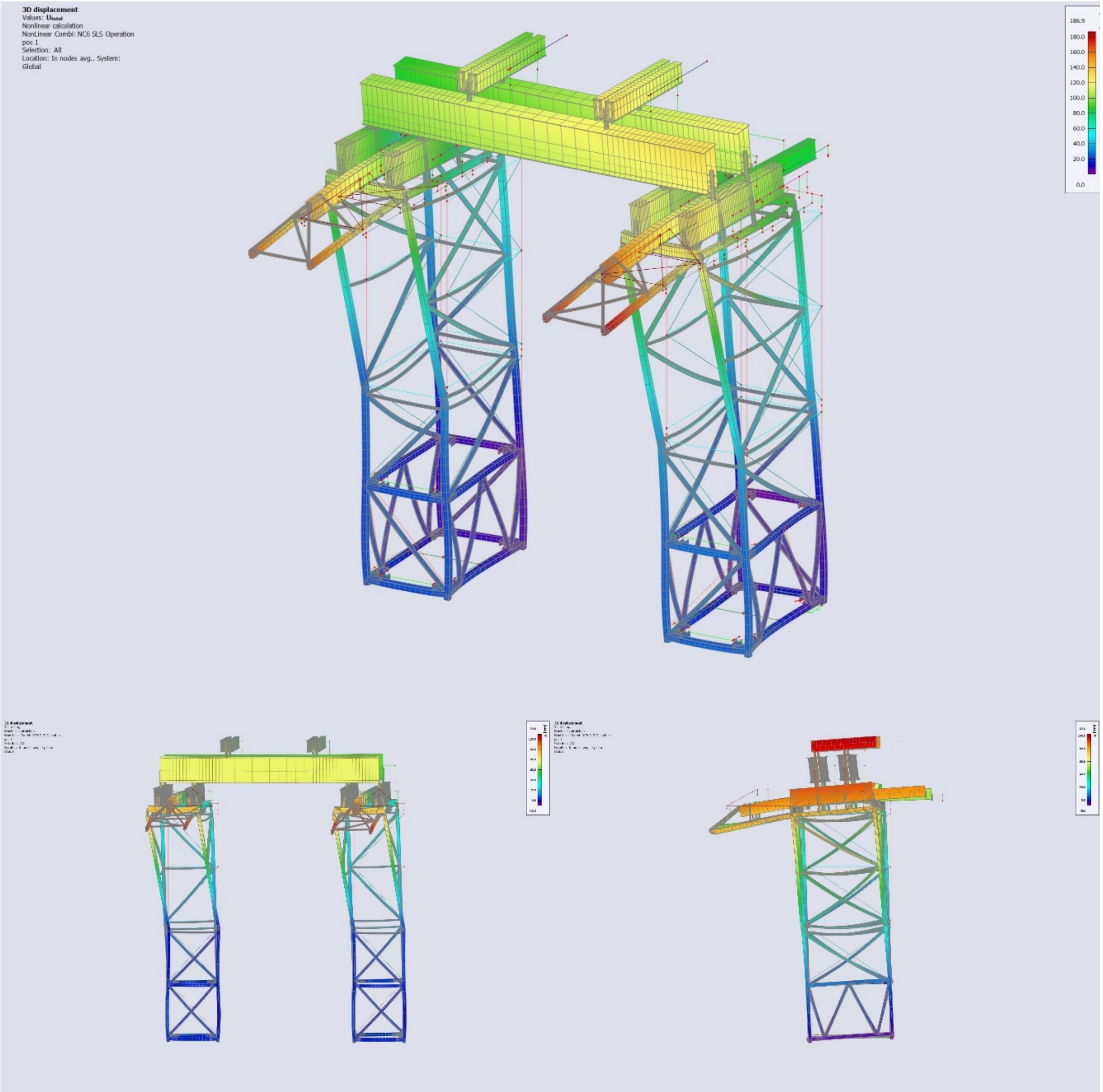


Figure 5-11 Deformations climbing frame, configuration 1

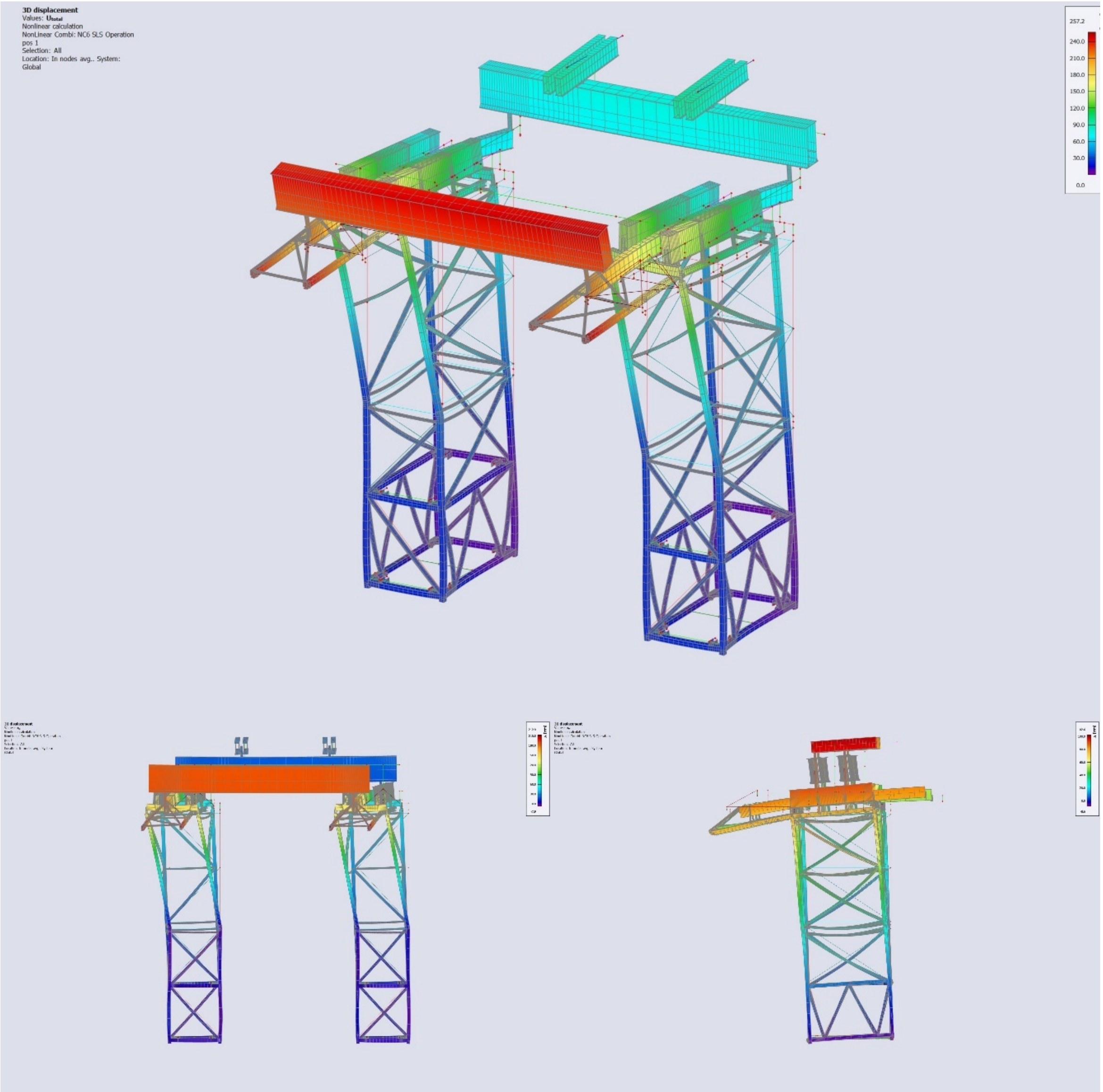


Figure 5-12 Deformations climbing frame, configuration 2

5.2.7 Reaction forces

The maximum reaction forces for both configurations are shown in Table 5-12. Annex D.2.7 shows the reaction forces on all supports. It can be seen that each climbing cage has only two vertical supports. The supports in Y direction only have compressive reaction forces. The supports in X direction can concur forces in both directions. This matches the conditions mentioned in Chapter 5.2.2.

Table 5-12 Maximum reaction forces final design

DIRECTION	COFIGURATION 1	CONFIGURATION 2
X	1778 kN	1672 kN
Y	2451 kN	2218 kN
Z	7047 kN	6813 kN

5.3 CONNECTIONS

The critical connection needs to transfer the reaction forces of the previous section to the towers. This connection is designed in detail. Other connections have been reasonedly assumed to be feasible, when properly designed.

Elements that, when assembled, do not fit in a container must make use of pinned connections, according to requirement R12.3. Therefore, most of the connections are pinned. The connection of the stability beam to the bottom horizontal X brace is welded. However, when connected, this element fits in a container.

5.3.1 Climbing cage to tower connection

As mentioned before, the guidance edge on the MLS chords will be used to transfer the horizontal loads, both in X and Y direction. Figure 5-13 shows this connection in 3D, it is a schematic overview generated by SCIA, Table 5-13 explains what beam is what colour. The free end of the stability beam is connected to the MLS chord. As the climbing frame must slide over the tower this connection is a pressure only connection. A detailed top view drawing can be seen in Figure 5-14. The connections are modelled as hinges in SCIA, this cannot be seen in the Figure.

The connection is calculated using hand calculations based on Eurocode EN 1993-1-8 welds and pin connections.

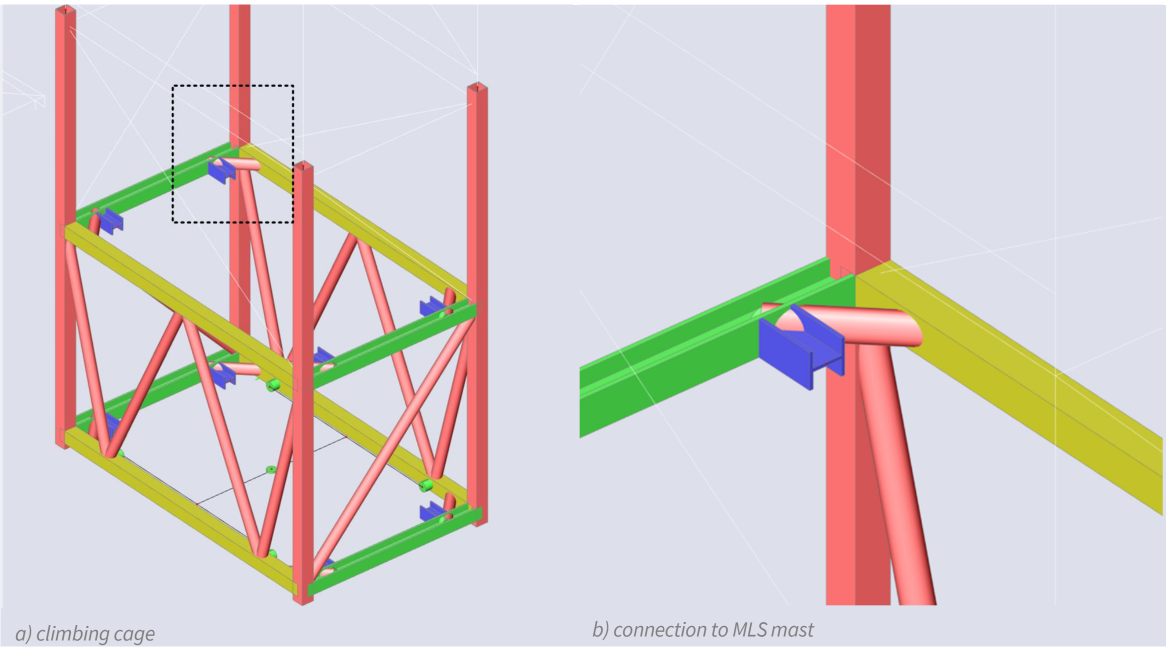


Figure 5-13 3D render climbing cage to MLS chord connection, made by SCIA

Table 5-13 Corresponding colours to beams in Figure 5-13

MEMBER	COLOUR IN FIGURE 5-13 B)
Stability beam	Blue
Bottom horizontal X brace	Green
Bottom chord	Red/ pink
Bottom diagonal brace	Red/ pink
Bottom horizontal Y brace	Yellow

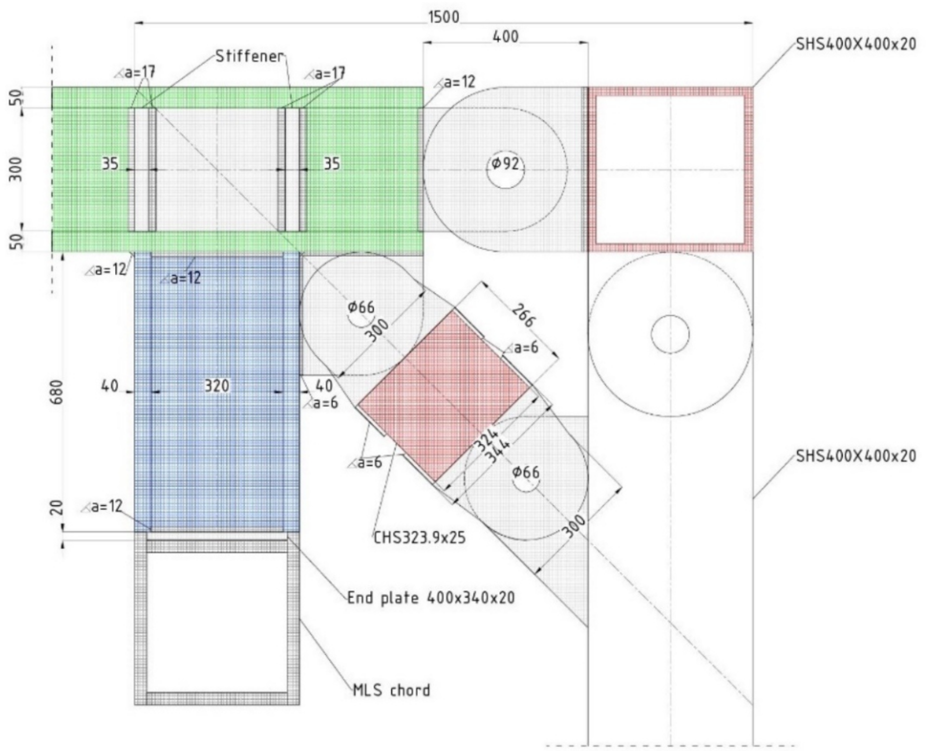


Figure 5-14 Top view total connection

5.3.1.1 Stability beam to MLS chord

Figure 5-15 a) shows the stability beam in detail. The flanges are in contact with the guidance edge flanges. Here the force transfer in Y direction will be assured. The force transfer in X direction will happen via the end plate that is welded to the webs (and a part of the flanges) of the stability beam. The plate is solely meant for force transfer in X direction because of the strength of the MLS chord. Annex D.3 validates both the pressure only connection and the MLS chord, globally and locally. The forces that need to be transferred are:

- 1778 kN in X direction
- 2451 kN in Y direction

5.3.1.2 End plate to stability beam

The end plate is welded to the end of the stability beam, see Figure 5-15 b). The weld is checked in Annex D.3.1.4. Annex D.3.1.4 also contains the check for the shear load on solely the web of the stability beam. The force that needs to be transferred are:

- 1778 kN in X direction

5.3.1.3 Stability beam to bottom horizontal X brace

This is a welded connection with stiffening plates in the bottom horizontal X brace. The web of the bottom horizontal X brace must be thickened because of shear stresses accumulation. All can be seen in Figure 5-15 c), all is validated in Annex D.3.1.5. The forces that need to be transferred are:

- 1778 kN in X direction
- 2451 kN in Y direction
- 1173 kNm around Z axis

5.3.1.4 Bottom horizontal X brace beam to column

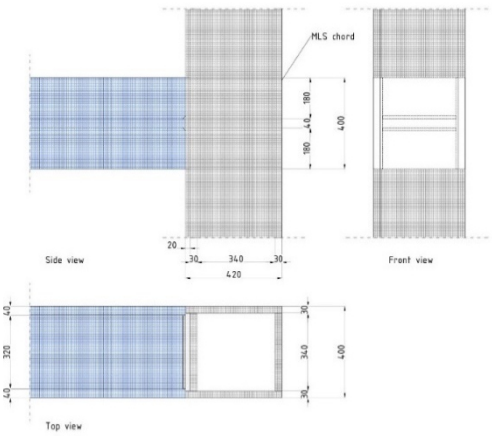
The connection between the bottom horizontal X brace beam to the column is a pinned connection, see Figure 5-15 d). See Annex D.3.1.6 for the validation. The forces that need to be transferred are:

- 1429 kN in X direction
- 940 kN in Y direction

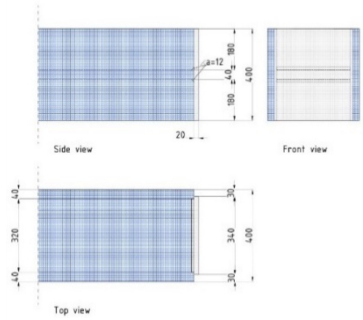
5.3.1.5 Bottom diagonal brace to stability beam and bottom horizontal X brace

The bottom diagonal brace has two slotted-in tongue plates. These plates surround an end plate that is welded to both the stability beam and the bottom horizontal X brace. The end plate and the tongue plate are connected via a pin. All this can be seen in Figure 5-15 e). See Annex D.3.1.7 for the verification of the connection. The force that needs to be transferred are:

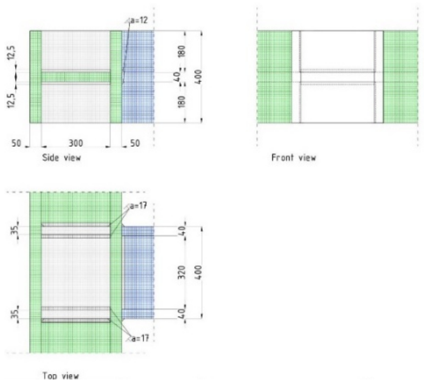
- 943 kN in x direction



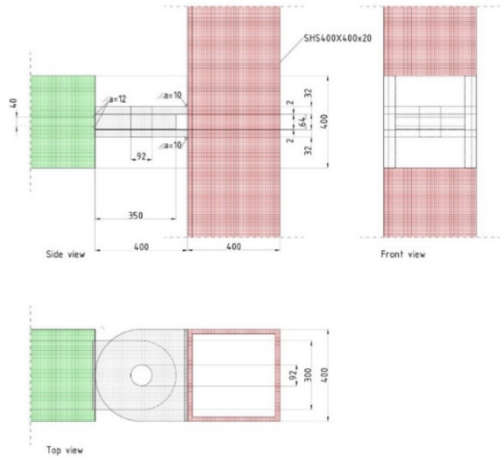
a) Stability beam to MLS chord connection



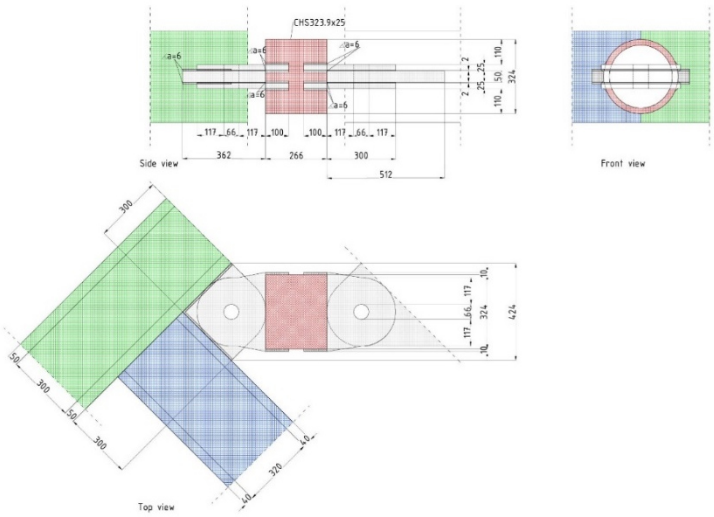
b) End plate to stability beam connection



c) Stability beam to bottom horizontal X brace connection



d) Bottom horizontal X brace to bottom column connection



e) Bottom horizontal diagonal brace to stability beam and bottom horitzontal X brace connection

Figure 5-15 Individual connections

5.3.2 Dywidag connections

The top frame members are connected via dywidag connections. All dywidag connections are formed around the MLS top frames. The gantry beam and the guidance beam are connected to that member. The highest tensile force occurs when the climbing frame hangs on the MLS top frames. This force is 2314 kN, which is shared over four dywidag bars. Meaning that a Mammoet dywidag bar of 36 mm nominal diameter will suffice [6].

5.3.3 MLS connections

Most remaining connections are like the connections designed for the MLS masts. It is assumed that they can be designed strong enough.

5.3.4 Rails

The rails will be connected via bolts to the top flange of the top frames Y. This is needed as the beam trolley needs to be able to roll over the total length of the beams.

5.4 MECHANICAL DETAILS

Two important mechanical details are the MLS hoisting system and the vertical climbing frame. Both will be elaborated in this Chapter.

5.4.1 MLS hoisting system

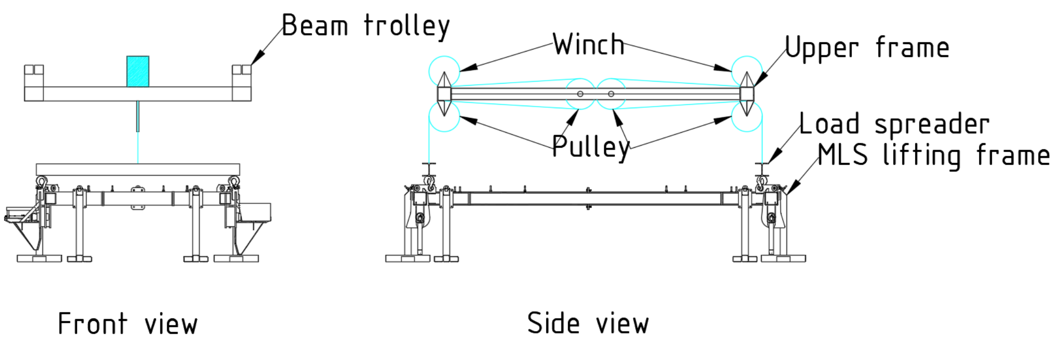


Figure 5-16 MLS hoisting system

5.4.1.1 Lifting

The height of the MLS mast and the rigging determines the height of the opening in the climbing frame. This height should be minimized. Therefore, two lifting frames are being used. The upper lifting frame is attached to the rails (via beam trolleys). This frame will not translate in vertical direction. The upper lifting frame is equipped with two winches. The winches are connected to a load spreader that is connected to the lower lifting frame. This lower lifting frame is the MLS lifting frame Mammoet has developed to lift an MLS mast section directly above the chord connection eyes. All this can be seen in Figure 5-16.

A strong rope without reeving is chosen per winch, as this increases the lifting speed. This fastens the (de)erection of the gantry.

Rigging stability

In X direction the rigging is stable since the load is picked up above the C.o.G. in one point [15], see Figure 5-16 (left). In Y direction the rigging is stable since the load is picked up above the C.o.G. with two independent winches, see Figure 5-16 (right). The latter also prevents rotations around the Z axis.

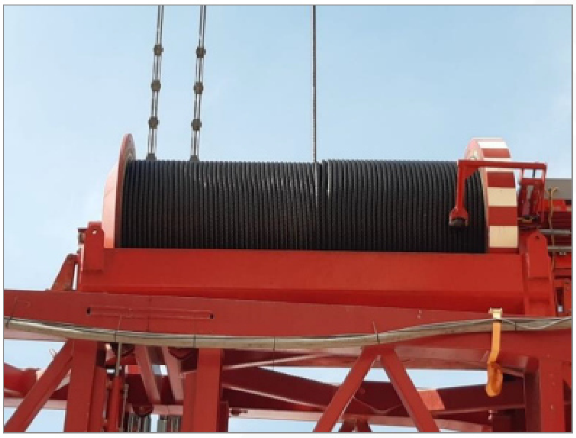
Two independent winches are used. This is preferred over four lifting points as the loads in each of the lifting points cannot be known if one lifts from four points. Then, it must be assumed that only two of the four points are able to lift the load, meaning that all four lifting points are overqualified.

Load spreader

Since Mammoet developed and used the MLS lifting frame it is assumed to be strong enough. Therefore, the load spreader is the first to verify. An HEB300 beam would suffice as load spreader, for the verification see Annex D.4.1.1.

Upper lifting frame

The rope must travel from the winch to the middle of the frame, over a turning disk that can translate in X direction, back to the end of the frame, over a steady disk. This to deal with the winding of the wire over the winch drum. This does not happen straight as the windings on the drum happen both over and next to each other, see Figure 5-17 a). Annex D.4.1.2 calculated the required length that is needed to lift from a static point.



a) Winch [15]

b) Verlinde geared trolley CHD20000-3-B [22]

Figure 5-17 Winch and beam trolley

5.4.1.2 Horizontal movement

The horizontal movement will be provided by beam trolleys. Each rail contains two beam trolleys that are spaced 8 meters apart. Mennens is a manufacturer of beam trolleys. Their 'Verlinde geared trolley CHD20000-3-B' has a capacity of 20 tons. This trolley is suitable for beams with flange widths ranging between 188 to 310 mm [22]. The trolley can be seen in Figure 5-17 b). By means of pulling the chain the trolley can ride back and forth [23]. The motor pulling the chain will be located at the upper lifting frame.

5.4.1.3 Available height and width

The total height of all elements combined is roughly 13.9 meters, the available height in the climbing frame is 14.3 meters. Meaning that 0.4 meters of margin is available. The width of the MLS mast section is 4.6 meters. The available width in the climbing frame is 5.8 meters. Meaning that 1.2 meters of margin is available. The maximum deformations, see Chapter 5.2.6, are within both margins, meaning that the mast sections can be inserted during the whole climbing operation. This is detrimental to the functionality of the design.

5.4.1.4 Guidance

When lifting a load close to another object, like the gantry towers, it needs guidance. The distance between the MLS mast and the gantry tower is 2.5 meters. With 2% side load at a height of +/- 150 meters guidance is needed for +/- 30 meters, after that it is not necessary.

5.4.2 Vertical moving frame

Figure 5-18 shows the vertical moving frame. Starting at one winch, the rope travels to the pulley on the vertical moving frame, it travels back down to the pulley, above the winch. After completing seven reevings it travels to the pulley on the other end of the frame. Here seven reevings are done too. Finally, the rope is anchored to the bottom of the climbing frame. Annex D.4.1.1 calculates the reeving. Annex D.4.1.2 validates the vertical moving frame.

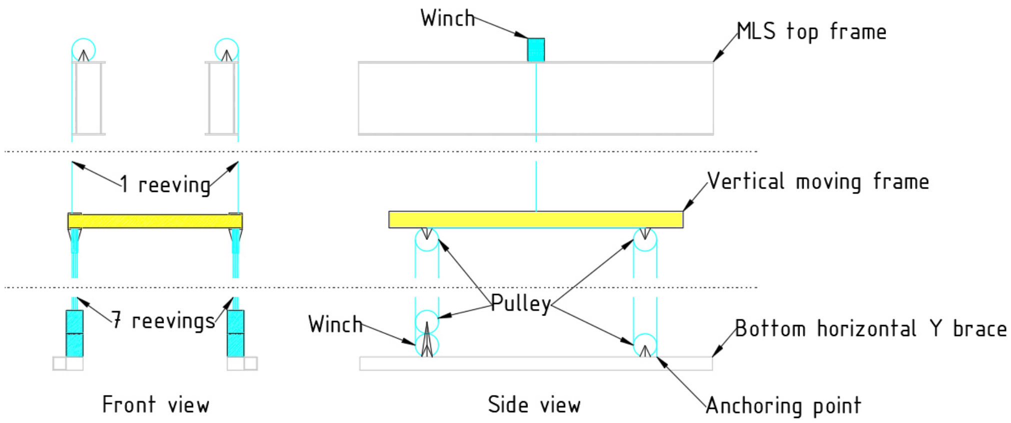


Figure 5-18 Vertical moving frame with reeving

5.5 ASSEMBLY AND DISASSEMBLY

The starting point of the assembly will be arriving at an empty building site where a vessel needs to be lifted. First the climbing cages will be assembled, this process will be similar to that of the assembly of an MLS mast. A hydraulic assist crane will be needed for this assembly. When the cages are assembled, the upper structure can be installed on top. First, the top frame Y beams. On top of them the MLS top frames and the guidance beam will be installed. The rails will be installed next. This will need some assistance because the cantilevering rails will need to be connected to the guidance beam with the upper braces. When that is done, the gantry beams will be lifted on top of the MLS top frames. Finally, the strand jack beams, and accompanying lift equipment are installed.

Now the erection of the towers can begin. Via the lifting system the MLS sections will be lifted and rolled in the climbing cage. Climbing will happen via the vertical moving frame.

When the towers are erected, the gantry is ready to lift the vessel.

After the gantry has lifted the vessel, a skidding system jacks up both gantry beams and skids them in opposite Y directions. Then, the hoisting system will lift and roll the MLS mast sections out of the tower. After which the vertical moving frame will be used to climb down.

Assembling and disassembling the gantry requires a hydraulic crane. From the Annex A.1.1 it can be seen that an LTM1500-8.1 could lift the gantry beam. This operation requires a minimum radius of 8 meters, see Figure 5-19. This crane can lift the mast sections too. Figure 5-19 shows that the radius to provide the mast sections from a single location and relocate only once needs to be 11.6 meters. The relocation of the crane is after installing the first gantry beam.

Disassembling the gantry requires two other crane positions, these can be seen in the right bottom picture of Figure 5-19. Annex D.4 calculates that the climbing frame will not tip over after removing one gantry beam. If, like mentioned for determining requirement R14, it is not possible to access the rear of the gantry (which is the bottom side in Figure 5-19), the climbing frame can be put on SPMTs which turn the structure. By doing so, all elements can be picked-up from one side, see Figure 5-20. This does require more space in longitudinal direction.

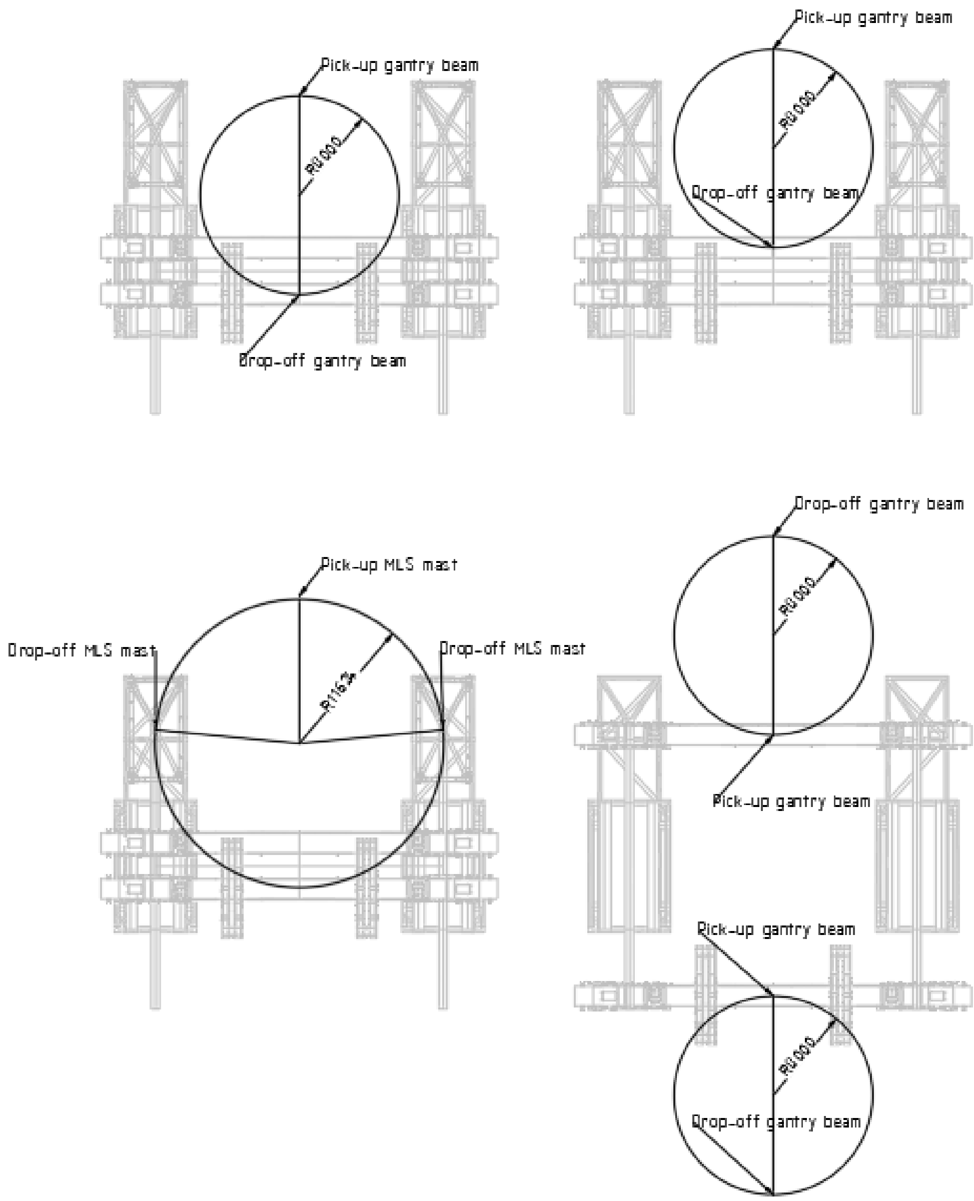


Figure 5-19 Crane positions final design

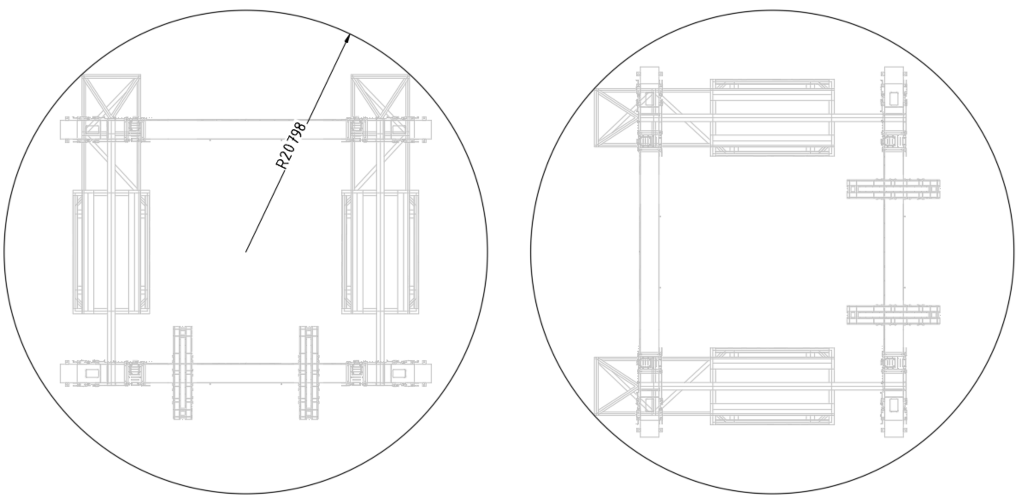


Figure 5-20 Radius final design on SPMT

5.6 DEMANDS

Annex D.4 calculates that the space required for the total operation, so the assist crane lifts and splitting the gantry, is approximately 1050 m^2.

Current prices for a ton of steel range between €750.- and €1000.- [24]. Meaning that an investment of €450,000 to €600,000 is needed for the raw material (600 Te). Moreover, the manufacturing of the beams adds costs. Also, the connections will have a contribution to the costs. Mammoet roughly calculates with a value of €10.- per kilo of steel, this includes raw material costs, fabrication, connections, and motion controls. Applying this over the self-weight results in an investment of €6,000,000.-.

According to Mammoet, they can erect one MLS mast section in approximately 6 hours, where this can be reduced when a trained crew is doing the job [11]. It is assumed that two MLS masts can be erected in one day. One climbing cage is roughly twice as big as one MLS mast section the duration is assumed to be one day per climbing cage. Assuming five days will be needed for the other beams and rails to be installed. Meaning that the total frame can be ready in seven days. It is assumed that lifting two new MLS sections in place will take one day. Climbing eleven meters is also assumed to take one day. If two MLS sections can be assembled per day, it will take 31 days to erect the whole gantry.

6 CONCLUSION

In this Chapter the main question will be answered. The main question was:

Which novel design is capable of (dis)assembling a gantry without the need for a large crane?

The most promising novel design for (dis)assembling a gantry without the need for a large crane is by using a climbing frame. The paradox of using a gantry to omit the use of a large crane but, in order to assemble and disassemble the gantry, a large crane is needed, can be broken. Because, by using this design for the (dis)assembly of a gantry, only a hydraulic crane is needed. The climbing frame itself is able to lift and install a new mast section. After installation the frame can climb on this new mast section to lift and install the next mast section. This omits the use of crawler cranes.

Compared to the crawler crane that was needed to perform the critical lift for the Jubail gantry, this design saves roughly 2100 m^2 of space. This is a reduction of 65%. Approximately €6,000,000.- is needed for the realization of this design. Meaning that during the ninth project the costs for the crawler crane are earned back. It is estimated that the assembly of the climbing frame and the erection of the gantry is comparable to the assembly of the crawler crane and the erection of the gantry. Meaning that it saves space and money while maintaining the current installation speed.

What is new about this design is applying a combination of existing best practice solutions on a larger scale. Lifting and inserting new mast sections and climbing along with the tower is known from tower cranes. And the climbing procedure is inspired by the FOCUS crane from Mammoet. However, this design can handle mast sections that are twice the size and weight.
Getting the upper structure out of the way was never a problem for the current way of working. However, for some alternative (dis)assembly procedures this was. The solution of using a guidance beam equipped with skid shoes is an innovative way of getting the upper structure out of the way.

The validation of the final design follows from hand calculations and a finite element model. Fundamental design features, such as the climbing system, the MLS hoisting system, and skidding of the upper structure are designed and validated too. Making that a proof of concept can be concluded.

The final design can be seen in Figure 6-1. This frame consists of two climbing cages (1.) that are connected via the gantry beams (2.). Each climbing frame is equipped with a hoisting system (3.) that is used to lift and roll the new MLS mast sections in place. The two towers that form the gantry will be erected simultaneously with the total upper structure on top. Climbing will happen via the vertical moving frame (4.). This frame is equipped with retractable pins that grab on jack-up blocks that are present on the MLS mast sections. The vertical moving frame can translate in vertical direction by means of winches that are connected to the climbing frame. It was required that the disassembly procedure happened using the same system as the assembly procedure. This would be impossible if the upper structure is still over the vessel. Therefore, after the gantry lifted the vessel, the gantry beams must split. To split the gantry beams, two cantilevering guidance beams (5.) are present. The climbing frame is designed such, that when the gantry is in its final position, the Jubail gantry design is established. A case study of this gantry was performed as it was the highest self-standing gantry Mammoet built. The only difference is that the self-weight of the climbing frame is acting on the gantry too. Meaning that solely the original MLS top frame beams (6.) and the added self-weight are decisive in the validity of the gantry's main purpose; lifting the vessel. This also means that the climbing frames are solely meant for climbing.

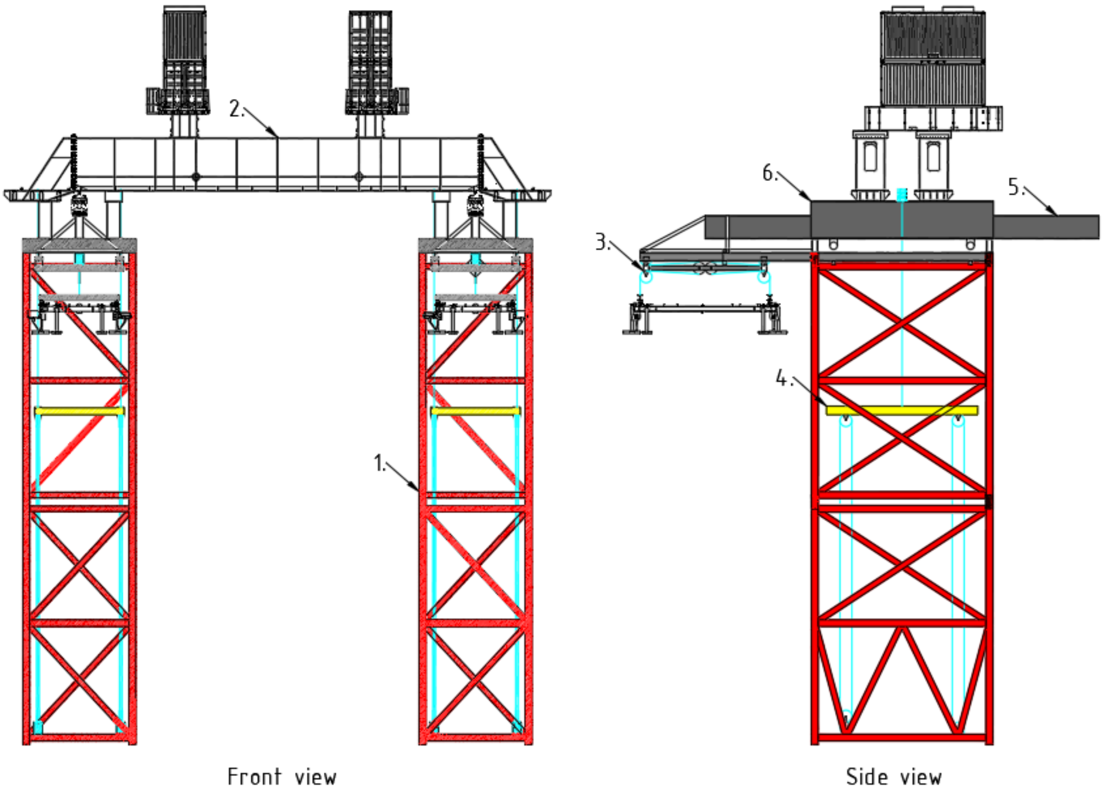


Figure 6-1 Final design climbing frame

The design is established via the following design route. First, a set of requirements is put together. This set consists of requirements that follow from the Jubail case study, the crane capacities of Mammoet cranes, and the ways of working within Mammoet. The total gantry life is divided into five stages. The stages are; tower erection, raise upper structure, upper structure out of the way, lower upper structure, and tower de-erection. Best practice solutions were developed for each stage. Based on the requirements, a selection of best practice solutions was considered in a morphological chart. This chart was used to develop seven concepts. Via a qualitative multi criteria analysis, containing six design criteria, three concepts scored within 10% of each other. A second round of assessment concluded that the best concept is a climbing frame at the top that splits the gantry beams after the lift.

7 DISCUSSION

Starting the discussion of this thesis is the scope limitation. The other duties of the assist crane on the building site are not considered. If a crawler crane is needed for other lifting jobs, it can be planned such that the (dis)assembly of the gantry will not have a major impact on the planning and budget of the total project. For example, the vessel needs tailing during the installation. A crawler crane can be used for that, meaning that one already has such a crane on site. The need for this design will be limited by the other duties of the assist crane. However, with this design one does not depend on these situations. It provides freedom in choosing a tail frame on an SPMT, if that is preferred. Moreover, applying this design enables working at denser locations as it requires less space than the current solution.

Another scope limitation is that this design is a tailored solution for a portal-like, self-standing gantry based on a case study. This includes specific ways of working and previous design choices within Mammoet. The set of requirements can be different for the other gantries or similar structures, which will result in a different selection of best practice solutions that form the morphological chart, leading to other possible concepts. However, the starting point of the route to come to a final design can be applied in a wider range of gantries and similar structures. The current set of best practice solutions is an (almost) complete set of possibilities, with accompanying rationale why it is not (yet) possible/ considered. Meaning that in the future one can perform the same analysis with a different set of requirements.

Doing a qualitative multi criteria analysis is inherently subjective. The outcome can differ depending on the (group of) person(s) assigning the weight factors and determining the scores for the concepts. The subjectivity is decreased by considering multiple people for the determination of the weight factor and quantitative reasoning (where possible) behind the scoring of the concepts. Moreover, a second round of assessment is performed where the top three concepts were evaluated on their differences. Comparing the pros and cons determined the best concept.

The best concept contains, on a local scale, a different set of functions. To get to the best design for this concept the whole process had to be repeated on a local scale. So, a new set of requirements, a new set of best practice solutions, etc.. Now, multiple design choices are based on anecdotal arguments. The focus for the final investigation phase was aimed at proving the concept, rather than a new design process.

The proof of concept is partly based on assumptions, for example the similarities in connections Mammoet already designed. This means that the design is a preliminary design. To become a final design each element must be designed in detail and checked by another engineer.

Based on the final design more reliable estimations about the spatial, monetary, and durational aspects of this design can be given. The values mentioned in this thesis are meant as an indication.

8 RECOMMENDATIONS

As mentioned in the discussion, some concepts were not considered based on a qualitative multi criteria analysis. To make a fair comparison between the concepts they require to be worked out like it has been done for the final design. Therefore, it is recommended to investigate other solutions to the problem. Promising solutions according to this thesis are; the climbing crane, a climbing frame that remains at ground level, and a system that translates the whole gantry.

To come to the best design, a new (local) design process needs to be carried out for each individual concept. This design process includes a new set of requirements, a new set of best practice solutions, and a new morphological chart. It might be that climbing via gears or jacks, results in a better concept.

The first improvement for this design is a model that includes the towers of the gantry. This will give necessary insight into the interaction between the tower and the climbing frame. Also, the exact loads need to be determined. Now, a conservative factor to consider fabrication weight and a conservative factor for the side loads has been used. Moreover, the added motion controls add weight to the system. All these need to be considered. Next to that, all connections need to be designed instead.

It is advised to investigate securing a clamped connection after the gantry beams have split. This configuration is decisive in the design. If a clamped connection can be secured both ULS and SLS results will turn out more favourable.

The guidance beam and the columns of the climbing cage are larger than eleven meters, meaning that they do not fit in a container. To comply with requirement R12.4 it should be investigated whether they can be designed such that they would fit.

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ANNEX A.

A.1. GANTRY - GENERAL

A.1.1. GUYS

If a gantry is not inherently stable guy lines, or guys, can be used. As said before, a guy is a wire that is attached to either the top or along the length of the towers. This limits the sway of the towers and decreases the buckling length. A counterweight or the foundation is attached to the other end of the wire. By using counterweights, the horizontal force is transferred by friction. The force in the guy changes the friction resistance, so special care must be taken in the design. Both methods require more workspace [6].

Active and passive guys can be used. The difference being that active guys have a variable length whereas passive guys have a fixed length. An active guy has a lifting device, which is attached to either the tower or the fixed point on the ground. The latter is preferred as it is more accessible. They can be used during the erection of a self-climbing gantry. Moreover, they can be pre-tensioned. If not needed, a passive guy is less expensive. Both types can be definitive or temporary [6].

A.1.2. MAST SECTIONS

The towers of the gantry are built out of mast sections. These are standard sections that can be stacked to reach the required height. By increasing the height, the capacity decreases. This is due to global stability problems. Multiple different mast sections are used in Mammoet. However, the most used are the MSG (Mammoet Sliding Gantry) and the MLS (Mammoet Lattice Sections).

A.1.2.1. MSG mast

The MSG mast is the most widely used mast section within Mammoet. Three MSG mast section types exist, see Figure A-1. Type one's outer dimensions are 2430 x 2570 mm, and it has a length of 5700 mm. These measures are standard container sizes. Type three is similar, however, twice the length. Type two is slenderer compared to the other two. Besides the different types, also a (partly) double tower configuration is possible. Here two mast sections are placed next to each other and connected via a coupling box [4]. It can be stacked by adding an adapter. The chords of all types have steel grade S690 QL. The braces are of steel grade S355 J2H. Types one, two and three respectively weigh 7.1, 6.5 and 12.9 tons [25].

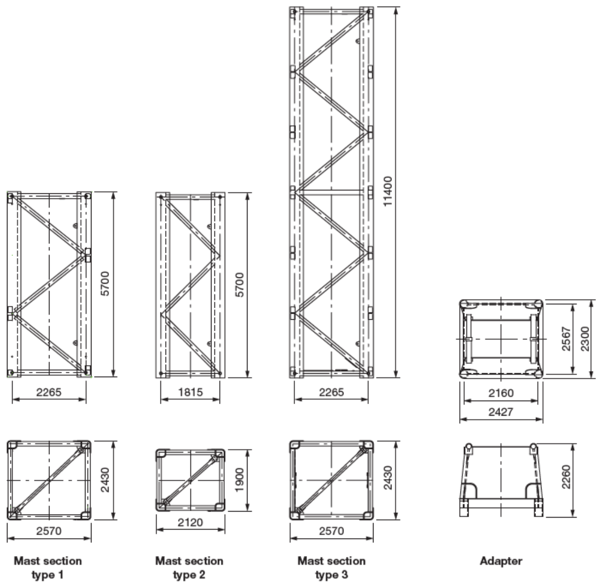


Figure Annex A-1 MSG mast sections [25]

A.1.2.2. MLS mast

A stronger mast type is the MLS mast, see Figure A-2. This mast section needs to be assembled and disassembled when needed. This increases the erecting time to one section per day [4]. This mast section also comes in three types. All are 11,000 mm long. Type one has its outer dimensions set at 4000 x 4000 mm. Type two 4000 x 8000 mm, or 8000 x 4000 this depends on the orientation. Type three is 8000 x 8000 mm. Like the MSG mast sections, the chords have steel grade S690 QL and the braces have S355 J2H [26]. Types one, two and three respectively weigh 35.6, 38.7 and 41.8 tons [10].

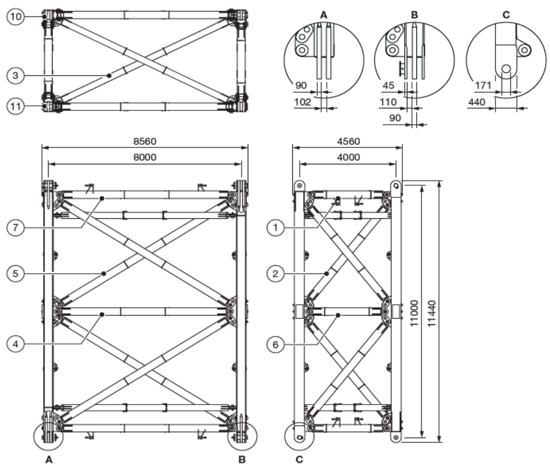


Figure A-2 MLS mast section 4000 x 8000 [10]

A.1.3. Gantry beam

Like the mast sections, lots of different gantry beams exist differing in length and capacity. Both in one piece and modular. The beams are usually double profile beams to be much more resistant to torsion and have double the resistance in shear and bending, compared to a single profile beam [6]. Currently the biggest gantry beam in use by Mammoet is the Kvaerner beam, see Figure A-3. It is 35 m long, has a moment of inertia of 1.15 m^4 around the strong axis and weighs 175 tons. To lift the heaviest loads two gantry beams can be placed next to each other to double the capacity.

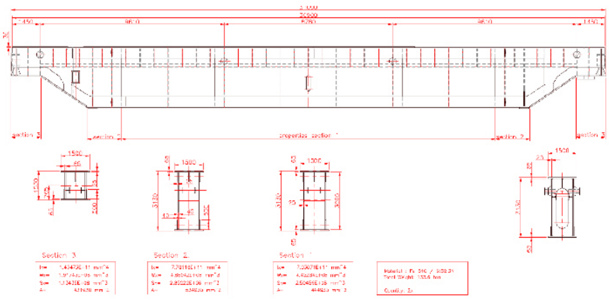


Figure A-3 Kvaerner gantry beam (old configuration) [4]

A.1.4. Lifting device

Mainly strand jacks are used as lifting devices for a gantry. They are compact and have a high lifting capacity [27]. Twisted wires form strands of 15.7- or 18.0-mm strands. Several strands can be combined to lift up to 900 tons with one strand jack. By combining strand jacks the capacity increases. Figure A-4 shows the set-up of a strand jack.

1. Lifting block
2. Strand jack
3. Strands
4. Strand reel or guide
5. End block

The workings of a strand jack are shown in Figure A-5. When lifting the upper wedges (shown in red) are closed, so that they clamp the strand. The upper anchor head (shown in blue) is pushed by hydraulic power. When the head is at its highest point the lower wedges close and the upper wedges open. The upper anchor head goes down and is ready for a new lift [6]. This process continues until the whole lifting operation is done. With increasing load, the lift speeds decreases. DaSTec and PAUL strand jacks with a capacity of 640 tons have a speed of 11.2 m/h [28].

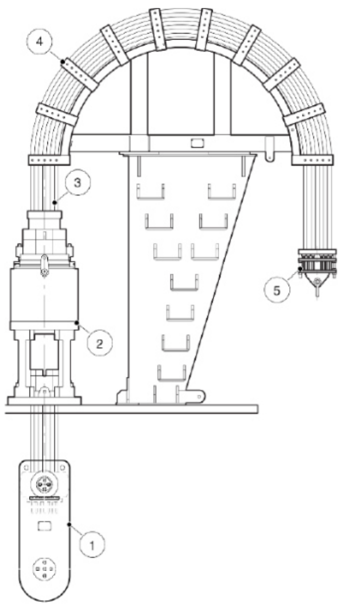


Figure A-4 Strand jack [6]

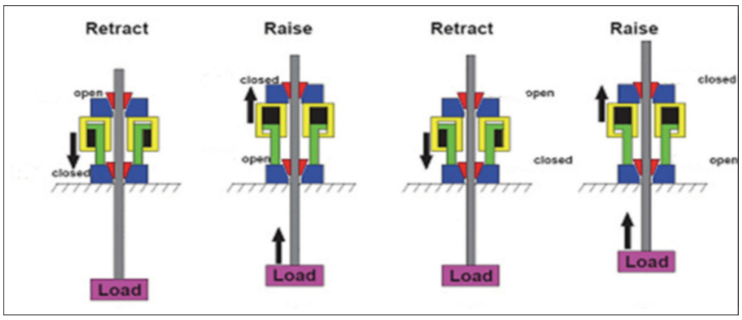


Figure A-5 Strand jack workings [48]

A.1.5. Skidding system

Since strand jacks are only able to execute a one directional movement they can be placed on skidding systems. A skidding system is a track on which a skid shoe is assembled, the strand jack rests on the skid shoe. By means of a hydraulic cylinder the shoe is pushed over the track. The cylinder locks itself in the track, then it extends and pushes the shoe. After pushing it retracts towards the new locking position. This continues until the load is at the desired place. Pulling the load over the track is possible, however, the capacity of the cylinder is half the pushing capacity. The weight of the load results in friction. To minimize the friction force Teflon pads in combination with stainless steel and lubricant is used [6]. Skidding can generate horizontal forces in the gantry.

A.1.6. Powerpacks

To power all hydraulic systems used in the structure a hydraulic powerpack is needed. Powerpacks regulate pressure, this can only be done when the pack is stable [6].

A.2. CASE STUDY

A.2.1. MLS mast capacities

Table A-1 MLS member capacities [11]

MEMBER	CAPACITY [KN]	CRITICAL CHECK
Chord	23,805	Chord buckling
Chord	16,200	Tension on chord eye
Horizontal brace 4 m	3290	Member buckling
Diagonal brace 4 m	3390	Pin bending
Horizontal brace 8 m	2555	Member buckling
Diagonal brace 8 m	2789	Member buckling

A.2.2. Total weight upper structure

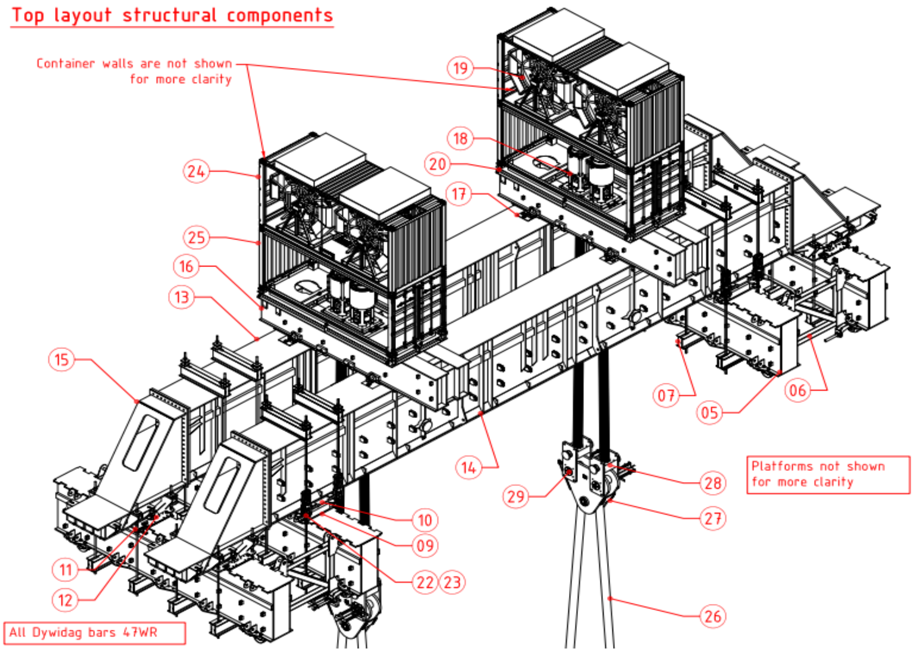


Figure A-6 Upper structure Jubail gantry [51]

Table A-2 Total weight upper structure

NUMBER	DESCRIPTION	QUANTITY	WEIGHT (EACH)	TOTAL KG	TOTAL WEIGHT
5	MLS top beam (incl. connection eyes)	4	21,060	84,240	456.97 Te
6	MLS top frame bracing 4000	4	1,140	4,560	
7	Dywidagbeam L = 2.1 m	28	485	13,580	
9	Swivel Topframe -GB P55 Lowerblock	4	2,890	11,560	
10	Swivel Topframe -GB P55	4	2,660	10,640	
11	Connection plate	2	200	400	
12	Connection plate gantry beam	8	115	920	
13	PDV gantry beam T38	1	75,700	75,700	
14	PDV gantry beam T44	1	77,600	77,600	
15	PDV gantry beam extension	4	11,900	47,600	
16	Strand jack support beam	2	16,500	33,000	
17	Connection plate Tuban beam	4	350	1,400	
18	Strand jack SSl830 Cap. 900 Te Type 4	4	4,500	18,000	
19	Strand reel dia2.9m	8	669	5,352	
20	Support Frame 20' container P55 Type-1 L=6.12m	2	3,036	6,072	

21	Strand wire dia18	120	261	31,320
22	Connection dywidag cap. 60 Te	16	26	416
23	Connection dywidag cap. 60 Te (mod)	16	26	416
24	20' Open top container modified	2	2,128	4,256
25	20' high cube container	2	2,236	4,472
26	Grommet 1203 Te 10.00 m	2	4,480	8,960
27	Lifting beam Cardan triangle plate -6	2	5,000	10,000
28	Anchor blocks SSL830 type 1	4	800	3,200
29	Lifting link cap. 450 Te EWL 0.75 m	8	413	3,304

A.2.3. TOWER CALCULATIONS

A.2.3.1. Properties

Table A-3 MLS tower properties

MLS TOWER		
Property		**Unit**
A_{tower}	177,600	mm^2
h_{tower}	8000	mm
b_{tower}	4000	mm
$I_{y,tower}$	$2.8*10^{12}$	mm^4
$I_{z,tower}$	$7.1*10^{11}$	mm^4
$W_{el,y,tower}$	$7.1*10^{8}$	mm^3
$W_{el,z,tower}$	$3.6*10^{8}$	mm^3
E	210,000	N/mm^2
L_{tower}	132,000	mm
$f_{y,tower}$	690	N/mm^2
γ_{M0}	1	-
γ_{M1}	1	-
γ_{M2}	1,25	-
G_{tower}	464.4 4555.8	Te kN

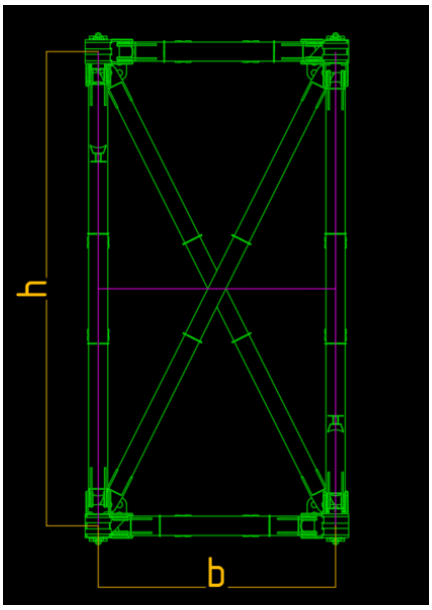


Figure A-7 MLS mast dimensions

A.2.3.2. Compressive resistance

From the buckling resistance of one chord the theoretical total load on the mast section is:

$$N_{Rk,MLS} = 4 * 23{,}805 = 95{,}220\ kN$$
$$N_{Rk,MLS} = 9706\ ton$$

A.2.3.3. Buckling

However, buckling needs to be verified too. The buckling resistance of the tower can be calculated via formula (1).

$$N_{Rk,buck} = \chi * N_{Rk} \tag{1}$$

Where:

- N_{Rk} is the compressive resistance of the member.
- χ is the buckling factor taking into account imperfections. This is calculated via:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \tag{2}$$

Where:

- ϕ is a helping factor, calculated by:

$$\Phi = 0.5\left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right) \tag{3}$$

Where:

- α is an imperfection factor that depends on the manufacturing process.
- λ is the relative slenderness, calculated by:

$$\bar{\lambda} = \sqrt{\frac{f_y * A}{N_{cr}}} \tag{4}$$

Where:

- N_{cr} is the critical buckling force. This force is calculated via:

$$N_{cr} = \frac{\pi^2 EI}{{L_{buck}}^2} \tag{5}$$

Where:

  - L_{buck} is the buckling length. This length depends on the boundary conditions of the member.

However, Steven Oomen found that another formula should be applied when a distributed load is applied along the column as well. For a clamped-free column, see Figure A-8, formula (6) should be used [26]:

$$F_V + \frac{G_{tower}}{3.17} \leq \frac{2.46 * EI}{H^2} \tag{6}$$

In general, this would be:

$$F_V + \frac{G}{\zeta} \leq \frac{\pi^2 EI}{{L_{buck}}^2} \tag{7}$$

Where:

- F_V is the vertical point load on the member.
- G is the self-weight of the member.
- ζ is the ratio between the buckling load along the tower (Q_{cr}) and the buckling load at the top (F_{cr}). Because for the tower like Figure A-8 the ζ would be:

$$\zeta = \frac{7.83 * 4}{\pi^2} = 3.17$$

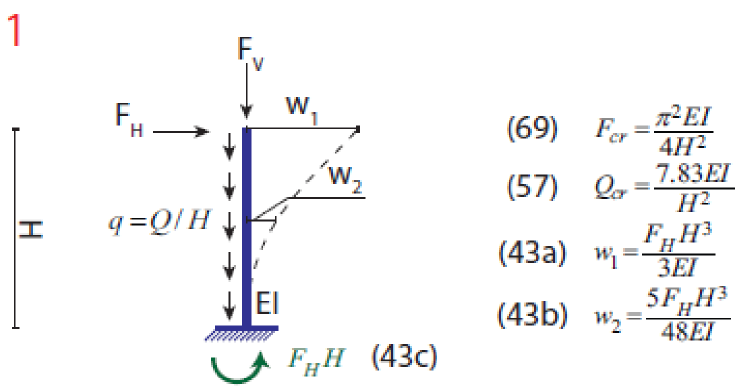


Figure A-8 Buckling system [26]

Here the imperfections must be taken into account too. It can be seen that the part after the equal sign is the buckling load. Thus, this should be reduced to $N_{Rd,buck}$:

$$F_V + \frac{G_{tower}}{\zeta} \leq N_{Rd,buck} \tag{8}$$

A.2.3.4. Around y axis

The upper structure generates stability. However, around this local y axis, thus the global X axis, this effect is neglected. So, only an analysis without upper structure must be done. The buckling length is twice the tower height because the mast can be simulated by the system in Figure A-8. The tower height is 132,000 mm, so the buckling length is 264,000 mm. The buckling curve for a welded box with relatively thick welds is curve c according to Table 6.2 from NEN-EN 1993-1-1 [29] [26]. The corresponding imperfection factor is:

$$\alpha = 0.49$$

This would result in the following:

$$N_{cr,y,tower} = 83{,}266\ kN$$
$$\bar{\lambda} = 1.21$$
$$\Phi = 1.49$$
$$\chi = 0.43$$
$$N_{Rk,buck,y,tower} = 52{,}433\ kN$$
$$N_{Rk,buck,y,tower} = 5342\ Te$$
$$F_V \leq 50{,}973\ kN$$
$$F_V \leq 5196\ Te$$

According to SCIA Engineer:

$$N_{Rk,buck,y,tower} = 80{,}000\ kN$$
$$N_{Rk,buck,y,tower} = 8155\ Te$$

A.2.3.5. Around z axis

Around the local z axis, the global Y axis, an analysis with and without upper structure must be done. Because the towers can be erected individually and simultaneously with the upper structure on top. The latter does add stability to the towers.

Without upper structure

Again, the buckling length is the tower height because the mast can be simulated by the system in Figure A-8. The buckling length is 264,000 mm.

This would result in the following:

$$N_{cr,z,tower} = 21{,}114\ kN$$
$$\bar{\lambda} = 2.41$$
$$\Phi = 3.94$$
$$\chi = 0.14$$
$$N_{Rk,buck,z,tower,without} = 17{,}345\ kN$$
$$N_{Rk,buck,z,tower,without} = 1768\ Te$$
$$F_V \leq 15{,}902\ kN$$
$$F_V \leq 1621\ Te$$

According to SCIA Engineer:

$$N_{Rk,buck,z,tower,without} = 20{,}800\ kN$$
$$N_{Rk,buck,z,tower,without} = 2120\ Te$$

With upper structure

The buckling length is the tower height because the mast can be simulated by the system in Figure A-9. The tower height is 132,000 mm.

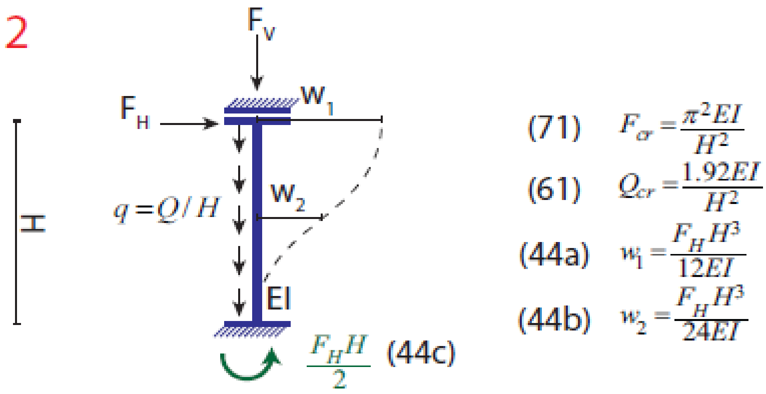


Figure A-9 Buckling system [26]

This would result in the following:

$$N_{cr,z,tower} = 84{,}456\ kN$$
$$\bar{\lambda} = 1.20$$
$$\Phi = 1.47$$
$$\chi = 0.43$$
$$N_{Rk,buck,z,tower,with} = 52{,}889\ kN$$
$$N_{Rk,buck,z,tower,with} = 5391\ Te$$

For the tower like Figure A-9 the ζ would be:

$$\zeta = \frac{1.92}{\pi^2} = 0.19$$
$$F_V \leq 29{,}471\ kN$$
$$F_V \leq 3004\ Te$$

According to SCIA Engineer:

$$N_{Rk,buck,z,tower,with} = 27{,}000\ kN$$
$$N_{Rk,buck,z,tower,with} = 2752\ Te$$

A.2.3.6. Bending moment

The bending moment capacity can be checked by:

$$M_{Rk,i} = f_y * W_{el,i} \tag{9}$$

This would mean:

$$M_{Rk,y,tower} = 690 * 7.1 * 10^8 * 10^{-6} = 489{,}900\ kNm$$

$$M_{Rk,x,tower} = 690 * 3.6 * 10^8 * 10^{-6} = 248{,}400\ kNm$$

However, it must be checked that the strength of the chords is sufficient. The formula to calculate the bending moment capacity based on the strength of the chords:

$$M_{Rk} = N_{Rk,buck,chord} * 2D \tag{10}$$

Where D is the distance between the chords. This results in:

$$M_{Rk,y,tower} = 23{,}805 * 2 * 8 = 380.880\ kNm$$

$$M_{Rk,x,tower} = 23{,}805 * 2 * 4 = 190.440\ kNm$$

The bending moment capacity is thus dependent on the strength of the chords.

A.2.3.7. Buckling and bending moment

According to Eurocode 1993 1-1 the combined Unity Checks are:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (11)$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (12)$$

First, some factors will be either zero or one:

- $\gamma_{M1} = 1.0$
- $\Delta M_{y,Ed} = \Delta M_{z,Ed} = 0$, because there is no shift in C.o.G. because the cross section is not class 4.
- $\chi_{LT} = 1.0$, because lateral torsional buckling does not apply for these mast sections as they are closed hollow sections [26].

Making the equations:

$$\frac{N_{Ed}}{\chi_y N_{Rk}} + k_{yy}\frac{M_{y,Ed}}{M_{y,Rk}} + k_{yz}\frac{M_{z,Ed}}{M_{z,Rk}} \leq 1 \quad (13)$$

$$\frac{N_{Ed}}{\chi_y N_{Rk}} + k_{zy}\frac{M_{y,Ed}}{M_{y,Rk}} + k_{zz}\frac{M_{z,Ed}}{M_{z,Rk}} \leq 1 \quad (14)$$

Here, k_{yy}, k_{yz}, k_{zy}, and k_{zz} are interaction factors. They can be calculated via two methods, in this thesis method two is chosen. That leads to the following formulas for k_{yy}, k_{yz}, k_{zy}, and k_{zz}, for elastic calculations:

$$k_{yy} = C_{my} * \left(1 + 0.6 * \bar{\lambda}_y * \frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}\right) \leq C_{my} * \left(1 + 0.6 * \frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}\right) \quad (15)$$

$$k_{yz} = k_{zz} \quad (16)$$

$$k_{zy} = 0.8 * k_{yy} \quad (17)$$

$$k_{zz} = C_{mz} * \left(1 + 0.6 * \bar{\lambda}_z * \frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\right) \leq C_{mz} * \left(1 + 0.6 * \frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\right) \quad (18)$$

Both formulas are similar, the only difference is the local direction, indicated by a y or a z. For the explanation the general term i is used. So:

- $C_{m,i}$ is the equivalent moment distribution factor around the local y-axis. Assuming a constant or linear moment distribution this value is calculated via:

$$0.6 + 0.4 * \psi \geq 0.4 \quad (19)$$

  Where:
  - ψ is the ratio between the bending moment at the bottom of the tower and the bending moment at the top of the tower.
- λ_i is the relative slenderness in local i-direction. This value is calculated via:

$$\lambda_i = \sqrt{\frac{A * f_y}{N_{cr,i}}} \quad (20)$$

  Where:
  - A is the area.
  - f_y is the yield strength.
  - $N_{cr,I}$ is the critical buckling force in local i-direction.
- N_{Ed} is the applied load, not yet known.
- χ_i is the reduction factor for buckling in the local i-direction.
- N_{Rk} is the resistance against axial compression.
- γ_{M1} is the material factor.

Assuming that only a constant bending moment distribution occurs and a clamped-free tower, the following holds:

$$\psi = 1.0$$
$$C_{my} = 1.0$$
$$C_{mz} = 1.0$$
$$N_{cr,y,tower} = 83{,}266\ kN$$
$$N_{cr,z,tower} = 21{,}114\ kN$$
$$\lambda_y = 1.21$$
$$\lambda_z = 2.41$$
$$\chi_y = 0.43$$
$$\chi_z = 0.14$$
$$k_{yy} = 1.0 + 1.79 * 10^{-8} * N_{Ed}$$
$$k_{yz} = 1.0 + 1.07 * 10^{-7} * N_{Ed}$$
$$k_{zy} = 0.8 + 1.43 * 10^{-8} * N_{Ed}$$
$$k_{zz} = 1.0 + 1.07 * 10^{-7} * N_{Ed}$$

When these factors are put in equations (13) and (14) along with the bending moment resistances $M_{Rk,y,tower}$ = 380.880 kN and $M_{Rk,z,tower}$ = 190.440 kN the results are:

$$U.C. = 2.46 * 10^{-8} * N_{Ed} + \frac{(1.0 + 1.79 * 10^{-8} * N_{Ed})M_{Ed,y}}{380{,}880{,}000{,}000} + \frac{(1.0 + 1.07 * 10^{-7} * N_{Ed})M_{Ed,z}}{190{,}440{,}000{,}000} \quad (21)$$

$$U.C. = 7.42 * 10^{-8} * N_{Ed} + \frac{(0.8 + 1.43 * 10^{-8} * N_{Ed})M_{Ed,y}}{380{,}880{,}000{,}000} + \frac{(1.0 + 1.07 * 10^{-7} * N_{Ed})M_{Ed,z}}{190{,}440{,}000{,}000} \quad (22)$$

Loads and moments are preferred to be plugged in as kN and kNm, this changes the equations to:

$$U.C. = 2.46 * 10^{-5} * N_{Ed} + \frac{(1.0 + 1.79 * 10^{-5} * N_{Ed})M_{Ed,y}}{380{,}880} + \frac{(1.0 + 1.07 * 10^{-4} * N_{Ed})M_{Ed,z}}{190{,}440} \quad (23)$$

$$U.C. = 7.42 * 10^{-5} * N_{Ed} + \frac{(0.8 + 1.43 * 10^{-5} * N_{Ed})M_{Ed,y}}{380{,}880} + \frac{(1.0 + 1.07 * 10^{-4} * N_{Ed})M_{Ed,z}}{190{,}440} \quad (24)$$

A.2.3.8. Resistances tower

Table A-4 MLS tower resistances

CHECK		UNIT
Compressive resistance	95,220	kN
Buckling around y	51,528	kN
Buckling around z (*without* upper structure)	15,902	kN
Buckling around z (*with* upper structure)	28,928	kN
Bending moment y	380,880	kNm
Bending moment z	190,440	kNm
Buckling and bending moment	See equations (23) and (24)	

A.2.4. GANTRY BEAM PROPERTIES

Figure A-10 shows the cross section. Note that this beam has cross-section class 4 [9]. So, the effective properties must be used.

Table A-5 PDV T38 gantry beam properties, effective

GANTRY BEAM PDV T38, EFFECTIVE		
Property		**Unit**
$A_{gb,eff}$	204,700	mm^2
$h_{gb,eff}$	3316	mm
$b_{gb,eff}$	1891	mm
$t_{f,gb,eff}$	38	mm
$t_{w,gb,eff}$	19	mm
$I_{y,gb,eff}$	$380.0*10^9$	mm^4
$I_{z,gb,eff}$	$65.6*10^9$	mm^4
$W_{el,y,gb,eff}$	$219.2*10^6$	mm^3
$W_{el,z,gb,eff}$	$69.5*10^6$	mm^3
E	210,000	N/mm^2
L_{gb}	23,794	mm
f_y	345	N/mm^2
γ_{M0}	1	-
γ_{M1}	1	-
γ_{M2}	1,25	-
G_{gb}	99.5 976.1	Te kN
$G_{gb+rig+hb}$	132.3 1297.9	Te kN

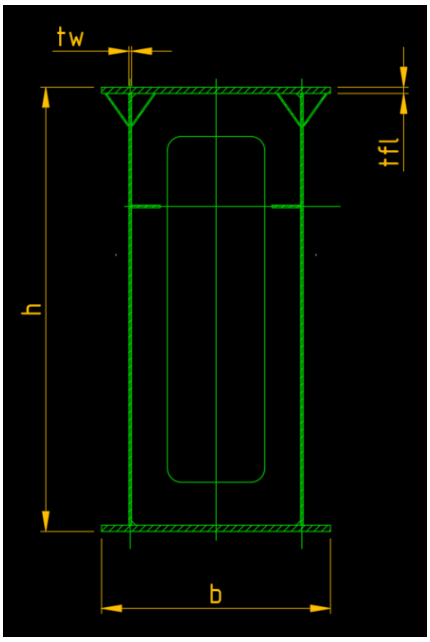


Figure A-10 Gantry beam PDV38 dimensions

Table A-6 Resistances PDV T38 gantry beam [9]

CHECK		UNIT
Compressive resistance	70,622	kN
Bending moment	75,624	kNm
Bending moment	23,988	kNm
Shear in Z direction	20,565	kN
Shear in Y direction	16,094	kN

A.2.5. Loads and load cases

A.2.5.1. Load cases

The load cases are displayed in Table A-7.

Table A-7 Load cases Jubail gantry [9]

NAME	DESCRIPTION
Load case 1	Self-weight
Load case 2	Secondary weight
Load case 3	Lifting load
Load case 4	Transverse load X-direction (1%)
Load case 5	Longitudinal load Y-direction (2%)
Load case 6	Wind on mast X-direction (14 m/s)
Load case 7	Wind on mast Y-direction (14 m/s)
Load case 8	Wind on load X-direction (14 m/s)
Load case 9	Wind on load Y-direction (14 m/s)
Load case 10	Global imperfection X
Load case 11	Global imperfection Y
Load case 12	Global imperfection XY

A.2.5.2. Load combinations

Combining the load cases from above results in the combinations in Figure A-11.

Name	Description	Type	Load cases	Coeff. [-]	Global imperfection	Load case
NC1	SLS -Operation base case	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC3 - Lifting Load	1.15		
NC2	SLS - Operational Dir. X	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC3 - Lifting Load	1.15		
			LC4 - transverse load X-direction (1%)	1.00		
			LC5 - Longitudinal load Y-direction (2%)	1.00		
			LC6 - wind on mast X-direction (14m/s)	1.00		
			LC8 - wind on load X-direction (14m/s)	1.00		
NC3	SLS - Operational Dir. Y	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC3 - Lifting Load	1.15		
			LC4 - transverse load X-direction (1%)	1.00		
			LC5 - Longitudinal load Y-direction (2%)	1.00		
			LC7 - wind on mast Y-direction (14m/s)	1.00		
			LC9 - wind on load Y-direction (14m/s)	1.00		
NC4	SLS - Operational Dir. X-Y	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC3 - Lifting Load	1.15		
			LC4 - transverse load X-direction (1%)	1.00		
			LC5 - Longitudinal load Y-direction (2%)	1.00		
			LC6 - wind on mast X-direction (14m/s)	0.71		
			LC7 - wind on mast Y-direction (14m/s)	0.71		
			LC8 - wind on load X-direction (14m/s)	0.71		
			LC9 - wind on load Y-direction (14m/s)	0.71		
NC5	ULS - base case	Ultimate	LC1 - Self Weight	1.35	None	
			LC2 - Secondary weight	1.35		
			LC3 - Lifting Load	1.73		
NC6	ULS - Operational Dir. X	Ultimate	LC1 - Self Weight	1.35	Deform. from loadcase	LC10
			LC2 - Secondary weight	1.35		
			LC3 - Lifting Load	1.73		
			LC4 - transverse load X-direction (1%)	1.50		
			LC5 - Longitudinal load Y-direction (2%)	1.50		
			LC6 - wind on mast X-direction (14m/s)	1.50		
			LC8 - wind on load X-direction (14m/s)	1.50		
NC7	ULS - Operational Dir. Y	Ultimate	LC1 - Self Weight	1.35	Deform. from loadcase	LC11
			LC2 - Secondary weight	1.50		
			LC3 - Lifting Load	1.73		
			LC4 - transverse load X-direction (1%)	1.50		
			LC5 - Longitudinal load Y-direction (2%)	1.50		
			LC7 - wind on mast Y-direction (14m/s)	1.50		
			LC9 - wind on load Y-direction (14m/s)	1.50		
NC8	ULS - Operational Dir. X-Y	Ultimate	LC1 - Self Weight	1.35	Deform. from loadcase	LC12
			LC2 - Secondary weight	1.35		
			LC3 - Lifting Load	1.73		
			LC4 - transverse load X-direction (1%)	1.50		
			LC5 - Longitudinal load Y-direction (2%)	1.50		
			LC6 - wind on mast X-direction (14m/s)	1.06		
			LC7 - wind on mast Y-direction (14m/s)	1.06		
			LC8 - wind on load X-direction (14m/s)	1.06		
			LC9 - wind on load Y-direction (14m/s)	1.06		
NC9	SLS - Survival X	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC6 - wind on mast X-direction (14m/s)	8.95		
NC10	SLS - Survival Y	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC7 - wind on mast Y-direction (14m/s)	8.95		
NC11	SLS - Survival X-Y	Serviceability	LC1 - Self Weight	1.00	None	
			LC2 - Secondary weight	1.00		
			LC6 - wind on mast X-direction (14m/s)	6.30		
			LC7 - wind on mast Y-direction (14m/s)	6.30		
NC12	ULS - Survival X	Ultimate	LC1 - Self Weight	1.35	Deform. from loadcase	LC10
			LC2 - Secondary weight	1.35		
			LC6 - wind on mast X-direction (14m/s)	13.43		
NC13	ULS - Survival Y	Ultimate	LC1 - Self Weight	1.35	Deform. from loadcase	LC11
			LC2 - Secondary weight	1.35		
			LC7 - wind on mast Y-direction (14m/s)	13.43		
NC14	ULS - Survival X-Y	Ultimate	LC1 - Self Weight	1.35	Deform. from loadcase	LC12
			LC2 - Secondary weight	1.35		
			LC6 - wind on mast X-direction (14m/s)	9.49		
			LC7 - wind on mast Y-direction (14m/s)	9.49		
NC15	ULS - Survival X stability	Ultimate	LC1 - Self Weight	0.90	Deform. from loadcase	LC10
			LC2 - Secondary weight	0.90		
			LC6 - wind on mast X-direction (14m/s)	13.43		
NC16	ULS - Survival Y stability	Ultimate	LC1 - Self Weight	0.90	Deform. from loadcase	LC11
			LC2 - Secondary weight	0.90		
			LC7 - wind on mast Y-direction (14m/s)	13.43		
NC17	ULS - Survival X-Y stability	Ultimate	LC1 - Self Weight	0.90	Deform. from loadcase	LC12
			LC2 - Secondary weight	0.90		
			LC6 - wind on mast X-direction (14m/s)	9.49		
			LC7 - wind on mast Y-direction (14m/s)	9.49		

Figure A-11 Load combinations [9]

Also, three stability combinations are taken into account, see Figure A-12.

Name	Load cases	Coeff. [-]
S1 - X-direction	LC1 - Self Weight	1.35
	LC6 - wind on mast X-direction (14m/s)	1.50
	LC3 - Lifting Load	1.73
	LC4 - transverse load X-direction (1%)	1.50
	LC8 - wind on load X-direction (14m/s)	1.50
	LC5 - Longitudinal load Y-direction (2%)	1.50
	LC2 - Secondary weight	1.35
S2 - Y-direction	LC1 - Self Weight	1.35
	LC3 - Lifting Load	1.73
	LC4 - transverse load X-direction (1%)	1.50
	LC7 - wind on mast Y-direction (14m/s)	1.50
	LC9 - wind on load Y-direction (14m/s)	1.50
	LC5 - Longitudinal load Y-direction (2%)	1.50
	LC2 - Secondary weight	1.35
S3 - 45 direction	LC1 - Self Weight	1.35
	LC6 - wind on mast X-direction (14m/s)	1.06
	LC3 - Lifting Load	1.73
	LC4 - transverse load X-direction (1%)	1.50
	LC8 - wind on load X-direction (14m/s)	1.06
	LC7 - wind on mast Y-direction (14m/s)	1.06
	LC9 - wind on load Y-direction (14m/s)	1.06
	LC5 - Longitudinal load Y-direction (2%)	1.50
	LC2 - Secondary weight	1.35

Figure A-12 Stability combinations [9]

A.2.6. Foundation loads

Table A-8 Operational foundation loads [9]

POINT	LOAD COMBINATION	R_X [KN]	R_Y [KN]	R_Z [KN]
A1	NC2	-17	-45	2955
	NC3	42	-16	7237
	NC4	7	-43	4851
A2	NC2	1	-40	-797
	NC3	63	-62	162
	NC4	28	-54	-1091
B1	NC2	24	-67	8486
	NC3	**73**	-71	7900
	NC4	57	-87	8995
B2	NC2	-27	-56	4508
	NC3	26	-30	979
	NC4	-2	-56	2714
C1	NC2	-53	-78	2502
	NC3	3	-59	6099
	NC4	-33	-79	4145
C2	NC2	-19	-54	-1089
	NC3	36	-70	-700

	NC4	6	-68	**-1565**
D1	NC2	3	-86	10,706
	NC3	51	-97	9685
	NC4	34	**-107**	**11,135**
D2	NC2	**-61**	-87	6739
	NC3	-8	-64	2648
	NC4	-38	-88	4826

Table A-9 Survival foundation loads [9]

POINT	LOAD COMBINATION	R_X [KN]	R_Y [KN]	R_Z [KN]
A1	NC9	-172	-173	**-7193**
	NC10	73	-66	5423
	NC11	-89	-156	-2412
A2	NC9	-104	-112	-7224
	NC10	127	-128	-2105
	NC11	-4	-138	-6737
B1	NC9	-123	-131	9212
	NC10	**140**	-148	5323
	NC11	36	**-226**	10,051
B2	NC9	-194	-188	9187
	NC10	60	**-58**	-2161
	NC11	-75	-183	3822
C1	NC9	-171	-172	-6245
	NC10	71	-68	5399
	NC11	-91	-157	-1735
C2	NC9	-105	-110	-6276
	NC10	126	-128	-2100
	NC11	-8	-137	-6102
D1	NC9	-122	-131	10,824
	NC10	139	-149	5416
	NC11	34	-226	**11,245**
D2	NC9	**-196**	-189	10,832
	NC10	59	-59	-2077
	NC11	-80	-185	4985

A.2.7. CRANE AREA

Figure A-13 shows a top view of the lift that required the largest area. As mentioned, the radius of the super lift (SLR) was 15,000 mm and the maximum radius to the frame was 31,500 mm. It is assumed that the crane slewed 90°. This creates two quarter circles. To account for the dimensions of crane itself, the outer corners of the quarter circles are connected. This results in the shape of Figure A-14. Where each area is:

$$A_I = \frac{1}{4} * \pi * r^2 = \frac{1}{4} * \pi * 15.0^2 = 176\ m^2$$
$$A_{II} = \frac{1}{2} * 15.0 * 31.5 = 236\ m^2$$
$$A_{III} = \frac{1}{4} * \pi * r^2 = \frac{1}{4} * \pi * 31.5^2 = 779\ m^2$$
$$A_{tot} = A_I + 2 * A_{II} + A_{III} = 176 + 2 * 236 + 779 = 1427\ m^2$$

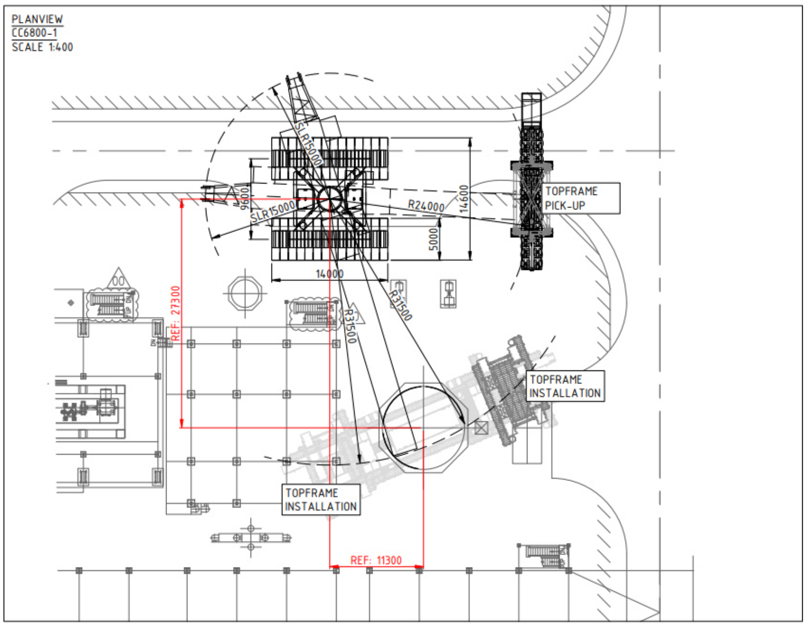


Figure A-13 Lift that required largest area [12]

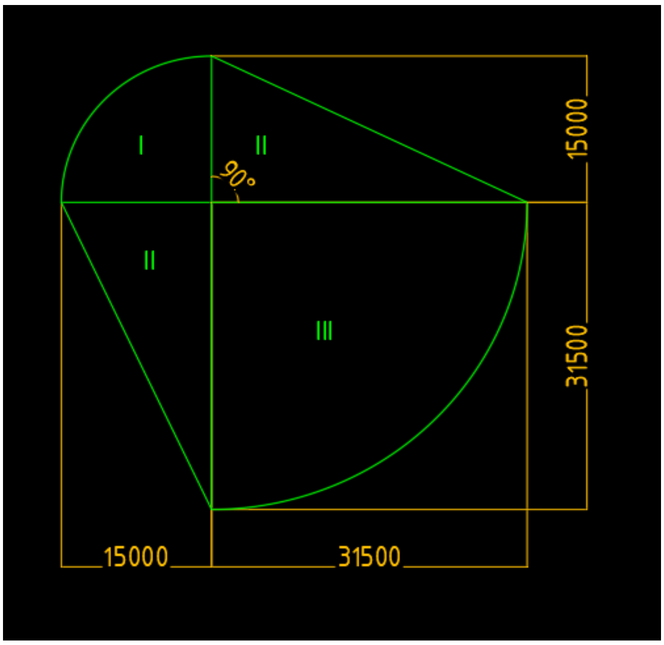


Figure A-14 Shape of required area

When assembling or disassembling, the total mast plus jib is laid down. This is a length of 174 meters. A hydraulic crane needs space around the mast and jib to place them. This roughly requires 10 meters around the mast and jib. Meaning that the total area needed for (dis)assembly of the crane is 1740 m^2.

A.3. CRANES

A.3.1. INPUT AUTOCRANE

The input parameters are:

- Load, in tons.
- Radius, the distance from the crane to the load, in meters.
- Hook height, in meters.
- Horizontal distance from the crane to the edge of the load, in meters.
- Distance from the edge of the hook to the boom of the crane, in meters.

The following is used for this input:

- Following from the case study are the 4000 x 8000 mm MLS masts. The weight of one MLS mast, including rigging, is 51.2 ton [12]. This will function as the lower limit for the cranes, since it cannot be avoided that these sections have to be lifted on site. The upper limit for the weight is the heaviest lift for the Jubail gantry. The heaviest weight was the gantry beam. Including rigging the weight was 132.3 ton [12]. Also, exactly in between these two, a 91.8-ton load, is included. It must be noted that the weights used, include a hook block. The hook block weights are based on the Jubail gantry. The 51.2-ton load used a 3.5-ton hook block, the 132.3-ton load used an 8.1-ton hook block. As the 91.8-ton load is an interpolation, the hook block is too. The use of these hook blocks is an assumption, as each specific crane usually brings its own hook block [14].
- The radius is kept at the minimal distance needed for the crane. A symbolic number of three meters is chosen, this made sure the minimum radius for the crane was taken by AutoCRANE.
- All loads will be checked for a hook height of 30, 60, 90, 120, 150 and 180 meters.
- The distance from the hook to the edge of the load also follows from the MLS mast. Half the minimal distance, including rigging, is 2042.5 mm [12]. This is rounded off to two meters.
- The ‘Engineering Foundation Course’ at Mammoet advised to use one meter as minimal distance between the edge of the load and the crane boom.

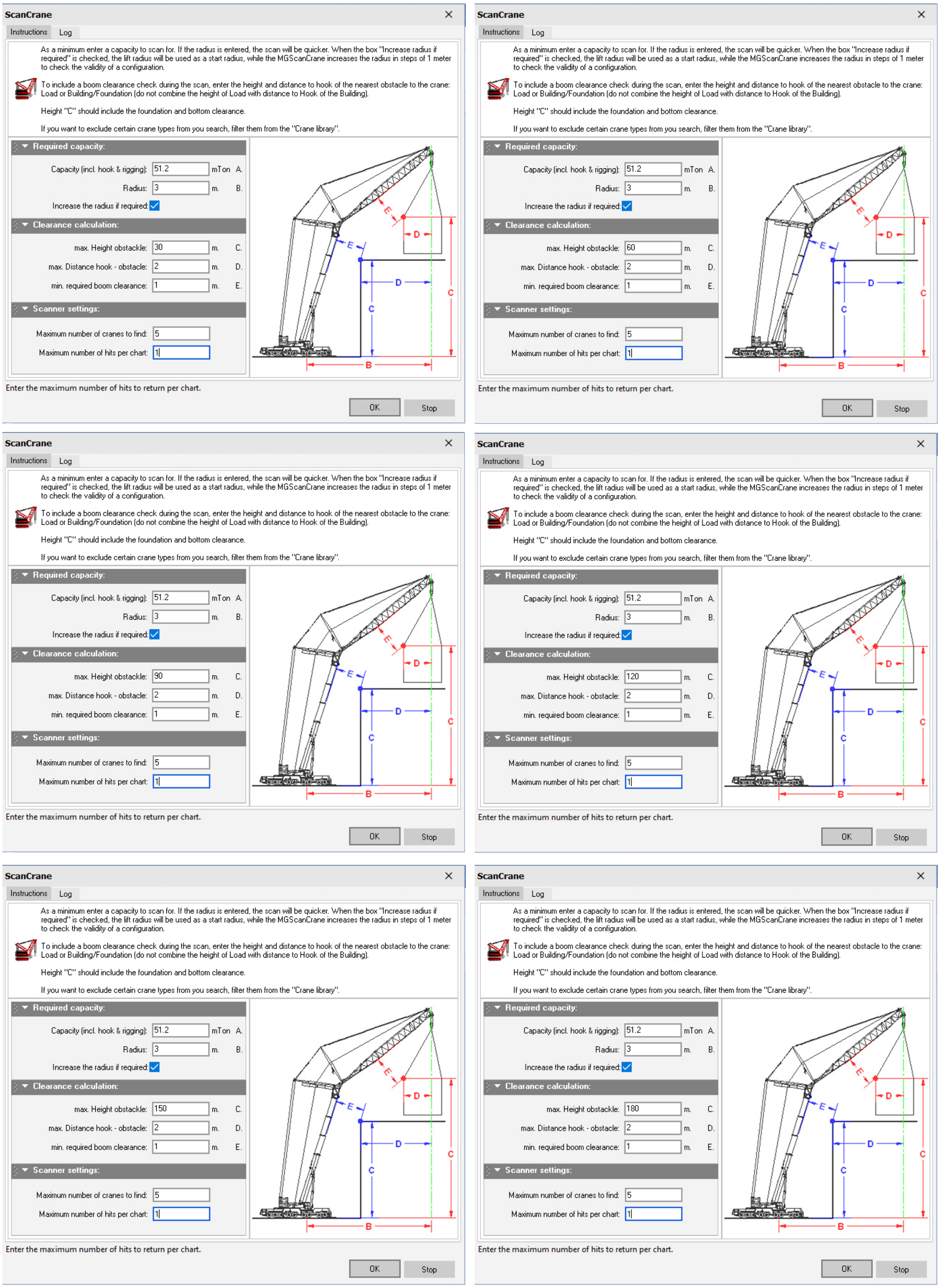


Figure A-15 AutoCRANE input 51.2 ton

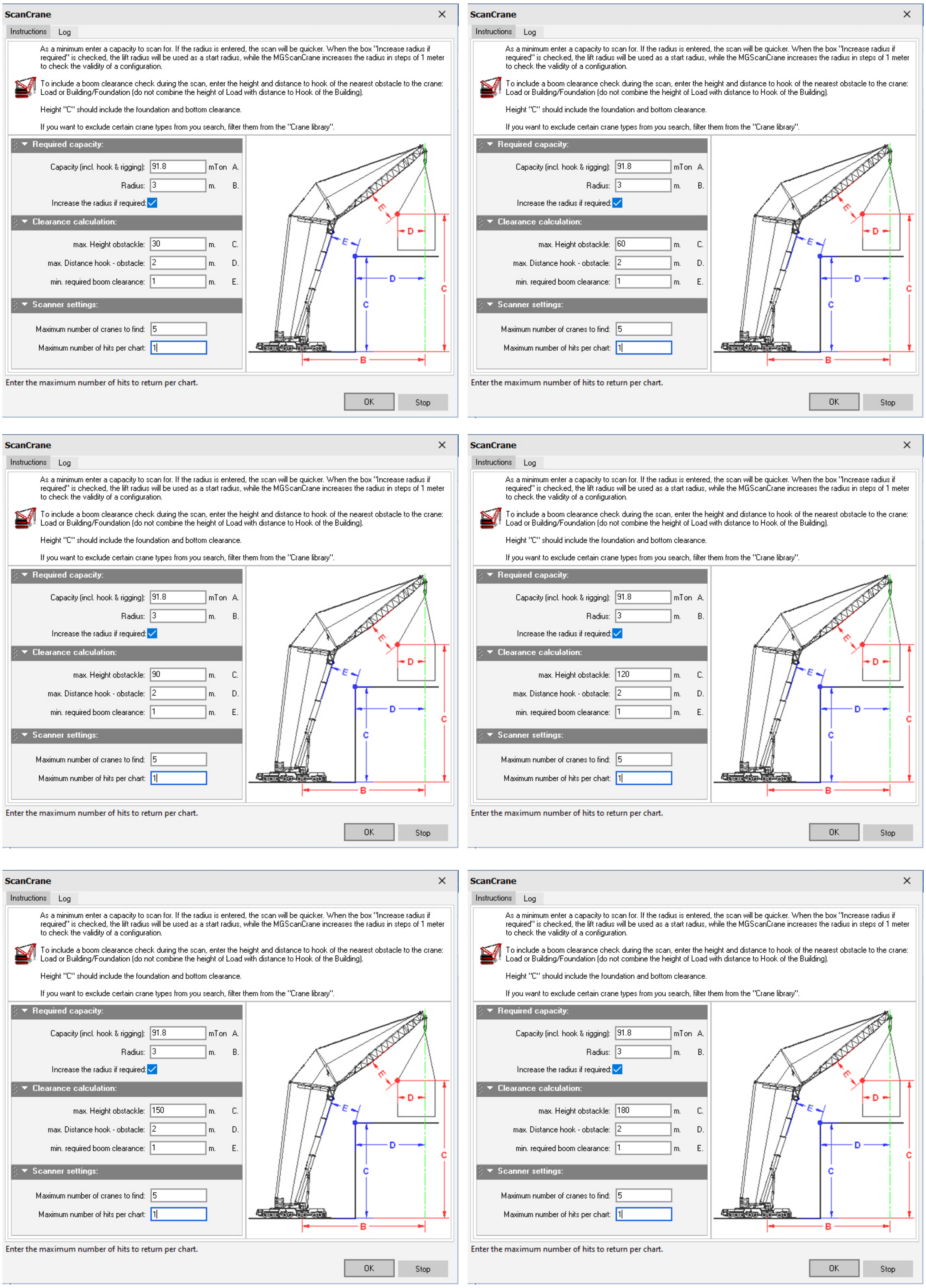


Figure A-16 AutoCRANE input 91.8 ton

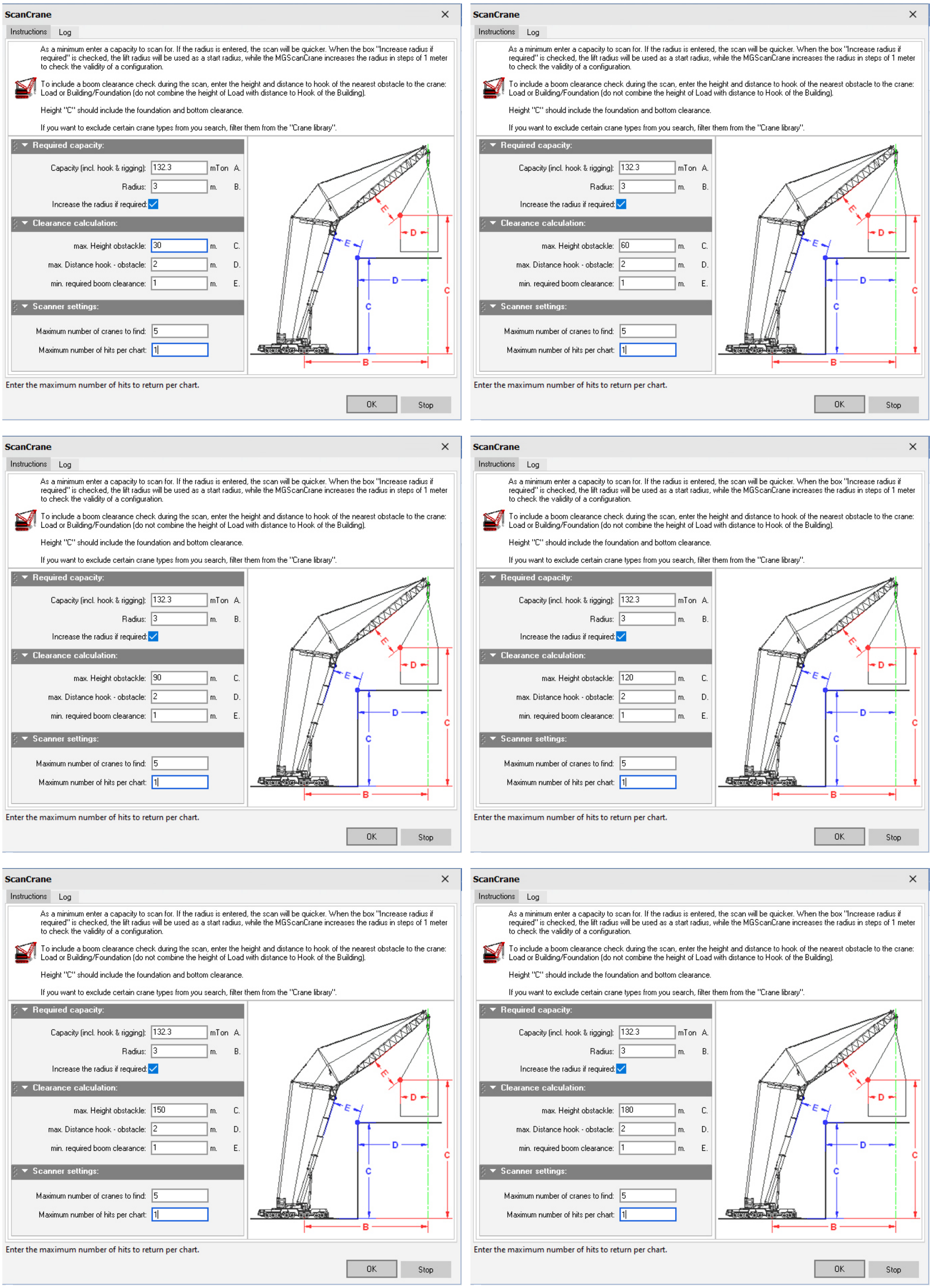


Figure A-17 AutoCRANE input 132.3 ton

A.3.2. SUMMARIZED CRANE TABLES

Table A-10 Cranes for 51.2 ton load short

HEIGHT [M]	CRANE	TYPE	RADIUS [M]	SL RADIUS [M]	ACTIVE CHART	MAIN BOOM [M]	JIB [M]	COUTER WEIGHT	PERCENTAGE [%]	OTHER POSSIBLE [%]
30	AC250-5	Hydraulic	8.0	-	HA	38.6	-	24.3	95.7	-
	AC220-5	Hydraulic	9.0	-	HA	38.0	-	29.9	98.0	-
	LTM1160-5.1	Hydraulic	7.0	-	T	35.7	-	27.4	99.8	-
	ATF220G-5	Hydraulic	8.0	-	SH	38.3	-	35.0	93.9	-
	LTR 1220	Hydraulic	7.0	-	T	39.1	-	70.0	89.4	-
60	LR1250(2014)	Crawler	8.2	-	Mode-3	59.0	7.0	78.3	92.3	-
	CKE2500-2	Crawler	15.0	-	Luffing Jib	45.7	21.3	114.0	95.7	-
	LTM1400-7.1	Hydraulic	16.0	-	TYSN	36.0	28.0	70.0	96.6	-
	LR1280W	Crawler	15.0	-	Mode-2	80.0	-	117.5	96.9	-
	LR1280	Crawler	13.0	-	SW	49.1	20.0	101.1	98.1	-
90	LR1300SX	Crawler	16.0	-	Mode-4	65.0	35.0	171.0	93.1	-
	LTM1650-8.1	Hydraulic	26.0	-	T3YVEN	51.0*	38.5	125.0	99.6	97.5
	LTM11200-9.1	Hydraulic	22.0	-	T7YVEF	82.5*	6.5	62.0	96.6	36.1
	LTM1800-9.1	Hydraulic	26.0	-	TYVEN-800	49.1	42.0	94.0	98.5	76.6
	LTM1750-9.1	Hydraulic	24.0	-	TYVEN	49.1	42.0	84.0	98.7	94.5
120	LTM11200-9.1	Hydraulic	28.0	-	T3YV2VEN	52.2*	60.0	182.0	96.6	-
	M16000 ANSI	Crawler	31.0	15.0	SWSL	84.0	48.0	267.5	100.0	-
	LR1550	Crawler	26.0	10.5	SWSL	77.0	56.0	160.0	73.1	-
	LR1500	Crawler	22.0	9.0	SDWB	84.8	48.0	170.0	90.5	-
	LG1550	Lattice boom	30.0	-	SW	49.0	84.0	200.0	98.5	70.1
150	CC2800-1	Crawler	33.0	13.0	HSWSL_S7	96.0	66.0	260.0	99.4	-
	TC2800-1	Lattice boom	33.0	13.0	HSWSL S7	96.0	66.0	260.0	99.4	-
	LG1750SX	Lattice boom	27.0	-	SXZLD4	168.0	-	175.0	94.8	34.1
	LR1600-2	Crawler	26.0	10.0	SL4DFB	138.0	24.0	215.0	96.6	85.3
	CC3800-1	Crawler	32.0	2.0	SFSL	108.0	54.0	280.0	94.5	79.4
180	LG1750SX	Lattice boom	32.0	17.0	SX2D4F2/F3 B	164.5	24.0	575.0	82.6	82.6
	CC38.650-1	Crawler	38.0	13.0	SFSL_4	60.0*	84.0	275.0	92.3	-
	LR1800-1.0	Crawler	40.0	14.0	HSDWBV	102.0	90.0	330.0	98.7	-
	M21000 ANSI	Crawler	36.6	21.9	SWSL	103.6	91.4	824.9	63.4	59.1
	PC6800	Lattice boom	38.0	15.0	SWSL	96.0	96.0	170.0	82.3	80.6

Table A-11 Cranes for 91.8 ton load short

HEIGHT [M]	CRANE	TYPE	RADIUS [M]	SL RADIUS [M]	ACTIVE CHART	MAIN BOOM [M]	JIB [M]	COUTER WEIGHT [TE]	PERCENTAGE [%]	OTHER POSSIBLE [%]
30	LR1250(2014)	Crawler	11.0	-	Mode-1	44.0	-	108.3	98.6	-
	LR1280W	Crawler	11.0	-	Mode-2	47.0	-	117.5	96.2	-
	LR1280	Crawler	11.0	-	S	49.1	-	121.5	97.5	-
	LTM1400-7.1	Hydraulic	8.0	-	TY	41.1	-	30.0	98.7	92.8
	LTM1450-8.1	Hydraulic	9.0	-	TN	21.4	14.0	34.0	94.5	92.0
60	LR1300SX	Crawler	13.0	-	Mode-4	47.0	20.0	151.0	98.8	-
	LTM1500-8.1 ANSI	Hydraulic	16.8	-	TY3SN (T-50)	42.1	21.0	135.0	97.8	-
	LTM1800-9.1	Hydraulic	11.0	-	TYV23ENZF-800	49.1*	6.1	64.0	97.3	88.5
	LTM1750-9.1	Hydraulic	16.0	-	TYVEN	38.2*	21.0	74.0	98.1	79.2
	AC700	Hydraulic	17.0	-	WIHISSL60	35.5	30.0	80.0	99.8	-
90	M16000 ANSI	Crawler	20.0	11.0	SSL	108.0	-	267.5	98.8	90.4
	LTM11200-9.1	Hydraulic	20.0	-	T37YVEN	52.2*	36.0	82.0	99.8	64.6
	CC2400-1	Crawler	17.0	9.0	SWSL	72.0	24.0	240.0	85.4	-
	LR1500	Crawler	27.0	11.0	SDB	102.0	-	300.0	97.1	88.4
	LG1550	Lattice boom	23.0	-	SW3H	56.5	49.0	200.0	96.6	70.6
120	CC2800-1	Crawler	19.0	11.0	SSLLSL+LF2_MaxSGL_S7	120.0	12.0	210.0	98.7	97.1
	TC2800-1	Lattice boom	21.0	13.0	HSWSL	108.0	24.0	160.0	98.7	85.8
	LR1600-2	Crawler	31.0	13.0	SDB	132.0	-	365.0	96.6	86.6
	CC3800-1	Crawler	17.0	11.0	LSL-LF3	117.0	12.0	215.0	98.2	85.8
	LG1750SX	Lattice boom	22.0	-	SX3LD4	139.4	-	220.0	95.6	42.9
150	LG1750SX	Lattice boom	32.0	17.0	SXLD4B	164.5	-	575.0	74.6	61.2
	CC38.650-1	Crawler	22.0	15.0	SWSL_4	108.0	54.0	340.0	99.8	90.0
	LR1750-2-SX	Crawler	30.0	17.0	SXLD4B	168.0	-	670.0	85.8	62.4
	LR1700-1.1	Crawler	23.0	13.0	HSL3DFBV	144.0	15.0	250.0	99.7	96.6
	LG1750	Lattice boom	28.0	18.0	SL9D2FB	136.5	18.0	620.0	90.9	84.2
180	LR11000	Crawler	35.0	18.0	PDW3B	90.0	96.0	410.0	97.2	91.4
	LR11350	Crawler	28.0	15.0	SDWBW	114.0	72.0	200.0	83.5	58.5
	CC8800-1	Crawler	46.0	19.0	SFSL	108.0	90.0	455.0	95.6	79.1
	PTC35	Ring	59.0	-	SWSL2 DS-LD	116.7	75.1	1300.0	27.9	27.0
	CC8800-1 BB	Crawler	38.0	24.0	BSWSL	120.0	78.0	355.0	97.1	87.4

Table A-12 Cranes for 132.3 ton load short

HEIGHT [M]	CRANE	TYPE	RADIUS [M]	SL RADIUS [M]	ACTIVE CHART	MAIN BOOM [M]	JIB [M]	COUTER WEIGHT	PERCENTAGE [%]	OTHER POSSIBLE [%]
30	LR1300SX	Crawler	10.0	-	Mode-1	50.0	-	161.0	96.9	-
	LTM1500-8.1	Hydraulic	8.0	-	TY3 (T-50)	47.3	-	60.0	95.2	-
	LTM1500-8.1 ANSI	Hydraulic	12.2	-	TN (T-50)	15.9	21.0	135.0	99.9	96.9
	AC500-1Y	Hydraulic	9.0	-	HA-SSL	42.5	-	100.0	93.8	-
	AC500-2	Hydraulic	9.0	-	HASSL0	42.5	-	80.0	94.5	-
60	LTM11200-9.1	Hydraulic	12.0	-	T7YVEF	59.1*	6.5	42.0	99.5	66.8
	LR1350-1	Crawler	19.0	9.0	SDB	72.0	-	293.0	96.6	81.9
	LG1550	Lattice boom	14.0	-	SW	42.0	28.0	160.0	74.3	69.6
	CC2400-1	Crawler	12.0	-	SW	48.0	24.0	200.0	98.7	89.1
	M16000 ANSI	Crawler	12.2	-	SW	48.0	24.0	161.5	96.4	75.2
90	LTM11200-9.1	Hydraulic	16.0	-	T3YV2VEN	46.4*	30.0	162.0	93.2	-
	M16000 ANSI	Crawler	17.0	15.0	SWSL	42.0	72.0	267.5	90.9	-
	TC2800	Lattice boom	20.0	13.0	SFSL	78.0	24.0	160.0	99.5	87.6
	LR1550	Crawler	18.0	10.5	SWSL	63.0	35.0	290.0	94.5	-
	LG1550	Lattice boom	18.0	10.5	SWSL	63.0	35.0	200.0	92.5	-
	TC2800-1	Lattice boom	30.0	15.0	SSLLSL	102.0	-	260.0	99.5	49.7
120	LG1750SX	Lattice boom	28.0	17.0	SX2ZD4F2/F3 B	154.0	12.0	575.0	92.5	61.8
	TC2800-1	Lattice boom	15.0	13.0	SSLLSL LF2 max SGL S7 LHA	114.0	12.0	160.0	98.4	-
	CC38.650-1	Crawler	27.0	13.0	LSL_13	129.0	-	400.0	99.5	83.2
	LR1750-2 SX	Crawler	24.0	17.0	SXZLD4B	157.5	-	670.0	95.9	69.8
	LR1700-1.0	Crawler	28.0	13.0	HSL3DBV	135.0	-	390.0	97.4	87.6
150	LG1750SX	Lattice boom	34.0	17.0	SX2LD4B	163.9	-	575.0	95.9	88.2
	LR1750-2 SX	Crawler	32.0	17.0	SX3ZLD4B	167.4	-	670.0	99.5	90.0
	LR1800-1.0	Crawler	34.0	19.0	HSL2ZDBV	165.0	-	490.0	98.3	-
	M21000 ANSI	Crawler	42.7	21.9	SWSL	103.6	61.0	665.3	88.7	-
	PC6800	Lattice boom	25.0	15.0	SWSL	96.0	60.0	170.0	94.3	90.0
180	LR11350	Crawler	32.0	20.0	PDWBW	102.0	84.0	300.0	98.7	84.3
	PTC35	Ring	59.0	-	SFSL2 DS-HD	134.8	57.2	1300.0	57.6	46.1
	CC8800-1 BB	Crawler	49.0	19.0	BSFSL	120.0	78.0	635.0	97.3	84.3
	CC12600	Crawler	42.0	20.0	SWSL	114.0	84.0	370.0	73.9	63.0
	LR12500-1.0	Crawler	45.0	25.0	HDWB2_Prelim	100.0	90.0	420.0	87.8	85.5

A.3.3. Complete crane tables

Table A-13 Cranes for 51.2 ton load

HEIGHT [M]	CRANE	TYPE	RADIUS [M]	SHORTER [M]	LONGER [M]	SL RADIUS [M]	CAPACITY [TE]	ACTIVE CHART	BOOM TYPE	JIP TYPE	ADAPTOR	MAIN BOOM [M]	JIB [M]	CENTER - CWT [TE]	CWT [TE]	SL-CWT [TE]	PERCEN AGE [%]
30	AC250-5	Hydraulic	8.0	2.0	-	-	53.5	HA	180.0	-	-	38.6	-	-	24.3	-	95.7
	AC220-5	Hydraulic	9.0	3.0	-	-	52.2	HA	180.0	-	-	38.0	-	-	29.9	-	98.0
	LTM1160-5.1	Hydraulic	7.0	2.1	-	-	51.3	T	184.0	-	-	35.7	-	-	27.4	-	99.8
	ATF220G-5	Hydraulic	8.0	3.0	1.0	-	54.5	SH	(92-46-46-46-46-0)	-	-	38.3	-	-	35.0	-	93.9
	LTR 1220	Hydraulic	7.0	-	1.0	-	57.3	T	-	-	-	39.1	-	-	70.0	-	89.4
60	LR1250(2014)	Crawler	8.2	-	1.8	-	55.5	Mode-3	-	0906-1)	-	59.0	7.0	36.0	42.3	-	92.3
	CKE2500-2	Crawler	15.0	3.0	1.0	-	53.5	Luffing Jib	-	-	-	45.7	21.3	24.0	90.0	-	95.7
	LTM1400-7.1	Hydraulic	16.0	-	-	-	53.0	TYSN	92-46-46-0	-	-	36.0	28.0	-	70.0	-	96.6
	LR1280W	Crawler	15.0	6.5	-	-	52.9	Mode-2	-	-	-	80.0	-	32.0	85.5	-	96.9
	LR1280	Crawler	13.0	4.0	-	-	52.2	SW	-	-	-	49.1	20.0	36.0	65.1	-	98.1
90	LR1300SX	Crawler	16.0	3.3	2.0	-	55.0	Mode-4	-	2316-1	-	65.0	35.0	57.0	114.0	-	93.1
	LTM1650-8.1	Hydraulic	22.0	-	-	-	52.4	T5YVEN	0+92+92+92+	-	-	63.6*	24.5	-	135.0	-	97.7
	LTM1650-8.1	Hydraulic	24.0	-	-	-	52.5	T3YV2EN	92+92+92+	-	-	51.0*	31.5	-	125.0	-	97.5
	LTM1650-8.1	Hydraulic	26.0	-	-	-	51.4	T3YVEN	92+92+92+	-	-	51.0*	38.5	-	125.0	-	99.6
	LTM1800-9.1	Hydraulic	20.0	-	5.0	-	66.8	TYV23EN-800	92+92+92	-	0.5	49.1*	24.5	-	94.0	-	76.6
	LTM1800-9.1	Hydraulic	21.0	-	-	-	53.5	TYV2EN-800	92+92+92	-	1.9	49.1*	28.0	-	74.0	-	95.7
	LTM1800-9.1	Hydraulic	26.0	-	-	-	52.0	TYVEN-800	92+92+92	-	1.9	49.1	42.0	-	94.0	-	98.5
	LTM11200-9.1	Hydraulic	22.0	-	-	-	53.0	T7YVEF	100-100-100-100-50-50-50	-	9.1	82.5*	6.5	-	62.0	-	96.6
	LTM11200-9.1	Hydraulic	16.0	-	3.0	-	67.0	T7YEV2F	100-100-100-100-50-50-50	-	19.1	82.5*	6.5	-	62.0	-	76.4
	LTM11200-9.1	Hydraulic	16.0	-	18.0	-	142.0	T3YV2VEN	100-100-50	-	20.2	46.4*	30.0	-	162.0	-	36.1
	LTM11200-9.1	Hydraulic	20.0	-	14.0	-	92.0	T37YVEN	100-100-100	-	10.2	52.2*	36.0	-	82.0	-	55.7
	LTM11200-9.1	Hydraulic	20.0	-	14.0	-	94.0	T3YVEN	100-100-100	-	10.2	52.2*	36.0	-	82.0	-	54.5
	LTM1750-9.1	Hydraulic	21.0	-	1.0	-	54.2	TYV2EN	92+92+92+	-	1.9	49.1*	28.0	-	74.0	-	94.5
	LTM1750-9.1	Hydraulic	24.0	-	-	-	51.9	TYVEN	92+92+92+	-	1.9	49.1	42.0	-	84.0	-	98.7
120	LTM11200-9.1	Hydraulic	28.0	-	16.0	-	53.0	T3YV2VEN	100-100-100	-	20.2	52.2*	60.0	-	182.0	-	96.6
	M16000 ANSI	Crawler	31.0	-	-	15.0	51.2	SWSL	-	-	-	84.0	48.0	54.4	150.6	62.5	100.0
	LR1550	Crawler	26.0	-	10.0	10.5	70.0	SWSL	-	W	-	77.0	56.0	-	160.0	-	73.1
	LR1500	Crawler	22.0	2.0	2.0	9.0	56.6	SDWB	-	-	-	84.8	48.0	40.0	130.0	-	90.5
	LG1550	Lattice boom	26.0	-	12.0	10.5	70.0	SWSL	-	-	-	77.0	56.0	90.0	200.0	-	73.1
	LG1550	Lattice boom	28.0	-	7.0	-	69.0	SWH	-	WH5t	-	49.5*	70.0	-	180.0	-	74.2
	LG1550	Lattice boom	30.0	-	-	-	52.0	SW	-	W	-	49.0	84.0	-	200.0	-	98.5
	LG1550	Lattice boom	28.0	-	11.0	-	73.0	SW3H	-	W3H	-	49.5*	70.0	-	200.0	-	70.1

150	CC2800-1	Crawler	33.0	-	1.0	13.0	51.5	HSWSL_S7	-	-	-	96.0	66.0	-	160.0	100.0	99.4
	TC2800-1	Lattice boom	33.0	-	1.0	13.0	51.5	HSWSL S7	-	-	-	96.0	66.0	-	160.0	100.0	99.4
	LG1750SX	Lattice boom	34.0	10.0	42.0	17.0	144.0	SX3LD4B	-	-	-	163.9	-	-	175.0	400.0	35.6
	LG1750SX	Lattice boom	34.0	10.0	42.0	17.0	145.0	SX2ZLD4B	-	-	-	163.9	-	-	175.0	400.0	35.3
	LG1750SX	Lattice boom	34.0	10.0	42.0	17.0	138.0	Sx2LD4B	-	-	-	163.9	-	-	175.0	400.0	37.1
	LG1750SX	Lattice boom	34.0	10.0	42.0	17.0	148.0	SX3ZLD4B	-	-	-	163.9	-	-	175.0	400.0	34.6
	LG1750SX	Lattice boom	32.0	6.0	48.0	17.0	123.0	SXLD4B	-	-	-	164.5	-	-	175.0	400.0	41.6
	LG1750SX	Lattice boom	32.0	6.0	48.0	17.0	127.0	SXZLD4B	-	-	-	164.5	-	-	175.0	400.0	40.3
	LG1750SX	Lattice boom	27.0	1.0	-	-	54.0	SXZLD4	-	-	-	168.0	-	-	175.0	-	94.8
	LG1750SX	Lattice boom	27.0	1.0	1.0	-	54.5	SXLD4	-	-	-	168.0	-	-	175.0	-	93.9
	LG1750SX	Lattice boom	24.0	-	68.0	17.0	150.0	SX2D4F2/F3 B	-	F2	-	115.5*	12.0	-	175.0	400.0	34.1
	LG1750SX	Lattice boom	26.0	-	7.0	-	73.0	SXD4F2/F3	-	F2	-	144.1	12.0	-	175.0	-	70.1
	LG1750SX	Lattice boom	26.0	-	54.0	15.0	138.0	SXD4F2/F3 B	-	F2	-	144.1	12.0	-	220.0	400.0	37.1
	LG1750SX	Lattice boom	26.0	-	6.0	-	69.0	SX2D4F2/F3	-	F2	-	147.0	12.0	-	175.0	-	74.2
	LG1750SX	Lattice boom	26.0	-	6.0	-	69.0	SX3D4F2/F3	-	F2	-	147.0	12.0	-	175.0	-	74.2
	LG1750SX	Lattice boom	26.0	-	58.0	17.0	150.0	SX3D4F2/F3 B	-	F2	-	147.0	12.0	-	175.0	400.0	34.1
	LG1750SX	Lattice boom	28.0	-	-	-	57.0	SX3ZD4F2/F3	-	F2	-	154.0	12.0	-	175.0	-	89.8
	LG1750SX	Lattice boom	28.0	-	-	-	57.0	SX2ZD4F2/F3	-	F2	-	154.0	12.0	-	175.0	-	89.8
	LG1750SX	Lattice boom	28.0	-	52.0	17.0	143.0	SX2ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	35.8
	LG1750SX	Lattice boom	28.0	-	52.0	17.0	148.0	SX3ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	34.6
	LG1750SX	Lattice boom	28.0	-	2.0	-	57.0	SXZD4F2/F3	-	F2	-	154.6	12.0	-	175.0	-	89.8
	LG1750SX	Lattice boom	28.0	-	56.0	17.0	127.0	SXZD4F2/F3 B	-	F2	-	154.6	12.0	-	175.0	400.0	40.3
	LR1600-2	Crawler	22.0	-	2.0	10.0	60.0	SL13DFB	-	-	-	147.0	12.0	65.0	150.0	-	85.3
	LR1600-2	Crawler	26.0	-	-	10.0	53.0	SL4DFB	-	-	-	138.0	24.0	65.0	150.0	-	96.6
	LR1600-2	Crawler	27.0	3.0	1.0	13.0	53.5	SDWB	-	-	-	102.0	54.0	65.0	150.0	-	95.7
	CC3800-1	Crawler	21.0	-	4.0	11.0	58.0	LSL-LF2	-	-	12.0	144.0	12.0	50.0	165.0	-	88.3
	CC3800-1	Crawler	20.0	-	3.0	11.0	64.5	LSL-LF3	-	-	11.0	147.0	12.0	50.0	165.0	-	79.4
	CC3800-1	Crawler	20.0	-	2.0	11.0	62.5	LSL-LF4	-	-	11.0	150.0	12.0	50.0	165.0	-	81.9
	CC3800-1	Crawler	22.0	-	6.0	13.0	57.5	SWSL	-	-	-	108.0	54.0	50.0	165.0	-	89.0
	CC3800-1	Crawler	32.0	-	-	2.0	54.0	SFSL	-	-	-	108.0	54.0	50.0	165.0	65.0	94.5
180	LG1750SX	Lattice boom	32.0	-	36.0	17.0	62.0	SX2D4F2/F3 B	-	F3	-	164.5	24.0	-	175.0	400.0	82.6
	LG1750SX	Lattice boom	32.0	-	36.0	17.0	62.0	SX3D4F2/F3 B	-	F3	-	164.5	24.0	-	175.0	400.0	82.6
	LG1750SX	Lattice boom	32.0	-	36.0	17.0	62.0	SX2ZD4F2/F3 B	-	F3	-	164.5	24.0	-	175.0	400.0	82.6
	LG1750SX	Lattice boom	32.0	-	36.0	17.0	62.0	SX3ZD4F2/F3 B	-	F3	-	164.5	24.0	-	175.0	400.0	82.6
	LG1750SX	Lattice boom	32.0	-	36.0	17.0	62.0	SXZD4F2/F3 B	-	F3	-	165.1	24.0	-	175.0	400.0	82.6
	LG1750SX	Lattice boom	32.0	-	36.0	17.0	62.0	SXD4F2/F3 B	-	F3	-	165.1	24.0	-	175.0	400.0	82.6

CC38.650-1	Crawler	38.0	-	2.0	13.0	55.5	SFSL_4	SFSL_4	-	-	60.0*	84.0	50.0	225.0	-	92.3
LR1800-1.0	Crawler	40.0	-	-	14.0	51.9	HSDWBV	LongStay	-	-	102.0	90.0	70.0	210.0	50.0	98.7
M21000	Crawler	54.0	-	40.0	22.0	86.6	SWSL	-	-	-	103.6	91.4	91.0	224.0	510.0	59.1
M21000 ANSI	Crawler	36.6	-	33.5	21.9	80.7	SWSL	#80	-	-	103.6	91.4	90.7	224.4	509.8	63.4
PC6800	Lattice boom*	38.0	-	4.0	15.0	62.2	SWSL	-	-	-	96.0	96.0	-	170.0	-	82.3
PC6800	Lattice boom*	46.0	-	1.0	15.0	63.5	SFSL	-	-	-	96.0	96.0	-	250.0	-	80.6

Table A-14 Cranes for 91.8 ton load

HEIGHT [M]	CRANE	TYPE	RADIUS [M]	SHORTER [M]	LONGER [M]	SL RADIUS [M]	CAPACITY [TE]	ACTIVE CHART	BOOM TYPE	JIP TYPE	ADAPTOR	MAIN BOOM [M]	JIB [M]	CENTER - CWT [TE]	CWT [TE]	SL-CWT [TE]	PERCENTAGE [%]
30	LR1250(2014)	Crawler	11.0	5.2	-	-	93.1	Mode-1	-	-	-	44.0	-	36.0	72.3	-	98.6
	LR1280W	Crawler	11.0	4.8	-	-	95.4	Mode-2	-	-	-	47.0	-	32.0	85.5	-	96.2
	LR1280	Crawler	11.0	4.7	-	-	94.2	S	-	-	-	49.1	-	36.0	85.5	-	97.5
	LTM1400-7.1	Hydraulic	8.0	1.0	-	-	93.0	TY	46-46-46-92	-	-	41.1	-	-	30.0	-	98.7
	LTM1400-7.1	Hydraulic	10.0	3.3	0.4	-	98.9	TY	0+92+92+46+	-	-	41.1	-	-	60.0	-	92.8
	LTM1400-7.1 ANSI	Hydraulic	10.0	0.9	3.7	-	97.1	TN	46+46+0+0+	-	-	25.6	14.0	-	100.0	-	94.5
	LTM1450-8.1	Hydraulic	9.0	6.0	-	-	99.8	T	0+0+0+46+	-	-	55.6	-	-	34.0	-	92.0
	LTM1450-8.1	Hydraulic	9.0	-	-	-	97.1	TN	0+0+46+0+0+0+	-	-	21.4	14.0	-	34.0	-	94.5
60	LR1300SX	Crawler	13.0	2.0	-	-	92.9	Mode-4	-	2316-1	-	47.0	20.0	57.0	94.0	-	98.8
	LTM1500-8.1 ANSI	Hydraulic	16.8	-	-	-	93.9	TY3SN (T-50)	92+92+46	-	-	42.1	21.0	-	135.0	-	97.8
	LTM1800-9.1	Hydraulic	11.0	1.0	-	-	94.3	TYV23ENZF-800	92+92+92+	-	0.5	49.1*	6.1	-	64.0	-	97.3
	LTM1800-9.1	Hydraulic	11.0	1.0	1.0	-	103.7	TY23E3NZF-800	92+92+46+	-	0.5	43.7*	6.0	-	74.0	-	88.5
	LTM1800-9.1	Hydraulic	14.0	4.0	-	-	95.0	TY2ENZF-800	92+92+92+	-	0.5	49.1*	6.0	-	84.0	-	96.6
	LTM1800-9.1	Hydraulic	16.0	-	-	-	93.1	TYVEN-800	92+92+0+	-	1.9	38.2*	21.0	-	74.0	-	98.6
	LTM1800-9.1	Hydraulic	16.0	-	1.0	-	99.0	TYV2EN-800	92+46+0+	-	1.9	32.7*	21.0	-	84.0	-	92.7
	LTM1800-9.1	Hydraulic	17.0	-	-	-	95.2	TYV23EN-800	92+46+0+	-	0.5	32.7*	21.0	-	94.0	-	96.4
	LTM1750-9.1	Hydraulic	11.0	1.0	1.0	-	99.8	TYV23EF	92+92+92+	-	0.5	49.1*	6.0	-	64.0	-	92.0
	LTM1750-9.1	Hydraulic	11.0	-	1.0	-	94.0	TYV23E3F	92+92+92+	-	0.5	49.1*	6.0	-	74.0	-	97.7
	LTM1750-9.1	Hydraulic	14.0	5.0	-	-	94.4	TYV2EF	92+92+92+	-	0.5	49.1*	6.0	-	84.0	-	97.2
	LTM1750-9.1	Hydraulic	16.0	-	-	-	93.6	TYVEN	92+92+0+	-	1.9	38.2*	21.0	-	74.0	-	98.1
	LTM1750-9.1	Hydraulic	15.0	-	3.0	-	115.9	TYV2EN	46+46+0+	-	1.9	27.2*	21.0	-	84.0	-	79.2
	AC700	Hydraulic	17.0	-	-	-	92.0	WIHISSL60	90-90-0-0	-	-	35.5	30.0	-	80.0	-	99.8
90	M16000 ANSI	Crawler	20.0	3.2	5.0	11.0	92.9	SSL	-	-	-	108.0	-	54.4	150.6	62.5	98.8
	M16000 ANSI	Crawler	26.0	-	4.0	11.0	101.5	SWSL	-	-	-	78.0	24.0	54.4	150.6	152.1	90.4
	LTM11200-9.1	Hydraulic	16.0	-	13.0	-	142.1	T3YV2VEN	100-100-50	-	20.2	46.4*	30.0	-	162.0	-	64.6
	LTM11200-9.1	Hydraulic	20.0	-	-	-	94.0	T3YVEN	100-100-100	-	10.2	52.2*	30.0	-	82.0	-	97.7
	LTM11200-9.1	Hydraulic	20.0	-	-	-	92.0	T37YVEN	100-100-100	-	10.2	52.2*	36.0	-	82.0	-	99.8
	CC2400-1	Crawler	17.0	4.0	4.0	9.0	107.5	SWSL	-	-	-	72.0	24.0	40.0	160.0	40.0	85.4
	LR1500	Crawler	27.0	13.0	-	11.0	94.5	SDB	-	-	-	102.0	-	40.0	170.0	90.0	97.1
	LR1500	Crawler	32.0	16.0	1.0	13.0	97.3	SL22DB	-	-	-	102.0	-	40.0	170.0	120.0	94.3
	LR1500	Crawler	17.0	-	1.0	-	95.4	SL8F	-	-	-	81.0	18.0	40.0	170.0	-	96.2
	LR1500	Crawler	16.0	-	2.0	9.0	103.8	SDWB	-	-	-	84.8	18.0	40.0	130.0	30.0	88.4
	LR1500	Crawler	17.0	-	1.0	-	95.8	SW	-	-	-	66.8	36.0	40.0	170.0	-	95.8

	LG1550	Lattice boom	23.0	3.0	14.0	13.0	116.0	SDB	-	-	-	105.0	-	-	160.0	250.0	79.1
	LG1550	Lattice boom	16.0	4.0	6.0	-	100.0	SL3H	-	-	-	91.0	6.0	-	160.0	-	91.8
	LG1550	Lattice boom	18.0	-	6.0	10.5	130.0	SWSL	-	W	-	63.0	35.0	-	160.0	-	70.6
	LG1550	Lattice boom	22.0	-	2.0	-	100.0	SWH	-	WH5t	-	49.5	42.0	-	160.0	-	91.8
	LG1550	Lattice boom	20.0	-	4.0	-	114.0	SW	-	W	-	49.0	49.0	-	160.0	-	80.5
	LG1550	Lattice boom	23.0	-	1.0	-	95.0	SW3H	-	W3H	-	56.5	49.0	-	200.0	-	96.6
120	TC2800-1	Lattice boom	26.0	10.0	1.0	13.0	93.5	SSLLSL max SGL S7	-	-	-	138.0	-	-	160.0	50.0	98.2
	TC2800-1	Lattice boom	21.0	-	2.0	13.0	93.0	HSWSL	-	-	-	108.0	24.0	-	160.0	-	98.7
	TC2800-1	Lattice boom	15.0	1.0	-	11.0	102.2	SSLLSL LF2 max SGL S7 LHA	-	-	-	114.0	12.0	-	160.0	-	89.8
	TC2800-1	Lattice boom	19.0	2.0	5.0	11.0	107.0	SSLLSL LF2 max SGL S7	-	-	-	120.0	12.0	-	160.0	-	85.8
	CC2800-1	Crawler	19.0	2.0	-	11.0	93.0	SSLLSL+LF2_MaxSGL_S7	-	-	-	120.0	12.0	-	160.0	50.0	98.7
	CC2800-1	Crawler	26.0	10.0	-	13.0	94.5	SSLLSL_MaxSGL_S7	-	-	-	138.0	-	60.0	180.0	100.0	97.1
	CC2800-1	Crawler	21.0	-	1.0	13.0	93.0	HSWSL_S7	-	-	-	108.0	24.0	60.0	180.0	-	98.7
	LR1600-2	Crawler	31.0	13.0	1.0	18.0	98.0	SL2DB	-	-	-	132.0	-	65.0	150.0	100.0	93.7
	LR1600-2	Crawler	31.0	13.0	-	13.0	95.0	SDB	-	-	-	132.0	-	65.0	150.0	150.0	96.6
	LR1600-2	Crawler	20.0	-	-	10.0	100.0	SL13DFB	-	-	-	117.0	12.0	25.0	150.0	50.0	91.8
	LR1600-2	Crawler	20.0	-	2.0	10.0	104.0	SL4DFB	-	-	-	120.0	12.0	65.0	150.0	50.0	88.3
	LR1600-2	Crawler	16.0	-	2.0	13.0	106.0	SDWB	-	-	-	102.0	24.0	65.0	150.0	-	86.6
	LR1600-2	Crawler	24.0	-	-	10.0	98.0	SDWVB	-	-	-	102.0	24.0	65.0	150.0	100.0	93.7
	CC3800-1	Crawler	26.0	11.0	2.0	13.0	94.0	LSL-2	-	-	-	138.0	-	50.0	205.0	65.0	97.7
	CC3800-1	Crawler	17.0	-	-	11.0	93.5	LSL-LF3	-	-	11.0	117.0	12.0	50.0	165.0	-	98.2
	CC3800-1	Crawler	17.0	-	-	11.0	93.5	LSL-LF2	-	-	11.0	120.0	12.0	50.0	165.0	-	98.2
	CC3800-1	Crawler	21.0	5.0	3.0	11.0	106.1	LSL-LF4	-	-	19.0	120.0	12.0	50.0	165.0	65.0	86.5
	CC3800-1	Crawler	16.0	-	3.0	13.0	107.0	SWSL	-	-	-	102.0	30.0	50.0	165.0	-	85.8
	CC3800-1	Crawler	23.0	-	1.0	11.0	103.0	SFSL	-	-	-	102.0	30.0	50.0	165.0	65.0	89.1
	LG1750SX	Lattice boom	30.0	8.0	26.0	15.0	214.0	SX3LD4B	-	-	-	132.4	-	-	220.0	400.0	42.9
	LG1750SX	Lattice boom	30.0	8.0	26.0	15.0	209.0	SX2LD4B	-	-	-	132.4	-	-	220.0	400.0	43.9
	LG1750SX	Lattice boom	29.0	7.0	29.0	15.0	199.5	SXLD4B	-	-	-	133.0	-	-	220.0	400.0	46.0
	LG1750SX	Lattice boom	24.0	2.0	3.0	-	99.0	SXLD4	-	-	-	136.5	-	-	245.0	-	92.7
	LG1750SX	Lattice boom	22.0	-	-	-	96.0	SX3LD4	-	-	-	139.4	-	-	220.0	-	95.6
	LG1750SX	Lattice boom	22.0	-	1.0	-	97.0	SX2LD4	-	-	-	139.4	-	-	220.0	-	94.6
	LG1750SX	Lattice boom	24.0	-	34.0	17.0	177.0	SX3ZLD4B	-	-	-	156.9	-	-	175.0	400.0	51.9
	LG1750SX	Lattice boom	24.0	-	36.0	17.0	156.0	SX2ZLD4B	-	-	-	160.4	-	-	175.0	400.0	58.8
	LG1750SX	Lattice boom	24.0	-	3.0	-	106.0	SX3D4F2/F3	-	F2	-	115.5	12.0	-	175.0	-	86.6
	LG1750SX	Lattice boom	24.0	-	3.0	-	106.0	SX2D4F2/F3	-	F2	-	115.5	12.0	-	175.0	-	86.6
	LG1750SX	Lattice boom	24.0	-	46.0	17.0	150.0	SX3D4F2/F3 B	-	F2	-	115.5	12.0	-	175.0	400.0	61.2

	LG1750SX	Lattice boom	24.0	-	46.0	17.0	150.0	SX2D4F2/F3 B	-	F2	-	115.5	12.0	-	175.0	400.0	61.2
	LG1750SX	Lattice boom	24.0	-	3.0	-	106.0	SXD4F2/F3	-	F2	-	116.1	12.0	-	175.0	-	86.6
	LG1750SX	Lattice boom	24.0	-	42.0	15.0	150.0	SXD4F2/F3 B	-	F2	-	116.1	12.0	-	220.0	400.0	61.2
	LG1750SX	Lattice boom	28.0	-	34.0	17.0	148.0	SX3ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	62.0
	LG1750SX	Lattice boom	28.0	-	32.0	17.0	143.0	SX2ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	64.2
	LG1750SX	Lattice boom	28.0	-	34.0	17.0	127.0	SXZD4F2/F3 B	-	F2	-	154.6	12.0	-	175.0	400.0	72.3
150	LG1750SX	Lattice boom	34.0	10.0	24.0	17.0	138.0	SX2LD4B	-	-	-	163.9	-	-	175.0	400.0	66.5
	LG1750SX	Lattice boom	34.0	10.0	24.0	17.0	148.1	SX3ZLD4B	-	-	-	163.9	-	-	175.0	400.0	62.0
	LG1750SX	Lattice boom	34.0	10.0	24.0	17.0	145.0	SX2ZLD4B	-	-	-	163.9	-	-	175.0	400.0	63.3
	LG1750SX	Lattice boom	34.0	10.0	24.0	17.0	144.1	SX3LD4B	-	-	-	163.9	-	-	175.0	400.0	63.7
	LG1750SX	Lattice boom	32.0	6.0	29.0	17.0	127.0	SXZLD4B	-	-	-	164.5	-	-	175.0	400.0	72.3
	LG1750SX	Lattice boom	32.0	6.0	30.0	17.0	123.1	SXLD4B	-	-	-	164.5	-	-	175.0	400.0	74.6
	LG1750SX	Lattice boom	24.0	-	46.0	17.0	150.0	SX2D4F2/F3 B	-	F2	-	115.5*	12.0	-	175.0	400.0	61.2
	LG1750SX	Lattice boom	26.0	-	36.0	15.0	138.0	SXD4F2/F3 B	-	F2	-	144.1	12.0	-	220.0	400.0	66.5
	LG1750SX	Lattice boom	26.0	-	36.0	17.0	150.0	SX3D4F2/F3 B	-	F2	-	147.0	12.0	-	175.0	400.0	61.2
	LG1750SX	Lattice boom	28.0	-	34.0	17.0	143.0	SX2ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	64.2
	LG1750SX	Lattice boom	28.0	-	34.0	17.0	148.1	SX3ZD4F2/F3 B	-	F2	-	154.6	12.0	-	175.0	400.0	62.0
	LG1750SX	Lattice boom	28.0	-	34.0	17.0	127.0	SXZD4F2/F3 B	-	F2	-	154.6	12.0	-	175.0	400.0	72.3
	CC38.650-1	Crawler	36.0	19.0	-	15.0	94.5	LSL_13	LSL_13	-	-	159.0	-	50.0	225.0	165.0	97.1
	CC38.650-1	Crawler	35.0	17.0	-	17.0	92.3	LSL_15	-	-	-	168.0	-	50.0	225.0	165.0	99.5
	CC38.650-1	Crawler	22.0	-	-	15.0	92.0	SWSL_4	SWSL_4	-	-	108.0	54.0	50.0	225.0	65.0	99.8
	CC38.650-1	Crawler	32.0	-	2.0	11.0	102.0	SFSL_4	SFSL_4	-	-	60.0*	60.0	50.0	225.0	65.0	90.0
	LR1750-2-SX	Crawler	32.0	6.0	26.0	17.0	133.0	SX3ZLD4B	-	-	-	167.4	-	95.0	175.0	400.0	69.0
	LR1750-2-SX	Crawler	32.0	6.0	26.0	17.0	125.0	SX3LD4B	-	-	-	167.4	-	95.0	175.0	400.0	73.4
	LR1750-2-SX	Crawler	30.0	4.0	26.0	17.0	107.0	SXLD4B	-	-	-	168.0	-	95.0	175.0	400.0	85.8
	LR1750-2-SX	Crawler	30.0	4.0	28.0	17.0	113.0	SXZLD4B	-	-	-	168.0	-	95.0	175.0	400.0	81.2
	LR1750-2-SX	Crawler	26.0	-	32.0	15.0	147.0	SX3D4F2F3B	-	F2	-	147.0	12.0	95.0	220.0	400.0	62.4
	LR1750-2-SX	Crawler	26.0	-	34.0	15.0	128.0	SXD4F2F3B	-	F2	-	147.6	12.0	95.0	220.0	400.0	71.7
	LR1750-2-SX	Crawler	28.0	-	32.0	17.0	141.0	SX3ZD4F2F3B	-	F2	-	154.0	12.0	95.0	175.0	400.0	65.1
	LR1750-2-SX	Crawler	28.0	-	34.0	17.0	119.0	SXZD4F2F3B	-	F2	-	154.6	12.0	95.0	175.0	400.0	77.1
	LR1700-1.0	Crawler	22.0	-	1.0	10.5	93.9	HSL3DFB	-	-	-	144.0	15.0	50.0	190.0	50.0	97.8
	LR1700-1.0	Crawler	22.0	-	2.0	10.5	95.0	HSL3ADFB	-	-	-	144.0	15.0	50.0	190.0	50.0	96.6
	LR1700-1.0	Crawler	23.0	-	-	13.0	93.5	HSL3ADFBV	-	-	-	144.0	15.0	50.0	150.0	50.0	98.2
	LR1700-1.1	Crawler	23.0	-	-	13.0	92.1	HSL3DFBV	-	-	-	144.0	15.0	50.0	150.0	50.0	99.7
	LG1750	Lattice boom	30.0	-	14.0	18.0	109.0	SL12D2BF	-	-	-	140.0	15.0	-	220.0	400.0	84.2
	LG1750	Lattice boom	28.0	-	24.0	18.0	101.0	SL9D2FB	-	-	-	136.5	18.0	-	220.0	400.0	90.9

180	LR11000	Crawler	33.0	-	2.0	25.0	100.4	PDW3B2	P	-	-	102.0	84.0	50.0	210.0	100.0	91.4
	LR11000	Crawler	35.0	-	-	18.0	94.4	PDW3B	P	-	-	90.0	96.0	50.0	210.0	150.0	97.2
	LR11350	Crawler	28.0	-	5.0	15.0	109.9	SDWBW	-	-	-	114.0	72.0	-	200.0	-	83.5
	LR11350	Crawler	28.0	-	4.0	15.0	109.9	SDWB	-	-	-	114.0	72.0	-	200.0	-	83.5
	LR11350	Crawler	28.0	-	8.0	30.0	109.9	SDWB2	-	-	-	114.0	72.0	-	200.0	-	83.5
	LR11350	Crawler	32.0	-	8.0	15.0	118.0	PDWBW	-	-	-	102.0	84.0	-	200.0	100.0	77.8
	LR11350	Crawler	32.0	-	20.0	30.0	156.9	PDWP2	-	-	-	102.0	84.0	-	200.0	100.0	58.5
	CC8800-1	Crawler	34.0	-	7.0	19.0	116.1	SWSL	SWSL	-	-	108.0	84.0	60.0	295.0	-	79.1
	CC8800-1	Crawler	46.0	-	-	19.0	96.0	SFSL	SFSL	-	-	108.0	90.0	60.0	295.0	100.0	95.6
	PTC35	Ring	59.0	25.3	35	-	330.8	SFSL2 DS-HD	HD(1600)	1.5/ 1.5	-	134.8	57.2	-	1300.0	-	27.8
	PTC35	Ring	59.0	25.3	37	-	339.5	SFSL2 DS-LD	LD(600)	1.5/ 1.5	-	134.8	57.2	-	1300.0	-	27.0
	PTC35	Ring	56.0	5.7	40.3	-	339.5	SWSL2 DS-HD	HD(1600)	2/ 1.5	-	134.8	63.3	-	1300.0	-	27.0
	PTC35	Ring	59.0	8.8	43	-	329.3	SWSL2 DS-LD	LD(600)	2.5/ 2	-	116.7	75.1	-	1300.0	-	27.9
	CC8800-1 BB	Crawler	38.0	-	-	24.0	94.5	BSWSL	-	-	-	120.0	78.0	60.0	295.0	-	97.1
	CC8800-1 BB	Crawler	48.0	-	1	24.0	100.0	BSFSL	-	-	-	120.0	78.0	60.0	295.0	140.0	91.8
	CC8800-1 BB	Crawler	36.0	-	4	19.0	105.0	SWSL	-	-	-	108.0	90.0	60.0	295.0	-	87.4
	CC8800-1 BB	Crawler	46.0	-	-	19.0	96.0	SFSL	-	-	-	108.0	90.0	60.0	295.0	100.0	95.6

Table A-15 Cranes for 132.3 ton load

HEIGHT [M]	CRANE	TYPE	RADIUS [M]	SHORTER [M]	LONGER [M]	SL RADIUS [M]	CAPACITY [TE]	ACTIVE CHART	BOOM TYPE	JIP TYPE	ADAPTOR	MAIN BOOM [M]	JIB [M]	CENTER - CWT [TE]	CWT [TE]	SL-CWT [TE]	PERCENTAGE [%]
30	LR1300SX	Crawler	10.0	3.6	-	-	136.5	Mode-1	2821-1	-	-	50.0	-	57.0	104.0	-	96.9
	LTM1500-8.1	Hydraulic	8.0	-	-	-	139.0	TY3 (T-50)	92+92+92+	-	-	47.3	-	-	60.0	-	95.2
	LTM1500-8.1 ANSI	Hydraulic	8.5	-	-	-	136.5	TY3 (T-50)	92+92+92+	-	-	47.2	-	-	75.0	-	96.9
	LTM1500-8.1 ANSI	Hydraulic	12.2	-	-	-	132.4	TN (T-50)	0+0+0+	-	-	15.9	21.0	-	135.0	-	99.9
	AC500-1Y	Hydraulic	9.0	2.0	1.0	-	141.0	HA-SSL	90-90-90-0	-	-	42.5	-	-	100.0	-	93.8
	AC500-2	Hydraulic	9.0	2.0	-	-	140.0	HASSL0	90-90-90-0	-	-	42.5	-	-	80.0	-	94.5
60	LTM11200-9.1	Hydraulic	12.0	-	-	-	140.0	T7Y	100-100-100-50-50-50-50	-	-	76.7	-	-	62.0	-	94.5
	LTM11200-9.1	Hydraulic	12.0	2.0	-	-	133.0	T7YVEF	50-50-50-50-50-50-50	-	9.1	59.1*	6.5	-	42.0	-	99.5
	LTM11200-9.1	Hydraulic	15.0	7.0	-	-	134.5	T3YV2VEF	100-100-100	-	18.7	52.2*	6.5	-	112.0	-	98.4
	LTM11200-9.1	Hydraulic	12.0	-	5.0	-	198.1	T3YVEN	50-100-50	-	10.2	40.6*	18.0	-	82.0	-	66.8
	LTM11200-9.1	Hydraulic	12.0	-	10.0	-	195.1	T3YV2VEN	100-50-0	-	20.2	34.7*	18.0	-	162.0	-	67.8
	LTM11200-9.1	Hydraulic	14.0	-	4.0	-	175.0	T37YVEN	100-50-0	-	10.2	34.7*	24.0	-	82.0	-	75.6
	LR1350-1	Crawler	14.0	2.0	2.0	11.0	139.0	SDWB	-	-	-	42.0	24.0	8.0	85.0	200.0	95.2
	LR1350-1	Crawler	19.0	9.0	-	9.0	137.0	SDB	-	-	-	72.0	-	8.0	85.0	200.0	96.6
	LR1350-1	Crawler	19.0	9.0	3.0	9.0	161.5	S2DB	-	-	-	72.0	-	8.0	85.0	200.0	81.9
	LG1550	Lattice boom	22.0	6.0	12.0	13.0	190.0	SDB	-	-	-	70.0	-	-	160.0	250.0	69.6
	LG1550	Lattice boom	22.0	6.0	12.0	13.0	190.0	SLDB	-	-	-	70.0	-	-	160.0	250.0	69.6
	LG1550	Lattice boom	14.0	-	4.0	-	178.0	SW	-	W	-	42.0	28.0	-	160.0	-	74.3
	LG1550	Lattice boom	14.0	-	4.0	10.5	183.0	SWSL	-	W	-	42.0	28.0	-	160.0	-	72.3
	CC2400-1	Crawler	20.0	10.0	-	13.0	138.0	SSL	-	-	-	72.0	-	40.0	160.0	80.0	95.9
	CC2400-1	Crawler	15.0	5.0	-	11.0	136.0	SSL/LSL	-	-	-	78.0	-	40.0	160.0	40.0	97.3
	CC2400-1	Crawler	16.0	5.0	2.0	11.0	148.5	SWSL	-	-	-	42.0	24.0	40.0	160.0	40.0	89.1
	CC2400-1	Crawler	12.0	-	-	-	134.0	SW	-	-	-	48.0	24.0	40.0	160.0	-	98.7
	M16000 ANSI	Crawler	20.0	8.4	1.0	11.0	142.6	SSL	-	-	-	72.0	-	54.4	150.6	62.5	92.8
	M16000 ANSI	Crawler	12.2	-	-	-	137.2	SW	-	-	-	48.0	24.0	27.2	134.3	-	96.4
	M16000 ANSI	Crawler	13.7	-	3.3	11.0	175.9	SWSL	-	-	-	48.0	24.0	54.4	150.6	-	75.2
90	LTM11200-9.1	Hydraulic	16.0	-	2.0	-	142.0	T3YV2VEN	100-100-50	-	20.2	46.4*	30.0	-	162.0	-	93.2
	M16000 ANSI	Crawler	17.0	-	1.0	15.0	145.5	SWSL	-	-	-	42.0	72.0	54.4	150.6	62.5	90.9
	TC2800	Lattice boom	21.0	7.0	1.0	15.0	135.5	SSL	SSL/LSL	-	-	108.0	-	-	160.0	50.0	97.6
	TC2800	Lattice boom	20.0	2.0	2.0	13.0	151.0	SWSL	-	-	-	72.0	24.0	-	160.0	-	87.6
	TC2800	Lattice boom	20.0	-	-	13.0	133.0	SFSL	-	-	-	78.0	24.0	-	160.0	-	99.5
	LR1550	Crawler	18.0	-	-	10.5	140.0	SWSL	-	-	-	63.0	35.0	90.0	200.0	-	94.5
	LG1550	Lattice boom	18.0	-	-	10.5	143.0	SWSL	-	W	-	63.0	35.0	-	200.0	-	92.5

	TC2800-1	Lattice boom	30.0	16.0	-	15.0	133.0	SSLLSL	-	-	-	102.0	-	-	160.0	100.0	99.5
	TC2800-1	Lattice boom	30.0	17.0	-	11.0	136.0	SSLLSL max SGL S7	-	-	-	102.0	-	-	200.0	150.0	97.3
	TC2800-1	Lattice boom	30.0	17.0	-	13.0	139.0	HSSL S7	-	-	-	102.0	-	-	160.0	150.0	95.2
	TC2800-1	Lattice boom	30.0	16.0	1.0	13.0	145.0	SSLLS_MaxSGL	-	-	-	102.0	-	-	160.0	150.0	91.2
	TC2800-1	Lattice boom	13.0	1.0	3.0	-	179.0	SHLH LF2 max SGL S7 LHA	-	-	-	84.0	12.0	-	120.0	-	73.9
	TC2800-1	Lattice boom	11.0	-	7.0	11.0	266.0	HSSL LF2 S7 LHA	-	-	-	84.0	12.0	-	160.0	-	49.7
	TC2800-1	Lattice boom	12.0	-	2.0	11.0	165.0	SSLLSL LF2 max SGL S7 LHA	-	-	-	84.0	12.0	-	160.0	-	80.2
	TC2800-1	Lattice boom	21.0	3.0	1.0	11.0	142.5	HSWSL S7	-	-	-	72.0	24.0	-	160.0	-	92.8
	TC2800-1	Lattice boom	21.0	3.0	1.0	13.0	143.5	SWSL	-	-	-	72.0	24.0	-	160.0	-	92.2
	TC2800-1	Lattice boom	22.0	-	3.0	13.0	156.0	SFSL	-	-	-	72.0	30.0	-	160.0	50.0	84.8
120	LG1750SX	Lattice boom	30.0	8.0	16.0	15.0	209.0	SX2LD4B	-	-	-	132.4	-	-	220.0	400.0	63.3
	LG1750SX	Lattice boom	30.0	8.0	16.0	15.0	214.1	SX3LD4B	-	-	-	132.4	-	-	220.0	400.0	61.8
	LG1750SX	Lattice boom	29.0	7.0	19.0	15.0	199.5	SXLD4B	-	-	-	133.0	-	-	220.0	400.0	66.3
	LG1750SX	Lattice boom	24.0	-	20.0	17.0	177.1	SX3ZLD4B	-	-	-	156.9	-	-	175.0	400.0	74.7
	LG1750SX	Lattice boom	24.0	-	20.0	17.0	147.0	SZLD4B	-	-	-	157.5	-	-	175.0	400.0	90.0
	LG1750SX	Lattice boom	24.0	-	20.0	17.0	156.0	SX2ZLD4B	-	-	-	160.4	-	-	175.0	400.0	84.8
	LG1750SX	Lattice boom	24.0	-	26.0	17.0	150.0	SX2D4F2/F3 B	-	F2	-	115.5	12.0	-	175.0	400.0	88.2
	LG1750SX	Lattice boom	24.0	-	26.0	17.0	150.0	SX3D4F2/F3 B	-	F2	-	115.5	12.0	-	175.0	400.0	88.2
	LG1750SX	Lattice boom	24.0	-	26.0	15.0	150.0	SXD4F2/F3 B	-	F2	-	116.1	12.0	-	220.0	400.0	88.2
	LG1750SX	Lattice boom	28.0	-	11.0	17.0	148.0	SX3ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	89.4
	LG1750SX	Lattice boom	28.0	-	11.0	17.0	143.0	SX2ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	92.5
	TC2800-1	Lattice boom	15.0	1.0	1.0	13.0	134.5	SSLLSL LF2 max SGL S7 LHA	-	-	-	114.0	12.0	-	160.0	-	98.4
	CC38.650-1	Crawler	27.0	12.0	-	13.0	133.0	LSL_13	LSL_13	-	-	129.0	-	50.0	225.0	125.0	99.5
	CC38.650-1	Crawler	35.0	20.0	-	15.0	135.5	LSL_11	LSL_11	-	-	135.0	-	50.0	225.0	205.0	97.6
	CC38.650-1	Crawler	35.0	20.0	-	19.0	133.7	LSL_9	LSL_9	-	-	135.0	-	50.0	225.0	165.0	99.0
	CC38.650-1	Crawler	35.0	20.0	-	21.0	137.3	LSL_15	LSL_15	-	-	135.0	-	50.0	225.0	165.0	96.4
	CC38.650-1	Crawler	21.0	5.0	5.0	15.0	156.5	SWSL_3	SWSL_3	-	-	96.0	30.0	50.0	225.0	65.0	84.5
	CC38.650-1	Crawler	21.0	5.0	6.0	15.0	159.0	SWSL_4	SWSL_4	-	-	96.0	30.0	50.0	225.0	65.0	83.2
	LR1750-2 SX	Crawler	29.0	7.0	17.0	15.0	189.5	SX3LD4B	-	-	-	135.9	-	95.0	220.0	400.0	69.8
	LR1750-2 SX	Crawler	26.0	4.0	20.0	15.0	181.0	SXLD4B	-	-	-	136.5	-	95.0	220.0	400.0	73.1
	LR1750-2 SX	Crawler	24.0	-	20.0	17.0	163.0	SX3ZLD4B	-	-	-	156.9	-	95.0	175.0	400.0	81.2
	LR1750-2 SX	Crawler	24.0	-	26.0	17.0	138.0	SXZLD4B	-	-	-	157.5	-	95.0	175.0	400.0	95.9
	LR1750-2 SX	Crawler	24.0	-	24.0	15.0	150.0	SX3D4F2F3B	-	F2	-	119.0	12.0	95.0	220.0	400.0	88.2
	LR1750-2 SX	Crawler	24.0	-	25.0	15.0	150.0	SXD4F2F3B	-	F2	-	119.6	12.0	95.0	220.0	400.0	88.2
	LR1750-2 SX	Crawler	28.0	-	10.0	17.0	141.0	SX3ZD4F2F3B	-	F2	-	154.0	-	95.0	175.0	400.0	93.8
	LR1750-2 SX	Crawler	22.0	-	9.0	15.0	145.0	SXSD3WB	-	-	-	84.0	49.0	95.0	220.0	400.0	91.2

	LR1700-1.0	Crawler	28.0	10.0	-	13.0	135.9	HSL3DBV	-	-	-	135.0	-	50.0	190.0	150.0	97.4
	LR1700-1.0	Crawler	28.0	10.0	-	15.0	136.4	HSL3DB	-	-	-	135.0	-	50.0	190.0	150.0	97.0
	LR1700-1.0	Crawler	19.0	-	-	10.5	136.1	HSL3DFB	-	-	-	114.0	15.0	50.0	190.0	50.0	97.2
	LR1700-1.0	Crawler	19.0	-	1.0	10.5	136.3	HSL3ADFB	-	-	-	114.0	15.0	50.0	190.0	50.0	97.1
	LR1700-1.0	Crawler	21.0	-	1.0	13.0	136.6	HSL3ADFBV	-	-	-	114.0	15.0	50.0	190.0	51.0	96.9
	LR1700-1.0	Crawler	21.0	-	3.0	13.0	136.5	HSL3DFBV	-	-	-	114.0	15.0	50.0	150.0	100.0	96.9
	LR1700-1.0	Crawler	16.0	-	1.0	10.5	146.0	HSDWB	-	-	-	102.0	24.0	50.0	150.0	50.0	90.6
	LR1700-1.0	Crawler	20.0	-	4.0	13.0	151.1	HSDWBV	-	-	-	102.0	24.0	50.0	150.0	100.0	87.6
150	LG1750SX	Lattice boom	34.0	10.0	6.0	17.0	138.0	SX2LD4B	-	-	-	163.9	-	-	175.0	400.0	95.9
	LG1750SX	Lattice boom	34.0	10.0	6.0	17.0	148.0	SX3ZLD4B	-	-	-	163.9	-	-	175.0	400.0	89.4
	LG1750SX	Lattice boom	34.0	10.0	6.0	17.0	144.0	SX3LD4B	-	-	-	163.9	-	-	175.0	400.0	91.9
	LG1750SX	Lattice boom	34.0	10.0	6.0	17.0	145.1	SX2ZLD4B	-	-	-	163.9	-	-	175.0	400.0	91.2
	LG1750SX	Lattice boom	24.0	-	24.0	17.0	150.0	SX2D4F2/F3 B	-	F2	-	115.5*	12.0	-	175.0	400.0	88.2
	LG1750SX	Lattice boom	26.0	-	12.0	15.0	138.0	SXD4F2/F3 B	-	F2	-	144.1	12.0	-	220.0	400.0	95.9
	LG1750SX	Lattice boom	26.0	-	12.0	17.0	150.0	SX3D4F2/F3 B	-	F2	-	147.0	12.0	-	175.0	400.0	88.2
	LG1750SX	Lattice boom	28.0	-	11.0	17.0	143.0	SX2ZD4F2/F3 B	-	F2	-	154.0	12.0	-	175.0	400.0	92.5
	LG1750SX	Lattice boom	28.0	-	11.0	17.0	148.0	SX3ZD4F2/F3	-	F2	-	154.0	12.0	-	175.0	400.0	89.4
	LR1750-2 SX	Crawler	32.0	6.0	1.0	17.0	133.0	SX3ZLD4B	-	-	-	167.4	-	95.0	175.0	400.0	99.5
	LR1750-2 SX	Crawler	26.0	-	14.0	15.0	147.0	SX3D4F2F3B	-	F2	-	147.0	12.0	95.0	220.0	400.0	90.0
	LR1750-2 SX	Crawler	28.0	-	11.0	17.0	141.0	SX3ZD4F2F3B	-	F2	-	154.0	12.0	95.0	175.0	400.0	93.8
	LR1800-1.0	Crawler	34.0	13.0	-	19.0	134.6	HSL2ZDBV	HSL2Z	-	-	165.0	-	70.0	170.0	250.0	98.3
	M21000 ANSI	Crawler	42.7	-	10.6	21.9	149.2	SWSL	#80	-	-	103.6	61.0	90.7	224.4	350.2	88.7
	PC6800	Lattice boom	25.0	1.0	1.0	15.0	140.3	SWSL	-	-	-	96.0	60.0	-	170.0	-	94.3
	PC6800	Lattice boom	34.0	-	-	15.0	147.0	SFSL	-	-	-	96.0	60.0	-	250.0	80.0	90.0
180	LR11350	Crawler	32.0	-	-	20.0	134.0	PDWBW	-	-	-	102.0	84.0	-	200.0	100.0	98.7
	LR11350	Crawler	32.0	-	6.0	30.0	156.9	PDWB2	-	-	-	102.0	84.0	-	200.0	100.0	84.3
	PTC35	Ring	59.0	25.3	26.0	-	229.7	SFSL2 DS-HD	HD(1600)	-	-	134.8	57.2	-	1300.0	-	57.6
	PTC35	Ring	56.0	5.7	38.0	-	232.9	SWSL2 DS-HD	HD(1600)	-	-	134.8	63.3	-	1300.0	-	56.8
	PTC35	Ring	59.0	8.8	31.0	-	287.0	SWSL2 DS-LD	LD(1600)	-	-	116.7	75.1	-	1300.0	-	46.1
	CC8800-1 BB	Crawler	38.0	-	5.0	19.0	156.9	BSWSL	-	-	-	120.0	78.0	60.0	295.0	140.0	84.3
	CC8800-1 BB	Crawler	49.0	-	-	19.0	136.0	BSFSL	-	-	-	120.0	78.0	60.0	295.0	280.0	97.3
	CC12600	Crawler	42.0	-	6.0	20.0	179.0	SWSL	SWSL/LWSL	-	-	114.0	84.0	-	370.0	-	73.9
	CC12600	Crawler	46.0	-	12.0	20.0	210.0	SFSL	SFSL/LFSL	-	-	114.0	84.0	-	370.0	250.0	63.0
	LR12500-1.0	Crawler	45.0	-	3.0	20.0	154.7	HDWB_Prelim	LongStay	-	-	100.0	90.0	150.0	320.0	-	85.5
	LR12500-1.0	Crawler	45.0	-	3.0	25.0	150.7	HDWB2_Prelim	LongStay	-	-	100.0	90.0	100.0	320.0	-	87.8

A.3.4. Interview Arjon Bakker – Crane Engineer Mammoet – 16/04/2024

Rens: Wie ben je, wat doe je?
Arjon: Ik ben Arjon Bakker, werk bij Mammoet. Ik ben begonnen met afstuderen bij de ontwerpafdeling, toen heette dat nog solutions. Toen ben ik daarna bij kranen terecht gekomen en uiteindelijk daarin doorgegroeid. Maar nu zijn we daarin meer overal, dus niet meer kranen en transport, maar algemeen. Daar zit ik nu. Ik ben nu de meeste, of nou ja niet de meest, maar een van de meer ervaren te worden na 10 jaar. Dus dat is wel anders natuurlijk.
Rens: Te gek! Dan denk ik dat je veel van mijn vragen goed kunt beantwoorden.
Arjon: Ik hoop het.
Rens: Ik zal even uitleggen waar ik mee bezig ben. Basically wat er aan mij is gevraagd als afstudeeropdracht is: Kun jij kijken naar wat wij doen met gantries? Want het grootste probleem waar ze nu tegenaan lopen is dat voor de opbouw van een gantry eigenlijk tegenwoordig al best wel flinke kranen nodig. Omdat ze op best wel grote hoogte moeten komen en die gantry beam is natuurlijk best wel zwaar. Kun jij daarnaar kijken en een mooie oplossing voor verzinnen? Dat is in een nutshell wat ik moet gaan doen. Dus ik heb al allemaal dingen zitten bedenken van: Hoe kun je nou dat ding opbouwen zonder dat je überhaupt een kraan nodig hebt? Of in ieder geval een lage kraan. Maar eerst heb je een definitie nodig van: Wat is nou een grote kraan en wat is nou een kleine kraan? En wat kunnen die kranen, tot welke hoogte, tot welk gewicht? Dus daarvoor heb ik dan met hulp van AutoCRANE een paar loadcases op bepaalde hoogtes gerund. En daar heb ik tabellen van gecreëerd en bepaalde dingen snap ik wel. En bij bepaalde dingen heb ik nog vraagtekens. Waarom is dat zo? En ook dingen van: Waarom werkt dat wel of waarom zou dat dan misschien in de praktijk niet werken? Want vorige week of twee weken geleden toen ik even bij je langskwam zei je: 'Het is leuk dat AutoCRANE dat allemaal geeft, maar
Arjon: Geen handige operationele situatie.
Rens: Nee precies. Dus daar heb ik een paar vragen over.
Arjon: Goed.
Rens: Oke, dan de eerste vraag: Wat is een kleine kraan, wat is een grote kraan?
Arjon: Dat is lastig, dat ligt aan wie je het vraagt. Als jij aan de mannen van de PTC vraagt wat is een grote kraan en wat is een kleine kraan, dan vinden zij elke rups (kraan) die ze zien al klein. En in het normale veld als je buiten kijkt vinden mensen een 100 tons kraan al groot. Dus die vraag is te open daarin. Als je het echt zou moeten verdelen zou je kunnen zeggen: Mobile kranen zien we als klein en vanaf rups kranen en ring kranen is het groot. Maar vergis je niet dat de grootste mobiele kraan, die 1750, dat is gewoon een slagschip, dus dat is ook al 'groot'. Dus dat is geen vast antwoord. Maar als je moet zeggen: Klein, groot? Zeg dan: Mobiel, rups.
Rens: En mobiel is dan ook met een hydraulische mast?
Arjon: Ja, iets waar je niet alle onderdelen voor hoeft aan te voeren om te gaan hijsen.
Rens: Ja, want dat brengt me tot de volgende vraag: Zo'n mobiele kraan heeft vanaf een bepaalde hoogte een jib nodig.
Arjon: Als die geen mast meer heeft, ja.
Rens: Hoe werkt dat? Hoe plaats je die erop? Is da teen kwestie van: Die doe je erop en dan schuif je hem uit? En hoeveel tijd kost dat?
Arjon: Hoeveel tijd het kost ligt ook aan hoeveel je erop wilt hebben natuurlijk. Je kan van 12 meter jib tot 50 meter jib bouwen, dat kost meer tijd natuurlijk. Hoe je hem erop bouwt is op een mobiele kraan krijg je eerst een connectie deel. Daaraan gaan dan die jib delen. Die bouw je in principe in het verlengde van de kraan of mast op. En daarna is het met alle tui connectors een kwestie van optrekken en uitschuiven. Hoelang duurt dat? Ligt aan de jib configuratie, maar ga uit van een dag. Maar als het heel kort is ben je een halve dag bezig.

Rens: Oke en is het dan ook zoals met een rups kraan dat je, want die moet natuurlijk helemaal in de lengte worden uitgelegd. Maar die hydraulische kraan heeft natuurlijk het voordeel dat zijn main boom ingekort kan zijn. Kun je hem aan de ingeschoven mast opbouwen?
Arjon: Ja juist, dat doen ze juist. Ingeschoven bouw je hem op. Dan moet je die hele jib wel plat leggen. We hebben wel situaties waarin we elke keer een stuk jib eraan doen en dan trekken we hem op. En dan weer een stuk jib eraan en dan trekken we hem weer op. Maar dat is echt als er geen ruimte is, dat doe je liever niet. Dat is gevaarlijk werk op hoogte, want je moet met een bewegend iets pinnen in gaan slaan. Maar in de basis: Ja, hoofdmast ingetrokken, jib opbouwen.
Rens: Oh oke, dus in principe zou je kunnen concluderen dat zo'n hydraulische mast minder ruimte nodig heeft om op te bouwen met jib dan zo'n hele rups kraan?
Arjon: Ja natuurlijk. Want die hele hoofdmast moet platliggen en dan heb je ook een backmast die je helemaal moet gaan opbouwen.
Rens: En dan zo'n rupskraan, want zo'n mobiele kraan kan gewoon on-site komen, daar z'n ding doen. Zeg dat hij 1 à 2 dagen nodig heeft om op te bouwen.
Arjon: Ja, ligt aan de configuratie.
Rens: Maar dat gaat geen week duren?
Arjon: Nee, over het algemeen niet. De kraan zelf, zonder jib, heb je vaak in een paar uur al opgesteld. Dat ligt aan de grootte ook. En die komt met zijn eigen ballast wagen en nog een support wagen met extra ballast die die nodig heeft. En dan staat ie wel snel klaar. De jib wordt natuurlijk per vrachtwagen aangevoerd.
Rens: En zo'n rupskraan wordt natuurlijk helemaal aangevoerd op een vrachtwagen en die moet ook helemaal in elkaar gezet worden. Hoelang duurt dat ongeveer?
Arjon: Ligt aan de configuratie, dat is heel lastig om te zeggen. We houden meestal aan, als je alleen de body al hebt, dan heb je al een onderwagen body, twee rupsen en een bovenwagen. Dan ga je los het A-frame erop leggen, die moet er soms nog op. Dan zet je dat in elkaar. En dan begin je met de backmast eerst bouwen meestal. Nou, dat is ook al 30 meter structure wat je in elkaar zet. Dan stop je meestal. Dan begin je aan de hoofdmast te bouwen. Dus ja, hoelang duurt dat? Je bent al gauw een week aan het bouwen. Dat duurt wel even ja. Het ligt echt aan de configuratie, het is zo verschillend, daar kun je niet iets vast op neerzetten.
Rens: Nee oke, maar het duurt wel aanzienlijk langer voor een rupskraan dan voor een mobiele kraan?
Arjon: Zeker.
Rens: En ik had ook gezien dat er een soort van tussen oplossing is, dat is dan wel een mobiel gebeuren, maar die heeft vervolgens wel een lattice boom. Zit dat ook qua opbouwtijd tussen de mobiele kraan en de rups in?
Arjon: Verschilt niet zo veel, alleen de onderwagen is sneller. Bijvoorbeeld de LG, zo noemen ze dat, een LG kraan, die komt aanrijden en die zet z'n poten uit en dan is hij klaar. Dus daar hoeft geen rups aan en een rups aan en opdrukken en noem het allemaal maar op. Maar ja, alsnog moet daar: een bovenwagen bovenop en dan de mast bouwen. Elk voordeel heeft zijn nadeel, je bent wat sneller. En een voordeel ervan, vind ik, omdat je met poten werkt in plaats van met rupsen kun je beter je lastspreiding kwijt. Maar, je kunt niet weg, je kunt niet van je plek af met last. Daarvoor worden natuurlijk de rupskranen gebruikt. Want ik moet overal kunnen zijn. Dus dat is heel project afhankelijk. Ik heb een keer een project getekend, toen had ik vijf opties. Vijf kranen opties uitgetekend omdat ze allemaal zouden komen. En uiteindelijk kwam de eerste optie, dat was jammer, maar het kan.
Rens: Want dat is ook een groot voordeel dan vervolgens van een rupskraan, die kan verplaatsen met last.
Arjon: Ja, die doe je meestal op een kraanbaan. Bijvoorbeeld als je aan een kade staat, wil je eerder een rupskraan pakken dan een vaste kraan. Want die kan zeggen 'nou nu op de achterkant van een schip', noem maar even wat, 'even 10 ton weghalen.' 'Wat doen we morgen?'. 'Morgen doen we aan de voorkant 20 ton hijsen.' 'Oke, rijden.' En dan rolt ie gewoon de andere kant uit. Dat is heel project specifiek, maar dat is wel waar rupskranen voor bedoelt zijn.
Rens: En die anderen staan dus echt gewoon vast?

Arjon: Ja die staan vast. En als je die wilt verplaatsen dan moet ie helemaal afgebroken worden. Verplaatsen en weer helemaal opgebouwd worden.
Rens: Die jib moet er dan echt vanaf?
Arjon: Je hebt tabellen met 'driving with equipement in place', maar dan haal je zulke hoge as-lasten dat de meeste sites dat niet aankunnen.
Rens: Oh ja, want het was iets van 12 ton per as.
Arjon: Ja, voor openbare weg.
Rens: Ja en daar ga je dan natuurlijk wel overheen.
Arjon: Ja ruim. Je gaat wel richting de 25 ton de as. En dan mag je niet hard rijden, de poten moeten uitgeschoven blijven. Zo'n boek is dat *gebaard een dikte van +/- 7 cm*. Kan je wel eens een keertje inkijken, dan zie je dat ook.
Rens: Oke, dat is inderdaad wel een opgave.
Arjon: Maar dat is niet wenselijk hoor.
Rens: Net noemde je ook al ballast. Wat is het nadeel van heel veel ballast?
Arjon: In welke zin?
Rens: Nou ik heb hier bijvoorbeeld *pakt AutoCRANE tabellen* op best wel een hoogte kon ik dan nog een hydraulic kraantje gebruiken, of kraantje, hij staat wel heel erg behoorlijk uitgestald.
Arjon: Ja dit is een 11200, daar hebben we er één van.
Rens: En die had dan best wel wat ballast nodig. En ik heb hier verder nog, kijk hier heb je super lift heb je dan wel 400 ton nodig. Maar wat is het nadeel van zoveel ballast meenemen?
Arjon: Kosten. Je moet bedenken wat je nu vergelijkt is appels met peren. Want dit is een mobiele kraan met 182 ton en een kraan met 400. Maar dit wat jij uit AutoCRANE hebt gekregen. Je hebt hier een LG1750SX, en een SX is al een dikkere mast. LG is een lattice boom. Alleen die 400 ton die uit AutoCRANE komt, die activeert waarschijnlijk niet eens. Dus jij klikt hem aan als dit is het sterkste wat er is. En dan zegt AutoCRANE 'prima, dat is goed.'. Maar AutoCRANE zegt ook 'let op, dit is een indicatieve tool.' Je moet zelf blijven nadenken. Als je deze kraan invoert in Licon, met deze configuratie en last die jij had bedacht. Denk ik, want jij zal zo rechtop staan, dat hier maar 200 ton van geactiveerd wordt. Dus dat betekent dat die gewoon zo achterover staat. En dan staat die gewoon vast. Superlift op de grond. Dus als je dan in Licon deze verandert naar 200 ton, zal de capaciteit mogelijk ook iets teruglopen. Dat is het spelletje met superlift. Daar kan ik twee uur over vol praten.
Rens: Oh echt? Shit haha.
Arjon: Haha. Superlift kranen moet je zien als een wipwap. Je kunt heel sterk zijn door al het gewicht hier te doen, maar dan kun je niks. En als je er wat afhaalt wel, maar dan wordt hij wel minder sterk. Maar ja, dan kun je wel draaien, wat je meestal wel wilt als je aan het hijsen bent.
Rens: Ja ja ja, want vorige week had ik een kranen introductie en daar zei Rutger inderdaad al van 'dat moet in evenwicht'. Hij had het ook over een superlift die in- en uit kan rijden, dat hij op wielen staat.
Arjon: Ja een superlift kar, dat hebben we bijna niet. We hebben een tray en die moet je zwevend houden. Een kar heb je wel, volgens mij heeft de 88 er eentje . Maar we hebben ze niet veel. Wat we soms wel eens doen als er echt een project is waarin we weten dat we een hijsje moeten doen, verkassen en weer een hijsje met 600 ton superlift. Dan gaan we hem bijvoorbeeld op SPMT's zetten. Die hele tray op SPMT. Maar dat is ook niet allemaal wenselijk. Dat zijn allemaal dingen echt project specifiek. Je kijkt eigenlijk altijd dat je je tray zwevend houdt. Dit kan gewoon zijn dat jij die tabellen hier uit AutoCRANE hebt, dat je dat uiteindelijk nooit waar kan maken. Dat is het gevaarlijke.
Rens: Dat is vervelend haha.
Arjon: Ja dat is jammer haha. Geloof me, ik heb dat belletje al zo vaak gehad.
Rens: Dus omdat hij dan gewoon op de grond blijft staan?
Arjon: Ja, hij blijft gewoon staan. Maar stel jij moet ook echt alleen daar zijn en heel sterk hijsen: dikke prima. Maar weet dan wel dat je er 200 ton extra op hebt liggen die je eigenlijk niet nodig had.
Rens: En die moet dus wel getransporteerd worden.

Arjon: Ja, neem eens aan. Buitenland: mag je 20 ton op een vrachtwagen laden. 200 ton, reken maar uit hoeveel vrachtwagenchauffeurs je nodig hebt. *gebaard dat dat prijzig is*. Kijk Carolus die daar op Hoogvliet zit met zware kranen. Die heeft nog een aantal vrachtwagens die mogen met meer rijden. Dan ben je blij. 40 ton heeft hij er volgens mij een paar. Maar dat gaat niet zomaar. Dus elke plak ballast die jij aan wilt laten voeren, die niet nodig is, is eigenlijk een vrachtwagen, geld, brandstof down the drain.
Rens: Dat zit dus vooral in het transport?
Arjon: Ja, en doodstaal, daar maken we geen winst op. Ja, als je het op het project laat liggen en de klant betaalt, dan maak je er winst op. Maar niet als kraanballast.
Rens: Interessant. En dan had ik ook ergens gezien dat er wel een radius voor de superlift was maar dat er geen superlift gewicht was. Wat is dat?
Arjon: Dan is de tray niet aangekoppeld. Maar je moet zien, je hebt je hoofdwagen en een backmast, daaraan hangen pendens naar beneden en daar hangt je superlift aan. Je hebt tal van hijsjes dat jij de backmast, stel 19 meter hebt staan, 19 meter superlift, geen superlift eraan hebt zitten. Dat je iets oppakt heel dichtbij. Dan draai je, doe je de superlift vast, staat die dus achterover. Want je staat op minimum radius, met eigenlijk te weinig gewicht. Dan ga je eerst helemaal aftoppen totdat die superlift loskomt en dan kun je instaleren. Dat wordt heel vaak gedaan. Omdat je dichtbij geen superlift nodig hebt, maar ver wel. Dus dan begin je je hijs op 25 meter, 19 meter superlift, zonder superlift, en dan maak je hem vast en dan top je misschien wel af op 50 meter, met superlift.
Rens: En komt er dan wel gewicht op die superlift?
Arjon: Ja ja, weet ik veel 200, 250, 300 ton. Dan komt er iets op die tray, die je natuurlijk vastkoppelt. En als je eenmaal het stuk hebt neergezet, dan zal die kraan weer achterovergaan. Dan ligt die waarschijnlijk weer vast en dan moet je hem weer losmaken en dan kun je weer optoppen. Dat noemen we een hengelspelletje. Dat is even hengelen. Maar je hebt ook klussen dat je met superlift contant staat omdat je juist heel sterk moet zijn de hele tijd. Dus weer: project specifiek.
Rens: Waarom is dit zo'n ingewikkeld probleem?
Arjon: Het lijkt heel ingewikkeld omdat je er pas net inzit. Toen ik hier net een jaar in dienst zat dacht ik ook 'lift? Kraan? Huh?' haha. En nu is het allemaal easypeasy, of nouja easypeasy, je zit nog steeds af en toe te schelden als het niet lukt haha. Want het blijft een vervelend spelletje.
Rens: Wat is het nadeel van zo'n superlift?
Arjon: Niks, het is groot, het is lomp en je moet goed nadenken wat je doet. Als jij in een positie eindigt en daarna moet je weer ergens anders weer wat oppakken. Dan moet je eerst die hele superlift tray afstapelen, omzetten en weer opbouwen en dan pas kun je weer verder. Dat kost gewoon een paar uur tijd.
Rens: En het kost heel veel ruimte?
Arjon: Ja, maar die heb je sowieso nodig met je kraan. Als je ergens een rups gaat neerzetten heb je per definitie ruimte nodig. Dus dat is niet zo zeer een probleem. En als het een probleem wordt, dan weet je dat van tevoren dat het krap is en dat je goed moet gaan puzzelen. Ik bedoel, we zetten ook een rupskraan op een postzegel. Dus het kan wel.
Rens: Oke, dus ruimte is niet persé het probleem?
Arjon: Niet altijd. Natuurlijk, een rupskraan heeft meer ruimte nodig dan een mobiele kraan, ook bij het opbouwen. Maar een mobile kraan kan niet hijsen wat een rupskraan kan hijsen. En dan heb je het verschil tussen rupskranen en ringkranen. Kijk hier naar buiten, de PTC, dat is achterlijk, daar kan ik drie rupskranen op kwijt. Maar ja, die doen niet wat de PTC kan. Dus alles heeft voordelen en nadelen. Ja, een rupskraan is groter, je hebt meer ruimte nodig.
Rens: En wat is het probleem van helemaal op de kleinste of helemaal op de grootste radius staan?
Arjon: Als jij op minimum radius gaat staan met maximale capaciteit, ten eerste je kunt niet verder terug. Dus als je een beetje staat te bewegen of net even 20 cm dichterbij wilt. Dat gaat niet, dus eigenlijk moet je daar een beetje van wegblijven. Tenzij je alleen zwenkt op minimum radius. Maar je moet bijvoorbeeld geen installatie dingen gaan doen op minimum radius. Want als dan jouw supervisor beneden zegt 'doe maar een

haartje optoppen’, ja jammer, ik kan niet verder. En dan? Ga je dan je rups verplaatsen, nee want je hebt er wel 200 ton in hangen. Ik noem maar even wat. Dus daar kun je niks mee. En hetzelfde geldt voor het maximum. De maximum radius gaat ook richting de maximumcapaciteit. Daar moet je ook een beetje mee bedenken van: wil ik dat? Ten eerste. En hetzelfde is, je staat op maximum, wat ga je doen? Ik blijf er altijd een meter vanaf, of twee zelfs. Tenzij het echt duidelijk is dat je het gaat redden. En dat project specifiek hoe die staat, dat je dan kunt zeggen ‘ik rups even een half metertje naar voren, dan kom ik er wel. Maar als jij tegen minima en maxima blijft zitten, dat zijn gebieden waarop een gegeven moment de computer in de kraan gewoon zegt ‘klaar’. En dan doe je niks meer. Dat is hetzelfde als mensen die zeggen ‘ja, maar je staat op 99 procent’. Ik zeg ‘prima, als het dan één keer waait en die last doet een beetje dit *gebaard schommelende last*, of de last is zwaarder, dan gaat het alarm af in de kraan en dan ben je klaar.’ Dan doet hij het niet meer.
Rens: Op welk percentage zou je je kraan het liefst willen hebben?
Arjon: Ligt eraan. Is het item gewogen, heb je het eerder vast gehad, hoe zeker zijn we van het stuk, waar is de installatie locatie, wat zijn de omgevingsstandaarden? Heel vaak mogen wij niet eens zo hoog want een Shell zegt bijvoorbeeld ‘Nou ik wil zeker niet dat je boven de 90 procent komt.’ ‘Oke prima, 90 procent.’ Dus ja, waar voel je je comfortabel bij?
Rens: En wat is een beetje een gangbare range dan?
Arjon: Ik heb op 99.8 gestaan, hij kan tot 100. Op 100 doet ie het nog, net. Het is net hoe je het wilt.
Rens: Oke, ik had er hier bijvoorbeeld eentje en die stond dan echt op 60 ofzo.
Arjon: Ligt aan de job, kijk liever heb je het niet. Als je bijvoorbeeld hebt dat een 130 tons kraan het niet redt en een 230 tonner, dus gewoon even 100 ton capaciteit meer. Redt het eigenlijk met twee vingers in z’n neus, ja dan is het niet anders. Maar dan moet je wel gaan bedenken dat je minder ballast meegeeft. Want dat is weer die vrachtwagen, dat is weer kosten, dat is weer handeling, dat is risico dat je vingers ertussen zitten. Je moet verder denken dan alleen het kraantje zelf. Dan gaan je minder ballast aanvoeren. En dan ga je kijken, tot welke range kom ik als ik de 40 ton extra ballast die die kraan heeft, of 20 ton. Als ik die eraf haal, sta ik dan op 80 procent in plaats van 60, dikke prima. Dan is die kraan nog te sterk eigenlijk voor het werk. Dus, nee er is geen gewenst getal.
Rens: En is dat ook wat je net zei dan de reden dat, bijvoorbeeld hier staat dan nog die LTM11200, die staat dan nog op 120 meter. Maar bij 90 meter en 60 meter zag ik dat rupskranen ook de job kunnen doen. Dus is dan de reden; zij kunnen dat ook en dat kost dan misschien maar wat extra, maar dan besparen we op de ballast die nodig is?
Arjon: Nee, want een rupskraan aanvoeren kost veel meer geld dan een mobiele kraan aanvoeren. Maar jij hebt hier zo’n specifieke mobiele kraan. Die kun je bijna uit je vergelijking halen. Want daar hebben we er maar één van, in Duitsland. Ik heb hem in mijn leven denk ik maar één keer gezien, en volgens mij zelfs rijdend, niet werkend. Dat is de sterkste die er überhaupt rondrijdt op de wereld volgens mij. Ik ga nooit verder dan een 1750 mobiel. Dit is zo specifiek. Ik heb hem in 10 jaar tijd denk ik vier keer ingetekend. En dan kwam die niet eens want dan ging er uiteindelijk een rupskraan heen. Dus dat zegt wel hoe exotisch die kraan is. Kijk, als dat nou echt the man for the job is en je krijgt ervoor betaald. Ja, laat hem aanvoeren, maar meestal wordt die niet gebruikt. Hele specifieke kraan is dat.
Rens: Oke, vet. Je hebt natuurlijk ook af en toe back masts en struts enzo, dat is ook allemaal extra tijd. Is dat super veel langer of valt dat wel mee?
Arjon: Of je nu 30 meter hoofdmast of 30 meter backmast opbouwt, het blijft 30 meter mast opbouwen. Alle pendens en dingen die eraan hangen die zitten er ook bij, die moeten gemaakt worden. Ja, dat kost tijd, dat kun je niet in minuten of uren zeggen. Dat kost gewoon dagen. We zijn gewoon dagen aan het werk, ‘een backmast op te bouwen? Kies maar een dag’. Soms gaat het snel, soms niet. Ligt net aan het project. Dat is wat het is.
Rens: En je hebt voor het opbouwen van zo’n crawler crane bijvoorbeeld ook weer een hijskraan nodig toch?

Arjon: Ja, correct. Mobiele kranen zet je daar meestal voor. En bij hele grote projecten heb ik ook wel eens gezien dat er een klein rupsje voor wordt ingezet. Dat kost dan weer te veel, dus het is meestal makkelijker om een mobiele kraan op een paar posities neer te zetten. En die de helemaal in elkaar te laten zetten.
Rens: En heb je dan ook een mobiele kraan nodig om een mobiele kraan op te bouwen die een jib nodig heeft?
Arjon: Ja.
Rens: Oke, dat kan hij niet zelf?
Arjon: Nee, zeker niet, want hij gaat in principe plat met z'n mast en daarna ga je opbouwen.
Rens: Oke, het is dus niet dat hij dat zelf dan klaar kan leggen?
Arjon: Nee, ik heb het er wel eens over gehad. Maar dan moet je alles zé specifiek goed uit laten komen, dan wordt het echt wel heel erg een feestje en meestal is dat niet handig want je moet toch alles gaan verleggen.
Rens: Een kraan mag je dus niet verplaatsen?
Arjon: Welke?
Rens: Als je helemaal opgetopt bent.
Arjon: Nee, een mobiele kraan ga je niet meer verplaatsen. Het is niet: pootjes intrekken en rijden zegmaar. Dat wordt hem niet.
Rens: Wat betekenen al dit soort codes? *wijst naar 'active chat' kolom in tabel*
Arjon: Dat is de configuratie. HA is in het Duits Haupt Auslege. T is weer voor LTM, heeft ook weer een specifieke naam, hoofdmast is dat alleen. Ik zou ook echt alle namen op moeten zoeken hoor voor de codes.
Rens: Weet je waar dat staat?
Arjon: Oh wel nee, dat is gewoon ervaring. En bijvoorbeeld de Y, dat is de afspanning, zo'n Y-guy. Zo'n afspanning die achter op je mast zit. Een N is eigenlijk altijd een jib, dat is beweegbaar. Ntje dat is een beweegbaar. De VE in deze configuraties dat zijn het aantal verlengingen. Dus je hebt zegmaar de hoofdmast, dan heb je nog een stukje verlenging erop zitten en daarna komt pas de jib. Dat heet N. Maar als jij de brochures opendoet van deze kranen, dus weet ik veel, pak die van 1650 bijvoorbeeld, als je de brochure opendoet dan heeft Liebherr ook standaard in zijn brochures met kleurcodes aangegeven wat welke tabel is. Dus dan zie je N, dat zijn alle blauwe delen, alle jibdelen.
Rens: Oh dat staat in de brochures?
Arjon: Dat staat er wel bij in alle brochures.
Rens: Oh, in principe kan ik gewoon al deze…
Arjon: Die zou je in de brochures moeten zien, die laten ze meestal in de brochures zien. Maar zo heb je bepaalde dingen die weet je na jaren uit je hoofd. Bijvoorbeeld voor een 1750, daar heb je staan de TY2VEN, dan heb je echt alles erop zitten. Dan heb je de afspanning en de jib en de verlenging. Zo zijn dat van die codes, die weet je. Bij rupsen heb je op een gegeven moment SSL, dat is de superlift. Dan heb je nog de superlift, of SSL, de sterke mast. Dan heb je nog de superlift dat is dan meestal de SDB. Dat is dan weer de kop en D staat voor Derek, dat is de achterkant. Derek is ook wel de backmast. Nou, zo hebben al die codes een naampje. Maar als je aan mij vraagt 'Wat is SX2D4F?' Dan kan ik zeggen 'Nou, SX dat is de mast, dat is een sterke mast. D en F dat zal staan waarschijnlijk voor het beweegbare deel wat erop zit. Je hebt hier inderdaad een beweegbaar deel er ook op zitten, denk ik. 'shorter/ longer' wat bedoel je daarmee?
Rens: Dat is hoe ver hij nog van die radius af kon wijken dat die de capaciteit nog had zegmaar.
Arjon: F2F3 B, hier heb je in elk geval een jib erop zitten. Meestal is dat gedeelte van de code is van de jib en het voorste gedeelte is hoofdmast deel. Hier want LSL heeft ook alleen, wat je ziet, een hoofdmast. LSL is de dunnere mast. Dus die is lichter. Dus als jij minder zwaar moet hijsen maar wel heel ver, dan wil je niet heel veel gewicht verliezen in je eigen mast. Zo zitten die codes in elkaar. SWSL is beweegbaar. S is de hoofdmast, W is beweegbaar bij een rups en SL is vanwege superlift. Zo heb je al die codes, al die letters hebben hun eigen betekenis.
Rens: En dat kan ik gewoon in die brochures vinden. En dat geldt ook voor die jib-type?

Arjon: Ja, dat gebruiken we eigenlijk niet zo veel. Totdat je op een geven moment écht in de grote rupsen zit. Voor een mobiele kraan is het gewoon: dit is de jib, meer opties heb je niet. Bij kleine rupsen is het ook gewoon: dit is de jib. Ga je op een gegeven moment echt exotisch richting windmolen kranen noemen wij dat altijd. Dan ga je inderdaad de SX masten die jib dat. Zo heb je allerlei verschillen erin zitten.
Rens: En even voor de zekerheid, dit had ik in die training begrepen, dat is hoe ver je al je delen van je hydraulische systeem uitschuift?
Arjon: Ja, voor mobiele kranen.
Rens: En als het allemaal tegelijkertijd is dan was dat weer wat.
Arjon: Ja en je hebt ook 0+100+46, he noem maar op.
Rens: En hier heb je dan vier keer eigenlijk precies hetzelfde, het enige verschil is dan hier een 2tje daar een 3tje.
Arjon: Ja, dat is gewoon een heel specifiek mast type. Van die kranen weet ik dat ook niet uit mijn hoofd. Op een gegeven moment moet je dat ook eens bij de planning gaan vragen 'joh, welke hebben wij überhaupt op voorraad?'. Want al die configuraties hebben wij wel in onze systemen staan, want dat is voor als wij een kraan inhuren. Dat je gewoon kan zeggen 'welke configuratie? Heb je die?'. Het kan heel goed zijn dat wij alleen de SX2 tabel hebben, geen idee. Soms is één code dat er ook een extra lier bij zit, dat is zó kraan specifiek. Dat weet niet iedereen 100%, of de meeste weten dat niet 100% uit hun hoofd. Ik wil het ook allemaal niet uit mijn hoofd kennen. Dat is gewoon te veel om te leren kennen. Maar daar zit een verschil in. En heel vaak is het als je specifiek een kraan voor een job hebt. En je zegt 'Nou ik denk dat ik deze mast configuratie nodig heb.' Dan bel je eerst naar beneden en zeg je 'he Simon, dit heb ik in gedachten, klopt dat? Weet je wel met zo veel meter mast en zo veel jib' en heel vaak krijg je dan 'nee wij hebben liever die configuratie want die ligt al in Duitsland.' Spelletje is dat.
Rens: Ah ja precies, dus buiten dat het mogelijk is.
Arjon: Is het planning technisch een spel. Ik bedoel ik heb wel eens, wat ik zei, vier kranen ingetekend omdat elke keer was van 'nee die kraan is beschikbaar', alle plannen opnieuw. Twee weken later 'nee ik wil toch die kraan', alle plannen weer opnieuw. Omdat de klant wel dan zegt 'ik wil een kraan op tekening de komt'
Rens: Dat lijkt me best wel lastig.
Arjon: Ja daar zit ook de kunst in om de frustratie de baas te zijn en te denken 'oke, weer opnieuw'. En nee, dat lukt niet altijd. Maar dat heeft iedereen.
Rens: En hoe kan het nou dat bij dezelfde load hier maar twee kranen mogelijk zijn en hier heb ik er veel meer? Buiten dat dat met die codes niet zo veel verschilt. Maar hier heb ik toch wel weer dan bijvoorbeeld, echt zo'n crawler. Die heb ik dan weer met een T. Het zijn wel echt meerdere andere opties.
Arjon: Ja het is wel een beetje lastig voor mij om er wat van te zeggen, want je kan in dat programma zoveel filteren dat ik natuurlijk niet weet wat je uitgezet hebt, of aangezet hebt. Of ingevoerd hebt. Want jij hebt ingevoerd waarschijnlijk gewoon een X gewicht op een bepaalde hoogte, ga ik even van uit?
Rens: Ja.
Arjon: Want met welk gewicht houd je rekening? Of kijk je alleen naar capaciteit?
Rens: Van het referentie project, die Jubail gantry, heb ik een mastdeel gepakt inclusief de rigging. Dat gewicht heb ik als een soort minimum, hoever of hoe hoog kun je komen met bepaalde soort kranen.
Arjon: Het kan zijn dat het programma deze eruit filtert omdat hij zegt 'die vind ik veel te sterk, ik ga voor die optie'. Daarom zeg ik, filteren in een programma is goed voor in de basis, maar daarin moet je ook bedenken: wat vertelt dit programma nou? Want ik voer wat in, maar klopt dit nou wat eruit komt. En als je nog geen ervaring hebt, is dat heel lastig om te zien. Van wat ga ik hier nou eens mee doen? Want als je op een geven moment inderdaad iets op hoogte 150 meter hoog en daarna op 180 meter hoog. Ja dan pak je natuurlijk bijvoorbeeld gewoon een LR1800 en daar ga je al het werk mee doen. Dus dat is natuurlijk heel lastig om, hiertussen in. Eigenlijk hier die hoogtes wat je hier zegt, is eigenlijk al project specifiek, punt. En dan kun je gewoon al zeggen als iemand zegt 'ja maar ik moet naar 180 meter', prima, dit valt gelijk allemaal weg *gebaard naar alles onder de 180 meter*. Dus dat kun je eigenlijk niet zo 123 zeggen 'oh dit werkt wel, dit

werkt niet'. En op een gegeven moment kun je misschien voor je eigen rust. Kun je natuurlijk wel zeggen al die LG's die je hier hebt staan, die allemaal om en naar bij hetzelfde opgesteld zijn. Gewoon zeggen, alles eruit strepen, active chart 'diversen'. En dan doe je zelf in je notitieboekje opschrijven welke dat allemaal zijn. Of je laat één code staan en je zet zelf ergens een notitie achter deze regel 'ook deze codes', weet je wel. Want nu heb je een hele grote sheet, waar ik 90, nou 80% dezelfde kranen zie. En dat geeft voor jouzelf geen overzicht.
Rens: Nee precies, maar ik dacht 'ik doe ze er allemaal in, want stel er zit wel een verschil tussen'.
Arjon: Nee het is echt een heel specifieke mast configuratie. Maar ik zit even te kijken want je hebt overal percentages wel staan en ik ben bang dat jij dit allemaal in AutoCRANE doet en dan enter doet en dan hier invult?
Rens: Ja.
Arjon: Ja, voor stel je filtert alles eruit, dan heb je al de helft aan kranen erin staan. Maak je hier een extra tabel bij, hier bij radius, achter radius. Daar zet je je capaciteit. Want dat is het enige waar mensen naar kijken. Radius en capaciteit. En daarna eens een keer naar de configuratie. Want iedereen wilt weten hoeveel ton op hoeveel meter kan ik hijsen. En hoeveel procent zou ik dan staan? Want jij vult hier wel je loadcase in, dat is goed, alleen ik weet niet, ja dan moet ik terug gaan rekenen. 98%, ja dan zal die ongeveer 100 ton last kunnen hebben maar, wat zit hierin? Je zegt net, rigging, last. Wat weegt mijn haakblok? Dat is bij elke kraan ook verschillend.
Rens: Oh dat is niet deel van de rigging?
Arjon: Nee, zeker niet. Een kraanblok kan zomaar vanaf, bij de mobiele kraantjes één ton wegen. En bij de grote rupsen 18 ton. Het PTC-blok weegt volgens mij 60, zoiets, als het niet meer is. Daar kun je je vies in vergissen. Een rupsblok in de 8800 die ik in Vlissingen heb die woog 15.8 ofzo daar in de buurt, 16 zeg maar even. Dus daarin is dit heel mooi, maar moet je eigenlijk wel even toevoegen waar je hier nou van uit gaat. En het is goed dat je zegt 'ik ga voor elke kraan dezelfde load case doen', maar in deze percentage range zit ook nog als het goed is een stukje hijsblok, hoop ik, of heb je die op nul gezet?
Rens: Ik heb het hijsblok op nul gezet, omdat ik dacht dat het deel van de rigging was.
<u>*This proved to be not true, the hook block was already included*</u>
Arjon: Nee. Dus in die zin kun je alles wat nu boven de 95% zit mogelijk niet hijsen. Want er komt gewoon een haakblok bij van tien ton, als je pech hebt.
Rens: Ja want zo'n haakblok moet die hele rigging, dat is die 51.2, 51 . zoveel, moet dat gaan tillen. Dus dan heb je dus een 50 tons haakblok nodig bijvoorbeeld?
Arjon: Ja, en die heb je niet, want een standaard haakblok van een mobiele kraan is bijvoorbeeld een 125 tons haakblok, dus die hangt erin. En voor een rupskraan heb je wat meer verschillen natuurlijk. Maar ja, als je 51 ton hijst met een rupskraan. Dan kun je dat bijna op de runner doen. Een runner heeft meestal zo'n 50 ton capaciteit bij een grote rupskraan.
Rens: Oh, dat gaat wel sneller dan.
Arjon: Ja, maar ga je daarvoor een kraan mobiliseren? Nee, dus dan gaat er weer een gewoon haakblok in. En dan is het volgende waar je dan ook rekening, kijk dan wordt het wel een hele diepe studie, dus die moet je als notitie misschien er even bijzetten. Ik heb het haakblok zwaarder gekozen, want: Als jij het haakblok helemaal naar boven doet, want jij zit op grote hoogte te hijsen. Dan is het gewicht aan de achterkant, qua hijsdraad is meer dan wat er aan de voorkant hangt. Dus als jij een te licht haakblok kiest. En hier hangt dan al dat draad over de mast naar de lier. En de machinist zegt 'haakblok naar beneden'. Dan zegt deze hier op de wrijving van de schijf en er gebeurt niks. En dan zie je zo je lier afspoelen en je draad als een rommeltje. Dus je hebt ook met bepaalde configuraties, dit soort grote configuraties, want jij staat met 164 meter mast, met nog 24 meter jib erop. 200 meter omhoog. Heb jij een minimum haakblok gewicht, want anders komt je haak niet naar beneden. Ja dat zeg ik, je begeeft je op een hele slippery slope. Dus daarom moet je denk ik, deze opzet is goed. Je hebt daarmee kennis van zaken een beetje geproefd. Zet voor jezelf eerst even vast: waar kijk ik wel naar en wat niet buiten beschouwing. Kijk, als jij die kranen pakt en je pakt standaard gewoon

een zwaar haakblok, dan komt het over het algemeen wel goed. Maar ga niet in die tabellen zitten ‘nou ik pak lekker het kleinste haakblokje’ dan ben je jezelf rijk aan het rekenen. Want het haakblok moet wel naar beneden komen.
Rens: Shit, ik dacht dat het deel van de rigging was haha.
Arjon: Haha, ja het is goed dat ik het vraag. Dus voordat je deze tabellen maakt, doe, heel mooi ‘table 4 cranes’, maar doe ervoor even een ‘table 4.1: this is my loadcase’.
Rens: Ja nee, ik moet de hele afbakening enzo, dat was ik nog van plan om te zeggen van ‘dit is in principe dit en dit’
Arjon: Afbakening, ik ga ervan uit dat het standaard kraanhaakblok dat in AutoCRANE erbij zit. Kijk je gaat niet zeggen ‘ik ga 5 ton hijsen met een 1200 tons haakblok’, dat is onzin. Maar je moet wel een beetje reëel kijken naar de kraan die je pakt. Dat je een beetje een normaal haakblok pakt. En je kunt beter in je hijstabellen een te zwaar haakblok erin hangen. Dat er eigenlijk een week voor uitvoering wordt gezegd ‘ja dat hoeft niet, haal even twee ballast blokken van dat blok af. Dus dat blok weegt maar 15 ton in plaats van 20.’ Dan komen ze ook gelijk wat lekkerder uit in capaciteit. Tenzij je op last-minute engineering doet en gewoon te horen krijgt: die kraan gaat er heen met dit haakblok en het haakblok weegt 6.7 ton. ‘Oke, 6.7 ton’ dan heb je het exact allemaal. Dat zijn de twee notejes waarmee ik zat.
Rens: Ik denk dat het al richting de laatste vraag gaat hoor. Wat heeft de voorkeur, zeg maar zo’n mobiele kraan met alle toeters en bellen, dus alles erop en eraan, helemaal uitgesteld. Of dan gewoon een rupskraan die dan?
Arjon: Voor mij persoonlijk, of in het algemeen voor het bedrijf?
Rens: Algemeen.
Arjon: Ligt aan de duratie van het project. Als jij een ‘weekputje’ hebt: gewoon een mobiele kraan, ja dan een mobiele kraan. Heb jij meerdere weken, dan een rups. Een rups is duurder om aan te voeren. Dus dat is nogmaals, project specifiek. Sta jij aan de overkant bij Damen één week, daar je geen rupskraan voor aanvoeren want dan duurt het opbouwen en afbreken langer dan het hijsen. Een mobiele kraan is sneller op locatie. Die is ook meestal makkelijker opgebouwd, dan een rupskraan. Gronddruk technisch ook, is dat fijner. Persoonlijk zelf, ja, rupsen zijn machtig mooi, ze zijn groot, ze zijn lomp. Alleen heel vaak worden ze bij heel projecten alleen gebruikt voor capaciteit. Dus je gaat bijna niet rupsen ermee. Ik wel eens een project gehad waarbij ik 80 meter moest rupsen met last. Ja dan is het een ander verhaal. Dus persoonlijk, ja tuurlijk, ik vind een kraan met alles erop en eraan, dat is prachtig om te zien. Dat is echt prachtig om te zien. Maar voor een bedrijf is het projectafhankelijk. Dat zou ik, dan moet je echt met een, als je in zo’n project komt waarin twee opties echt kunnen. Of een rups op een vaste locatie met een bepaalde configuratie of een mobiel die kan het ook nog. Dan moet je op een gegeven moment gewoon naar de projectmanager en zeggen ‘zeg het maar, ik kan met deze twee kan ik het doen, jij weet hoelang we er gaan staan en wat voor een nevenwerk we nog meer doen.’ Want niet alles komt bij engineering terecht natuurlijk, wij kijken meestal de zwaarte hijsen en op een gegeven moment op de achtergrond hoor je: ‘ja als we die kraan hebben staan dan kan die ook wel even daar een stellinkje weg hijsen en dan kan hij nog even dit en dan kan hij nog even dat.’ Weet je wel, allemaal wat gewoon gebeurt als daar wordt gewerkt. Dus dat is een projectmanagers keuze op een gegeven moment en daarmee ook het overleg met de klant ‘ja, zeg maar als het uitloopt dan loopt het uit. Die kraan moet weg. Een rupskraan die kost zoveel aanvoeren.’ Het is gewoon een geld spelletje, dan.
Rens: Ja precies dus in principe is dat niet echt aan de engineer?
Arjon: Nou ja het is een beetje een samenspel. Planning, projectmanagement, kosten en daarin engineering: wat is technisch het beste, of makkelijkste of fijnste? Daar kun je geen vast antwoord op geven nee. Ja, hebt een aardige beerput opengetrokken, haha. En daarom zeg ik, ik kan nog twee uur met je zitten, leuk hoor. Maar het is wel een hele hoop werk wat je hebt gedaan, maar haakblok moet je wel mee gaan nemen ja. Maar ik zou dit ook niet weggooien. Ik zou dit ergens apart opslaan en zeggen als dan iemand vraagt ‘waar ben je al die uren aan kwijt?’, ‘ja sorry ik was het haakblok vergeten.’ Kan gebeuren. Maar dat hoort niet bij de rigging. Maar ook voor jouw load case, je hebt een voorbeeld genomen om iets uit te zoeken. Ik weet niet hoe

vast je moet houden aan die load case, maar ik zou bijvoorbeeld hiervan voor het lekkerder werken, zou ik daar gewoon 52 ton van maken of 50. En van deze zou ik gewoon 130 maken, of 140.
Rens: Maakt dat niet zoveel uit dan?
Arjon: Tuurlijk, op grote hoogte maakt elke ton uit. Alleen ik zit een beetje met dat .3. Als iemand één lik verf extra op dat stuk heeft gedaan weeg je al meer. Snap je wat ik bedoel? Dus je moet even, daarom zeg ik, ik weet niet jouw opdracht specifiek. He, ik heb hem niet gelezen, ik heb niet de voorwaardes gezien. Maar dan zou ik bijna zeggen 'maak hier dan 132 van en hier 51 half' in ieder geval wat meer rond dan 51.2. Maar dat is mijn gevoel, ik ken jouw opdracht niet. En die haakblokken moeten er wel echt in. Maar maak dan hier ook een tabel bij met capaciteit en haakblok gewicht.
Rens: Want de capaciteit is fijner om te weten dan het percentage?
Arjon: Beide. Eigenlijk moet je hebben als jij dit hebt en jij hebt een capaciteit en een haakblok erbij. Dan kan iedereen zien: oke ik sta dus op 95% met dit haakblok, maar wat nou als we tegen die tijd zeggen 'nou doe eens wat minder haakblok, dan kan mijn last misschien iets omhoog.' Dan heb je wat meer om over te praten. Dan nu alleen 98, 95% procent. Oke en met hoeveel last? Ik moet het al vragen omdat ik het niet zie, laat staan mensen die het moeten controleren. Want die lezen dit en die zien hoogte, ja welke hoogte? Dus hiervan zou ik maken 'lifting height'. Of 'hook height'. Even dat het voor mensen wat meer spreekt. En dan dit, filter een beetje dezelfde er gewoon uit. Dan wordt het echt heel snel overzichtelijk.
Rens: Ja want dit is dus eigenlijk vier keer precies dezelfde kraan?
Arjon: Ja. Ja, met een iets andere mast configuratie. De vraag is; hebben we die? Geen idee. Ook nog eens allemaal met nagenoeg dezelfde hoofdmast, jib configuratie, met dezelfde couterweight en met dezelfde super lift én dezelfde percentages. Dus eigenlijk..
Rens: Het enige verschil zit in een 2tje of een 3tje.
Arjon: Ja en dat is net welke mast je hebt. Dus eigenlijk is dit gewoon een klein boompje die je omgekapt hebt. Papier. Haha.
Rens: Haha.
Arjon: Snap je, daar moet je even naar kijken. Ik denk dat het voor jezelf ook heel veel rust geeft om het op die manier te doen.
Rens: Ja, het maakt die tabellen sowieso veel leesbaarder.
Arjon: Ja. Oh en een AC250 en een 220 hebben we niet meer. Dan moet je even kijken voor een LTM1230 en een LTM1250 hebben we volgens mij ook nog.
Rens: En waarom komen die er dan niet uit?
Arjon: Dat zeg ik, het filter programma.
Rens: Want ik hoopte in ieder geval zo min mogelijk filters te hebben, laat ik het zo zeggen.
Arjon: Ja, waarschijnlijk heb je ook nog ergens aangegeven: geef mij maar maximaal 10 opties. Daar staat ook ergens een vinkje voor. Geef me zoveel opties. Dan gaat hij ook niet verder zoeken. Als hij ze gevonden heeft dan heeft hij ze gevonden. Als dat nou toevallig een kraan is die alleen in Abu Dhabi rondrijdt, dan ga jij niet de kraan vinden die hier rondrijdt. Dat is een beetje het lastige. En wij hebben elke kraan nog in onze cranebrowser staan, in onze AutoCRANE, cranebrowser, hoe je het moet noemen. Omdat als wij een belletje krijgen uit Egypte. Ja, daar rijdt die misschien nog wel rond, ziet die er niet zo strak meer uit natuurlijk. Maar dat is er dan nog wel. Dus daarin, dat zeg ik, dat is het hele lastige.
Rens: Ja, er komt nog best wel wat werk aan haha.
Arjon: Haha, ja maar wat is werk, wat is werk. Het is als je erin verdiept, dan moet je dit niet een week voordat je gaat afstuderen horen. Dan kun je het beter nu horen. En daarin moet je inderdaad wel gaan zeggen, nou haakblok. Haakblok dat is gewoon iets, dat moet je gewoon meenemen anders heb je gewoon geen geloofwaardig verhaal meer. En of we kranen nou wel of niet hebben, oke prima dat is dan alleen voor deze markt hier, he. En niet als Mammoet breed, want misschien dat we ze in Duitsland rijdt die misschien wel. Ik weet dat we hem hier niet hebben in ieder geval. Dus daar moet je niet al te veel aan wegen.

Rens: Nee want het is ook wel interessant, alleen het kost natuurlijk veel tijd. Maar dat is wel goed, want nu kan ik wel gerichtere vragen stellen dan twee weken geleden dat ik even bij jullie langs was.
Arjon: Je moet even kijken, je hebt de ANSI-tabellen en de, hoe heet die andere? Je hebt twee tabellen, de een is 75% tabel, die hoef je hier niet te gebruiken in Europa. Dat is een gereduceerde tabel voor Amerika. Kijk je hebt hier ook weer LG, LG, LG, echt zo'n lap lijst. Hier zit dan nog wel wat verschil. Maar filter even eruit wat merendeel hetzelfde is. LR11000, mooie rups, mooie rups. Oh een P-boom tabel, mooi man! Een PDW. Dat is de grote mast. Dus dat is één mast en dan gaat die *gebaard dubbel zo dik* en dan weer terug naar één. Heel duur haha. Dus zoek van die kraan, nu je hem hier hebt staan. Zoek daar ook even zijn gewone tabel. Dan zie je gelijk het verschil in capaciteit. Daarom zeg ik: het filter komt met de meest exotische mastdelen, de exotische tabelletjes, gekke dingen. Soms helpt het om even een kraan te openen, in de duty chart gewoon alleen de beweegbare jib op te zoeken van die rupskraan en gewoon eens te zeggen op 30 meter, dubbelklik. En dan zie je op een gegeven moment, 30 meter mast, 24 meter mast. Oke dat is te laag. Dan ga je zelf filteren. Ik wil minimaal een hoofdmast van 80 meter. Whoep. En dan doe je weer dubbelklikken op 20 meter en dan zie je, oke dan kan ik zoveel ton hijsen. Het gevoel moet je even krijgen. Dat duurt lang, dat gevoel krijgen. Maar dat helpt misschien hierboven af en toe even.
Rens: Want hier kan ik eigenlijk nog geen goede conclusies uit trekken?
Arjon: In mijn ogen niet. Je hebt in deze tabellen wel staan dat er altijd wel een oplossing is voor Mammoet om te fixen. Maar dat we al.
Rens: Nee precies, maar ik zou hier nog niet de conclusie uit mogen trekken van. Want eigenlijk waar ik naartoe wil werken is. Uit het verhaal is wel een beetje gebleken dat de meest goedkope en snelle kraan is zo'n mobiele kraan. Tot welke hoogte en welk gewicht kan hij dan gaan. Je ziet al heel snel eigenlijk, 180 meter, dat gaat hem gewoon niet worden.
Arjon: 100 is al een uitdaging voor mijn gevoel.
Rens: En je zegt ook die 11200, of '1200', dat wordt hem ook gewoon niet omdat, ja daar heb je er maar één van.
Arjon: Ja volgens mij heb je er één rondrijden in Duitsland. Voor zover ik weet. Misschien dat we er twee hebben hoor, maar ik heb hem hier in Nederland echt nog nooit gezien.
Rens: Dus eigenlijk kan je wel een soort van de conclusie daaruit halen, nou tot 90 meter kun je best wel gaan met zo'n ding. Misschien kan wel ietsje hoger.
Arjon: Kijk, is het je nou ook opgevallen nu he. Je hebt al deze kranen, maar ik heb nog geen enkele keer de zwaarste mobile kraan gezien. De LTM1750. We hebben één mobiele kraan en ik zal je uitleggen waarom die er waarschijnlijk niet uit is gekomen. Kijk, de 1650 8.1 die heeft twee configuraties een T5 en een T3. Een T5, dat betekent dat hij 80 meter hoofdmast heeft. Dus in enkel die pijp heeft hij 80 meter. Een T3 heeft 50 of 60 meter mast. Dat is dus al een compleet verschillende kraan. Dan kijken we alleen al naar de voorkant van de tabel. Alles daarachter dat is redelijk hetzelfde. Een 1750, dus eigenlijk een sterkere kraan, dat is er eigenlijk nog een van de oudere generatie. Die heeft maar 50 meter mast en dan moet hij een jib op gaan bouwen. Dus die redt de hoogtes niet die jij wilt. Maar eigenlijk is het een sterkere kraan. Dus daarom wil ik zeggen; een sterkere kraan, hoeft niet persé beter te zijn voor je klus. En ook, ook al heb jij een T5 mast, sta je maar op 64 meter. Want als jij een hengel helemaal uitschuift, wordt die slap. En als jij alle deeltjes voor de helft inschuift wordt die heel sterk. Daarom zie je de meest sterke configuraties in de 46 range, 46-46-46 is gewoon heel sterk. Probeer het zelf thuis maar, pak pvc buizen die allemaal net een stukje kleiner zijn dan elkaar. Schuif ze allemaal voor de helft in, dat krijg je niet gebogen. Schuif je ze allemaal uit *gebaard breken*. Dat is de gedachtegang, de techniek achter de gedachtegang. Daar gaat heel veel rekenwerk aan te pas natuurlijk.
Rens: Maar dat die 1750 er niet uit is gekomen is dus niet gek?
Arjon: Nee, dat vind ik niet gek, helemaal niet met die gevraagde gewichten op die hoogtes. Maar daarom wil ik zeggen. Je hebt een tooltje gebruikt, wat in de basis goed is, want je hebt nog niet genoeg ervaring om te zeggen; ik pak die kraan, ik pak die kraan. Maar weet wel wat het tooltje doet. Een mens is zo goed als de

spullen die hij gebruikt. Daarom zeggen we hier ook heel vaak voor de grap als iemand iets heeft. ‘Dat ligt aan degene tussen de stoel en het scherm’ en dat is de lagging factor, de computer doet wat je zegt.
Rens: Ja. En een vraag die ook nog van boven kwam was, aan mij dan, was. Joh maar wat nou als die gantry die ik nu als referentie neem, die in Saudi-Arabië, die was, nou het hoogste deel was 145 meter ofzo.
Arjon: Die voor die Dubai Eye?
Rens: Ja. Nee, voor de Jubail. En die gantry die was 145 meter ongeveer. De vraag was; wat nou als we de gantry beam gewoon uit verschillende onderdelen laten staan? Dat we elke keer bijvoorbeeld maar 50 ton naar boven moeten hijsen. En dat we hem dan daarboven, ja dat is dan wel even vervelend om op 145 meter hoogte iets in elkaar te zetten. Maar stel; is dat mogelijk? En ja, dan heb je toch nog steeds een rups of een lattice boom nodig. Ja, dus leuk, maar heel veel winst ga je daar niet uithalen. Want je hebt nog steeds gewoon een gigantisch sterke kraan nodig.
Arjon: Nou ja, als je drie delen van 50 ton naar boven hijst, ik noem maar even een getal, of dat je één deel van 150 ton naar boven hijst. Dat maakt een enorm verschil voor een kraan.
Rens: Toch wel?
Arjon: Ja zeker. Ga jij maar eens zoeken of je 150 ton op die hoogte kunt krijgen, dan ga je uitkomen in de PTC-range, of heel grote rupsen.
Rens: Ja dat heb ik hier inderdaad. Op 180 meter, dan heb je die PTC inderdaad nodig en dan gebruik je die PTC niet op zijn volledige capaciteit.
Arjon: Ja dan denkt die PTC ‘huh, wat kom ik hier doen? Echt een trut hijsje.’
Rens: En hier, dit zijn wel echt hele sterke kranen dan? Die rups, want dit is voor 132 zoveel ton.
Arjon: Ja en een SX mast, sterke mast. En waarom noem jij een LG1750 SX een lattice boom en een LG1750-2 SX een crawler? LG is een potenkraan.
Rens: Misschien heb ik daar een klein foutje gemaakt.
Arjon: Je kunt het zo zien: LR is altijd een rups, LG is een potenkraan, exacte benaming moet ik ook opzoeken. En CC is weer van Demag, Terex/Demag en dat is ook een rups. Een LTM is een mobiel gewoon. En de AC’s waren ook mobiele kranen, maar die hebben we niet zo veel meer. Weet je wat ook heel mooi zou zijn eigenlijk nog? En dan houd ik op met uitbreiden van je tabel. Is je hebt de kranen beschreven, maar alleen de types. En ik weet dat een LTM van Liebherr is. En dat een CC van Terex/Demag is. Dan heb je nog de ATF-mobiel, dat is van Faun. Niet iedereen weet dat. Ik denk als je hier een tabelletje voor maakt. Of voor in het algemeen een tabelletje maakt. Even met hé let op, in mijn tabel heb ik niet het merk genoemd, want dan zie je 100 keer Liebherr, Liebherr, Liebherr, Liebherr staan, LTM is voor Liebherr. CC is voor Terex/Demag. Even de hoofdmerken waar je het typetje aan kunt koppelen. Dan weet iedereen als je dadelijk roept ‘ja dan pak ik een Liebherr.’ Dan weet iedereen ah dan gaan we in de LTM-ranges of in de LR ranges door. Want even, je gaat nu heel erg de verdieping in omdat je hier al wat langer zit. Maar iemand die jou moet controleren. Die daar ergens hoog op een kantoortje zit. Geen flauw idee.
Rens: En nu we het er toch net over hadden, zeg maar. Hier zie je dan die met die SX mast, die LG1750 SX, en dat is dan op 150 meter met 130 ton. En hier heb je 90 meter. En dan heb je eigenlijk gewoon een beetje hetzelfde. Sorry 150 meter 90 ton. Dan heb je eigenlijk best wel redelijk dezelfde kraan nodig. Toch? Maar het is wel 40 ton minder.
Arjon: Ja, configuratie. Deze? 90 ton op 150 meter, en?
Rens: 130 ton. Wat is daar dan het grote verschil tussen?
Arjon: Eens even kijken. Je hebt ze allebei op 34 meter radius geschoten? Nou ja het grote verschil is dat je met de 91 ton op 150 meter sta je op 66%. En die andere op 150 meter hoogte sta je op 95% procent. Dat is het verschil. De capaciteit, de kraan kan het. Dat is wat ik zeg, het tooltje zegt ‘het kraantje kan het’. Is het daardoor de beste keuze omdat je op 66 % staat? Misschien niet, misschien wil je een andere kraan gaan gebruiken.
Rens: Nee, maar in principe is het gewoon dezelfde kraan?

Arjon: Ja, het is wel dezelfde kraan. Jazeker, dezelfde kraan alleen. Eigenlijk zou die bij die 91 ton niet voor zijn volle capaciteit benut worden. Maar dan ga je waarschijnlijk weer minder super lift meenemen. Dan staat ie weer op 90%. Het spelletje.
Rens: Het is wel een interessant probleem.
Arjon: Je hebt te veel keuzes. Daarom is het soms handig als je een klus hebt om te bellen 'Welke kraan gaat erheen?' 'Ja die' 'Oke'. Of 'Welke hebben we beschikbaar in die periode?' 'Ja die'. Nou dan kun je in ieder geval naar alle drie even kijken. Die is het beste, kunnen we die dan reserveren? Nu ben je in een grote puinbak aan het graaien. En 'Oh nou die kan'.
Rens: Ja, het is eigenlijk te specifiek?
Arjon: Ja, sommige dingen zijn gewoon te specifiek.
Rens: Ik zal zo nog wel aan Ivo vragen wat hij precies wil. Want bepaalde conclusies kan ik al wel trekken. En sommige dingen totaal niet.
Arjon: Soms is 'nee' ook een antwoord.
Rens: Nee precies, alleen moet ik even kijken in hoeverre ik daar nog de diepte in ga en dat ga uitwerken.
Arjon: En misschien moet je ook wel overleggen dat je een pad kiest en de diepte in gaat en ongeacht wat je in dat diepe gat gaat vinden. Kies een weg en ga ervoor. Dat zou je ook met degene die je begeleidt moeten bespreken.
Rens: Ja ik ga dit inderdaad even met Ivo bespreken, want zo te horen kan ik behoorlijk diepgaan nog.
Arjon: Ja dat is het. En op een gegeven moment moet je gewoon zeggen, nou pak dan inderdaad dit, hier heb je dan het haakblok niet meegenomen. Maar dit heb ik eruit geleerd, één: rigging is geen haakblok. Nou prima. Maar pak dan eens even gewoon een bepaalde gegeven capaciteit kraan uit elke range. En ga daar nou eens alleen met die kranen eens kijken; wat is nou het verschil? En wat kan ik nou doen? Dan heb je denk ik veel sneller een overzicht. Dan lappen lijsten, denk ik.
Rens: Ja vet. Werk aan de winkel. Dankjewel!
Arjon: Geen probleem jongen. Nou uurtje precies.
Rens: Haha perfect.

A.4. MAMMOET

A.4.1. INTERVIEW LEO OORSCHOT – TECHNICAL ADVISOR ENGINEERING MAMMOET – 03/09/2024

Rens: ... Dus dat was een limitatie van wat ik wilde doen. En dan kijken van 'oke, maar wat zijn de capaciteiten van die kraan dan? Hoe hoog kan die, en welk gewicht kan die dan tillen?' En daaruit bleek eigenlijk dat, met jib, kan dat ding (hydraulische telescoop kraan) 90 à 100 meter en dan zou die net nog een MLS mast bovenop ergens kunnen zetten.
Leo: Ja een mastdeel weegt 24 ton ofzo?
Rens: Ja bijna 40.
Leo: Oh 40 ton al ondertussen. En die kan die dan nog tillen op 100 meter hoogte.
Rens: Maar dan die gantry beam, die lukt al niet.
Leo: Die grote balk niet meer, die is 100 ton.
Rens: Precies, daar werd de hoogte al gelimiteerd tot een meter of 30.
Leo: Ja, maar een telescoop kraan is heel veel goedkoper, omdat die goedkoper opbouwt, hij kan zelf rijden, je hebt niet een hele ploeg nodig.
Rens: En je kunt hem binnen een dag al gemobiliseerd hebben.
Leo: Ja, in een dag. Je kunt hem 's ochtends bestellen en dan komt die.
Rens: Ja, dus toen ging ik kijken. Want je ontkomt er natuurlijk niet aan om op je building site een hydraulische kraan te hebben, die heb je eigenlijk altijd wel. En anders is het niet een heel groot probleem. Dus dan gingen we kijken van 'oke, maar wat zijn dan allemaal mogelijkheden om tot grote hoogte te komen, met alleen maar het gebruiken van een hydraulische kraan?' Dus kijken naar een klimframe van onderen,

zoals in de FOCUS, die alles naar boven duwt. Of een klimframe op de bovenkant, zoals een torenkraan, die dan zichzelf naar boven bouwt. Of, ik had nog wat creatieve ideeën. Als je die torens ziet als scharnierend opgelegd op die gantry balk. En die gantry balk maak je wat langer. En je gaat aan de ene kant trekken, dan trek je de andere kant omhoog, zet je er een mastdeel onder, andere kant trekken, zet je er een mastdeel onder.
Leo: Die is creatief.
Rens: Die is heel creatief, het is ook een uitdaging. Qua krachten zou het werken, maar je hebt gigantisch veel funderingskrachten nodig en weet ik veel wat. En natuurlijk trekkrachten. Of dat je zoals een papiertje. Dan kun je zo duwen *beeldt paper push concept uit*, dan gaat de boven kant omhoog.
Leo: Ja ja, dat is in spreidstand. Die zijn weleens bedacht, gantries in spreidstand. Daar heb je veel ruimte voor nodig.
Rens: Ja daar heb je heel veel ruimte voor nodig. Bij mijn ouders had ik wat lego blokjes gepakt en wat scharniertjes. En dan had ik het zo gebouwd.
Leo: Ja en je steekt dus ook voorbij waar je aan het werk bent. Want vaak heb je maar een weggetje hier naartoe. Dus dat wiebelen met die masten omhoogtillen, dat is wel een hele creatieve, die heb ik nog nooit gezien. Dat opduwen wel. Maar dat met die masten, dan moet je een puntstuk hebben en dan steekt die eroverheen. En dan kun je hem als hefboom gebruiken. En dan kun je je mast monteren. En dan zwaait die hem zo naar binnen. Die is creatief.
Rens: Die had ik ook nagebouwd in lego.
Leo: Maar, je hebt elke keer een scharnierpunt nodig.
Rens: Ja dat, en je hebt twee funderingpunten nodig.
Leo: Ja je moet ook breder met je fundering, wat je in de grond stopt dat beton, dat moet ruim buiten je masten uitsteken want anders kun je niet aan twee kanten trekken.
Rens: Nee. En dit punt dat verandert steeds. Want je trekt hem op en dan wordt dit stuk door Pythagoras korter. En dan moet je een nieuwe er weer onder zetten, want dan gaat je constructie aan de wandel.
Leo: Ja, ja dat is allemaal ingewikkeld.
Rens: En je krijgt ook hele grote krachten in je mast.
Leo: Ja dat ligt aan je arm. Als je de arm nog wat langer maakt, dan kan het wel beter. Je kunt tillen. Kijk hij kan extreem lang zijn, het trek punt hoeft niet het til punt te zijn. Dus je kunt dichter bij de kilo's pakken en wat verder weg het trekpunt maken. Maar goed dat is eh... Wat op raffinaderijen is, als je een fabriek hebt en pijpenracks en een grote kolom en een huisje, hier staat een huisje zo. En een dakje. Dan staan hier de volgende twee. En dan staat hier weer wat. En dan moet deze vervangen worden. En hier loopt vaak de navelstreng van de raffinaderij, dat is vaak een pijprek waar aan alle kanten dingen vastzitten. Ja, je hebt over het algemeen alleen maar dit plekje. Of de straat loopt zo. En dan kun je net hier dat nieuwe ding neerleggen. Je mag er niet vanuit gaan dat je vrije ruimte hebt aan alle kanten. Sterker nog, een goed ontwerp moet aan drie kanten begrensd zijn en je moet vanaf een kant kunnen werken. Dan ben je breed inzetbaar, dan kun je altijd uit de voeten. Het komt weleens voor dat wij natuurlijk een greenfield raffinaderij bouwen, een gloednieuwe. Ja dan is de hele planning wel duidelijk, dan werk je van de ene hoek naar de andere en dan heb je veel meer vrijheid. Maar zodra je in bestaande raffinaderijen komt of andere industrie, dan zijn er al dingen. Het wordt allemaal drukker en groter en voller. En dan moet je er eigenlijk vanuit gaan dat je maar van een kant aan kan voeren.
Rens: Oh, dat is wel een goede inderdaad. Dat is ook wel een mooie.
Leo: Dus gewoon van een kant, dan moet die erin en eruit. En ja dus dat is een hele belangrijke.
Rens: En een andere manier natuurlijk, waar we ook even naar hebben gekeken, maar dat is dat je... maar dat is bijvoorbeeld een helikopter, wat zou dat nog kunnen doen. Gewoon wat out of the box, wat andere out of the box ideeën. Allemaal opgeschreven en dan op basis van allemaal eisen die Mammoet stelt of die voortkomen uit. Want in heb die Jubail gantry als referentie gebruikt. Dan ideeën afgeschoten al gelijk. Uiteindelijk komt er dan een morphological chart. Oh ja, want je bent dan op grote hoogte, maar omdat je

die kraan niet kunt gebruiken om die gantry beam uit de weg te halen, kun je niet op dezelfde manier naar beneden. Want dat vat dat staat in de weg.
Leo: Ja dat klopt, als je klaar bent en je hebt een vat geïnstalleerd dan zit die balk precies boven het vat.
Rens: Dus dat is probleem nummer twee; hoe gaat die bovenkant, hoe gaat dat aan de kant?
Leo: Oh, maar met sleetjes kun je een boel oplossen.
Rens: Precies. Dus toen heb ik alle bewegingsvrijheden genomen, beweging in deze richting, beweging in die richting. Maar ook rotaties, deze rotatie of deze rotatie. En dan ook weer alles, weet je wel, want je zou ook de gantry beam kunnen splitsen. Dat is dan een slecht idee, maar het zou kunnen. Dus dan kun je uit elkaar bewegen of je kunt deze twee, omdat het twee balken zijn, om dan wat meer evenwicht te bewaken.
Leo: En dan allebei voor en achter laten zakken.
Rens: Ja, en dan heb je een stabiele toren.
Leo: Dus dan heb je een torentje en dat is dan natuurlijk een grote toren, want die is 8 meter. Die toren is 8 meter breed. En dan zou die twee balken hebben en dan zou je inderdaad met een simpel dingetje zou je ze zo naar buiten kunnen hengelen. Dan kun je ze zo oppakken en dan kun je ze zo scharnieren, ja dat zou kunnen.
Rens: En waarschijnlijk, daar zet ik nu mijn geld op, want ik heb een, hoe heet dat, een multi criteria analyse gedaan. En daaruit is gekomen dat, de beste ideeën zijn eigenlijk om dan wel een klimframe op de bodem, zoals de FOCUS kraan, en dat je dan de hele gantry gaat sleeën. Dus de hele gantry.
Leo: Ja dus die gaat in z’n geheel met van alles en alles gaat die. Dus je fundering die stretch je een beetje naar voren. En dan slee je na gebruik je complete gantry de vrije ruimte in en dan naar beneden klimmen.
Rens: Ja, dus dat. Of dat je het klimframe nog steeds aan de onderkant en dat je dan dit gaat doen *beeld splitsen op de top uit*. Dus dat je het aan de zijkant beweegt. Of een klimframe op de bovenkant, zoals bij torenkranen, en dat je het uit elkaar beweegt. Dat zijn de drie opties die uit de multi criteria analyse als beste naar voren kwamen. En ik ga mijn geld inzetten op het idee dat het klimframe aan de onderkant zit en ditte, dat dus die gantry beams uit elkaar worden geskid.
Leo: Maar dan komt er eentje aan de verkeerde kant terecht.
Rens: Ja, dat wel.
Leo: Maar goed, op grondniveau kun je dat wel uit elkaar halen waarschijnlijk, maar dat is een dingetje. Want je benut een kant die je eigenlijk niet wilt benutten. Er zijn situaties dat het niet kan.
Rens: Ja komt dat zo nauw? Want in principe als dit net naast die, dat is eigenlijk al te veel?
Leo: Nou ja, zo’n balk is breder dan een meter, we nemen aan dat die container breed is, 2.5 meter. Dit ding staat natuurlijk op pootjes dus een klein beetje uitgebouwd. Dus een halve meter, plus twee en een halve meter, dan kom je drie meter naar buiten als je er strak langs naar beneden zou gaan. Dat is eh, ja dat is krap. Er zullen plekken zijn waarbij er geen drie meter ruimte is. Dat dat gewoon tegen andere dingen aan staat.
Rens: oke, dat is wel een goede inderdaad.
Leo: Kan ik me nog wel ergens bij voorstellen dat je die balk laat zakken en dat je hier een steuntje maakt en dat je die balk bijvoorbeeld op 30 meter hoogte legt. Die pijpenrekken, je zou eens wat foto’s van raffinaderijen moeten bekijken. Die dingen zijn niet allemaal 30 meter hoog. Dus huisjes en pijpenrekken en heel veel dingen die stoppen op een meter of 10. Dus daarboven is wel ruimte. Dus kan nog altijd zeggen, nou ik heb dat concept en als ik hier klemloop omdat hier een brug, een leidingenbrug loopt. Dan parkeer ik dat grote ding op dit mastdeel. Dan breek ik al het andere af, als dat kan. Als je van bovenaf klimt dan kom je naar beneden geklommen. Als je aan de onderkant aan het klimmen bent, dan kan het niet, dan zit die eraan vast. Maar als je van boven klimt en op een gegeven moment dan zou je daar toch overheen moeten om dat mastdeel, maar dan moet je maar 40 meter hoog ofzo. Maar dat is een speciaaltje, daar hoef je niet op te ontwerpen. We hebben nu een raffinaderij, Suncore in Canada, daar, even kijken hoor, moet ik het even schetsen. Er is vrije ruimte, zo. Het is een binnenpleintje en het is een vierkante. Hier is een soort poortje, daar kun je gewoon naar binnen. Ja die poort is echt 10 bij 10, dat is echt een royale poort. Maar in ieder geval, hier lopen leidingen overheen. Hier zitten allemaal constructies, hier zit een constructie hier is een

constructie. En hier moeten we een heel groot vat uitwisselen. Hier past onze PTC, past erin, het is een plein van 80 bij 80, het is best een grote ruimte. Dus hier kunnen we opbouwen. Behalve dat hier lopen allemaal pijpenbruggen. We hebben ervoor gekozen om op hoogte te gaan opbouwen.
Rens: En hoe ga je dat met je mast doen dan?
Leo: Ja, ze bouwen dus hier een tafel en hier bouwen ze een tafel en de mast gaat hierzo liggen. En we gaan de kraan ook wat hoger leggen. Er komt een zandbed van 2 meter ofzo, dat hogen ze op om. Want de kraan is zelf al 7 meter he. De mast pivot, het draaipunt van de mast, die zit op plus 7 meter. En ik dacht dat ze de kraan op een verhoogd zandbed gaan zetten, zodat we boven die pijprekken uitkomen. En dan gaan ze hier met containerstapels en balken en loadspreaders maken ze een werkveld. En dan gaan we hoog opbouwen. Liever niet, maar.
Rens: Jeetje, wat een onderneming.
Leo: Ja omdat die raffinaderij daar is alles ingebouwd en dat grote vat dat weg moet, de kraan kan het doen, de kraan kan er staan, maar hij kan z'n mast niet opbouwen of afbreken. Dat moet over pijprekken heen, nou ja oke.
Rens: Holy shit, haha.
Leo: Maar goed, dat is dus een uitzondering, want meestal kan het wel. Dus als jij stelt van ik wil aan twee kanten die gantry balk laten zakken, dan zal dat heel vaak wel kunnen. En een enkele keer dat het niet kan, daar zou je een plan B voor moeten hebben. Maar dat plan B dat is niet het standaard plan. Wat we bij Mammoet doen is dat we ontkennen de problemen niet die er zijn, maar we gaan ook niet continu voorzieningen en oplossingen meenemen voor iets dat voorlopig nog niet voorkomt. Zolang je nog niet de opdracht hebt op een locatie dat het niet kan. Moet je niet al te druk maken. Je moet een plan B hebben, voor als die klant komt met dat probleem. Maar dat wilt niet zeggen dat dat plan B helemaal uitgeengineerd en gekocht en betaald moet zijn. In feite moet die klant dat betalen.
Rens: En zoiets als dit is wel redelijk, dat kun je wel verwachten dat dat een keer gaat gebeuren.
Leo: Ja, dat komt veel voor, aan drie kanten ingebouwd. Aan de andere kant, we hebben ook plekken waar het allemaal wel kan. Zolang jouw eerste opdracht niet een site is waar jouw oplossing niet past. Je moet ergens een referentie hebben. Dus je kunt die Jubail gantry pakken, dan kun je z'n plotplan kijken. Hoe zag het er uit die omgeving. En als het daar al niet kan, dan kun je die andere pakken. Want er zijn er twee geweest al in Saudi-Arabië. Twee MLS gantries.
Rens: Oh oke, en die andere dan?
Leo: Wouter Gerritsen die heeft er een gedaan en Bert van der Meide heeft er een gedaan.
Rens: Oh, was dat niet dezelfde?
Leo: Oh heeft Wouter Bert afgelost? Eh nee, we hebben er twee. We hebben er al twee gedaan.
Rens: En je hebt er ook een in Antwerpen gedaan weet ik.
Leo: Ja, Antwerpen ook gedaan. Dan hebben we er al drie gedaan. We hebben er zeker weten twee in Saudi gedaan en eentje in Antwerpen. Dus we hebben drie MLSen gedaan. Dus als jij drie keer MLS plotplan opzoekt. En als het bij alle drie ruimte was om aan de achterkant een balk te laten zakken. Dan kun je ermee verder. Dan geldt het alleen als een plan B als Wouter van den Bos zegt 'Maar als daar nou wat staat?' 'Ja maar daar heb ik aan gedacht.' Haha. Het is op te lossen. Het kost wat meer moeite, dan kun je zeggen, een klant die zo'n lastige raffinaderij heeft die betaalt daar maar voor. Maar als bij twee van de drie het plan al niet werkt, dan is het geen goed plan.
Rens: Nee, dan is het geen goed plan. Dat is wel een goeie. Want precies dit soort inzichten daarvoor wilde ik eigenlijk dit gesprek ook met je hebben.
Leo: Twee keer Saudi en een keer Antwerpen en dat was Borealis ja.
Rens: Ja Borealis ja dat klopt ja.
Leo: Dat was de eerste want die hebben we toen in aanbieding gedaan omdat we geen werk hadden. Dus we hadden New York Wheel was verloren, die was gestopte en alles en ellende. Toen hadden we al die masten. En je hebt altijd een first user nodig, een project nodig. Om een systeem in de markt te kunnen zetten.

Niemand wil de eerste zijn. Dus bij Borealis zullen wij waarschijnlijk heel scherp aangeboden hebben. Misschien wel tegen kostprijs, zo van 'joh we hebben die gantry'. Die klant denkt ook 'Ik ben dan wel de eerste, maar dit is een koopje.' Dat is altijd, met een nieuw product, de eerste gebruiker die wil dat nooit. Tenzij die een enorme korting krijgt. Want er zitten altijd foutjes in of vertraging met opbouwen omdat de manuals nog niet klaar zijn of omdat de mensen nog niet weten hoe ze met de systemen moeten werken. Dus de first user is altijd, die weet dat ook, dus die gebruikt dat ook weer in de onderhanding. Dat is het commerciële spel.

Rens: En zeg maar, als je dan dat klimframe op de, maakt niet uit waar dat ook zit. Je zou dat op verschillende manieren kunnen doen, je kunt jacken, je kunt lieren gebruiken, zoals bij de FOCUS, die is met lieren dacht ik. Maar je zou ook met tandwielen kunnen werken, maar bij Mammoet gebruiken jullie eigenlijk bijna geen tandwielen, waarom is dat?

Leo: Eh haha oke. Die Gusto boten die hier steeds liggen, die hebben een hele lange poot met een tandradbaan erop. Die hoeft nooit uit elkaar die boot. Dus die tandbaan die wordt gemaakt in stukken, die wordt daar helemaal gesteld, afgelast, die zit voor de eeuwigheid vast. Als ik een gantry maak in stukken van 11 meter, dan krijg ik elke 11 meter een deling. Waar die tandradbaan begint en eindigt. Nou die is verschrikkelijk die deling. Die moet je super strak uitlijnen en daar zit altijd speling op, het past nooit precies. Het is echt heel vervelend. En hij is loodzwaar die tandbaan.

Rens: Ja?

Leo: Ja ja ja, want als jij die tandjes ziet. Als jij de Gusto schepen gaat, dat kan dus bij Gusto daar hebben we tegenwoordig contacten, Gusto MSC. Want die hebben natuurlijk vorig jaar hier gestaan. Dat is meestal een klont staal van 100 breed en 200 dik ofzo. En dar gaan die tanden uit. Dus die tanden die.

Rens: Want, 200 breed?!

Leo: 200 diep. Zo, dus die tanden die worden er helemaal uit gemachined. Hij is 100 mm breed, hij is 200 diep. Dus die tanden die zijn 80 naar voren en hier 100. Dus dat hele tanden profiel is een enorm gesmeed blok staal. Dus dat weegt, dat zijn serieuze kilo's. Dat is echt een heel zwaar onderdeel. En dan moet je daar dus ook nog een deling in gaan maken dus dat stopt gewoon en dan moet z'n broertje verder. Dus je moet gaan kijken, ik ga stoppen waarschijnlijk in zo'n dalletje. En daar komt dan een spleetje, dat moet strak ten opzichte van elkaar getolereerd worden. Dit tandsysteem, ga je dat vastmaken aan je masten? Zit dat er dan altijd aan vast of komt die los ernaast? We hebben weleens wat geschetst. Dit is je MLS mast, dit is je chord. Hier zitten die koppelingen waar die pennen doorheen kunnen. Als je dat tandsysteem er hier tegenaan wilt zetten. Dan moet je ze eigenlijk op elkaar gaan stapelen. Maar dan is die spleet weg, dan ben je gewoon aan het stapelen, he contact druk. Of je moet hem bevestigen aan je chords. Hoe dan? Want dat ding is af, die chords zijn af. Dus dat is niet fijn. Dus dat is best een puzzel. Het liefst zou je dat tandsysteem integreren in je masten. Maar die masten die hebben we al gebouwd. Dus dat is al een uitdaging.

Rens: En die zijn zo zwaar uitgevoerd omdat er natuurlijk gigantisch veel krachten op uitgevoerd worden.

Leo: Ja, want uiteindelijk klim je omhoog met twee of drie of vier tandwielen. He, die dingen, sprockets heten ze geloof ik. Dus je klimt langs die baan omhoog met maar een paar van die ronsels. Het is allemaal werktuigbouw dit. In beweegbare bruggen heb je dit ook. In beweegbare bruggen heb je ook van die tandbanen om basculebruggen te openen en te sluiten. Dus je hebt een paar van die ronsels, tandwielen, die lopen, die klimmen omhoog langs die tandbaan. En die kilo's die uit jouw hele systeem naar beneden komen, die moeten dus door drie tandjes op die baan en dat gaat dan allemaal naar beneden en dat zit dan ook nog vast aan een vakwerk zo. Ja ergens, zwaartekracht werkt altijd, maar dat moet dus lokaal door die tandjes. Het is serieus denk ik t10 centimeter breed en een bepaald oplegvlak. Tamelijk duur en heel erg sterk. Je kunt eens bij Gusto checken hoe die boten van hun eruit zien en als je er iets meer van wilt weten dan kunnen we wel eens dat Chinese meisje van Gusto benaderen. Want ik heb een rondleiding gedaan voor Gusto bij die grote kraan toen voor hun project. En toen zei ze 'Misschien komen we elkaar wel weer tegen'. Als ik nu zeg ik heb een student dan geef ik haar je adres en dan mail je gewoon. 'Ik heb jouw naam van Leo gehad, ik had eigenlijk een vraag over die tandsystemen. Heb je een voorbeeld van een typical systeem?' Want zo'n boot

van Gusto, zo'n potenboot, ze hebben ze in vier en zes, die systemen. Als ik er een met zes heb en die boot die weegt 50,000 ton. Dan kunnen die zes poten die boot omhoogklimmen. Dan weet je dus dat een zo'n systeem, dat is dan al gouw acht, negenduizend ton sterk. En dan kun je aan de detaillering en de onderdelen wel zien. En dan denk je 'Nou, dat is best een machine.' En dat is echt een maatje duurder en groter en zwaarder dan dat wij gewend zijn. En wat wij gaan klimmen. Dat is 200 ton. Dus ik kan dat systeem misschien enorm naar beneden schalen, dat zal wel lukken. Dus het kan allemaal veel kleiner, maar wat er bestaat, dat zijn dus die klimboten. En die zijn allemaal groot en looiig. En dan moet je het zelf gaan verzinnen weer.
Rens: Ja want ik zat te kijken inderdaad van 'oke, stel je hebt zo'n klimframe op de onderkant en je wilt alles naar boven duwen. Dan op het einde, als je ook een beetje toekomst bestendig wilt zijn enzo, dan moet je eigenlijk wel per mast, per toren sectie, moet je iets van 800 ton al het liefst wel kunnen willen.
Leo: Dat is helemaal niks.
Rens: Nee, maar dat is in principe een zo'n systeem dan toch? **Een tienfout in de war op dit moment**
Leo: Nou veel minder nog. Onze complete gantry weegt veel minder dan een poot van Gusto kan leveren. Dus het is een overkill. Die Gusto systemen zijn overkill maar volgens mij bestaan ze niet in hele kleine versies. Maar dat is natuurlijk, dat kun je uitzoeken.
Rens: Ja. En qua onderhoud, is dat nog een groot nadeel voor/ van tandwielen? Want het is lastig inderdaad wat je zegt met het hergebruiken.
Leo: Nee tandwielen hebben de grote plus dat ze mechanisch zijn. Wij hebben een sterke voorkeur voor mechanica, mechanische systemen boven elektronica. En alles wordt steeds elektronischer. Maar het liefst werken wij met hydrauliek, daar zit heel veel kracht in. Kijk, pneumatiek, lucht, dat is helemaal niet sterk. Hydrauliek is heel sterk. Hydrauliek en het alternatief is elektromotoren. Elektromotoren zijn relatief zwaar en daar kun je heel veel mee, maar dat zit natuurlijk gekoppeld met elektronica. En die elektromotoren zijn wel robuust, maar die elektronica is altijd link. Als de bliksem inslaat of als er allemaal water bijkomt. Terwijl tandwielen, die kun je in weer en wind gebruiken. Hydrauliek, met die slangen, daar kunnen ze buiten goed mee overweg. Daar kan veel kracht doorheen, dus met barren enzo en grote diameters. Dus het is zondermeer robuust. Want het zit ook op die boten dus dat is een plus. Alleen de systemen die Gusto heeft op die potenboten die zijn echt een maatje te groot voor wat wij nodig hebben. En ik vraag me af of ze nog kleiner te krijgen zijn.
Rens: Ja want dan zou je het helemaal zelf moeten ontwerpen bedoel je.
Leo: Ja ik kan me eigenlijk niet voorstellen dat. Kijk de kleine systemen, die zijn er denk ik niet, omdat die allemaal worden weggeconcurreerd door elektronica, elektromotoren en al het kleine spul. Want het is natuurlijk duur, tandwielen smeden, tandbanen maken. Het voordeel ervan, de robuustheid, betaalt zich uit bij zo'n grote boot op zee. Maar in het kleine werkveld zijn ze denk ik al weggeconcurreerd door de elektromotoren, de elektronica, het veel fijnere spul wat massaal en veel goedkoper geproduceerd kan worden. Ik heb niet gelijk een idee van wat kleine systeempjes. Maar die kunnen we aan Gusto vragen 'Wat is het kleinste systeem dat jullie hebben?' Wat je ook hebt is een systeem van cilinders, hydraulische cilinders, lange slag cilinders. Als je een lange slag hebt, met een zuiger en een stang. Die hebben als nadeel de afdichtingen. Daar gaat altijd, die stang die schuift in en uit. Je pompt er olie in aan de ene kant, nou dan schuift die uit. Je pompt aan de andere kant, nou dan schuift die weer in. Maar hij schuift hier door een openingetje met een afdichting. Het veroudert die afdichting, het moet namelijk iets zijn dat klemt dus rubber, pakking dingen, o dingen van alles en nog wat. Maar het mag niet 100 procent klemmen, het moet afdichten, het moet klemmen, maar als er zand en vuil in gaat zitten dan gaat het toch verweren. En door het zonlicht en ozon verweert het. Of door water, roestwater dat erlangs loopt. Er is altijd wat met die dingen.
Rens: Oke, dus die hydraulische systemen zijn ook niet perfect?
Leo: Nee, maar we gebruiken het heel veel. Telescoopkranen, een telescoopkraan die jij zo graag wilt gebruiken om op te bouwen. Die heeft natuurlijk een mast, die mast die schuift uit, dat is allemaal hydrauliek dat daarin zit. Maar die zijn nog beschermd. Maar die hele grote cilinder aan de onderkant, dat ding zit gewoon buiten in weer en wind. Die schuift gewoon uit, die moet gewoon onderhouden worden. Er zijn

gewoon hele grote, sterke cilinders te krijgen. Maar dat blijft, het moet eigenlijk veel werken, want dat is beter. Als het weinig werkt dan gaat het alleen maar roesten en verouderen en dat is eigenlijk helemaal niet goed. Hydrauliek werkt het beste als het veel gebruikt wordt. Dan blijft het in ieder geval elastisch en dan kun je het een beetje smeren en doen. Maar goed, dan zit je al een stap verder. Want de keuze van je systeem. Dit is een detail uitwerking. Je moet eerst een concept hebben. Maar het is wel iets om mee te nemen. Maar hydrauliek is een heel erg breed, daar kan heel veel mee en een gewoon echte mechanische oplossing is een hele fijne, want die werkt altijd.
Rens: Maar je hebt met dit natuurlijk wel dat, stel je moet een grote slag maken, dan moet de initiële lengte van dat ding ook groot zijn. Want wat is ongeveer de ordegrootte, want ik had al wat in die, wat is het, volgens mij die special devices handboek had ik eventjes gelezen. En daar was de grootste slag die jullie met een cilindersysteem konden iets van ordegrootte drie meter ofzo.
Leo: Ja dat is heel veel al hoor. Dat is heel veel. Bij de Oosterschelde kering hebben ze cilinders met een slag van tien meter.
Rens: Zo, maar dat zijn echt joekels van.
Leo: Ja dat zijn enorme cilinders, die hebben ook een vermogen gekost. Ja want de allergrootste schuiven die moeten tien meter naar beneden. Dus de Oosterschele heeft hele grote cilinders. En de Pieterschelde die heeft kantelbalken. Dat is dat schip dat, de pioneering spirit van Allseas, die hadden een systeem met telescoop cilinders van dertig meter ofzo, veertig meter. Maar daar zijn ze nu vanaf gestapt, die hebben ook een klimsysteem eronder. Een soort van, ik weet niet eens, dan zouden we foto's moeten bekijken. Mijn vrouw er gewerkt, toen waren ze allemaal met telescopen bezig met gigantische afmetingen maar dat viel niet te bouwen. Dus die zijn ook afgestapt van enorme telescoop cilinders. Er zijn grenzen aan wat je met een cilinder kan.
Rens: Ja want bijvoorbeeld, je kan niet de slag van een. Oh, een vraag eigenlijk: Zou je de slag van een MLS kunnen maken met een cilinder?
Leo: Dat is 11 meter, nee.
Rens: Dat is wel heel erg enthousiast.
Leo: Dus dat zou je in stappen moeten doen. En wat wel kan natuurlijk. Die MLS mast, als dit 11 meter is, dan kun je natuurlijk zeggen 'Nou met die oogies, ik wil een slag maken die halverwege stopt.' Want zo hebben wij een vijzelsysteem gehad. Dan doe je een, moet ik het goed zeggen. Je duwt hem een halve slag omhoog. Dan is er ruimte voor een halfje, dan gaat dat halfje erin. Daar zet je hem op af. Dan klimt de cilinder. Dan doe je weer een halve slag, dan gaat het halfje eruit en dan gaat de hele erin. Dat is heel omslachtig.
Rens: Ja. Maar dan zou je liever iets hebben dat die ene halve slag maakt dat die hem hier dan vastpakt zo. En dat dan je cilinder klimt en dat je dan nog een halve slag maakt. Ik weet niet hoe je dan.
Leo: Maar goed, als je 11 meter wilt klimmen moet je ook 11 meter overbruggen. Je hebt dan gewoon een opening, de krachten moeten 11 meter oversteken. Dus dat is best veel. 11 meter is veel. En die MLS masten hebben als het goed is nog opgelaste nokjes.
Rens: Ja dat klopt, die zitten in het midden inderdaad.
Leo: Ja daar kun je langs klimmen. Het idee was dat je op dat nokje zou kunnen klimmen met iets.
Rens: Maar hij heeft er maar een per chord.
Leo: Nee, drie toch?
Rens: Wat ik op die fabricatie tekeningen, wacht die heb ik hierzo.
Leo: Ik dacht dat ze er drie hadden want een heb je niks aan.
Rens: Kijk je hebt er hier in ieder geval een. Hier dat is deze.
Leo: Ja je kunt het niet zien hier. Dan moeten we even kijken. Want volgens mij hadden ze een. Want op een kan je niet klimmen. Ik dacht dat we boven, midden en onder hadden.
Rens: Dat zou wel heel erg, dat zou wel top zijn. Misschien heb ik die tekening wel niet goed bekeken.
Leo: Maar, daar kan altijd bij gelast worden. Dat is een modificatie, die is niet leuk, maar wel mogelijk.
Rens: En waarom is dat niet leuk?

Leo: Geld. Je masten zijn af en dan ga je ze modificeren. Maar dat kan natuurlijk, je kan als er een nok op zit, dan kunnen er ook twee op, er kunnen er ook drie op. Dit is het onderzoek, dit zijn de parts. Die moeten we niet hebben, we moeten een assembly hebben.
Rens: Nee ik heb alleen maar de parts volgens mij. Ik weet alleen niet welke welke is. Ik had er eentje van de.
Leo: Dit zijn de pennen. Nee je hebt toch ook hogere tekeningen, 10-05, 10-02 en 02 is een assembly of een welding machine. Even kijken.
Rens: Dit ziet eruit als een chord.
Leo: Ja even kijken. Hier zie ik een nok zitten. Detail 1. Ja het lijkt erop dat we maar een klimnok hebben.
Rens: Ja want hier is die gewoon plat.
Leo: Das nou jammer zeg.
Rens: Ja het verbaasde mij ook wel een beetje.
Leo: Oh, ik dacht dat we er drie, want een daar heb je niks aan. Een is geen dat is jammer. Dat is jammer en open eens tekening 01? Die heb je denk ik ook.
Rens: Ja ik heb deze van Walter gekregen is dat deze dan?
Leo: Ja, dat is 10-01. Ja er staat er echt maar een op. Kijk hier zit niks, hier zit wel wat, hier zit niks. Dat is nou jammer. Maar het kost niet de wereld he? Je hebt een mast deel van 40 ton die is helemaal af. Dingen bij lassen om het werkend te krijgen, dat is te doen. Dat is een kleine aanpassing. Maar ga je dan klimmen langs die masten of je gaat het systeem op die nokjes maken? Je gaat het systeem op die nokjes maken. Dan zit je nog steeds. Ik kan hier bijvoorbeeld een, twee. Ik natuurlijk zeggen, ik heb hier die middelste, dan moet je al een stramien hebben dat je over die koppeling heen komt. Dus dit stramien moet matchen, dit is 11.4 of 11.0 wat is ook alweer de systeemlengte van die dingen?
Rens: Ja volgens mij 11.4 inderdaad.
Leo: Je hebt 5200 zie je. En 600. Twee keer, dus nee dat is anders. 600 en 52 staat er toch? Dat is 10.4, dat is 11. Hart is 11. Dus je hebt ergens hier het midden, dan moet je elf meter. Dat moet je het in logische stukken opdelen. Zodanig dat er niks hier zit. Dan moet je het in vieren of in vijven delen. Maar dan moet je precies een brugje hebben over dit. Maar goed dat is gewoon een puzzeltje, dat is wel op te lossen. Want je hoeft natuurlijk iet in meters te klimmen. Je hoeft niet 11 meter in een keer te klimmen. Je kunt ook in stapjes van 2.75 ofzo. Zo mag je ook klimmen. 2.75 zal wel, de helft van 11 is 5 en een half, de helft van vijf en een half is 2.75. Maar dan kom je precies daaruit, dat is niet handig. Dus het zal iets moeten zijn, 11 gedeeld door vijf is 2.2. Dus dan zou je zeggen 2.2, 2.2 zit er net voor, net erover, 2.2. Zoiets zal het dan zijn. Dan zou je in slag van 2.2, dat is met een cilinder weer wel goed te doen. Twee meter is een, daar zijn wel cilinders voor te krijgen.
Rens: Ja, dan moet je alleen dus een manier verzinnen waardoor die cilinders dan wel, of die cilinders kunnen op hun plek kunnen staan. Maar dan moet je dat mast deel even vast kunnen houden.
Leo: Je moet hierop precies dat kleine richeltje kunnen klimmen, dat is best een dingetje. Dat is nog niet ideaal.
Rens: Ik had even de capaciteit. Die dingen hebben best een grote capaciteit. Die zijn echt iets, wat was het nou, 680 ton ofzo. Dacht ik. Wat, want jullie hebben natuurlijk alles dat multi purpose. Alles wordt hergebruikt, in andere configuraties gebruikt, in andere constructies gebruikt. Dan lijkt me dat plastische vervorming...
Leo: Nee wij rekenen elastisch. Uitgangspunt is elastisch rekenen.
Rens: Voor alles?
Leo: Ja. Elastisch rekenen want je wilt je materiaal hergebruiken en. De reden daarvoor is een hergebruik. En twee is, onze ULS combinaties treden op en zijn niet uitzonderlijk. Als je een gebouw hebt of een brug of een toren constructie, die worden uitgerekend op een honderd jaar storm, een aardbeving of volle belasting met zettingen en wind. Combinaties waar alles inzit. Die zijn dan de maatgevende combinaties, daar wordt het op ontworpen. Als dat echt optreedt dan mag falen niet optreden maar plastisch vervormen wel, zolang het maar blijft staan en niet instort. Dat geldt voor gebouwen en constructies in de Eurocodes, instorten mag niet, plastisch mag wel. Maar bij ons, als wij 2000 ton gaan hijsen op Unity Check 1. Die 2000 ton is echt 2000

ton. Die gantry die we uitrekenen op storm, ja die storm komt meestal niet. Die aardbeving die is er meestal niet. Maar die hijslast is er. Dus voor ons is een ULS combinatie hijsen, die is er ook echt. Dus als jij plastisch gaat rekenen met je maatgevende hijslast, dan ga je ook plastisch werken.
Rens: Dus dan weet je dat je vervormingen krijgt.
Leo: Je krijgt hem gewoon. Dus voor ons is een incidentele ULS combinatie die eens in de honderd jaar optreedt, dat is helemaal niet zo, bij ons treedt die gewoon op. Dus dat is het grote manco dat wij niet plastisch kunnen rekenen. In principe ontwerpen we er niet mee en we rekenen er niet mee, en als het echt niet anders kan. Want ook in onze ULS combinaties zit natuurlijk een stuk invloed wind, zit een stuk sideload in. We nemen vaak nog misalignment en excentriciteiten mee die er niet zijn. Maar 90 procent van je belasting geval is vaak gewoon gewicht. Daar kun je gewoon niet onderuit. Dus de manier waarop wij onze constructies gebruiken, heb je dus gewoon. Je hebt gewoon de maximale belasting. Daarvoor worden we ook ingehuurd, om iets te doen wat helemaal op de grens zit. Daar hebben we het equipment voor, maar dat kun je niet plastisch gaan inzetten. Dat kun je maar een keer doen.
Rens: Ja precies, want dan pas het niet meer.
Leo: Nee ja nee dan ga je het beschadigen, de lassen gaan kapot en nee dat doe je gewoon niet. Dus elastisch een, voor het hergebruik en twee omdat de maximale combinatie komt gewoon voor. Dat is gewoon heel realistisch.
Rens: En daaropvolgend. Lassen of, want de onderdelen die niet containerized hoeven zijn, dat is meestal gelast en de andere zijn meestal gebout of gepind.
Leo: Nou bouten niet he. Bouten daar hebben ze een hekel aan. Bouten dat mag je doen als dingen langer dan een jaar in elkaar gaan zitten. Want bouten is veel werk, want eigenlijk wil je bouten voorspannen, want dan komen ze niet los. Maar voorspannen daar heb je weer gekalibreerde tools voor nodig en als je voorgespannen hebt. Boven de 70 procent, moet je eigenlijk alles weggooien daarna. Want je bent plastisch aan het vormen. Een voorgespannen bout, die rekt, die vervormd plastisch, heb jij gehad met een vak, staalconstructies drie ofzo. Dus als wij bouten toepassen, dan doen we dat eigenlijk alleen voor systemen die echt langere tijd staan. Want je moet die bouten gewoon afschrijven en weggooien. Dus een, het is vervelend, voorgespannen bouten dan moet je je gereedschap kalibreren, je vlakken moeten officieel allemaal goed zijn. Die moet je voorspannen, haal je dat? Dus dat doe je alleen maar als je het langere tijd kan staan en als je na afloop de bouten weggooit. Als je korter staat, de meeste projecten van een gantry is een paar weken opbouwen, klus een of twee vaten zetten en weer afbreken. Als jij dan allemaal bouten elke keer moet een hergebruiken, mag het dus alleen bij hele lage krachten. Dus als jij hem dan handvast voorspant, dat is dus eigenlijk niet voorspannen, dan mag je de bout wel hergebruiken. Maar dan is het gewoon veel werk. En een pen is veel minder werk.
Rens: Oh dus eigenlijk is het bij jullie of lassen of pennen?
Leo: Ja. Ja, dat zie, dat hele ding is gepend, daar zit geen bout in. En bouten hebben ook het nadeel. Een bout heeft twee ringen nodig en een moer, allemaal losse onderdelen. Die ringen die laten ze uit hun handen vallen, die moeren roesten. Het is echt altijd gezeur met die bouten, zijn altijd weg. Alle bouten zijn altijd zoek na afloop van een project. Want continu moet je nieuwe bestellen. En het gaat ook gewoon langzamer. Een pen, een pen die heeft zo, zo zoekertje, zo en aan de achterkant een borst. Dan zit hier een gat, dus zo steken ze hem er doorheen. Dan kan vaak, hier aan de achterkant zit ruimte voor een cilindertje, dan kunnen ze hem met een pompie kunnen ze hem zo door een gat heen pompen, met die zoekrand lukt dat wel. Dan gaat hier een grote borgpen doorheen. Dan gaat er een splitpen op. Dat zijn eenvoudige handelingen. Of grote pennen die hebben ook in de smaak van. Klein zoekrandje, als die echt groot is. En dan gaat hier een deksel op. En dat deksel wordt dan met drie vier vijf boutjes vastgezet. En dan hoef je alleen de deksel te bouten en dat zijn allemaal tapbouten. Dus die hebben dan niet die moer, dat scheelt al. En die hebben ook maar een ringetje, dat scheelt ook. Maar dat is ook niet ideaal. Het makkelijkste is gewoon een grote pen een borgpen en een klip, zo’n elastische vering. En dan is alles geborgd.
Rens: Oke en wanneer, wat is het onderscheid tussen wanneer je gaat lassen en wanneer je gaat pennen?

Leo: Wacht even. Moet nog wat uitleggen.
Rens: Oh sorry.
Leo: Een M48, dat kan nog een bout zijn, M36 is ook nog een bout. Je gaat geen pin 36. Maar alles, nou vanaf, pak hem beet rond de 60 millimeter als ondergrens minimaal, dan is het eigenlijk gewoon een pen. Het heeft geen zin om een pennetje 40 te maken. Ja pennetje 40? Daar kan dus gewoon een bout M36 in of een bout M42, dat hoeft dan weer niet.
Rens: Oh en waarom dan niet? Wat is dan het voordeel dat je dan toch die bout erin steekt?
Leo: Ja, omdat je die koopt. Dat is gewoon een koopdeel. Je kunt die bouten natuurlijk krijgen met die kop met een hele lange steel en met een stukkie draad. En dan, dan heb je hier de steel, die is net zo sterk als een bout. Want die bout die kan je kopen als 8.8, als 10.9. En 8.8 is 640 sterk, 8 keer 8 keer 10, ja en als ik een pennetje koop van 40 en ik maak hem van 42-chromo-4 is die ook 600 sterk. Dat is hetzelfde.
Rens: Oh en dit is dan goedkoper?
Leo: Ja die ligt gewoon op de plank, die kan je zo krijgen. En een pen moet je laten maken. Maar vanaf 50 millimeter, dan gaan we bouten niet meer tekenen. Dan tekenen we pennen. Dat is een beetje een omslagpunt. En bij ons zijn de krachten meestal zo groot, dat je al gauw groter bent dan 60 millimeter. Dus pennen hebben qua montage de eerste voorkeur. Lassen op site kan sowieso niet. Lassen op site zijn altijd slecht. Geen controle op, weer en wind. En lassen op site hoe ga je dan weer afbreken? Dat kan niet. Als je iets gelast hebt, dan zit het vast. Dus lassen op site dat doen ze eigenlijk alleen voor zeetransport. Zeevasting. Daar worden dingen vast gelast en die worden bij aankomst, ergens anders, weer losgebrand.
Rens: Oh.
Leo: Ja als jij met de boot uit China komt, daar staan grote modules op of andere dingen. Dan moet jij gezekerd zijn voor een storm op zee. Dat je niet je spullen verliest. Dus dan worden daar braching en schoren opgezet en dat zijn vaak pijpen die gewoon afgelast worden. Een schetsplaat, er wordt gelast en hier in Nederland wordt het losgebrand.
Rens: Oke, ja, klinkt wel omslachtig.
Leo: Nee, nee, nee want dat is heel makkelijk. Wat ze doen. Je hebt het dek van de bak. Zo en dan zitten hier een paar grillage balken zo. En dan kan er hier een groot vat op liggen in een soort van zadel. En dan, dat zadel heeft zo'n flapje vaak. Want dan kun je hier je SPMT's eronder rijden. Iets groter tekenen zo. Dat moet erop gereden worden. Dus hier moet een SPMT onder staan met z'n wielen. En dan zit hier vaak een schetsplaat aan. En dan lassen ze op het dek een schetsplaat, een schetsplaat. Een dan pakken ze een pijp, die snijden ze een sleuf in. Hier snijden ze een sleuf in. Die laat je er gewoon zo overheen zakken. Over die sleuf. Dat hoeft niet eens precies pas te zijn, hoeft niet tot de laatste millimeter. En dat lassen ze op schuif zo vast. En ze lassen het daar zo vast, past altijd. En dat is gewoon een goedkope pijp, een pijp rond 300 ofzo or rond 400. Een normale buis van een paar meter lengte. Want dan kun je al je krachten als het ponton natuurlijk heel hard rolt, zo dwars. Ja dan gaan die krachten gelijk hup die buis in en dan het schip of dat ponton in. Dus dat is heel goedkoop, dus lassen aan het dek, dat is heel makkelijk dat kan altijd.
Rens: Maar daar verpest je het dek niet mee ofzo?
Leo: Een heel klein beetje maar. Je staat waarschijnlijk boven een webframe. Dus van bovenaf gezien is zo'n ponton in een aantal, allemaal kamers, met frames, dan kunnen ze pompen. Dus je zorgt natuurlijk dat je uitlijnt op die sterke punten. Dus die grillage loopt vaak op de kruispunten, daar kun je lekker veel kilo's naar beneden drukken. En dan zitten er van die bulkheads, dat zijn van die dwarsverbanden. Nou, daar kan je een schetsplaat op lassen. En dan gaat er gewoon zo'n pijpje tussen. Dus die pijp die wordt zo gemaakt, zo. Die sleuf dat kost helemaal niks. He, dat is die doorsnede, die laten ze eroverheen zakken en dan lassen ze hem zo. En dat steekt er zo in en dat past altijd. En dan kan je die schuiflas uitrekenen, dat is een hele goedkope manier om dingen even vast te zetten. En na afloop wordt alles weer losgebrand en ook het dek weer schoongemaakt. Ook dit wordt hier vastgezet, daar zitten vaak van die clips op. Dit is de dekvloer. Staat hier een profiel met z'n onder flens. Dan hebben ze vaak zo'n staalplaatje, zo. En stel dit is een flens van 25, dan snijden ze hier clipjes van 30. En als je die natuurlijk een beetje zo scheef zet. Dan is dat hapje, is ook 25 hoog.

Zo is ie 30 en zo gekeken is die 25. Dus dat past altijd. Dan lassen ze hem hier en dan lassen ze hem daar en dan zit het vast. En dit zijn allemaal plaatjes 20. Is allemaal heel standaard staal en hoe moet je anders iets aan een dek vastzetten? Dat is met bouten, met kettingen kan niet he? Als dit een module is van 3000 ton en je moet dat met kettingen vastzetten, dat kan helemaal niet.
Rens: Die zijn niet sterk genoeg?
Leo: Nee. Nee kettingen die zijn 5 ton en maximaal 10 ton ofzo, dan heb je een hele sterke ketting. Maar je kan zo 20 procent dwarskracht hebben. Dus lassen, eigenlijk op bouwplaatsen wordt niet gelast. Zeevasting is een van de weinige dingen waar op site gelast wordt en dat wordt ook gewoon losgebrand. Maar bij een gantry gaan wij niet lassen. Dus je moet iets verzinnen, of bouten of pennen. Of klemmen, dat kan ook.
Rens: Dat je het op schuif?
Leo: Ja, dat is, maar ook dan moet je iets hebben van een systeempje. Ja dat is, maar je moet een verbinding en pennen zijn gewoon ontzettend makkelijk.
Rens: ja en dat is dan voor de elementen die dus niet containerized zijn?
Leo: Buiten ja, ja in principe wil je alles in een container vervoeren, dat is heel efficiënt. Dat hoeft niet altijd. We hebben bruggen gebouwd, de levycrossings over de Mississippi. Dit is de Mississippi rivier en die hebben levies, dijken, sinds de overstroming van Katrina. Lang geleden bij New Orleans hebben ze heel de Mississippi van dijken voorzien. Maar alle industrie ligt nu achter de levies. Dus er staan hele grote raffinaderijen, plastic fabrieken, alles staat nu achter een dijk. Terwijl alles wordt aangevoerd met pontons over de Mississippi. En nu moet alles over die dijk naar de fabriek. Dus wij hebben nu een balken systeem ontworpen, dat heten levy crossings. En dat zijn vijf delen. En dat zijn kokers van drie meter breed en vier meter hoog. En die kunnen, fabrieken kunnen die bij ons huren en dan installeren wij dat. Er zit een hele lange oprit bij er zit een brug bij, een afrit bij. En de eerste fabriek die heeft hem nu geloof ik voor drie jaar gehuurd. Drie jaar lang ligt daar nu een enorme brug van ons. Van ik dacht dat we vier van die balken gebouwd hebben. Die zijn totaal niet containerized. Er liggen hier dekpanelen tussen waar je overheen kan rijden met SPMT's, dat is een miljoenen investering geweest. Die eerste fabriek die huurt hem nu voor een paar jaar. Als zij klaar zijn met verbouwen en ze hebben heel hun fabriek uitgebreid en al die modules die zijn er. Dan halen wij hem weer weg, want we mogen die dijken niet meer aanraken in Louisiana. Die zijn heilig die dijken. Ze hebben geen sluisjes gebouwd, ze hebben geen. In Nederland hadden we waarschijnlijk gezegd 'Nou, we maken een coupure, zo. En dan gaat hier wel een schuif in als bij hoog water van de Mississippi, dat je er gewoon tussendoor kan.' Nou zo slim waren ze niet. Ze hebben gewoon dijken gebouwd en we mogen ze niet meer aanraken. En de volgende fabriek, want er liggen heel veel fabrieken daar in Louisiana. Dus wij kunnen, iemand anders die zegt ik ga m'n fabriek upgraden, verbouwen, doen, die kan dat dan weer huren bij ons. Dat zijn de levy crossings. Deze dingen zijn dus niet containerized. Het zijn balken 70 meter gedeeld door vijf, 14 meter. Hij is al te lang voor een container, hij is te hoog voor een container en hij is te breed voor een container. Maar deze liggen er dus gewoon jaren.
Rens: Ja, dus dan maakt het minder uit.
Leo: Ja en die hebben ook een koppeling met bouten. De koppeling is, met een zijaanzicht. Ik ben er half bij betrokken geweest. Onderin zit een pen. De trek is een honderd procents koppeling. Boven drukken we. En daar zit dan een bout in. Daar zitten bouten M68 in en een stuk of 6 ofzo.
Rens: Oh ja en omdat dit natuurlijk een langer project is, is bouten dus...
Leo: Dus hier is het druk, hij kon volgens mij 100 procent druk hebben en 20 procent trek ofzo bovenin. In principe die trek kant, als wij gaan monteren, dan kraagt die uit als een hengel. Zo rij je naar de overkant, dus hij moest wel wat trek kunnen hebben. Maar het was 100 procent druk boven en 100 procent trek onder en boven wat minder trek capaciteit, dus daar kon je wel wat grotere bouten doen. Want het voordeel van bouten is spelingsvrij. Het nadeel van pennen is natuurlijk speling. Als ik een penverbinding maak. Met twee ogen. Dan zit hier een gat en daarachter zit ook een gat. Die pen die is een millimeter kleiner. Dus ik heb 1 millimeter speling. Maar die ene millimeter speling heb ik ook aan de achterkant, daar heb ik ook een millimeter. Uiteindelijk heb ik al twee millimeter. Als ik aan de bovenkant platen stuik tegen elkaar laat lopen.

Zo, die vlakken die kan ik bewerken. Zo hier loopt een balk zo een hele lange balk en daar een balk en dit is het bewerkte vlak. En ik ga hier een bout doorheen doen met een moer. Kijk, dat is nul speling. Met een pen met een gat verbinding is altijd speling. Dat is niet uit te sluiten. Dus daar moet je iets mee met die speling. Het is ook wel fijn speling, want dan kan het een beetje uitzetten.
Rens: Ja, maar het zorgt ook voor misallignment.
Leo: Ja, daarom zijn gantries ook altijd een beetje krom. Of rekenen wij ze een beetje krom. We bouwen ze vaak heel recht, maar we rekenen ze vaak heel krom.
Rens: Ja want ik hoorde gister van Ivo over die, tijdens dat je zo'n kortstondig project en omdat je niet te maken hebt met dat mensen zich superveilig voelen. Ja mensen moeten zich natuurlijk wel veilig voelen met het gebruik van zo'n constructie. Maar de serviceability eisen, die zijn veel minder streng.
Leo: Ja.
Rens: En daardoor zijn dit soort dingen dus ook veel meer gedoogd en veel meer...
Leo: Nou nee dit is gewoon praktisch, je kan niet anders, ik kan niet spelingsvrij monteren met pinnen. Hoe moet jij spelingsvrij monteren?
Rens: Je kan er epoxy tussen doen ofzo.
Leo: Ja en dan maak je hem los, en dan? Bij het volgende project zitten we al die epoxy los te bikken. Nee er zit gewoon speling in. En dat betekent dus ook, die gantry die kan jij dus scheef opbouwen. Dus jij start hier en je hebt jezelf vastgeklemd aan het beton. En hier zit een beetje speling, dus dat mastdeel zou theoretisch zo kunnen staan. En zijn broertje zou theoretisch zo kunnen staan. En zou theoretisch... Dus je kan jezelf scheef opbouwen. Als jij consequent. En je kan ook zo opbouwen. Kijk je kan ook in je speling net zo net een beetje staan, en zo net een beetje staan. Je kan net zo 2 millimeter ook zo dwars staan. Zo kun je ook scheef opbouwen. Die scheefheid heb je ook. Dus wij zeggen heel vaak, heel veel bouwnormen zeggen 1 gedeeld door 200 of 1 zelfs 1 gedeeld door 100 of 1 gedeeld door 200 als misallignement in bouwconstructies. Dat is typisch gebouwen. Wij stellen 1 op 500 of 1 op 1000 zelfs. Omdat ons spul is allemaal gemachined en bewerkt en de grap is dat dingen zich aardig uitmiddelen. Als jij zo aan het opbouwen bent, dat is dus niet zo. Het gaat vrij random, het gaat soms ook weer een beetje terug. Dus niet al je spelingen werken tegen je, dat is wel zo in een storm. Als dat ding echt omgaat, ja dan weet je zeker dat je zo staat. Maar goed je kan daaraan rekenen en wij stellen dat dat altijd veel minder is dan dat en de gantry Borealis die zat zelfs op 1 2000ste.
Rens: 2000ste!?
Leo: Ja ze meten buiten wel, maar ze schrijven het nooit op, maar die gantry was 116 meter hoog of 110 meter hoog. Hij stond minder dan 25 millimeter scheef. Met een lasertje meten ze tijdens het opbouwen alles. Op de foto's, dat ding is ook spat recht, dat wil niks zeggen op die lengte. Van horen zeggen 'Ja nee we hebben gemeten en hij staat recht.' Dat moeten we dan maar geloven. Dus wij dachten, grootteorde van minder dan 25 millimeter. Als je dan deelt, ja 1 2000ste dat is helemaal niks. Dus wij rekenen streng maar niet zo streng als de norm. De norm zegt dat dingen echt heel krom kunnen zijn, maar dat komt allemaal uit de staalbouw in de hallenbouw.
Rens: Maar jullie zijn dus eigenlijk strenger dan...
Leo: Nee wij zijn, ja wij bouwen veel strenger met veel kleinere toleranties op. Die enorme scheefstanden die ze in de hallenbouw hebben. Kijk dit soort dingetjes, dit is, er zit hier een deling, daar een deling en er zit daar een deling. Die zijn die varkensstallen, dat is echt gewoon gooi- en smijtwerk. Nou voor dit soort, daar staat, de ankers die zijn al scheef ingestort. Dat staalprofiel dat is dan gelast aan een voetplaat, waar een paar ankerbouten in moeten. Nou dat staalprofiel dat is net niet helemaal netjes, beetje kromgetrokken dus die voetplaat gaat een beetje zo scheef. En dan krijg je hier zo die deling, daar zit die ene kopplaat die hebben ze gelast op een staalprofiel die is een beetje zo kromgetrokken en z'n broertje is zo kromgetrokken. Dan weet je ook weer niet hoe het monteert. Dit soort dingen daar schrijven ze vaak 1 op 100 voor. Maar dat stort niet in he. Dat moet je maar meenemen die scheefstand in je rekening, in je rekeningen. Maar dat is voor ons onwerkbaar. Dit soort schuurtjes zijn 8 meter hoog, dus dan heb je het over 8000 gedeeld door 100. Een, twee nullen eraf, ja 80 millimeter. Als wij 1 op 100 doen en je bent 100 meter hoog. Dan heb je een meter dwars

uitbuiging. Dat slaat gewoon helemaal nergens op. Dat meten we niet, we zien het niet, het is er niet. Dus die 1 op 1000 of 1 op 500 dat zijn vaak de getallen die we in onze sommetjes stoppen.

Rens: Want er was bijvoorbeeld een, nou het was ook een wild idee hoor, maar het was om alles mee te nemen. Maar dat was dus eigenlijk wat je hier hebt getekend, dat was dus een toren en dan zou dus, met die gantry balken, zou dus bijvoorbeeld aan de zijkant opschuiven. En dan die andere toren blijft gewoon staan om vrij te komen van je vat. Maar dan had ik berekend inderdaad dat je uiteindelijk wel, die masten zijn sterk genoeg om dat allemaal te houden. Je fundering dat is natuurlijk wel een uitdaging. Maar het boog wel een meter door.

Leo: Nou dat is voor een mast heel veel.

Rens: Ja, dus dat is eigenlijk gewoon te veel?

Leo: Ik denk dat je daar reken technisch niet uitkomt hoor, met een meter krom. Dan gaan de spanningen best wel oplopen voordat je zo'n grote mast van 8 meter diepte of 4 meter diepte om hebt. Daar komt heel veel kracht bij. Want dat is eigenlijk gewoon, als jij onder ingeklemd bent. En je wilt een meter opzij, dan is delta, is FL tot de derde gedeeld door 3 EI. Dus bij een meter, 1000 millimeter, kun jij terugrekenen met je parameters wat je F wordt en dan het moment is F keer L, de hoogte. Nou dan ga je toch forse krachten krijgen hier onderin.

Rens: Ja het was ook echt gigantisch hoor, maar als ik die hele rot formule zeg maar met normaalkracht en moment erop enzo. Dan was de Unity Check 0.4 ofzo. Maar het zijn dus te grote vervormingen? Stel je zou dat dus hebben, wat zou dan daarvan, zou je zeggen 'Dat moet je gewoon niet doen, punt.'?

Leo: Als het kan, dan kan het. Maar het zal er zeker raar uitzien. Als dingen elastisch blijven dan mag het. Je mag, als je het niet kapot rekent dan reken je het niet kapot. Daar zijn we niet kinderachtig in. Onze kranen onze grote PTC-kraan die buigt ook anderhalve meter door. Ik heb een mast en ik heb een jib, met allemaal touwtjes ertussen van draden en kabels. En als ik 3000 ton hijs, dan ga ik dat oppakken. Al die tijd dat ik aan het hijsen ben, dan ben ik dit aan het vervormen. Die trekdelen die worden korter, die jib die komt krom te staan, die kabels die rekken uit. Zou je gewoon aan je kraanmodel 3000 ton hangen, dan komt die zo anderhalve meter vervormd die. Hij komt geloof ik een meter naar voren. Dat doen we niet want die machinist die haalt draad binnen, die trekt zijn kraan gewoon die meter naar achteren. Dus je staat als een hengel sta je krom. Dus die meter vervorming die zit gewoon in constructies, dat kan gewoon. En een telescoop kraan ook, zo'n telescoop kraan daar zijn ook foto's van, die zijn zelfs meer te zien. Die telescopen die staan dan ook gewoon een meter krom, zo'n werp hengel. Er zit daar ook speling tussen de delen, die schuifdelen. Maar het is ook gewoon allemaal heel dun hoogwaardig staal, 690 en 890 en 1100 soms. Dus dat is heel elastisch want je kan dus met t is 6 millimeter en de E van staal, kun je gewoon gaan uitrekenen. Maar dan kom je daarop. Dus vervormingen opzich is niet een ramp, een meter. Maar de krachten die erbij horen, ja maar die moet je als je die goed kan rekenen, dan is het goed he. Dus dat is niet verboden om dat te hebben.

Rens: Nee oke, dus dat zou niet een showstopper zijn?

Leo: Nee. Dat mag, je mag gewoon veel vervormen. Vervormen is, als niks aanloopt en niks vastklemt dan mag het. Als het maar elastisch is, want dan komt het toch weer terug.

Rens: Ja. Dan nog een klein vraagje over die hydraulic power packs waar je het over had. Ik had van Erik volgens mij gehoord dat die alleen maar werken als ze staan zoals ze staan. Dus als die omgedraaid zijn dan werkt die niet meer. Kun je me daar iets meer uitleg over geven?

Leo: Nou een power pack is degene die stroom levert, of hydraulische olie. Een power pack daar gooi je diesel in, dan gaat de pomp draaien en dan heb je een flow van olie. Dus de power pack levert de energie voor de aandrijving. En een power pack die drijft sleeën aan, skids, of strand jacks of vijzels of elektro, of een hydro motor. Maar nee dat vind ik vreemd, dat zou ook gewoon twee kanten op moeten werken. Dat zegt me niks.

Rens: Nee prima. Dan, die gantry, dat is een mastdeel. Als je trouwens, het uurtje is in principe voorbij, dus als je door moet.

Leo: Nee hoor ga door.

Rens: Oke. Dat zijn twee torens en elke toren heeft vier funderingspunten. Maar ze zijn niet allemaal, ze worden niet even zwaar belast. Degene in het midden worden tijdens de lift het meeste belast. En die aan de buitenkant bij een storm ofzo.
Leo: Nou dat ligt eraan hoe jij je oplegbalk maakt.
Rens: Maar bereken je ze wel allemaal hetzelfde? Of hou je er rekening mee dat die binnenste tijdens lift bijvoorbeeld, stel dat is de maatgevende. Bereken je dan die binnenste anders dan de buitenste of bereken je ze gewoon op de maatgevende?
Leo: Nou daar weet jij zelf het antwoord al op, je moet iets uitrekenen op de krachten die gaan optreden. Hier liggen die balken overheen. Hier ga ik hijsen. Als jij je balk precies in het midden oplegt, dan gaan de kracht gewoon netjes naar alle vier. Maar als jij je balken op de binnenste oplegt dan gaat er meer daar inzitten. Als de wind van deze kant waait, dan gaat die hier wat meer drukken en daar wat meer trekken en hij gaat hier wat drukken. Maar ja, de wind kan ook van die kant waaien, dus dan is het andersom, dus dan weer die buitenste toch he? Dat maakt allemaal niet uit. De wind kan ook nog zo waaien. Dus jij moet alle richtingen wind en aardbeving en hijsen moet je combineren. En dan krijg je gewoon min max getallen. En dan maken we een funderingstekening en omdat alle richtingen kunnen omdraaien komt er in de regel uit dat ze allemaal die min max getallen hebben.
Rens: Ja oke maar in het, misschien is dat het uitzonderlijke geval, maar dat de lift, dat die het meest kritiek is. Waardoor deze, sorry, waardoor de buitenste in principe lichter kunnen dan de binnenste. Zou je dat dan doen of zou je de buitenste ook gewoon zoals de binnenste gaan rekenen?
Leo: Nou ja wij hebben onze oplegging, die is hetzelfde. Onze startpunt, ons baseframe, dit dingetje die baseframe box. Die is hetzelfde. Maar wij geven krachten door aan de klant en als er echt significant meer kracht aan de binnenkant zit, dan aan de buitenkant dan zeggen we ‘Oke, deze moet vastgezet worden met tien keer een dywidag en deze kan met vier keer een dywidag toe.’ Dan doen we dat. En die klant moet sowieso dat stuk beton eronder uitrekenen, dus dat is hen pakkie an. Maar zodra het een beetje in de buurt ligt, wordt dat allemaal afgerond tot dezelfde getallen.
Rens: Ja en dit is opzich wel logisch inderdaad die ene, je kan niet voorspellen waar de wind naartoe gaat.
Leo: Nee. En ook de twee procent bijvoorbeeld met het omhoog brengen van een vat kun je twee procent drukken. Omdat de kraan die aan het tillen is en je gantry die kunnen niet gelijklopen, dus die twee procent die kan in die kant werken. Dus twee procent, maar het kan ook dat de kraan te sloom is en dan ben je je gantry naar achteren krom aan het trekken. Dan ben je heel snel aan het hijsen maar die kraan die gaat niet snel genoeg en dan trek je jezelf die kant op. Dus ook die twee procent kan twee kanten op werken. Dus wind kan alle kanten, tailkrachten kunnen alle kanten. Dus je zit al gauw met allemaal combinaties die in alle richtingen werken, dus dan kom je al gauw op dezelfde krachten uit. Maar als de binnenste echt zwaarder belast wordt, echt substantieel veel zwaarder dan de buitenste. Dan hoeven wij de klant niet op kosten te jagen, die vindt dat ook fijn. Dus dan krijgt die gewoon lagere krachten daar. Maar wij gaan niet een lager detail ontwerpen. Om de simpele reden dat dat verkeerd gaat. Stel dat wij een lichte en een zware uitvoering hebben, dan zul je precies zien dat ze hem verkeerd stellen en dan zit die lichte waar de zwaarste lasten zijn en de zwaarste aan de buitenkant. Dat is gewoon onhandig.
Rens: Oh, dus om wat veiligheid weer in te calculeren.
Leo: En gemak voor de ploeg buiten. Ze worden, ze moeten op heel veel dingen letten. Daarvoor zijn die masten. Rupskranen, zoals je ze koopt, je hebt een hele manual, die starten. Je kan tig configuraties krijgen in Liebherrs. Vaak zware masten onder en steeds lichter boven en dan hebben allemaal coderingen. Nou degene die met die kranen werken die weten hoe ze moeten opbouwen en die weten wel de configuraties. Dat staat dan ook wel op het plan, dan kijken ze de manual. Er moet hier een mastdeel type 2D in en hier een 2A en dan kan je hier 1C en hier op 1B, het zal allemaal wel. Dat gaan we met die gantry niet doen. Je kunt je natuurlijk voorstellen, dat je een gantry opbouwt, maar dat ga je niet doen want je hebt een MLS. Maar je had kunnen zeggen ik doe hier een plaatdikte 30, hier plaatdikte 25 en hier plaatdikte 20 en ik eindig plaat 15. Dat kan, je kan helemaal gaan optimaliseren. Hebben we niet gedaan, al onze MLS masten zijn hetzelfde. Dus die

zijn allemaal even dik, even sterk, even zwaar. Er zit dus bovenin een overkill aan materiaal want onderin, dat is waar je hem controleert. Maar je mag optimaliseren. Er zijn de klant daarin te willen (?). Bij rupskranen gaan ze daar heel ver in, want elke kilo die verder weg zit van het draaipunt dat kan je minder hijsen. Dus daar worden de masten echt helemaal geoptimaliseerd. Zware masten onder, lichte masten bovenin. Maar niet met de MLS gedaan. En de optimalisatie die jij misschien in je funderingspunten hebt. Daar kunnen we, als dat serieus is, kunnen we dat aan een klant geven. En anders laten we het zitten. Mara dat zit niet in de scope van jouw opdracht denk ik. Ik denk niet dat je daar iets mee kan verder.
Rens: Nee, maar het is goed om te weten. En dan, even voor de zekerheid. Die MLS masten, kipstabiliteit is dat een issue?
Leo: Hoe? Kip treedt alleen op bij buiging. Je hebt knikken, dat doe je bij druk en kippen doe je bij buiging, met een moment. En kip is een stuk complexer dan knikken. Knik is goed uitgezocht, kippen is een stuk ingewikkelder. Er moet dus sprake zijn van een buigend moment. En dat buigend moment dat moet in de sterke as werken. Want als je namelijk op je kant ligt kan je al niet meer kippen. Om de zwakke as kun je niet kippen. Oke, dan heb je een gantry. Een gantry is een uitkragende balk. Die zou dus alleen, hij is zo 8 meter, hij is zo 4 meter. Dus het sterke as moment, dat kan kip geven, het zwakke as moment kan al niet. Een zwakke kan die dus alleen zo wegknikken. En bij sterke as, dan zou je dus kip kunnen hebben, want dan zou die bij een groot moment zo. Zou die eventueel willen draaien, dus dan wil die zo uitkomen daar bovenin. Dus dan moet er een twist inzitten. Ik kan me er nog niet helemaal voor me halen hoe een uitkragende mast kan, het zal wel bestaan. Kip van een uitkragende mast, maar ik ben het nog niet reken technisch voorgekomen. Dus ik denk het niet. Oh ja en er is nog een reden dat het niet, nou dat is sowieso een reden. Bij kip, een I profiel, die heeft een kleine I_t, I_t is klein. Dus er is weinig dat die rotatie verhinderd. Bij kip zit dus altijd een twist is, dat is altijd rotatie in he. Kip is iets met rotatie. Heeft dus iets te maken met kleine I_t. Als jij een kleine torsiestijfheid hebt, dan ben jij, kan je makkelijk roteren ben jij kipgevoelig. Zo'n vakwerk met die rondlopende kokers, heeft een grote I_t. Dus dat ding is heel torsie stijf. Hij is dus niet gevoelig voor kippen. En er is een mooi klein regeltje dat als jij kokers hebt. Kokers met een b h verhouding van 2 staat tot 1, niet kipgevoelig. Dus dat gebruiken wij heel vaak om te zeggen 'Ja wij hebben masten van 8 bij 4, die zijn precies 1 op 2, niet kipgevoelig.'. Ergens zit er in de Eurocode een klein regeltje dat kokers met een verhouding 2 staat tot 1 niet kipgevoelig zijn. Hoef je dat nooit meer uit te rekenen. Dat is heel lekker. Maar nee dat ben ik ook nog nooit tegengekomen in mijn resultaten dat dit een zinnige was, nee.
Rens: Oke, nou dat is mooi. Dat scheelt een hele hoop rekenwerk.
Leo: Ik denk niet dat je de kipformules hoeft in te bouwen.
Rens: Nou dan had ik alleen nog een vraag over skidshoes en skidbeams. Maar dat had gister had Ivo al uitgelegd dat een skidshoe dat zelf kon jacken en dat een skidbeam gewoon.
Leo: Een skidbeam is gewoon een balk, die sleet. En een skidshoe heeft nog een vijzel dat die iets kan. Oke dus jij gaat kijken, de scope: Hoe kan je die grote bovenbalk erboven op krijgen. Met alleen een telescoopkraan. Dus dat betekent opbouwen laag.
Rens: Ja dan ligt die balk er al op.
Leo: En ook alle bordessen en alle hijsvoorzieningen en dan moet je kiezen: ga ik naar boven klimmen, zet ik mijn klimsysteem hoog, dat ik dat klimsysteem alleen maar 100 of 200 ton van de bovenkant hoef te tillen. Of zet ik mijn klimsysteem laag zodat ik op een gegeven moment 800 tot 1000 ton aan het tillen ben. En inclusief het stabiel houden van alles. Ja, dat is inderdaad een fundamentele keuze en daar moet je, daar kun je al op wegen op techniek, prijs, veilig werken, dus hoe werkt het operationeel. Want als jouw platform naar boven klimt, moeten jouw mensen ook mee naar boven. En alle spullen moeten ook mee naar boven. Als jouw platform op de grond blijft worden de krachten allemaal groter, maar je kunt lekker laag aan het werken de hele tijd. Oke en dan dat is allemaal aspecten. En je moet dus ook meewegen hoe bouw ik weer naar beneden, hoe breek ik af. Want er staat een vat in de weg. Eigenlijk is het opbouwen minder belangrijk dan het afbreken, het afbreken is moeilijker. Als jij een systeem hebt dat goed kan afbreken dan ben je, dan kun

je ook goed opbouwen. En dan heb je al met een multi criteria matrix, heb je al concepten bekeken. Maar je concepten zijn nog niet zo goed dat je nu al op een concept door kan denk ik?
Rens: Er zijn drie concepten die dus als beste uit die multi criteria analyse waren gekomen. En nu ben ik dus eigenlijk die klimframes, want ik had ook een klimkraan was een van de opties. En er was ook nog een systeem waarbij je dan een extra klimsysteem naast je toren maakte, die dan de hele bovenkant er op tilt. Sarens die heeft een bepaalde gantry.
Leo: Dat hoeft geen 2000 ton sterk te zijn, dat hoeft maar 200 ton sterk.
Rens: Ja precies.
Leo: Ja precies, je hebt een lightversie die je ernaast zet.
Rens: Dat was ook bijvoorbeeld een optie, maar dat heeft dan weer bepaalde nadelen waardoor die eruit is gevallen.
Leo: Oh wacht even. Wat kost een Liebherr kraan? Als wij twee Liebherrs kopen om die balken naar boven? Kijk wij zetten die MLS neer. Het is niet zo'n stomme gedachten hoor. Ik zet hier mijn hele grote MLS neer, die is 8 meter diep zo. En hier moet straks dat vat komen. Dit is die 8 meter, die 4 meter is zo. Hier komen dan die grote balken straks op. Dit is die funderingsplaat, van beton, groot ding. Hier zitten de pijpenrekken in de weg, daar kan ik allemaal niks. Dit is mijn kant waar ik aanvoer. Ik zeg tegen die klant 'Joh, maak jij eens even een pukkel hier eraan en maak jij eens een pukkel daar eraan.' Hier wil ik een Liebherr klimsysteem zetten. En dat Liebherr klimsysteem dat brace ik af.
Rens: Wacht sorry, wat is dit precies?
Leo: oh gewoon een torenkraan, nee wacht even ik kan. Als ik hier een Liebherr torenkraan neerzet en ik zet hier een Liebherr kraan neer. Die dingen die zijn zelf klimmend.
Rens: Ja.
Leo: En die kunnen maar een heel kort mastje hebben, dat kan he? En als die in staat zijn om dit hier bovenop te leggen. Dan kan ik die grote balk die kan ik er hierzo tussen fietsen. Dan kan ik met twee van die kranen, zou ik dat dan kunnen optillen. En dan kan ik het hierzo bovenop leggen. Als ik hier gewoon twee hele korte torenkranen neerzet. Zo, twee hele korte torenkranen. Hebben we nooit gedaan, want wij willen altijd alles zelf doen. Maar dan, dan hoef ik niet. Kijk deze systemen die kan ik vastmaken met torens. Ik zet vier masten neer en ik doe een steker, want ze zitten vast aan een gebouw. Ze gebruiken ze ook voor die bruggen over die dalen in Oostenrijk, in Duitsland. Die hele hoge torenkranen die kunnen gewoon 130, 140 meter hoog. Als jij maar om de paar meter, om de 30, 40 meter vastmaakt, kan je omhoogklimmen. Ja want onze MLS masten die zijn gigantisch. Daar kan zo'n torenkraan aan vast en dan zit die gewoon vast. Dan doet die helemaal niks meer. Dan zit die balk in de weg. Mijn verbindingen naar... Maar nog steeds. Maar wat kosten. Maar oke, dan doe ik het bovenaanzicht, dan doe ik het anders. Als dit mijn balken hierzo zijn, dan zet ik ze daar, dat ik er net zo bij kan. En die andere daar. Jij zou dus bij van der Spek, of bij Liebherr, of bij onze commerciële mannen toch eens kunnen vragen; 'Wat nou als ik gewoon...' Waarom zou ik zelf het wiel uitvinden als ik twee van die kranen kan huren. Want die staan op bouwplaatsen de hele wereld. Waar wij komen met onze gantries, staan ook bouwkranen. Kijk, het is out-of-pocket money, we hebben het zelf niet. Wij moeten hier gewoon betalen. Daar zit gewoon de winst in voor de verhuurder. Hij staat er het hele project, van dag een tot de allerlaatste dag bij het afbreken. Dus het is een serieuze kostenpost. Maar je hoeft niet een compleet systeem te ontwikkelen. Dus dit is eigenlijk de nuloptie. Dit is bestaand.
Rens: Maar zou dit niet net zo prijzig zijn als dat je er zelf een rupskraan neer knalt?
Leo: nee want een rupskraan. Nee want deze systemen zijn snel en goedkoop en ze worden ingezet voor lange duur bij bouwen van flats, hoogbouw. Overal in het Midden-Oosten staan massa's torenkranen. Aannemers hebben ze zelf, die kosten... Eigenlijk is dit gewoon de kilo's, zo'n torenkraan, wat zal die wegen? Die mastdelen zijn heel licht. Ik denk dat er misschien 200, 250 ton staal in zit.
Rens: Oh, dat valt wel mee inderdaad.
Leo: Ja, en een grote rupskraan met al zijn ballast, daar zit je zo op 1000 ton. Dat is best een machine hoor zo'n rupskraan. En die heb jij ook nodig. Want die rupskraan, deze dingen, als ze vast kunnen houden aan die

MLS mast. En je, ja kan die balk er net tussen door fietsen en dat stekertje, dat kan interessant zijn. Want, waarom denk ik eraan? Wij moesten, we moesten niks, maar we gingen een aluminium uit een gezonken schip maken. In de Oostzee daar lag een schip. En dat was geladen met aluminium en die was tijdens een storm gezonken. En toen hadden wij een ponton ingericht en daar hadden wij een kraantje met een grijpertje op gezet. En dat kraantje, dat was een heel simpel rupskraantje, dat hadden we gewoon gekocht. En daar hadden we de onderwagen, die hadden we ervan af gesloopt. En daar hadden we een spin gebouwd, met pootjes. We hadden zijn bovenwagen en zijn motor hadden we ervan afgesloopt. Het was dus gewoon alleen maar een loadspreader waar hydrauliek zat. Dat kraantje dat lieten we naar de zeebodem zakken. Met een hele lange slang, dus hier op het dek stond gewoon de cabine. Dan kon je gewoon vanuit je cabine stuurde je de elektrische signalen en hydrauliek naar beneden. Dus ik dacht 60 of 70 meter diep. En dat kraantje kon onderwater met camera's en lampencamera's kon die aluminium plakjes pakken uit dat ruim en die deed die dan in een bakje. En dat bakje dat haalden we dan met een ... omhoog. Maar dit was een koop kraantje, dat was een koop kraan, gewoon een heel simpel dingetje. Gewoon zo eentje Hitachi of iets simpels en. Gewoon hij kon 10 ton tillen, gewoon een ding dat je overal ziet op het land. Gewoon alles wat niet onderwater kon dat hadden we eraf gesloopt. En die hydrauliek die kon wel onderwater met slangen. Wij kunnen dat niet zelf bouwen. Die mensen die dit soort kraantjes bouwen die Hitachi en Kobelco en alles, ja die kunnen dat goedkoper dan wij. Waarom zou ik moeilijk doen als het makkelijk kan? Als ons klimsysteem gruwelijk duur wordt en dan moeten we heel die MLSen er nog voor gaan verbouwen. En ik kan twee kranen huren voor een miljoen. En ik daar het systeem. Ja dan ben ik elk project een miljoen kwijt. Maar als het maar een paar ton is. Je hebt dat nog niet zomaar terugverdiend, zo'n MLS klimsysteem.

Rens: Ja want dan zou je dus deze kranen zou je tegelijk. Die zou je opbouwen en terwijl deze dan opgebouwd is, tilt die dan een mast deel op en dan bouwt die weer op en dan kan die zich eraan klikken. Dus deze twee bouwen ook dit hele systeem op.

Leo: Ja, en zichzelf. Maar dat is natuurlijk niet spannend, maar het geeft je een referentie hoeveel budget je hebt voor je MLS. En daarom moet je hem meenemen. De nuloptie bestaand equipement, die geeft het budget, en dat is een hele relevante. Jij studeert af aan de universiteit, jij moet breed kijken. Dat is het budget wat beschikbaar is voor alle andere opties. Ja want als alles duurder is dan dit en dan moet je het natuurlijk betrekken op realistische inzet. Nou het lijkt erop dat die MLSen, die zetten we maar een keer per jaar in. Zo'n systeem gaat 20 jaar mee. Dan weet je ongeveer hoeveel het kost. Want je gaat een keer verbouwen, maar ook bij het inzetten, er moeten extra containers naartoe, dat is extra geld. Dus je moet die nuloptie. Hoef je niks in je MLSen te doen, hoef je alleen maar een paar hoekjes, een paar oogjes te maken. Dus dat zijn minimale verbouwkosten van je MLS en operationeel is die ook niet duur. Er hoeft niet een vijzelploeg naartoe of een of andere special devices ploeg. Nee, gewoon twee machinisten. Dus dit is goed te ramen, het inzetten van twee. Dus je kan eens bij sales gaan vragen wie er wel eens een ramingetje heeft aangevraagd van dat soort kranen. Dan kunnen ze jou wel op weg helpen hoe je dat zou moeten doen. Maar waarschijnlijk hebben wij dit ook wel eens aangevraagd voor een project. Want wij. Als dit goedkoper is, dan gaan we dat niet zelf doen, dan huren we dat in. Er moet wel iemand van sales jou kunnen helpen. Alex van der Meer is bijvoorbeeld zelf een engineer geweest en er zitten er meer. Hylke Geerts is ook engineer geweest. Er zitten er een paar engineers die zijn naar sales overgestapt. Die hebben hier wel gevoel bij. Die kunnen waarschijnlijk wel zeggen 'Nou dan moet je die eens bellen.' En dan helpen ze je wel met een soort van raming maken voor je project. En dan komt daar uit rollen van nou ja, eenmalige inzet voor een project dat vier maanden duurt. Ben je zes ton kwijt. Nou dan weet je, elke MLS gantry waar ik die kranen inzet zes ton. Maar goed als je dat dan twintig jaar hebt. 20 keer 0.6 is 12 miljoen, dat is best substantieel geld. Maar, zomaar even een heel systeem bouwen, dan ben je ook een paar miljoen verder en je hebt nog steeds kosten aan je systeem. Ik heb geen idee, maar anders zit je volledig in het blauwe te engineeren. Dan zeggen we op het eind 'ja wat kost het?' 'Ja hoezo?'. En dan komt er een wijsneus en die zegt 'Ik heb het even opgevraagd, maar ik kan voor drie ton kan ik twee torenkranen huren. Waarom heb je dat niet bekeken?' Jij moet een nuloptie meenemen met bestaand equipement en de rupskraan.

Rens: Dus jij zegt de rupskraan is niet per se de nuloptie?
Leo: Nou wacht even dan zeg ik, de rupskraan is 0.1 want die hebben we zelf. En de torenkraan is 0.2. De rupskraan is de interne en de 0.2 is de externe oplossing. Ik denk dat je ze allebei toe moet voegen aan je lijstje.
Rens: Ja hier heb ik al, de kostenraming van bij de Jubail gevonden.
Leo: Heel goed, dus dan is dat de interne oplossing. Niks doen aan je MLSen, gewoon wat kost het nou. En daar komt ook de vraag vandaan 'Goh waarom is die zo duur?'. Maar je moet even kijken naar 'Oke als we het niet met de rups doen, maar we doen het met twee torenkranen, zijn die sterk genoeg om die bovenbalken erop te leggen?' Dat moet haast kunnen want je blijft relatief dichtbij. Je zit hier misschien maar op 15 meter radius te werken, terwijl die kranen die hebben hele lange uitleggen. Maar op 15...
Rens: Ja die zitten soms op 30 radius.
Leo: Ja, dus op 15 moet je een eind kunnen komen. Dit staat op alle bouwplaatsen van de hele wereld. Ook in het midden oosten. Overal zie je die torenkranen staan. Die zijn breed beschikbaar. Ze hebben iets nodig om aan vast te zitten. Als we een paar oogjes moeten lassen op de MLS en die MLSen die zijn kneiter sterk, die geven voldoende steun. Dan is dit een reële. Want als dit goedkoper is dan de rupskraan, dan is dat al winst voor onze sales afdeling. Dan weten ze al, dat hebben ze vast niet zelf uitgezocht, misschien wel, maar ik denk het niet. Je moet ook aan je klanten gaan vragen. Op een krappe bouwplaats dan sta je hier misschien al in de verkeerde zone. Maar toch, het is het uitzoeken waard.
Rens: Ik vind het helemaal geen slecht idee. Want ik had bij die rups gevonden, die is, als je hem een maand zou hebben en dan is 0.8 maand al hoe heet dat op- en afbouw. Dan kost je dat bijna, nee meer dan 200,000 euro.
Leo: Euro 200,000 voor een rupskraan.
Rens: Ja, voor een maand.
Leo: Oh voor een maand, maar zo'n project duurt al gauw twee tot drie maanden. Dan zit je al op die zes ton. Dus als jij torenkranen kunt huren voor bodemprijzen en het past en het kan. Allemaal. Het kan ook zijn dat ze afvallen, want die rupskraan die kan zelf bewegen, die kan er naartoe die kan weer weg. Die heeft ook wel weer voordelen.
Rens: Ja het voordeel is ook dat deze, maar ik had die eruit gehaald hoor, maar deze kraan kan natuurlijk ook het tailen doen.
Leo: Ja, ja en dat kunnen zij totaal niet. Maar voor tailen zijn weer hele goedkope oplossingen beschikbaar met sleetjes en tailframepjes en ... en SPMTs.
Rens: Precies dat was ook weer de reden, dat ik de neven taken die zo'n kraan kan doen. Die heb ik ook buiten de scope gelaten. Omdat er dus goedkopere en je hebt meer vrijheid ook als je dat niet hoeft te doen. Dan zou je kunnen zeggen 'Oh maar wij hebben dit systeem er liggen, dat kost zoveel of dat vinden we gewoon fijn om mee te werken.' Dan kan dat.
Leo: Maar de grap is. Hier heb je een klimsysteem wat als je van onderen klimt van minimaal 200 ton en maximaal misschien wel 1000 ton grootteorde. Die rupskraan die zal ook nog steeds wel in stukjes opbouwen die wil je niet. Een telescoopkraan die kan het gewoon niet. Die twee torenkranen die zullen ook weer in stukjes opbouwen, maar die hebben wel het voordeel dat ze snel zijn. En zij klimmen maar 100 ton omhoog. Ze klimmen zichzelf omhoog. Dus het is echt een orde lichter en kleiner. Dat kan interessant zijn.
Rens: Ja precies want dat klopt inderdaad als je dat frame aan de bovenkant, dan moet elk frame moet 200 ton tillen want die hele bovenkant die weegt 450 ton. Ja en als je het aan de onderkant doet een zo'n toren met de helft van de bovenkant is ook al ordegrootte 6 700 ton.
Leo: Ja dus dit gaat misschien nog wel over die 1000 ton. Het nadeel van die torenkraan is weer werken op hoogte. Je mensen moeten omhoog, maar we hebben een alimac. Je staat hierboven toch weer dingen in elkaar te doen. Maar het hangt helemaal af van de prijs, wat kost een torenkraan voor gedurende twee, drie maanden? En dan heb je er zelfs twee van nodig, want waarschijnlijk een gaat gewoon niet lukken. Want

anders hadden we dat allang gedaan. Een torenkraan, we zijn niet stom, als een torenkraan echt heel goedkoop is dan hadden we hem al lang ingezet. Maar het is.
Rens: Maar het is natuurlijk wel een voordeel als je een interne oplossing hebt toch?
Leo: Intern, ja tuurlijk. Eigen geld dat is eigen geld. Want hij kost dan wel, het is onze eigen kraan, het is ons eigen project, we zetten hem zelf in, het is goed voor je bezetting. Nee eigen spullen zijn altijd goedkoper dan extern. Ik bedoel het is niet leuk voor het project, maar het is wel leuk voor je eigen equipment. Extern is out of pocket. En wij hebben, wij hebben drie klimkranen. Dat is waar ook. Drie klimkranen voor windturbines on shore. Oh god hoe heten die dingen. Ik heb al een paar keer beloofd om jou dingen toe te sturen, toen kon ik het allemaal niet vinden, maar die kan ik wel vinden. Datasheet, die heb ik. Die staan in Maleisië, Australië, datasheet opzoeken toesturen. Ik ben alweer vergeten hoe ze heten. Degene waarmee ik het moest uitzoeken die heeft inmiddels Mammoet verlaten, die is voor zichzelf begonnen, Nial Mc Durmat (?). Ik weet waar ik moet zoeken, dat ga ik jou toesturen. Wij hebben kraantjes die zichzelf omhoogklimmen en die zijn, die staan heel steil. Dat zijn.
Rens: Ja die lijken een beetje op...
Leo: Ze staan op, ze komen bij ALE vandaan. ALE had ze, mammoet had ze niet, maar ALE had ze. En die hebben we overgekocht, die kraantjes van hen.
Rens: Zijn het dit soort dingen?
Leo: Nee hij staat op de grond, hij heeft een eigen klimconstructie en hij staat op een padistol, een vierpoot. Die kan ik opzoeken voor je.
Rens: Oh wacht, volgens mij heb ik die ook gevonden.
Leo: Ja, we hebben er drie van. En dat is eigen spul. Maar hij tilt maar 15 ton.
Rens: Oh, dat is niet zo veel.
Leo: Nee, maar misschien dat die fabrikant dat is een Deense fabrikant.
Rens: Iets met een K ofzo?
Leo: Dat moet ik opzoeken, dat weet ik niet. Maar we hebben ze, en we gebruiken ze voor die bladen en voor op land, on shore windmolens. Er is altijd gezeur over die funderingen.
Rens: Oh en dit zelf was het klimgedeelte? Dus die klom zichzelf tot...
Leo: Ja hij kan zichzelf klimmen en hij had een relatief korte mast en hij tilde niet zo heel veel geloof ik. Maar we hebben ze gewoon zelf en ze konden best hoog.
Rens: Ja ik heb ze in AutoCRANE gezien. Ik heb ze in AutoCRANE voorbij zien komen.
Leo: Hoe heten ze nou? Niet Favelco, nee ze heten anders. Kroll! Nee, nee.
Rens: Ja wel, volgens mij wel.
Leo: Nee, Kroll is een hele grote die is het niet. Het is een andere. Ik weet het eigenlijk niet. Daar moet ik me. Ik weet waar ik moet zoeken in ieder geval. Haha.
Rens: Maar zo'n tekening, deze, zo'n soort kraan heb ik wel in AutoCRANE voorbij zien komen.
Leo: Ja, en dit lijkt natuurlijk, want dit ding is zelfstandig, zelf staand die hoeft niet eens af te schoren aan dat ding. Dat is makkelijk die kun je dicht op die grote masten zetten, want dan kan die grote balk erlangs.
Rens: Ja, maar als die maar 15 ton kan tillen.
Leo: Nou dan heb je er twee nodig en dan mag die balk niet zwaar. Maar goed misschien dat die ook bestaat als 25 ton. Een schijfje bijzetten, misschien kunnen we hem opbiefen.
Rens: Ja, maar die, deze lift, in ieder geval bij de Jubail daar was de zwaarste lift met rigging enzo en de haakblok, was 130 ton.
Leo: Oh dat is stoer. Dat gaat die niet tillen. Dat is wel een probleem voor die dingen. Maar nog steeds we hebben iets wat we zelf al hebben, die kraantjes. En die hebben helemaal niet zoveel werk meer.
Rens: Ja je hebt het toch liggen, dan.
Leo: Dat moet ik even opzoeken. Kijk, wat heb jij nog meer?
Rens: Ja nee ik ben. Ik weet niet of jij nog andere?
Leo: Nee als jij een keer wat hebt dan wil ik wel wat lezen, maar je hebt nog niet zoveel op papier staan toch?

Rens: Nou, ik zit nu op 125.
Leo: Hallo. 125 bladzijden.
Rens: Ja maar er zitten ook heel veel tekeningen en dingen zeg maar. Ik heb van al die, kijk, stel je doet een translatie in de X richting, hoe ziet dat er dan uit? Of je split die gantry beam. Ik heb van al die dingetjes een tekening. Of je gaat het met die hele toren doen weet je wel. Dus ik heb daar allemaal tekeningetjes van gemaakt.
Leo: Oke, stuur maar op.
Rens: Oke top, dankjewel! Ook bedankt voor het interview en de informatie. Ik heb echt veel geleerd!

A.5. MCA

A.5.1. WEIGHT FACTOR

Not all criteria mentioned above are equally important. To distinguish which criterion is more important, a table (Table A-17) is created comparing each criterion against another. Each time the criterion in the row is compared to the criterion in the column. The one that is more important scores one point. If two criteria are deemed equally important, they both score one point [18]. To maintain a clear overview the criteria are labeled A to F, see Table A-16.

Table A-16 Criterion labels

CRITERION	LABEL
Motion controls	A
New technique	B
New material	C
Power	D
(Not) working at height	E
Foundation	F

By summing all points together, a total score for the criterion is established. If a criterion were to have a sum of zero, it manually gets a one so that it still has an influence. All other factors will be multiplied by two. The (adjusted) sum is normalized by dividing it by the total points. Now, all weight factors are known. The whole procedure can be seen in Table A-17.

Table A-17 Establishing the weight factor

	A	B	C	D	E	F	SUM	WEIGHT FACTOR
A	X	1	1	1	0	0	3	0.18
B	0	X	1	0	0	0	1	0.06
C	0	1	X	1	0	0	2	0.12
D	0	1	1	X	0	0	2	0.12
E	1	1	1	1	X	1	5	0.29
F	1	1	1	1	0	X	4	0.24
						Sum	**17**	**1**

A.5.2. Rationale

The following rationale is determined by the author and is discussed with the supervisor from Mammoet.

'Motion controls' are deemed more important than 'new technique' as a new technique can be learned over time. S, the investment costs stop when the new skills are acquired. More motion controls need more repairs, this is an ongoing process. This is the same reason that 'motion controls' are more important than 'new material'. It is also more important than 'power' since more power only slows down the project, maintaining the motion controls in good shape requires maintenance. However, the comfort and safety of working on the ground is more important. So is 'foundation', this is a selling point towards the client; a larger and heavier foundation means more costs. Moreover, the foundation is only used shortly, after the lift is performed it only serves purpose for the vessel. So, environmental reasons play a role.

'New technique' is equally important as 'new material' as they are both initial investments instead of an ongoing investment. Both criteria scored one point for that reason. All other criteria are elevated over 'new technique'.

'New material' is equal to 'power' as they are correlated, more material means that it needs to withstand more load, more load requires more power. Als because of the initial investments and the ongoing comfort for the workers or the costs for a large and heavy foundation makes that '(Not) working at height' and 'foundation' are elite over 'new material'.

'Power' and 'foundation' have similar considerations, however 'foundation' is more important because a large and heavy foundation is a waste of material.

The comfort and safety of the workers is more important than any of the criteria. Working at building sites comes with risks. Eliminating as many risks as possible is part of the safety standard of Mammoet [30].

The subjectivity in deeming a criterion more important than another is reduced by multiple reviews of engineers within Mammoet. Their weight factors are added to the factors from Table A-17 in Table A-18. Their considerations can be found in Table A-19 to Table A-23. The results differ, therefore the mean is added. Since the weight factor from Table A-17is created by two people this value is double counted in the calculation of the mean.

Table A-18 Weight factors engineers

AUTHOR	WALTER DE JONG	WOUTER GERRITSEN	XIMENA MILIA	WILLEM-JAN COSTER	MARCO HOLLEBRANDSE	MEAN
0.18	0.13	0.22	0.13	0.06	0.26	0.17
0.06	0.19	0.11	0.06	0.03	0.19	0.10
0.12	0.32	0.11	0.32	0.32	0.13	0.21
0.12	0.19	0.11	0.26	0.19	0.06	0.15
0.29	0.13	0.28	0.03	0.13	0.03	0.17
0.24	0.03	0.17	0.19	0.26	0.32	0.21
					Sum	**1.00**

Table A-19 Establishing the weight factor Walter de Jong Stuctural Engineer Mammoet

	A	B	C	D	E	F	SUM	ADJUSTED SUM	WEIGHT FACTOR
A	X	1	0	0	0	1	2	4	0.13
B	0	X	0	1	1	1	3	6	0.19
C	1	1	X	1	1	1	5	10	0.32
D	1	0	0	X	1	1	3	6	0.19
E	1	0	0	0	X	1	2	4	0.13
F	0	0	0	0	0	X	0	1	0.03
						Sum	**15**	**31**	**1**

Table A-20 Establishing the weight factor Wouter Gerritsen Stuctural Engineer Mammoet

	A	B	C	D	E	F	SUM	ADJUSTED SUM	WEIGHT FACTOR
A	X	1	1	1	0	1	4	8	0.22
B	0	X	1	1	0	0	2	4	0.11
C	0	1	X	1	0	0	2	4	0.11
D	0	1	1	X	0	0	2	4	0.11
E	1	1	1	1	X	1	5	10	0.28
F	0	1	1	1	0	X	3	6	0.17
						Sum	**18**	**36**	**1**

Table A-21 Establishing the weight factor Ximena Milia Stuctural Engineer Mammoet

	A	B	C	D	E	F	SUM	ADJUSTED SUM	WEIGHT FACTOR
A	X	1	0	0	1	0	2	4	0.13
B	0	X	0	0	1	0	1	2	0.06
C	1	1	X	1	1	1	5	10	0.32
D	1	1	0	X	1	1	4	8	0.26
E	0	0	0	0	X	0	0	1	0.03
F	1	1	0	0	1	X	3	6	0.19
						Sum	**15**	**31**	**1**

Table A-22 Establishing the weight factor Willem-Jan Coster Stuctural Engineer Mammoet

	A	B	C	D	E	F	SUM	ADJUSTED SUM	WEIGHT FACTOR
A	X	1	0	0	0	0	1	2	0.06
B	0	X	0	0	0	0	0	1	0.03
C	1	1	X	1	1	1	5	10	0.32
D	1	1	0	X	1	0	3	6	0.19
E	1	1	0	0	X	0	2	4	0.13
F	1	1	0	1	1	X	4	8	0.26
						Sum	**15**	**31**	**1**

Table A-23 Establishing the weight factor Marco Hollebrandse Fabrication Superintendant Mammoet

	A	B	C	D	E	F	SUM	ADJUSTED SUM	WEIGHT FACTOR
A	X	1	1	1	1	0	4	8	0.26
B	0	X	1	1	1	0	3	6	0.19
C	0	0	X	1	1	0	2	4	0.13
D	0	0	0	X	1	0	1	2	0.06
E	0	0	0	0	X	0	0	1	0.03
F	1	1	1	1	1	X	5	10	0.32
						Sum	**15**	**31**	**1**

A.6. BEST PRACTICE

A.6.1. STAGE I TOWER ERECTION

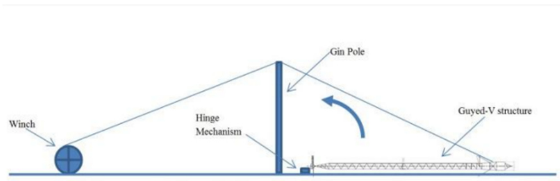
A.6.1.1. Additional crane

Currently, the assembly of the whole gantry is done with an additional crane, namely a stationary crane. One could think of alternatives for this. Possible alternatives are a climbing crane, a gin pole, and a drone/ helicopter.

a) Stationary crane [61]

b) Liftra LT1500 climbing crane [53]

c) Floating gin pole [63]



e) Gin pole [59]

d) Ericson skycrane [62]

Figure A-18 Tower erection methods - addtional crane

Stationary crane (current solution)

As mentioned, this is the current solution. However, now only a hydraulic crane can be used, see Figure A-18 a). According to the investigation done in Annex A.3 the height of the towers is then limited to 90-meter, requirement R9.1. The result is that this solution can only be used when a hammerhead gantry will be used. As hammerhead gantries will not be considered, see Annex A.6.3.8.

This solution <u>will not</u> be considered.

Climbing crane

An example of a climbing crane erecting the tower is in the wind turbine industry. Liftra designed the LT1500 which is a self-climbing crane that is next to the tower, see Figure A-18 b). The crane has claws that clamp around the wind turbine tower. Now the crane can lift the next tower section on top. After this, with its own hook the crane lifts its hoist block to the top of the tower section. With the hoist block the crane can pull itself to the new height. This process is repeated until the desired height is reached.

The principle of a crane climbing along and assembling the tower might be useful for a gantry. The MLS mast sections are different from wind turbine sections. Meaning that a redesign is necessary to make sure the crane can clamp to the MLS sections. It would be beneficial if one crane is able to erect both towers.

This solution <u>will</u> be considered.

Gin pole

There are two assembly procedures known to work with a gin pole. One is with a floating gin pole, see Figure A-18 c). This technique is used in the erection of powerline towers. A floating mast is in the middle of the tower. The mast is stabilized by ropes connected to the towers. Each time the tower elements are assembled the floating mast is pulled up and reconnected to the just built tower section to perform the new lift. The capacity of a floating gin pole is limited to around 120 kN [31]. Additionally, because the pole is held together with ropes, there are stability issues leading to a safety factor of 3.0. Meaning that the mast sections cannot be lifted as a whole, this does not comply with requirement R3.1.

Another way of working with gin poles is by using them as a lever arm. The whole tower is assembled horizontally after which it is pulled straight, Figure A-18 d). However, because of the height and the weight of the tower the gin pole itself would have to be constructed first. Already asking for a structure almost as tall as the gantry. This would introduce a new component in the paradoxical problem.

Both solutions will not be considered

Drone/ helicopter

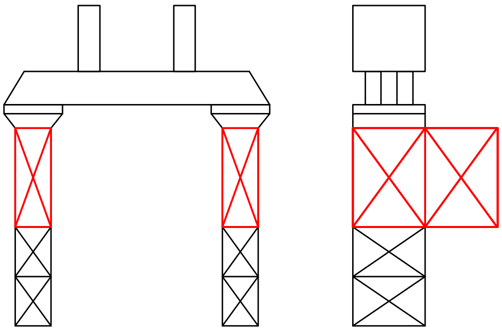
In principle drones and helicopters are able to lift objects. This technique is used in the installation of lattice powerline towers. There the Erickson S-64 assembled all tower segments [32]. Currently the strongest helicopter, the Mi-26 Halo helicopter, is able to lift items up to 20 ton [33]. Drones have a smaller capacity. However, they can be combined to increase the capacity. Currently, the strongest drone can lift 227 kg, 0.227 ton [34].

Considering the weight of a mast section including rigging and hook block, requirement R3.1, this does not come to the required capacity. When combining the strongest drones at least 51.2 / 0.227 = 226 drones are needed.

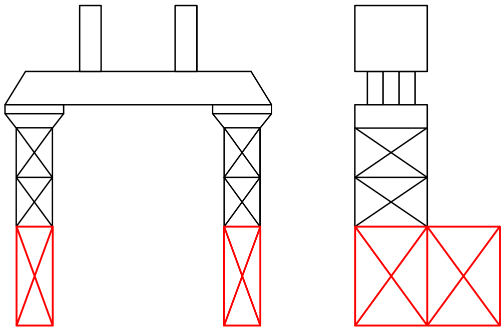
Both solutions will not be considered.

A.6.1.2. Climbing frame

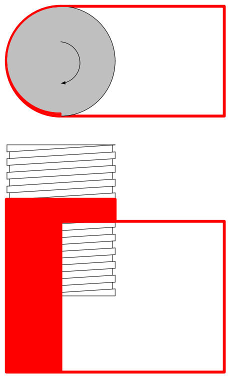
Four types of climbing frames are considered: top, bottom, twisting, and corkscrew, see Figure A-19.



a) Climbing frame top



b) Climbing frame bottom



c) Twisting frame

d) Corkscrew [60]

Figure A-19 Tower erection - climbing frames

Top

Tower cranes need to erect in a dense city or hundreds of meters high above the ground. Therefore, they need a self-erecting system. The crane has a climbing frame around its tower that can push itself, see Figure A-20. This creates space for a new tower segment. Tower cranes usually have the frame at the top of the tower, right under the lifting beam. They lift their new segment themselves and place it at the space for the new segment [35].

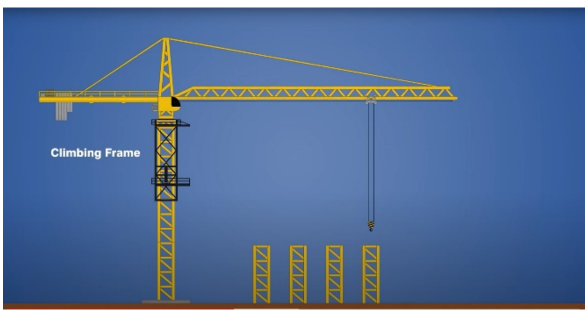


Figure A-20 Self erecting tower crane. Screenshot taken from 'How tower cranes build themselves' [35]

Although the MLS mast sections are bigger than tower crane mast sections, this solution has potential. For a build-up sequence see the end of this Paragraph.

This solution will be considered.

Bottom

The climbing frame can also stay at the base of the tower. The whole gantry is pushed upward to make space for new segments. New segments can be placed on ground level and do not need to be lifted in order to be placed in the tower. Mammoet uses such a system in their FOCUS crane, see Figure A-21. The principle is similar to the climbing frame placed at the top, only the location of the frame is different [36]. This requires more stability from the frame as it must stabilize the whole tower.

Figure A-21 FOCUS crane by Mammoet [19]

Similar to the FOCUS crane is the Frameup project in Sweden, see Figure A-22. In order to erect a student housing building, a steel frame was used to jack-up modular living units. Eight Modules of 2.778 x 3.832 x 3.203 meters, weighing 4.2 tons were stacked to form a six-story high building [37].

Figure A-22 Snapshot of the assembly of one 3D module [38]

For a build-up sequence see the end of this Paragraph.

This solution will be considered.

Twisting
When looking at bolts and screws, one observes that a translation occurs due to a twisting motion. This is because of the threat. When the climbing frame is designed as a bolt and the mast sections are designed as nuts, one can erect the tower by a twisting motion, see Figure A-19 c). The whole procedure can be found at the end of this Paragraph.

The mast sections will need a redesign to comply with this erection system. It does not comply with requirement R3.

This solution <u>will not</u> be considered.

Corkscrew
Looking at the corkscrew in Figure A-19 d). By means of two lever arms the cork in a bottle of wine is pulled out. When the climbing frame is equipped with the lever arms and the mast sections are designed with the necessary indents the mast sections can be pushed up.

The indents on the mast section and the arms on the climbing frame need to be big to be able to lift the eleven-meter-long mast section in one stroke. Therefore, this cannot be considered a good solution. The principle has similarities with gears, this is considered to be part of the top or bottom climbing frame. This is more detailed, so it will be elaborated when a more detailed design will be relevant.

This solution <u>will not</u> be considered.

A.6.1.3. Smart use of structure
The towers can be erected with smart use of the elements that are already present. Doing this can potentially save on new elements needed specifically for the assembly.

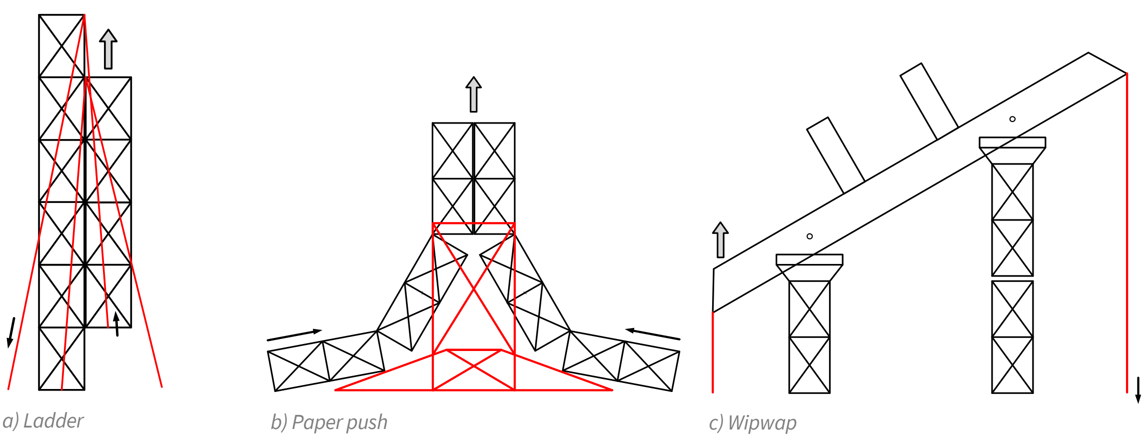


Figure A-23 Tower erection – Smart use of structure

Ladder
Extension ladders are an efficient way to extend the height while standing on the ground. The ladder exists of two (or more) ladders. By pulling a rope that rolls over a pulley at the top of the lower ladder to the bottom of the upper ladder, the upper ladder goes up. The upper ladder has a locking system that secures the ladder at each rung.

Applying this to the towers would ask for the towers to be split in two, see Figure A-23 a). One of the sections is pulled up by means of the principle described above. Once it is up a new section is placed underneath. Now, the other section is pulled up. When a new section is placed underneath this section the process can be repeated. A step-by-step sequence can be seen at the end of this Paragraph.

This method requires two 4x4 masts and will cause stability problems, see requirement R3.

This solution will not be considered.

Paper push
When one pushes the sides of a piece of paper towards the middle, one notices that the middle goes up. This principle was the inspiration for this concept. By applying a horizontal force one can push something up, see Figure A-23 b). A step-by-step sequence can be seen at the end of this Paragraph.

This concept requires the towers to be split into two. This means that at four locations new sections must be given, requiring two cranes instead of one. Both sides of one tower need to be pushed in a controlled manner otherwise the tower will be skewed. The mast sections cannot instantaneously turn from perfectly horizontal to an inclined position without a vertical force. Therefore, they require some guidance by means of a slope. All this requires a guidance/ stability frame.

So, this method requires 2 4x4 masts, needs a large guidance/ stability frame, more than one crane is needed, and the operational area is large. This is not according to requirements R3, R9, andR8.

This solution will not be considered.

Wipwap
By means of a lever arm one can also get things into the air. Here the gantry beam is used as a lever arm and the rotation point is at the intersection between the gantry beam and the tower, see Figure A-23 c). By pulling on one side of the beam the other side rises. Now, a new mast section can be placed underneath. After the sections are connected, one needs to pull on the other side to get the opposite side at the same height. When this side also has a mast section underneath the process can be repeated. For a build-up sequence see the end of this Paragraph.

A point of attention; due to the inclination the horizontal distance between the towers decreases. This means that the gantry will 'walk' away. This can be solved by using a sequence of one section on one side, two sections at the other side and one section on one side. Still, an extra foundation point is needed.

Because of engineering enthusiasm, this solution will be investigated.

A.6.1.4. Assembly sequences

Climbing frame top

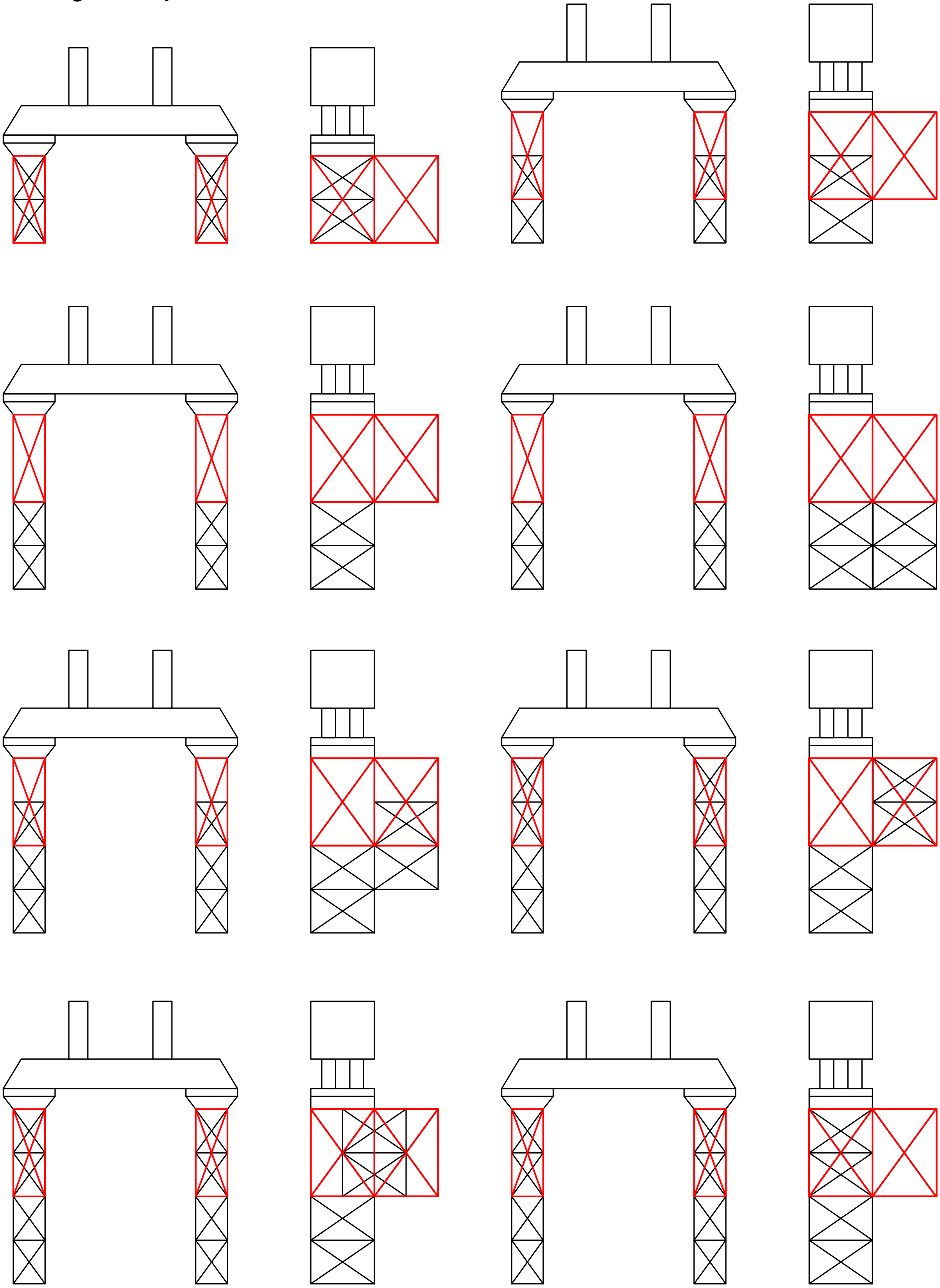


Figure A-24 Climbing frame - top sequence

Climbing frame bottom

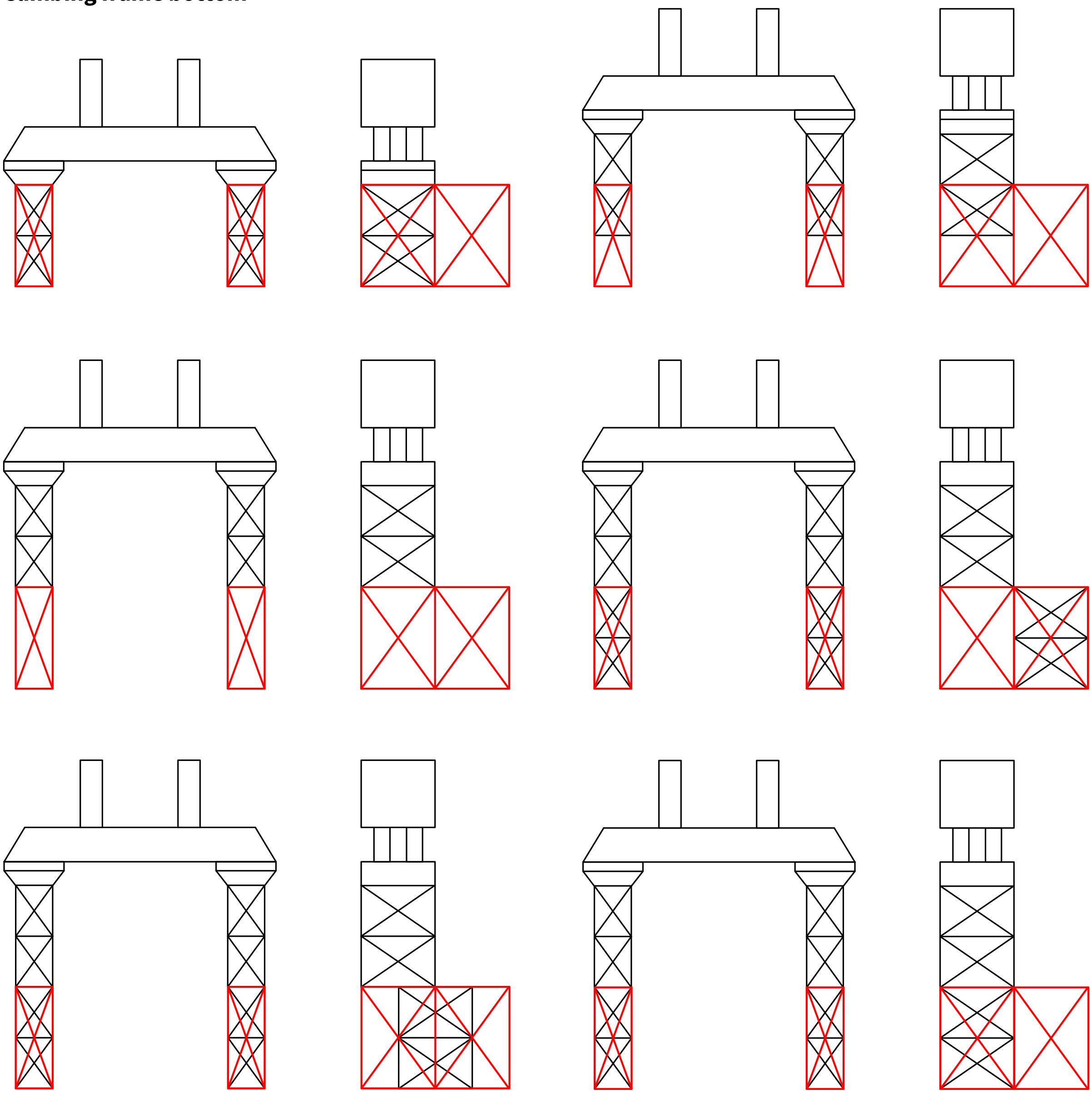


Figure A-25 Climbing frame - bottom sequence

Twisting frame

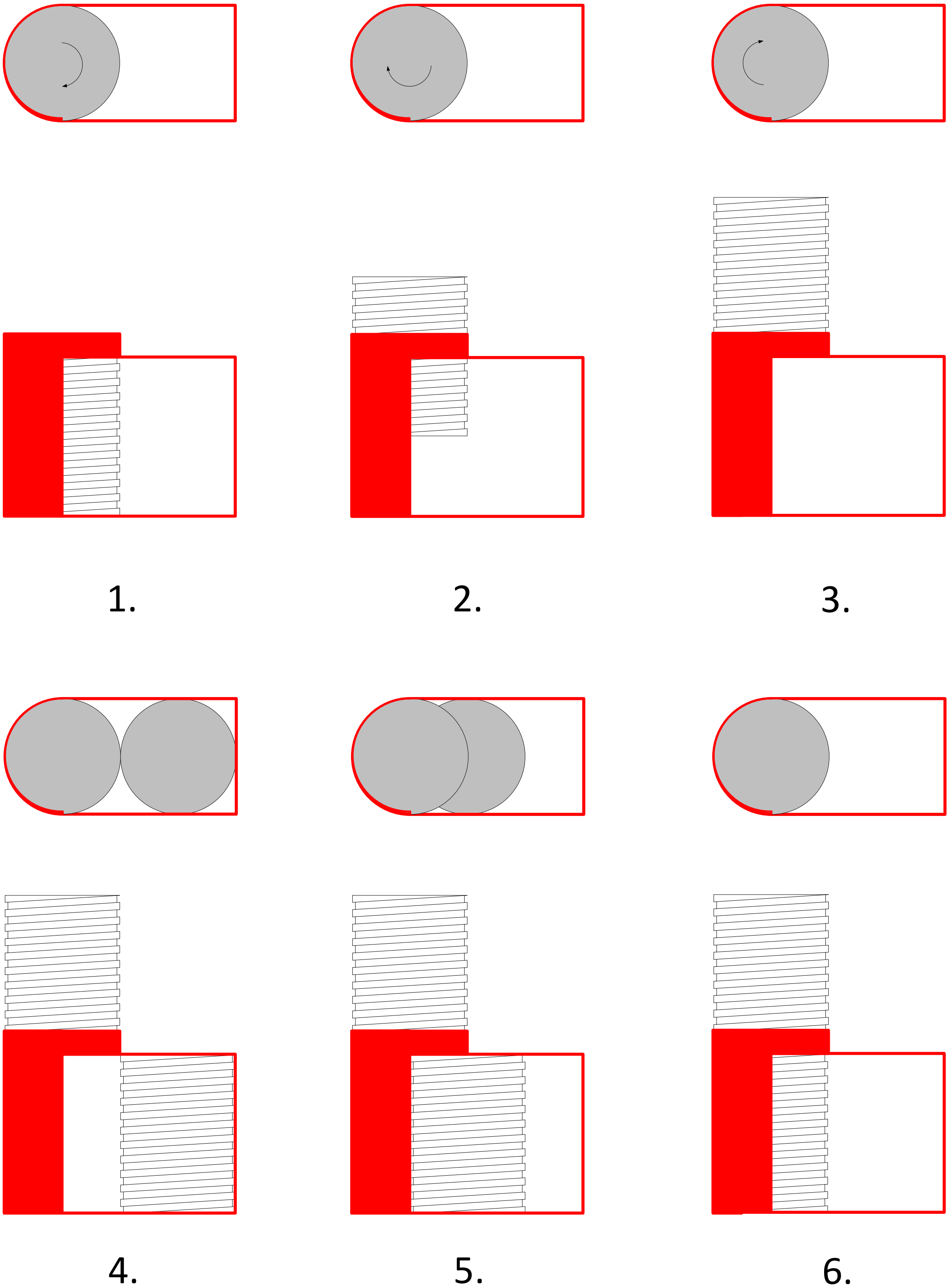


Figure A-26 Climbing frame - twisting sequence

Ladder

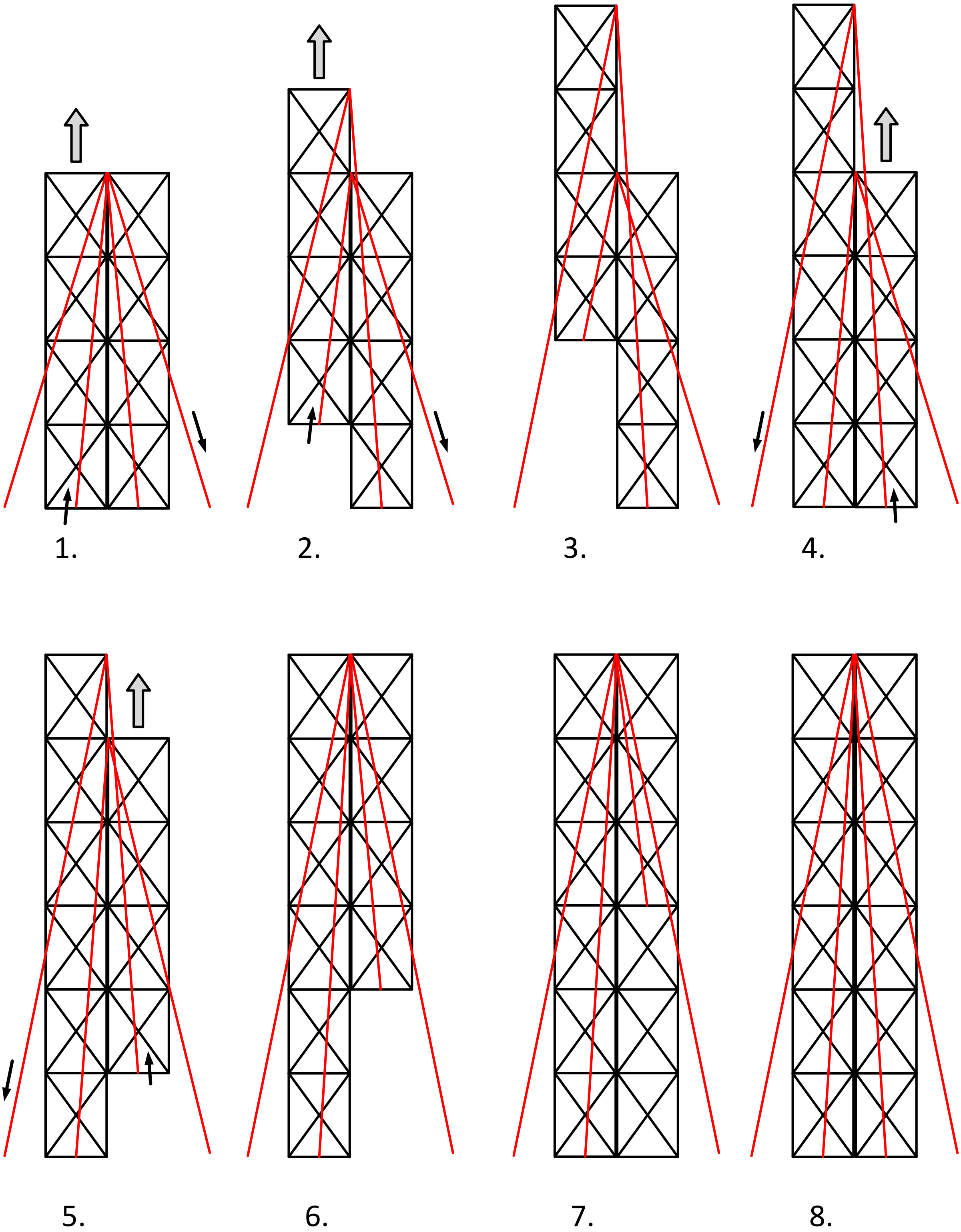


Figure A-27 Smart use of structure - ladder sequence

Paper push

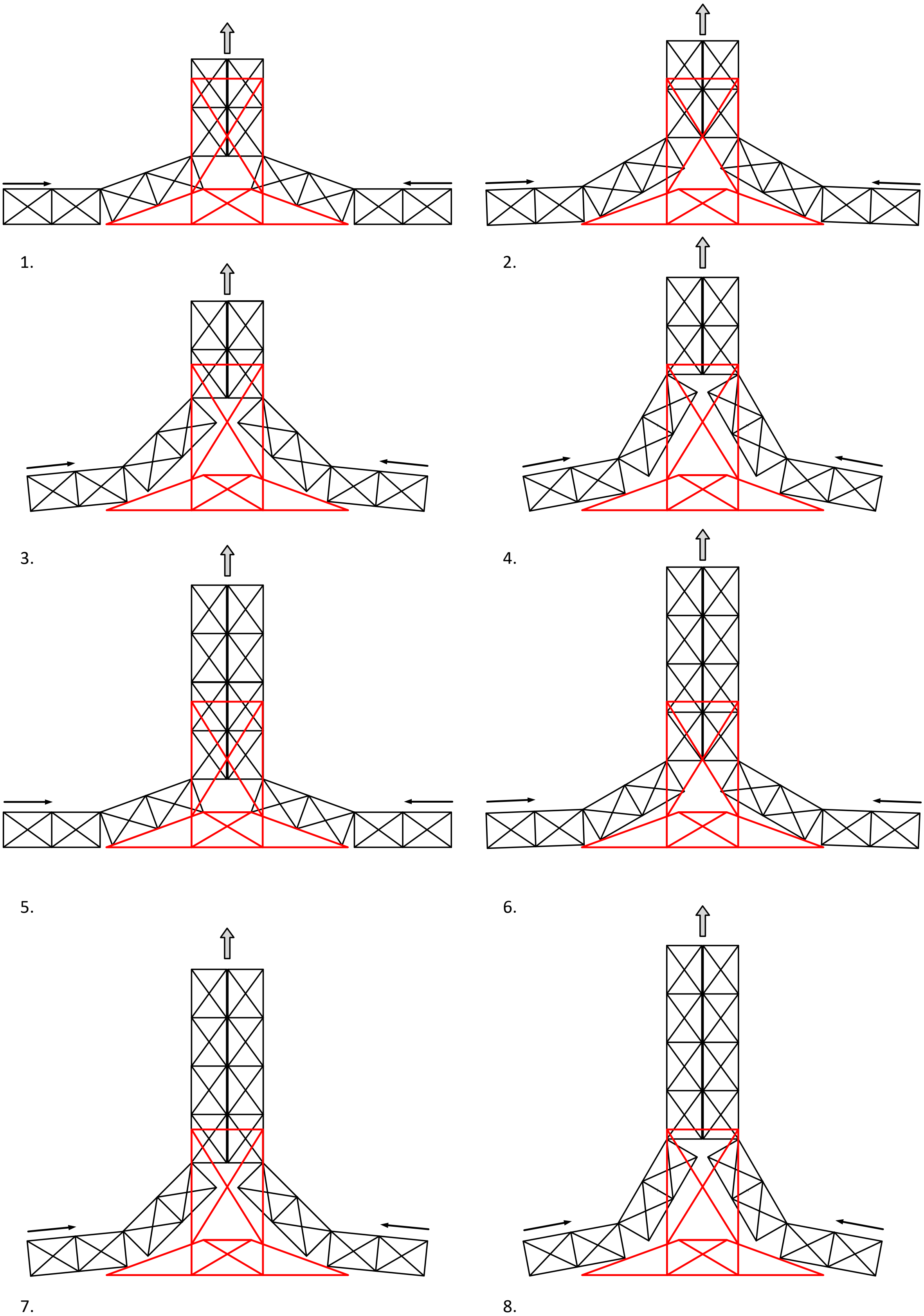


Figure A-28 Smart use of structure - paper push sequence

Wipwap

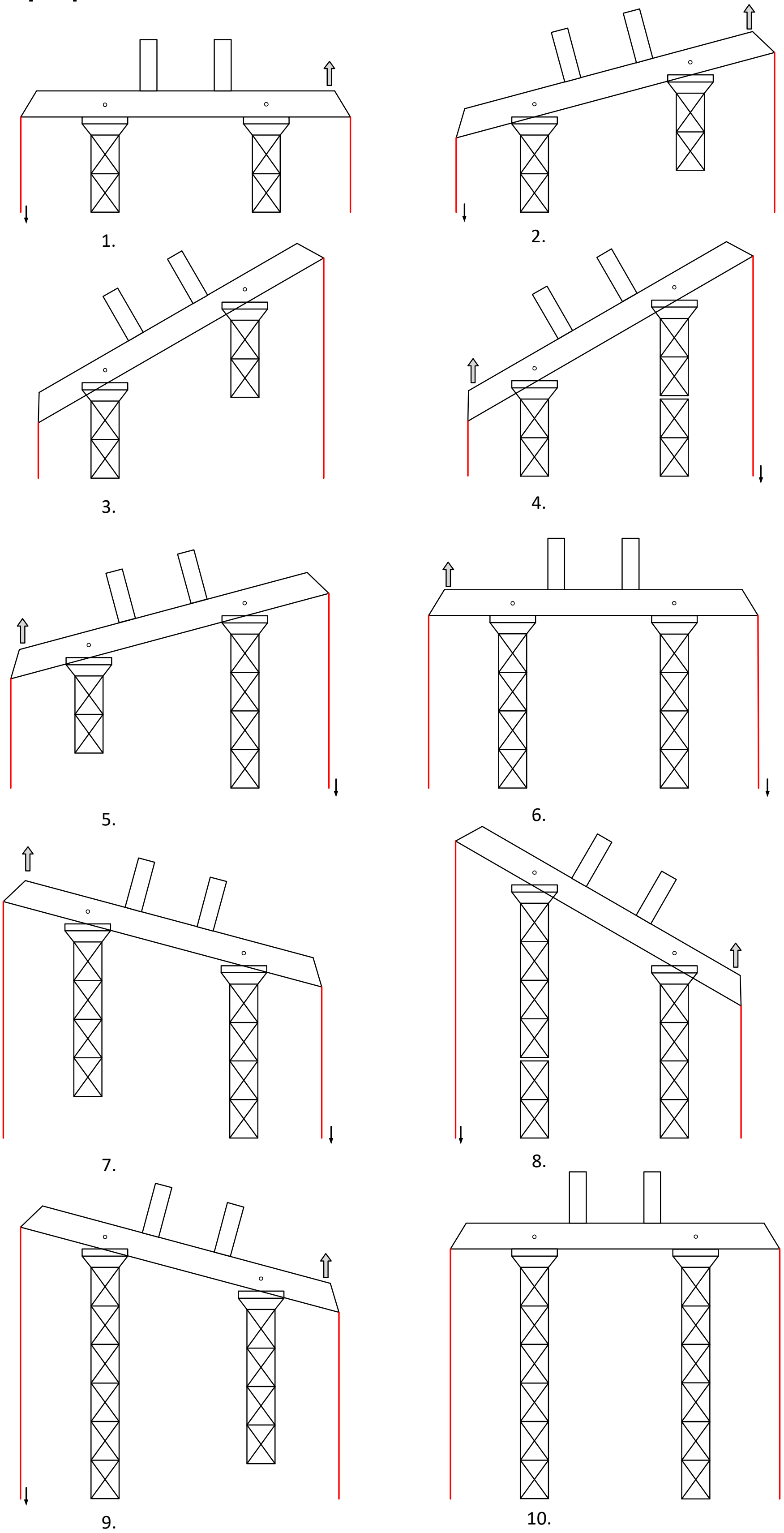


Figure A-29 Smart use of structure - wipwap sequence

A.6.2. Stage II Raise upper structure

A.6.2.1. Additional crane

From the five solutions that were presented in Annex A.6.1.1 three solutions are applicable for stage II, see Figure A-30.

a) Stationary crane [61]

b) Liftra LT1500 climbing crane [53]

c) Ericson skycrane [62]

Figure A-30 Raise upper structure – additional crane

Stationary crane

As mentioned before, the use of a stationary crane is only viable when combined with a hammerhead gantry. A hammerhead gantry must be higher than the trunnions. This is higher than 60 meters, so according to requirement R9.2 this means that the upper structure is limited to a weight of +/- 50 tons. This is not in line with requirement R4.

This solution <u>will not</u> be considered.

Crane on top

Another option is to put a crane on top of the tower. This crane lifts the gantry beam and the lifting devices to the desired location. The crane must not stand in the way of the upper structure, meaning that it needs to slide the upper structure under itself, or it needs to be outside of the tower. Besides that, the tower should guarantee the stability. If this option is chosen it would be wise to combine it with a climbing crane.

This solution <u>will</u> be considered.

Drone/ helicopter

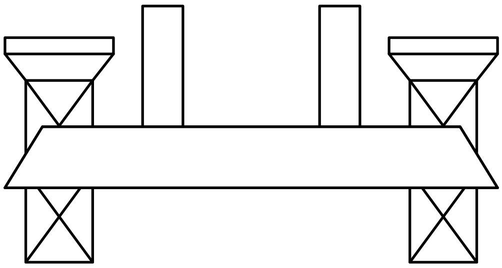
The heaviest part of the upper structure is heavier than one mast section. Therefore, also for this stage this solution lacks capacity.

This solution <u>will not</u> be considered.

A.6.2.2. Extra system along tower

When the towers are erected, they can be used to raise the upper structure. To do so, an extra system along the tower is needed. Here two options are considered, see Figure A-31.

a) Sarens Climbing Tower [39]



b) Extra system along the tower, excluding lift

Figure A-31 Raise upper structure - extra system along tower

Including lift

A competitor of Mammoet is Sarens. Their gantry beam travels upward while the vessel is hanging underneath it, see Figure A-31 a). In other words, they have a climbing upper structure that erects the vessel. Each tower can be equipped with one, two, three or four climbing bars, each having a climbing capacity of 450 tons. Depending on the tower configuration their gantries can lift up to 3600 ton [39].

As can be seen in Figure A-31 a) an extra upper structure is needed to keep the towers from falling inward. Getting that upper structure in place will require a new solution.

This solution will not be considered.

Excluding lift

Sarens' principle can also be used without lifting the vessel. By means of climbing bars the upper structure can climb to the top of the towers, see Figure A-31 b).

This solution will be considered.

A.6.2.3. N/A

The last solution would be that it is not needed to raise the upper structure. Then, the upper structure is already on top of the towers and pushed (along with the towers) to the desired height, see Figure A-32. The advantage would be that the whole upper structure can be assembled on ground level instead of 150 meters in the air.

This solution will be considered.

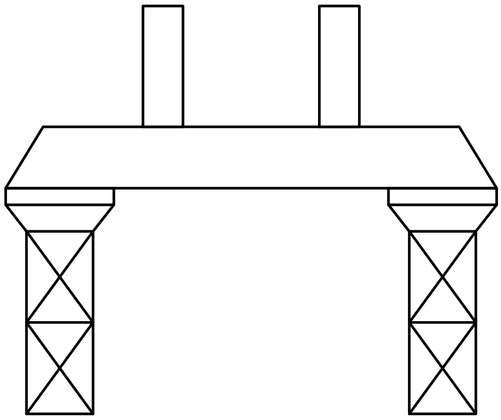


Figure A-32 Raise upper structure - N/A

A.6.3. Stage III Upper structure out of the way

A.6.3.1. Additional crane

The same three solutions from stage II are possible for using an additional crane in stage III, see A.6.2.1.

a) Stationary crane [61]

b) Liftra LT1500 climbing crane [53]

c) Ericson skycrane [62]

Figure A-33 Raise upper structure - N/A

Stationary crane

Again, the use of a stationary crane is only viable when combined with a hammerhead gantry. That means that it cannot remove the upper structure if it is over the vessel, see requirement R9.1.

Crane on top

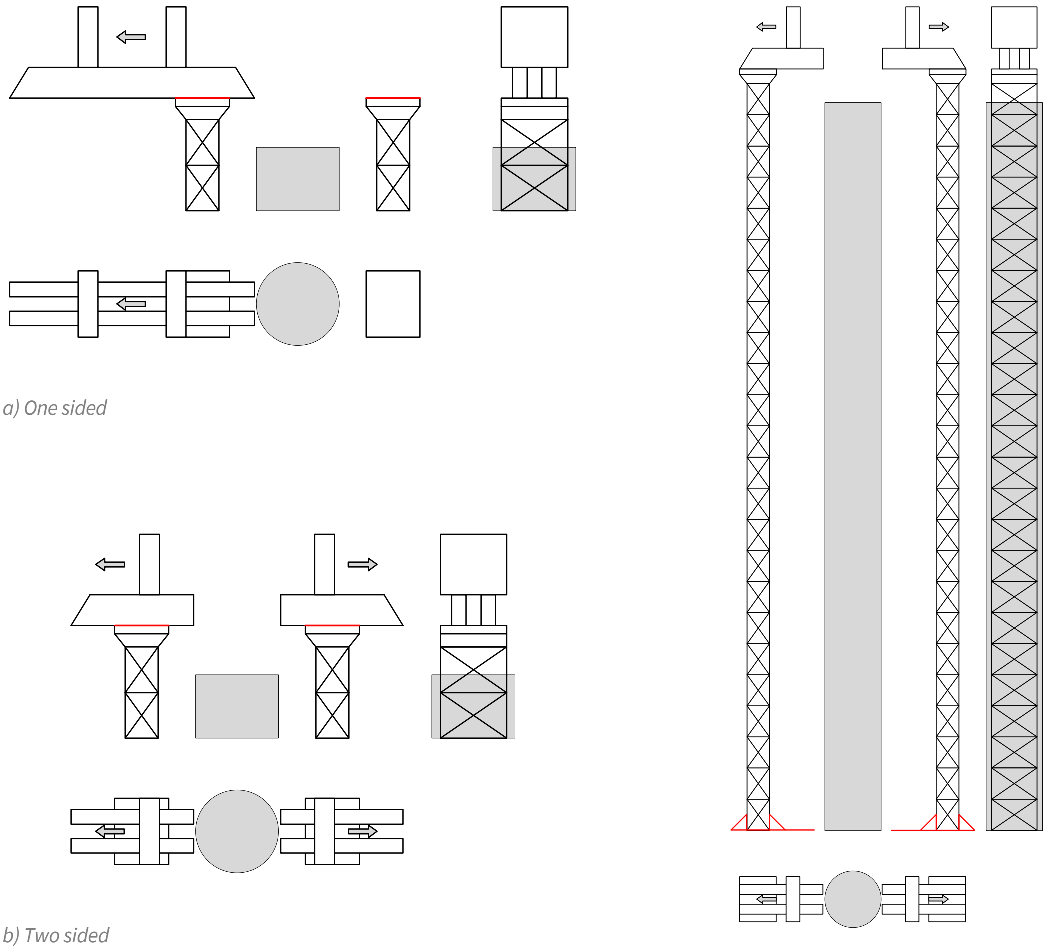
When a crane is on top of the tower. This crane can remove the upper structure. The crane must not stand in the way of the upper structure, meaning that it needs to slide the upper structure under itself, or it needs to be outside of the tower. Besides that, the tower should guarantee the stability. If this option is chosen it would be wise to combine it with a climbing crane.

Drone/ helicopter

The heaviest part of the upper structure is heavier than one mast section. Therefore, also for this function this option cannot be considered possible.

A.6.3.2. Translation X

In X direction three variants are possible: one sided and two sided, including and excluding the tower. For a build-up sequence see the end of this Paragraph.



a) One sided

b) Two sided

c) Two sided + towers

Figure A-34 Upper structure out of the way - translation X

One sided

The upper structure can be slid in the X direction in order to come free from the vessel, see Figure A-34 a). The position in the Figure is the critical position. Coming is the check for requirement R3.5 for compressive force and bending moment on the tower. It satisfies the calculation, however it does not comply with serviceability limit state criteria. Because it is not stable while disassembling.

The weight of the gantry beam and the lifting devices is roughly 3200 kN. This is the N_{Ek}. Assuming that the C.o.G. of the gantry beam and the lifting devices is in the middle of the gantry beam, means that the $M_{z,tower,Ek}$ is:

$$M_{z,tower,Ek} = L_{C.o.G.-tower} * N_{Ek}$$

The length between the C.o.G. and the tower is:

$$L_{C.o.G.-tower} = \frac{L_{gb}}{2} - \left(\frac{L_{gb} - 2 * \frac{1}{2} * b_{tower} - \emptyset_{vessel}}{2} + \frac{b_{tower}}{2} \right)$$

$$L_{C.o.G.-tower} = \frac{23}{2} - \left(\frac{23 - 2 * \frac{1}{2} * 4 - 10}{2} + \frac{4}{2} \right) = 5\ m$$

Resulting in a bending moment of:

$$M_{z,tower,Ek} = 3200 * 5 = 16{,}000\ kNm$$

Assuming no moment around the y axis:

$$M_{y,tower,Ek} = 0\ kNm$$

Results in requirement R3.5 to be:

$$U.C. = 0.19$$
$$U.C. = 0.35$$

Meaning that it checks. However, the system needs to be checked for stiffness. From structural mechanics Figure A-35 applies for the tower.

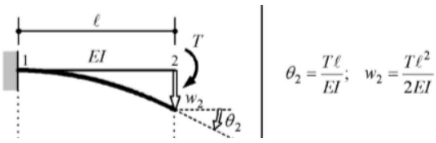


Figure A-35 Forget-me-not strcutural mechanics [40]

$$w_{i,tower,Ek} = \frac{M_{i,tower,Ek} * {L_{tower}}^2}{2EI_{i,tower}} \tag{25}$$

$$w_{z,tower,Ek} = \frac{16{,}000 * 132^2}{2 * 210{,}000 * 7.1 * 10^{11} * 10^{-9}} = 0.934\ m$$

While the maximum allowable deflection is:

$$w_{tower,Rk} = \frac{1}{200} * L_{tower} \tag{26}$$

$$w_{tower,Rk} = \frac{1}{200} * 132 = 0.660\ m$$

$$U.C. = \frac{w_{tower,Ek}}{w_{tower,Rk}} = \frac{0.934}{0.660} = 1.42$$

Meaning that it does not comply.

This solution <u>will not</u> be considered.

Two sided

When the upper structure is split into two, both sides can slide to the opposite direction, see Figure A-34 b). This is more stable than the previous option. However, splitting the gantry beam is not wanted, see requirement R4.4.

This solution will not be considered.

Two sided + towers

This principle can also be done but instead of sliding the upper structure, both towers are sliding, see Figure A-34 c). Again, splitting the gantry beam is not wanted, see requirement R4.4.

This solution will not be considered.

Out of the way sequences

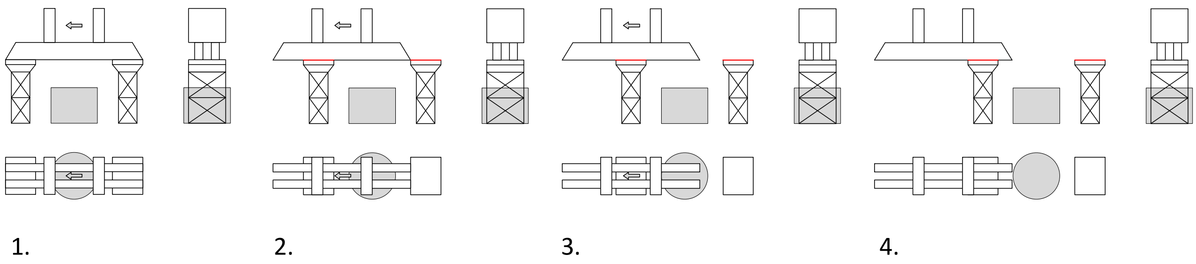


Figure A-36 Translation X - one sided sequence

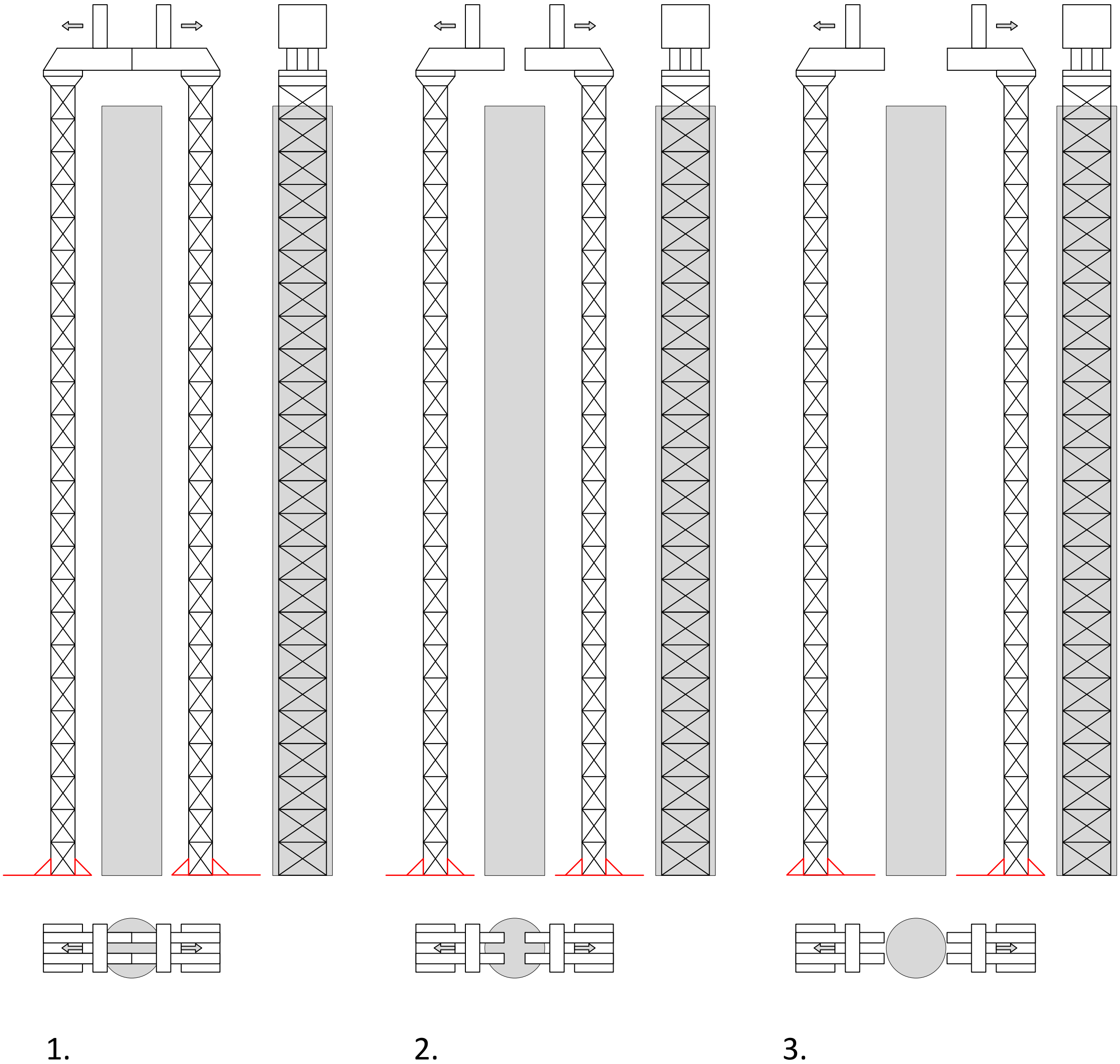


Figure A-37 Translation X - one sided + tower sequence

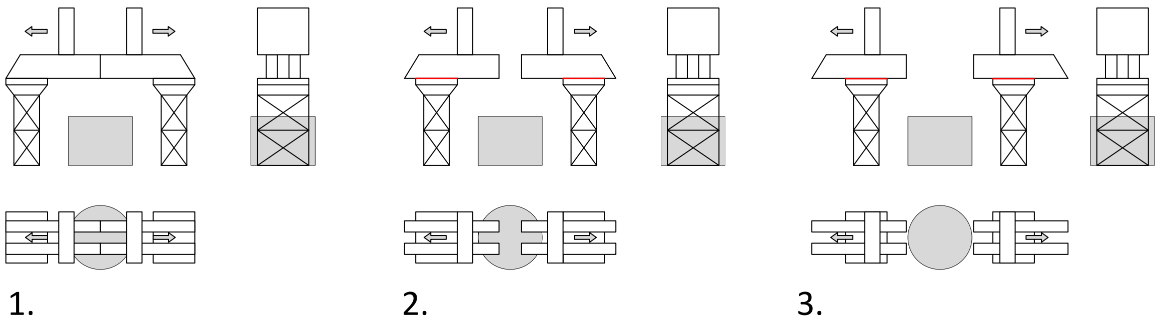
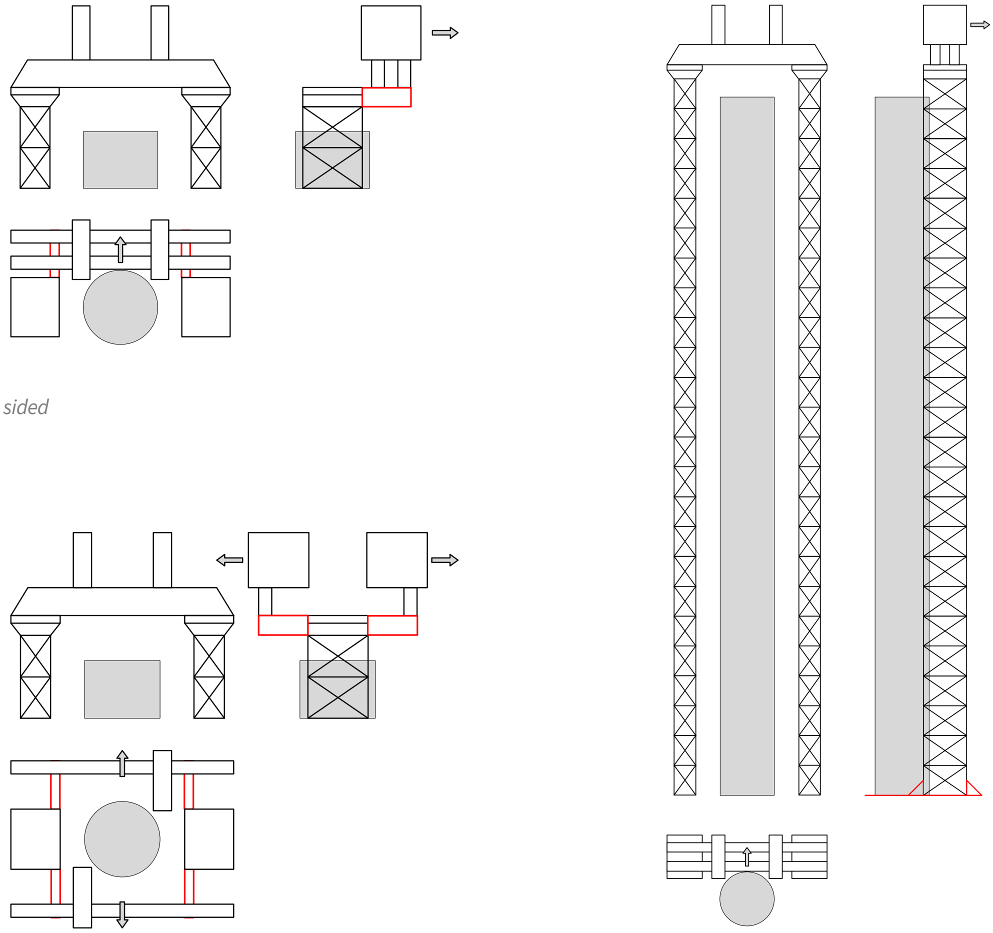


Figure A-38 Translation X – two sided sequence

A.6.3.3. Translation Y

The upper structure can also slide in the Y direction. Here the options; one sided, including and excluding the towers and two sided are possible. For a build-up sequence see the end of this Paragraph.



a) One sided

c) Two sided

b) Two sided + towers

Figure A-39 Upper structure out of the way - translation Y

One sided

By means of two additional beams the upper structure can slide away from the vessel, see Figure A-39 a). It is calculated that this solution satisfies both requirement R3.5, and the stiffness requirement. The latter under the condition that the guidance beam (in red) is stiff enough.

Half the weight of the upper structure acts as an eccentric load on the tower. The eccentricity is in the direction of the strong axis of the tower, so around the local y direction. The distance between the centerline of the upper structure and the centerline of the tower is half the width of the vessel plus half the length of the strand jack beam:

$$L_{tower-u.s.} = \frac{\emptyset_{vessel} - L_{strand\ jack\ beam}}{2}$$

$$L_{tower-u.s.} = \frac{10-8}{2} = 9\ m$$

Resulting in a compressive force and bending moment of:

$$N_{Ek} = \frac{4500}{2} = 2250\ kN$$

$$M_{y,tower,Ek} = N_{Ek} * L_{tower-u.s.} = 2250 * 9 = 20{,}250\ kNm$$

The bending moment in the other direction is assumed to be zero. Making the Unity Checks from requirement R3.5:

$$U.C. = 0.11$$

$$U.C. = 0.21$$

Meaning that it checks. Here the stiffness will be checked again, Figure A-35 will be used again.

$$w_{y,tower,Ek} = \frac{20{,}250 * 132^2}{2 * 210{,}000 * 2.8 * 10^{12} * 10^{-9}} = 0.300\ m$$

$$U.C. = \frac{w_{y,tower,Ek}}{w_{tower,Rk}} = \frac{0.300}{0.660} = 0.45$$

This solution <u>will</u> be considered.

One sided + towers

Another option is to slide the whole gantry in Y direction, see Figure A-39 b). Now the horizontal force, coming from the acceleration, must be checked.

This solution <u>will</u> be considered.

Two sided

Being more stable and having no acceleration problems can be done by splitting the upper structure in two and slide each part to the opposite side, see Figure A-39 c).

This solution <u>will</u> be considered.

Out of the way sequences

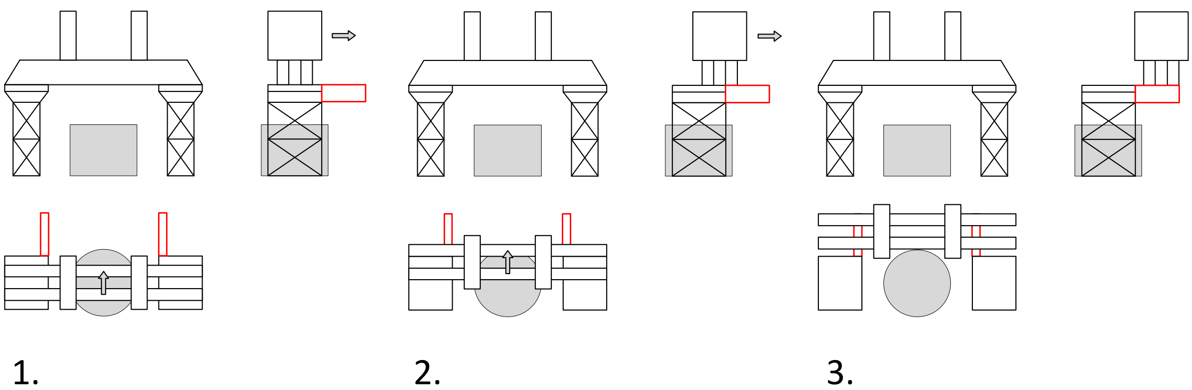


Figure A-40 Translation Y - one sided sequence

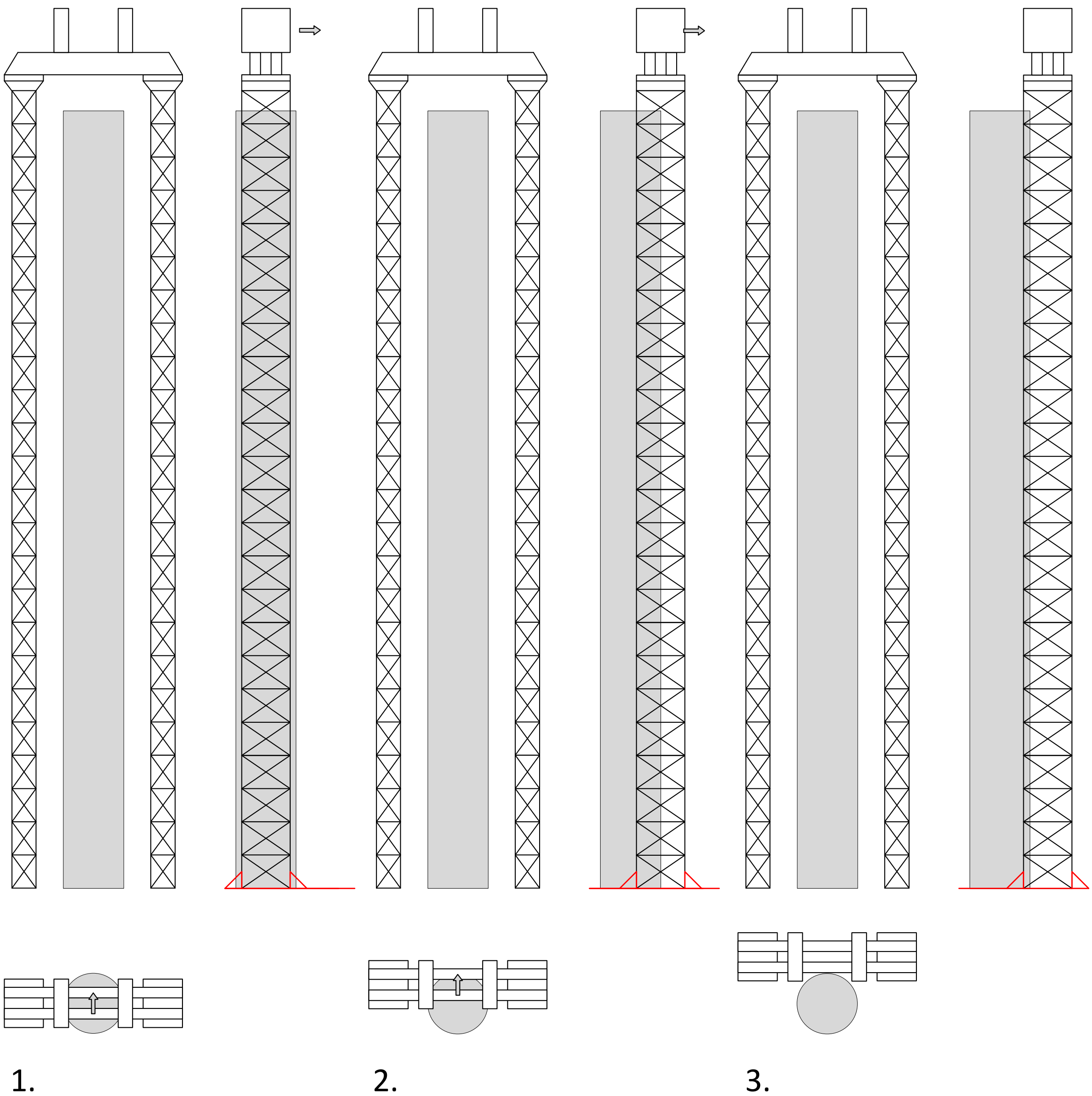


Figure A-41 Translation Y - one sided + tower sequence

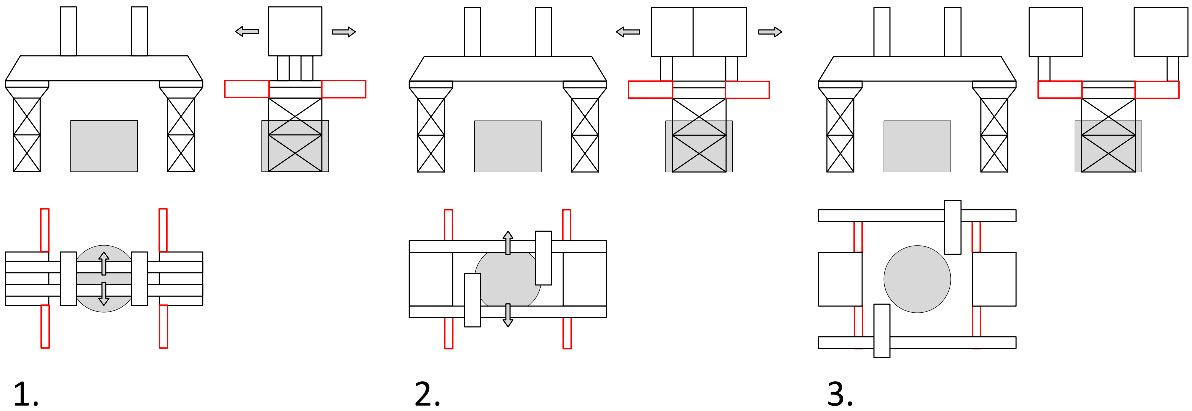
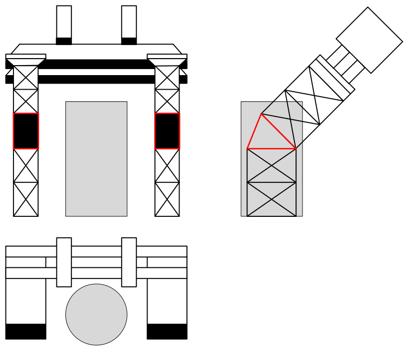


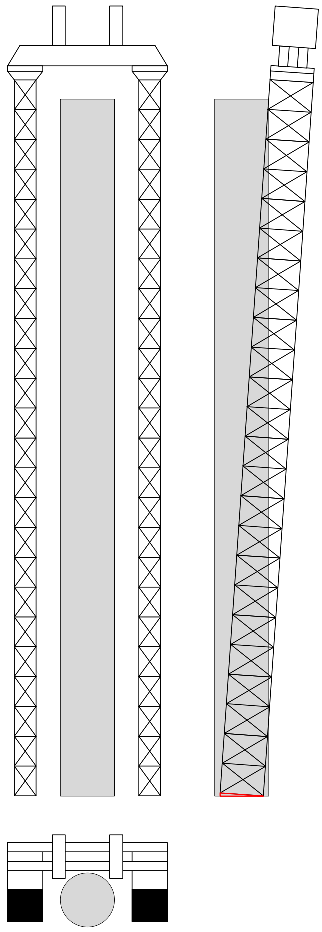
Figure A-42 Translation Y - two sided sequence

A.6.3.4. Rotation X

Besides translations, rotations can also solve the problem. Here the possibilities for rotations around the X axis will be considered. For a build-up sequence see the end of this Paragraph.



a) Top rotation



b) Whole tower

Figure A-43 Upper sturcture out of the way - rotation X

Top rotation

Rotating the upper structure with one mast section, see Figure A-43 a). Doing this results in high shear forces in the chords and high compressive forces in the braces which is not what they are designed for. Also, all dywidag connections will be loaded in shear.

This solution <u>will not</u> be considered.

Whole tower rotation

Also, the whole tower can be rotated, see Figure A-43 b). With a small rotation the upper structure comes free from the vessel. However, each time a mast section is removed from the bottom the upper structure will come closer to the vessel, so the rotation must be increased. Besides that, removing a mast section while maintaining the skew is challenging.

This solution <u>will not</u> be considered.

Out of the way sequences

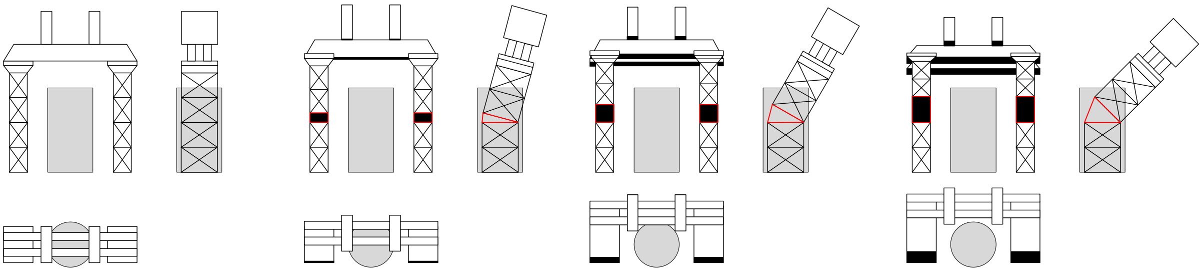


Figure A-44 Rotation X - top rotation sequence

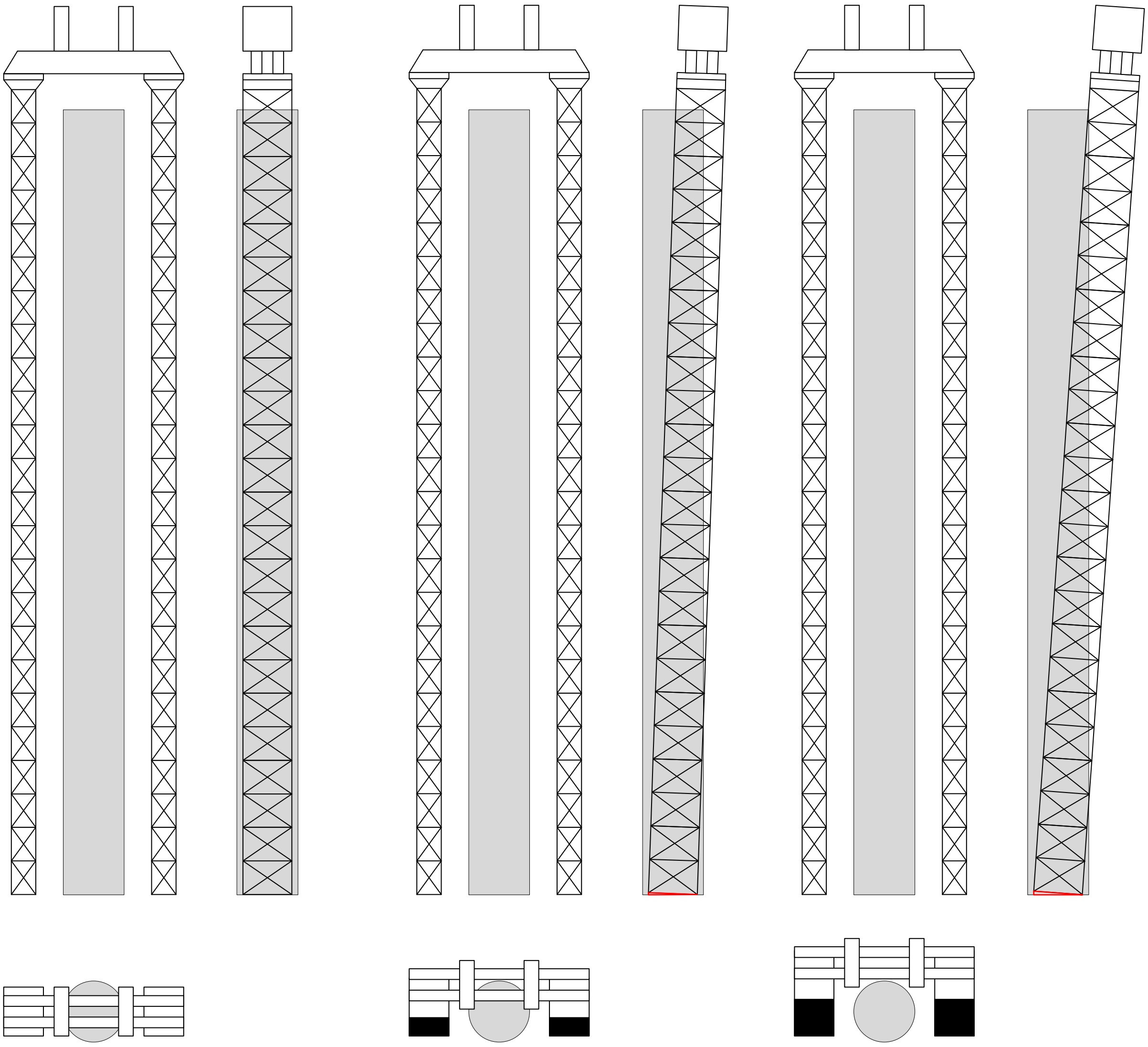
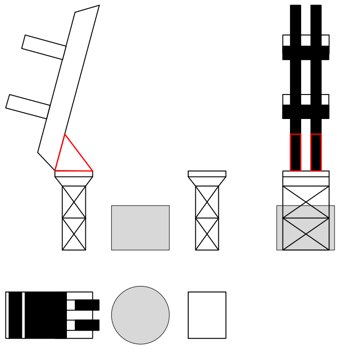


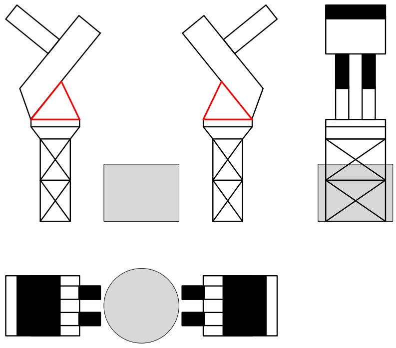
Figure A-45 Rotation X - whole towers sequence

A.6.3.5. Rotation Y

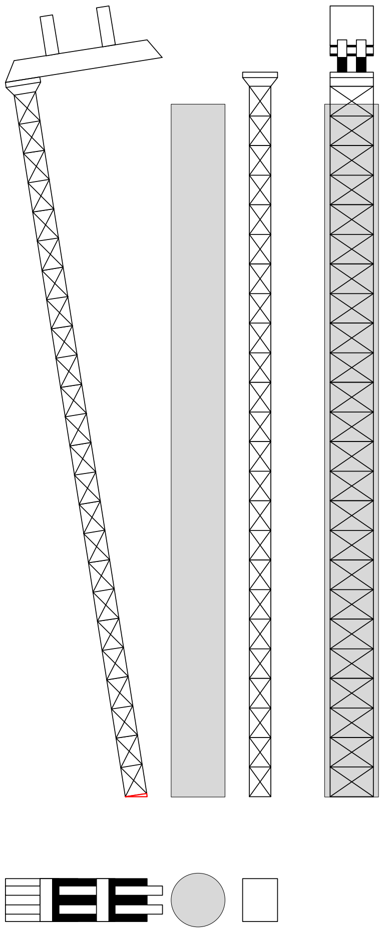
Also, rotations around the Y axis are possible. For a build-up sequence see the end of this Paragraph.



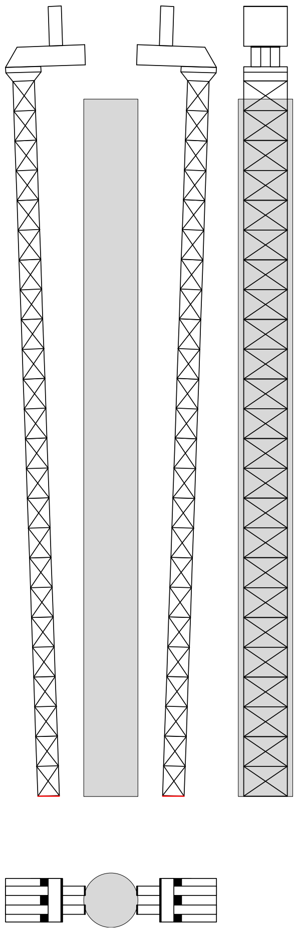
a) One sided



c) Two sided



b) One sided + tower



d) Two sided + towers

Figure A-46 Upper structure out of the way - rotation Y

One sided

This variant looks like a draw bridge, see Figure A-46 a). When the upper structure comes free from the tower stability becomes a problem, see Annex A.6.3.2.

This solution <u>will not</u> be considered.

One sided + tower

Again, also the whole tower can rotate, see Figure A-46 b). However, this will cause both stability problems and the same disassembly problem as the whole tower variant, see Annex A.6.3.2.

This solution will not be considered.

Two sided

The stability problem arising from the one-sided variant might be solved by splitting the gantry beam and rotate both sides, see Figure A-46 c). However, splitting the gantry beam is not wanted, see requirement R4.4.

This solution will not be considered.

Two sided + towers

Also not considered because of the splitting of the gantry beam and for causing disassembly problems is the two-sided variant with rotating towers, see Figure A-46 d).

This solution will not be considered.

Out of the way sequences

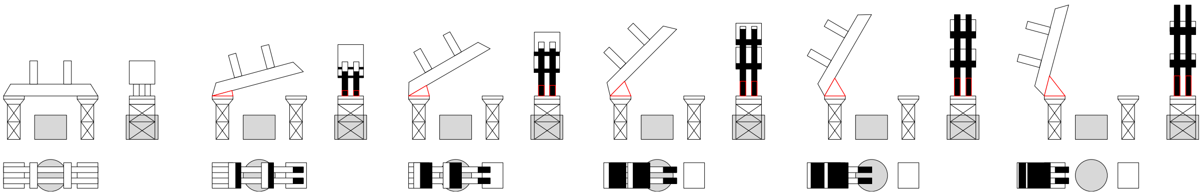


Figure A-47 Rotation Y - one sided sequence

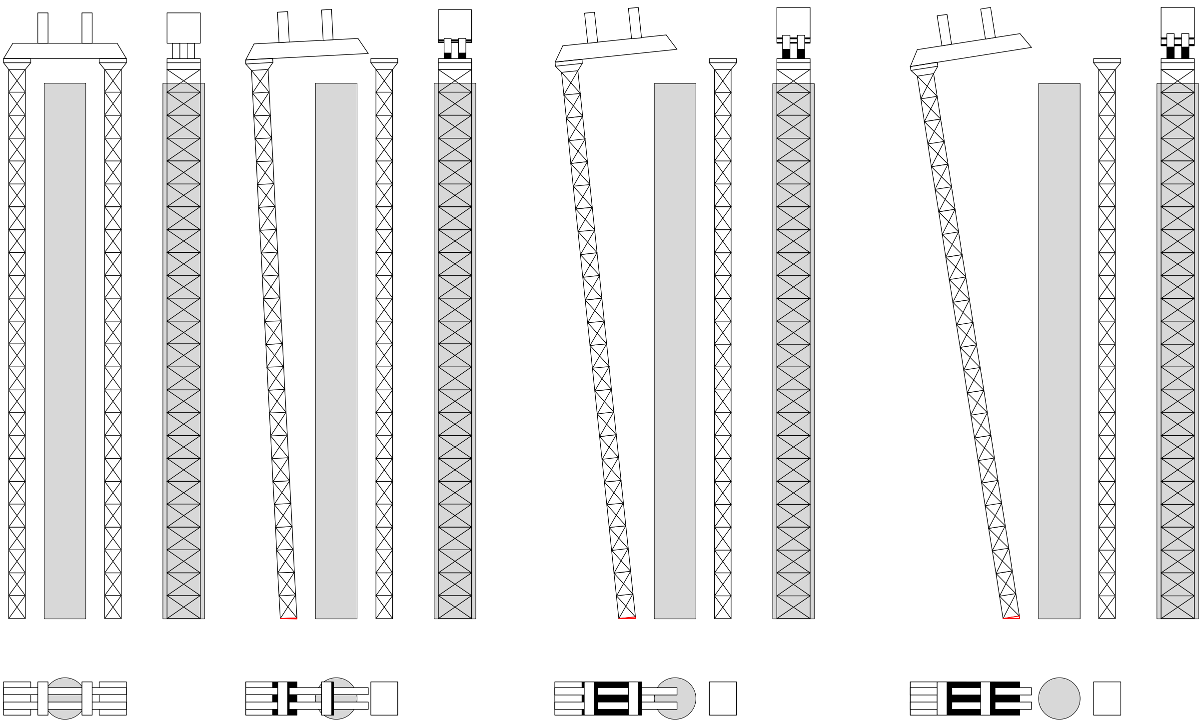


Figure A-48 Rotation Y - one sided + tower sequence

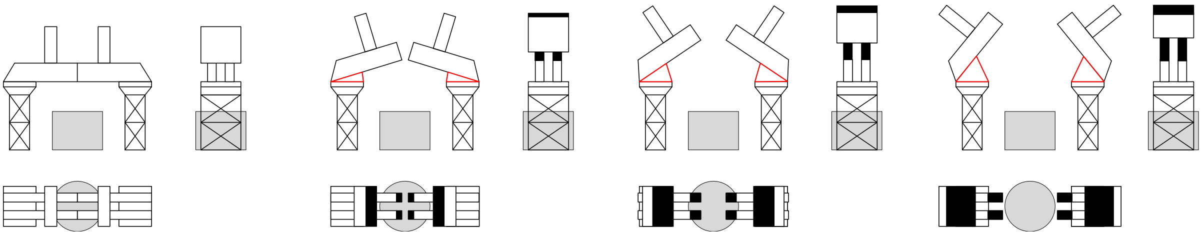


Figure A-49 Rotation Y - two sided sequence

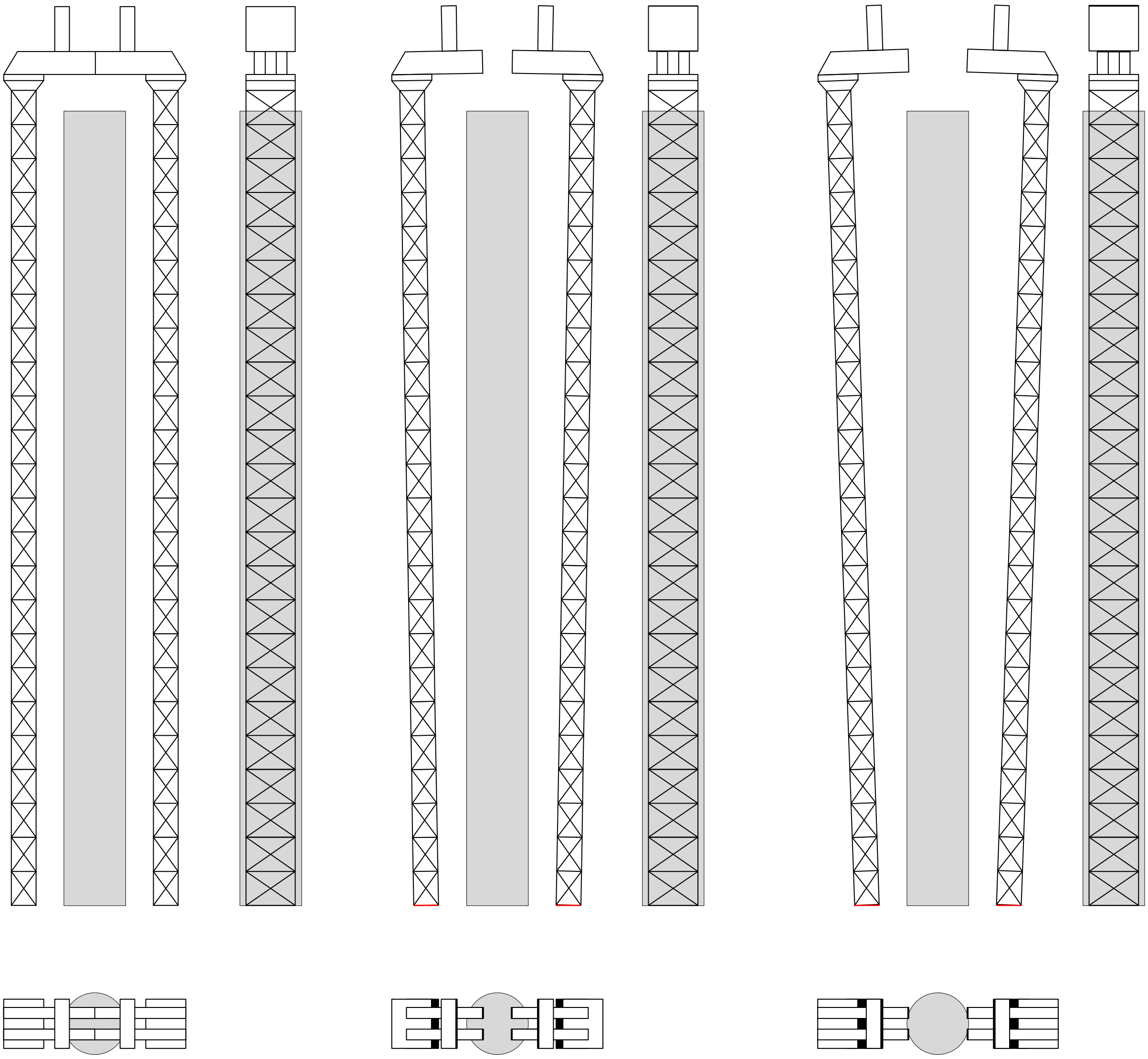


Figure A-50 Rotation Y - two sided + towers sequence

A.6.3.6. Rotation Z

The last possible motion is the rotation around the Z axis. For a build-up sequence see the end of this Paragraph.

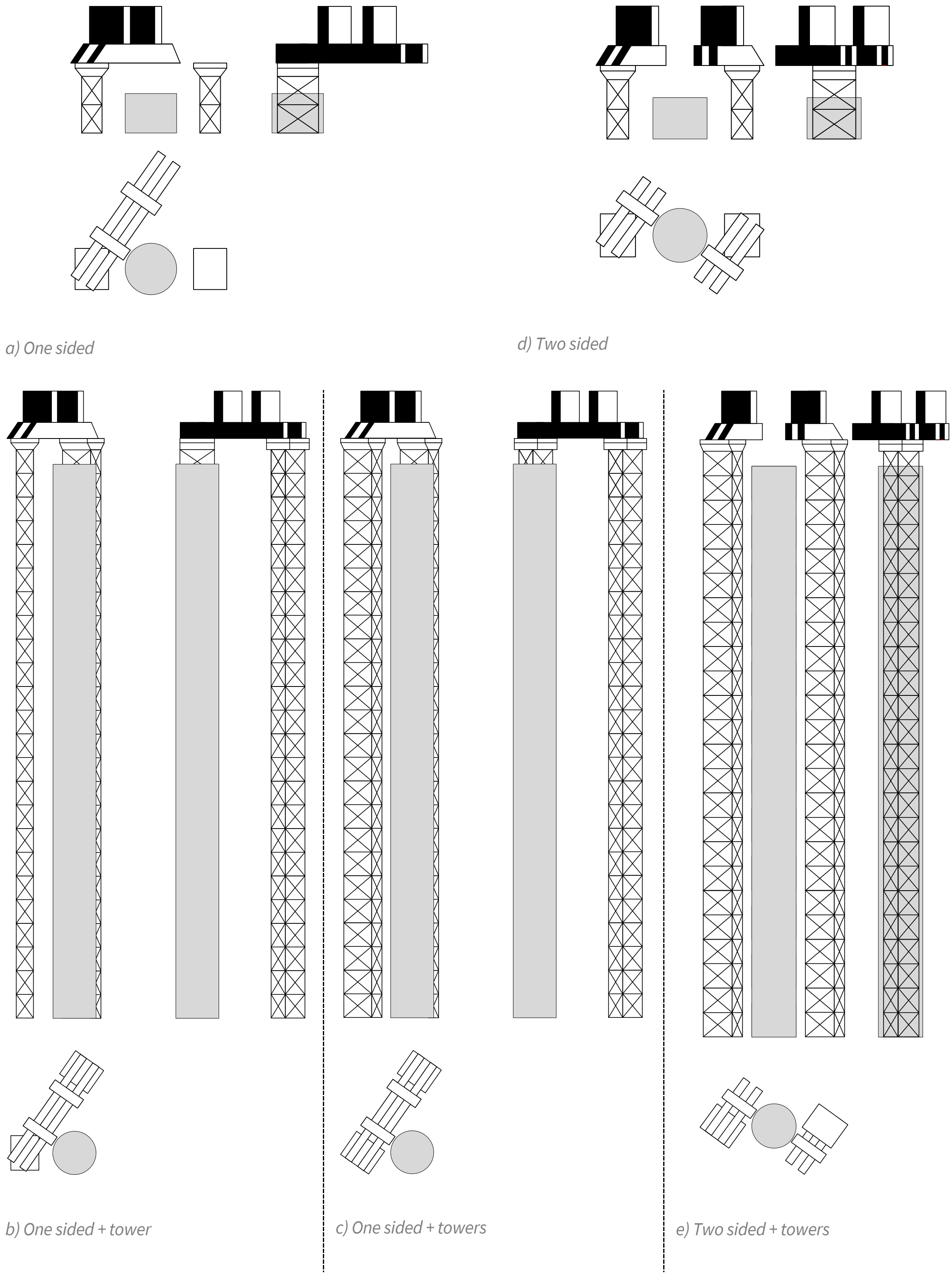


Figure A-51 Upper sturcture out of the way - rotation Z

One sided
Turning the upper structure around the Z axis can be seen in Figure A-51 a). When the upper structure comes free from the tower, stability problems will occur, see Annex A.6.3.2.

This solution will not be considered.

One sided + tower
This can be solved by sliding the tower too, see Figure A-51 b). However, sliding should be done over a curved path, see requirement R12.2.

This solution will not be considered.

One sided + towers
This variant is similar to the former one, whit the difference that here the left tower rotates while moving, see Figure A-51 c). However, also with this variant; sliding should be done over a curved path, see requirement R12.2.

This solution will not be considered.

Two sided
For completeness the two-sided rotation around the Z axis is added, see Figure A-51 d). Splitting the gantry beam is not wanted, see requirement R4.4.

This solution will not be considered.

Two sided + tower
The same holds for this variant. See Figure A-51 e).

This solution will not be considered.

Out of the way sequences

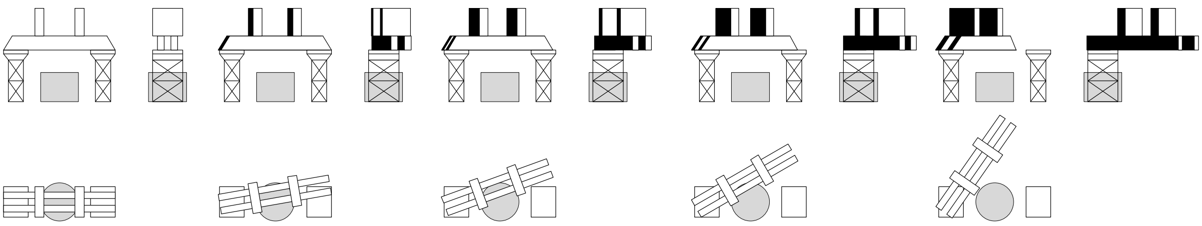


Figure A-52 Rotation Z - one sided sequence

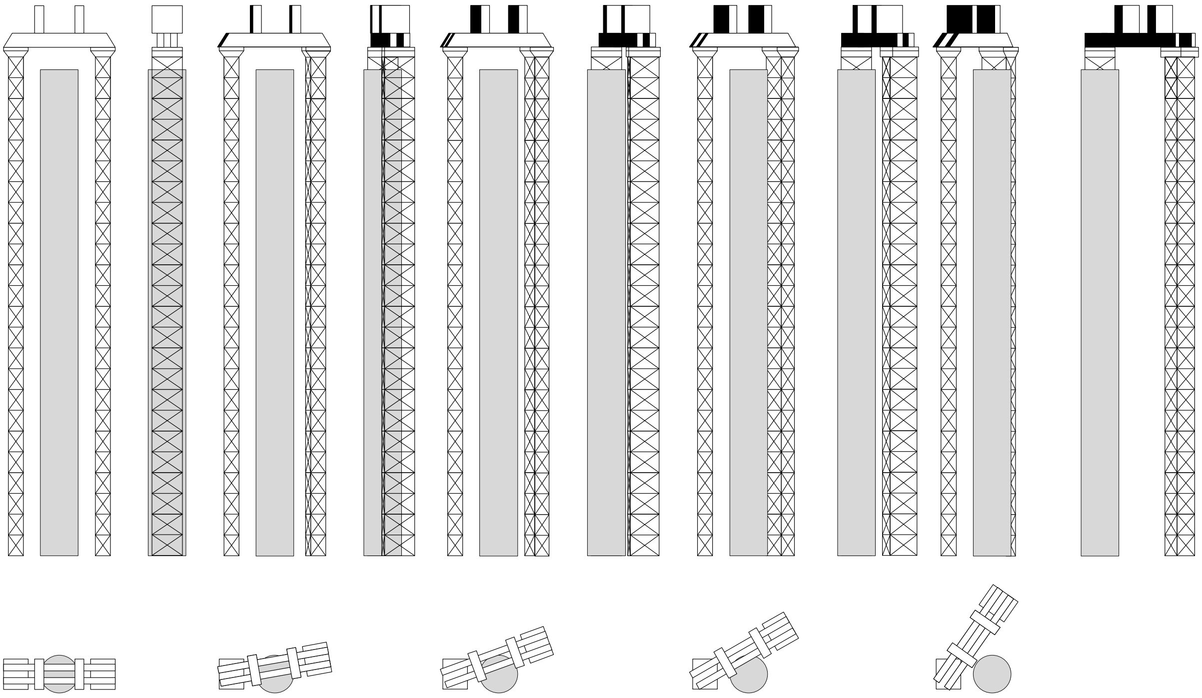


Figure A-53 Rotation Z - one sided + tower sequence

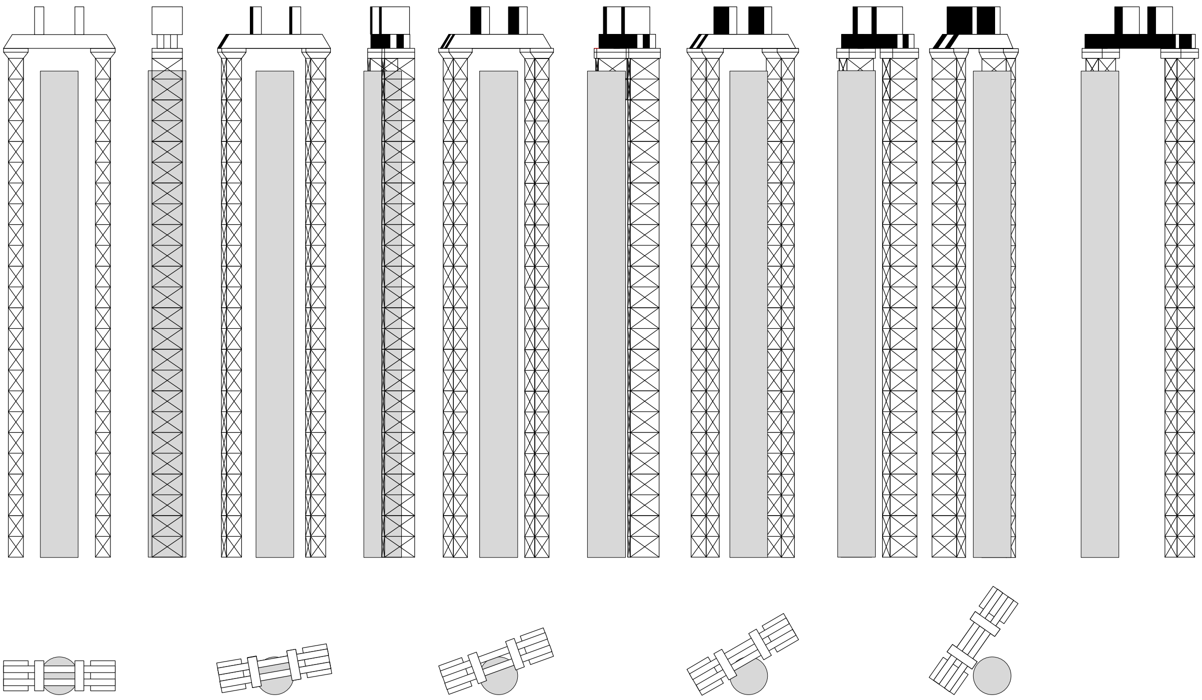


Figure A-54 Rotation Z - one sided + towers sequence

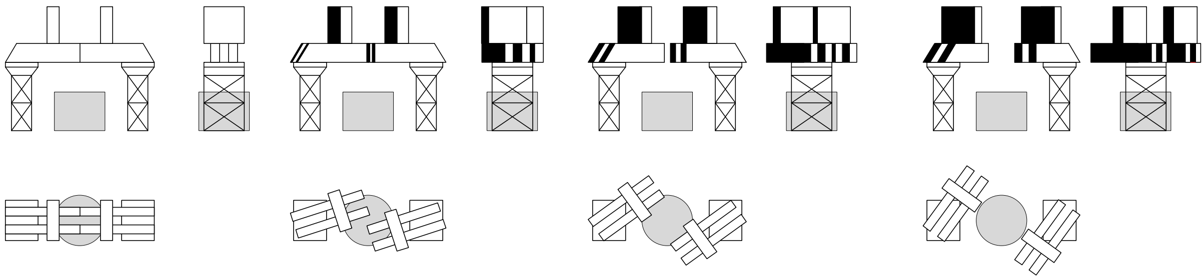


Figure A-55 Rotation Z - two sided sequence

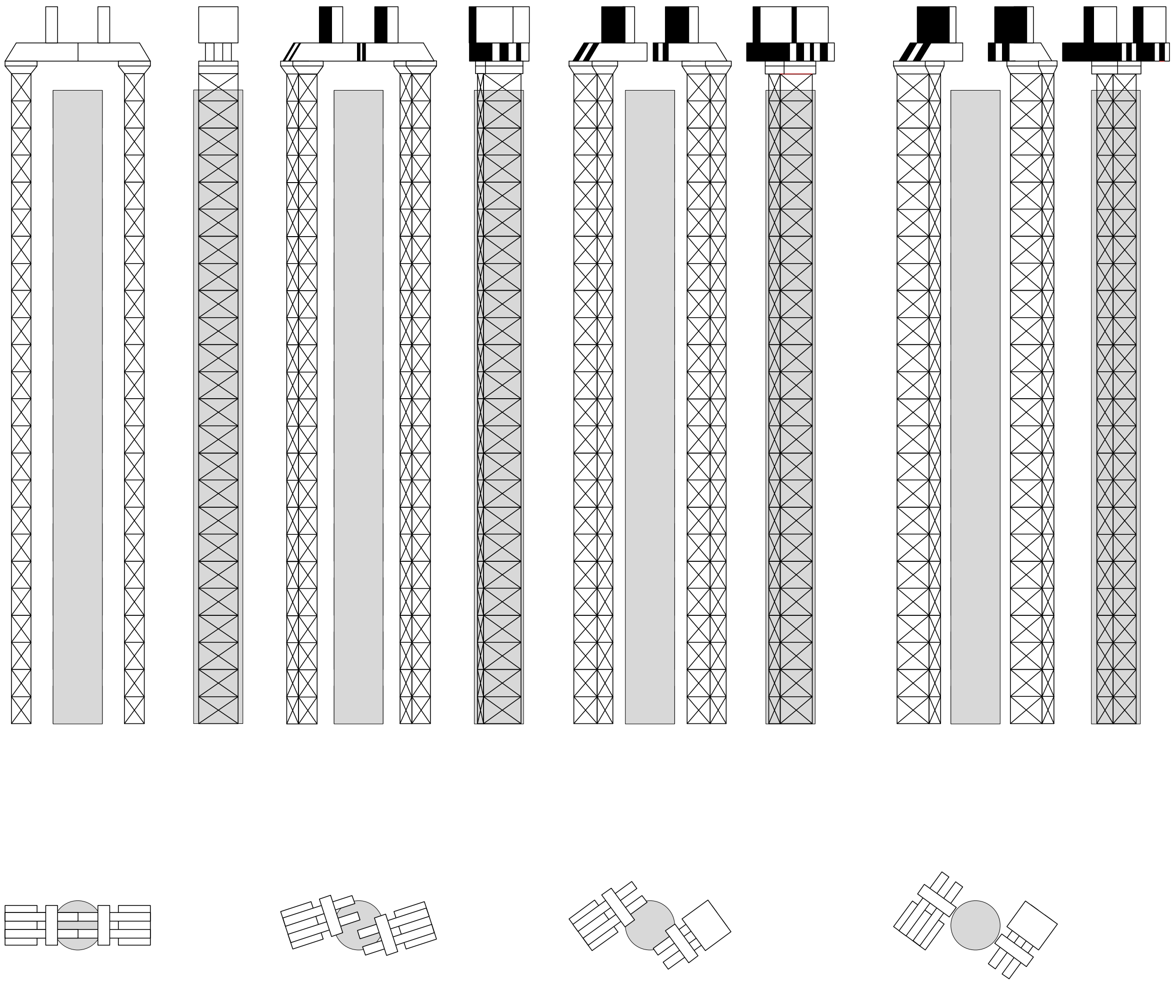
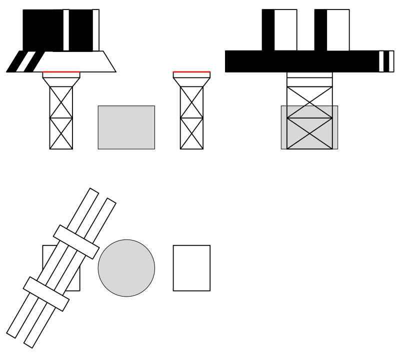


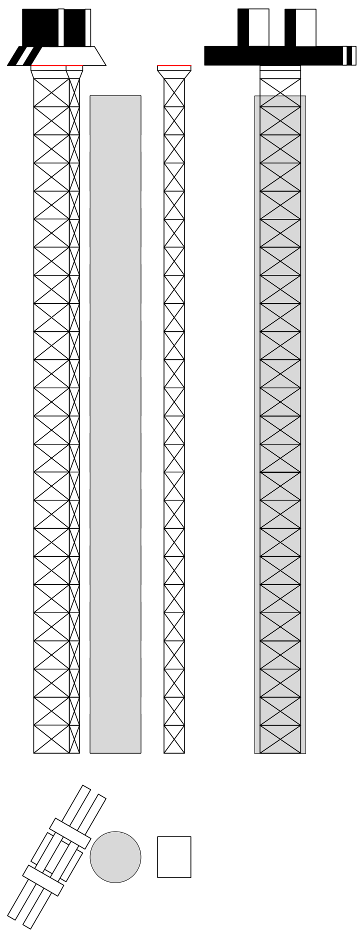
Figure A-56 Rotation Z - two sided + towers sequence

A.6.3.7. Combination

A combination of two motions might result in a better variant. For a build-up sequence see the end of this Paragraph.



a) Siding and turning



b) Sliding and turning + tower

Figure A-57 Upper structure out of the way – combination

Sliding and turning
When the upper structure is slid to the middle of the tower and then turns the tower is stable during disassembly, see Figure A-57 a). Annex A.6.3.2 calculates that the SLS will become a problem.

This solution will not be considered.

Sliding and turning + tower
This variant is similar to the former one however the whole tower rotates, see Figure A-57 b). For the same reason as sliding and turning, this will not be possible.

This solution will not be considered.

Out of the way sequences

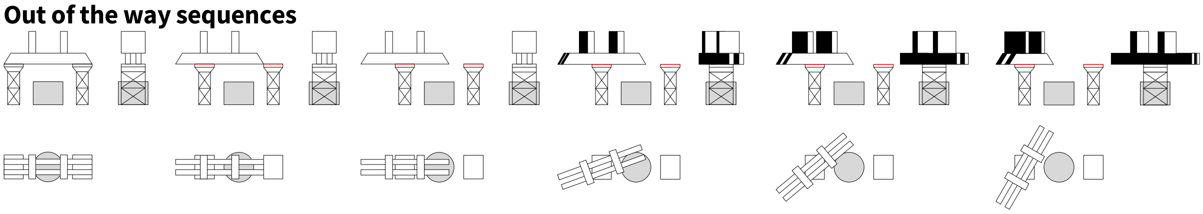


Figure A-58 Combination - sliding & turning sequence

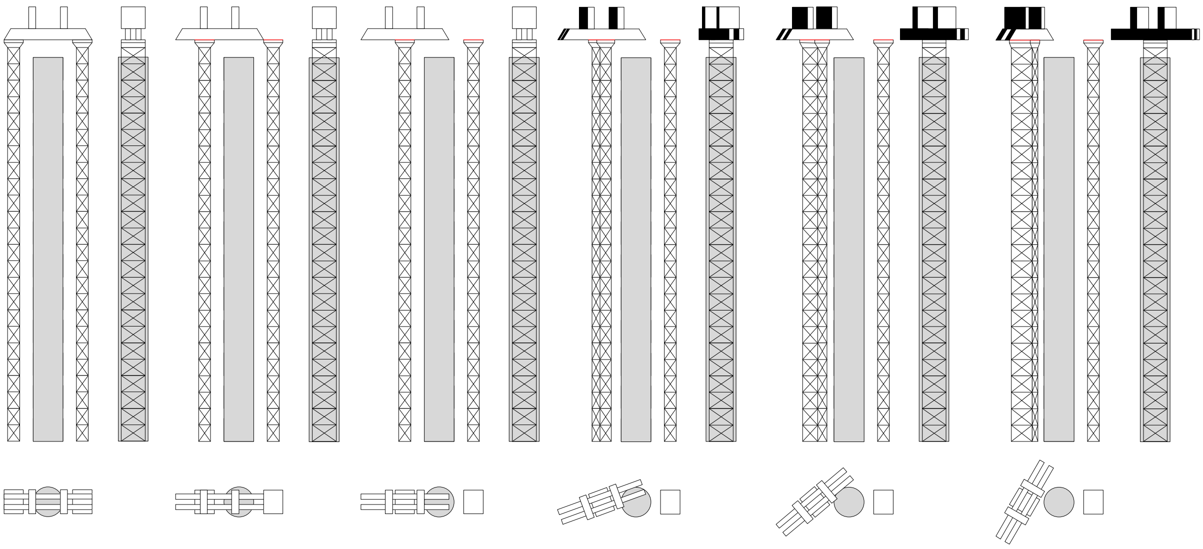
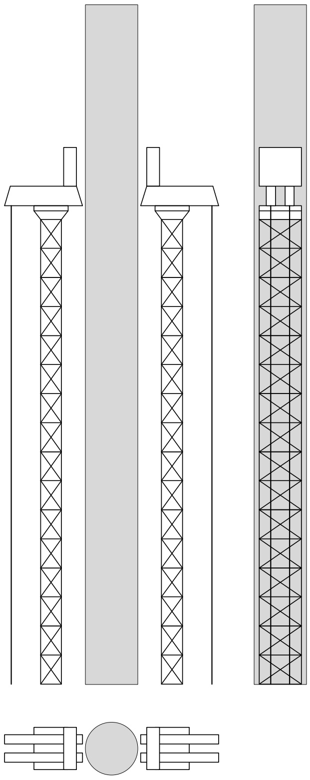


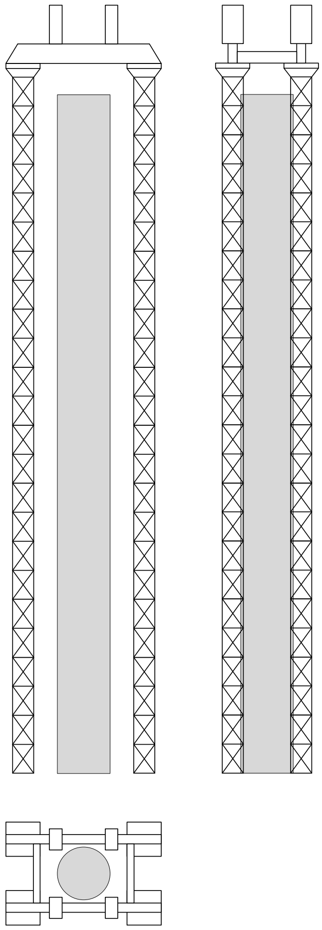
Figure A-59 Combination - sliding & turning + tower sequence

A.6.3.8. N/A

There are variants where the upper structure is not in the way. These do not need a solution.



a) Hammerhead



b) Four towers

Figure A-60 Upper structure out of the way - N/A

Hammerhead

As explained in Chapter 2.1 Mammoet uses hammerhead gantries. They can be smaller than the to-be-lifted object, see Figure A-60 a). Because of that, the upper structure is not in the way of the vessel. However, the height of the gantry is limited by 90 meters because of the crane capacity. This option is not future proof.

This solution will not be considered.

Four towers

When using four towers the upper structure can be out of the way too, see Figure A-60 b). The four towers are needed to ensure that while lifting the heavy vessel the forces go into the towers, otherwise two extra gantry beams are needed. However, this variant uses four 4x4 masts which is not wanted, see requirement R3.

This solution will not be considered.

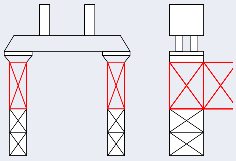
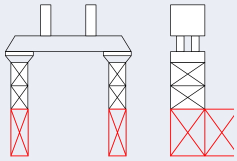
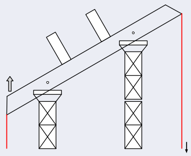
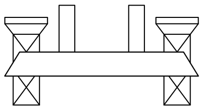
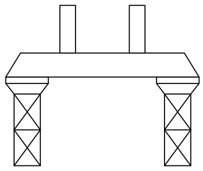
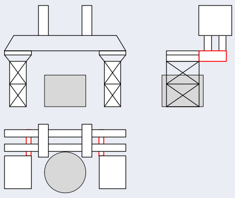
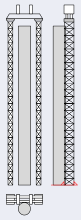
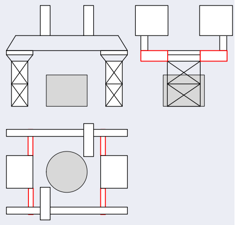
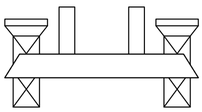
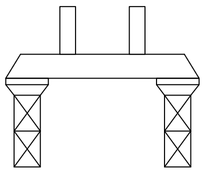
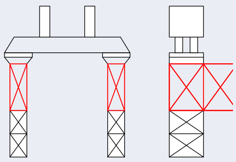
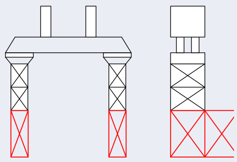
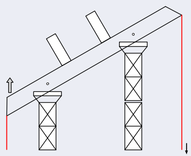
ANNEX B.

This Annex will discuss and score the most promising concepts developed by the morphological chart in Table 2-9. Each Paragraph shows the route through the morphological chart. After that the scoring per criterion is elaborated.

As said in Chapter 2.6 each concept will get a score from 1 to 5, see Table 2-6, on all criteria. After that the weight factor is multiplied with the score to account for the importance of each criterion. By adding all factored scores, a total score per concept is established. The concept with the highest total score is the best and will be designed.

B.1. CONCEPT I CLIMBING CRANE

Table B-24 Morphological chart concept I

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls

The crane needs to be able to slew around, luff, hoist mast sections and the upper structure and lift itself. So, four motion controls are needed per crane. Two cranes are needed therefore the total number of motion controls is eight. This results in a score of 3.

New technique

A self-climbing crane is not part of the fleet of Mammoet. Therefore, it requires new knowledge. So, the score is 1.

New material

Both cranes consist of a boom and a gripping mechanism. The boom can be quite slender as the heaviest lift is half the gantry beam, see next criterion. The gripping mechanism is a complicated part. Because of that the costs for new material will be higher. That makes a score of 2.

Power

It is assumed that the crane will perform the same lifts as the crane did in the reference project. That means that the heaviest lift is the gantry beam, weighing approximately 130 tons including rigging and hook block. If two cranes are used, this is brought down to 65 tons. Therefore, it scores 5.

(Not) Working at height

The whole upper structure needs to be assembled at height. So, this scores a 1.

Foundation

The foundation does not need considerable adjustments because of this concept. Merely, the weight of the climbing cranes will have a small impact. Also, picking up the mast sections and the upper structure will have an impact as they will be an eccentric load on the towers. Making that it scores 4.

Score

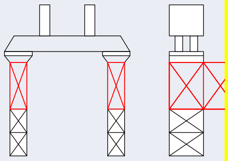
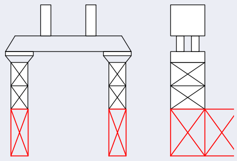
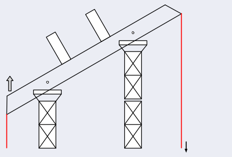
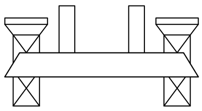
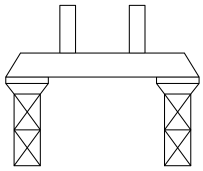
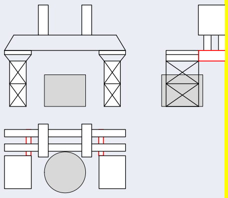
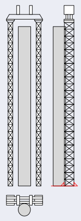
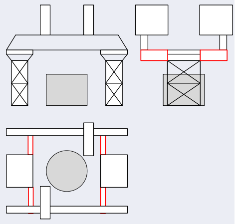
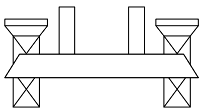
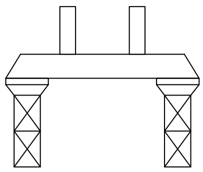
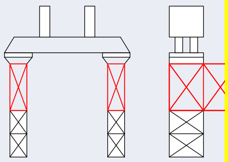
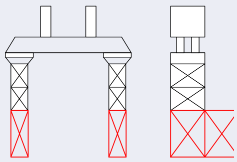
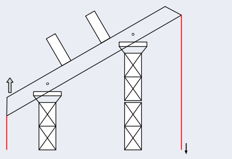
All scores are put in Table B-25. All scores will be multiplied by their weight factor, these outcomes are added together to form the total score for the concept.

Table B-25 Score concept I

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	3	0.18	0.53	0.17	0.50
New components	1	0.06	0.06	0.10	0.10
New material	2	0.12	0.24	0.21	0.41
Power	5	0.12	0.59	0.15	0.75
(Not) Working at height	1	0.29	0.29	0.17	0.17
Foundation	4	0.24	0.94	0.21	0.82
		Total score	**2.65**	**Total score**	**2.76**

B.2. CONCEPT II CFT EXTRA SYSTEM

Table B-26 Morphological chart concept II

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls

The motions that occur in this concept are: pushing of the climbing frame, lifting of the new mast sections and the upper structure, and getting the new mast sections and the upper structure in place. In total three per frame. Making six for two frames. Therefore, it scores 4.

New technique
A climbing frame is being used in Mammoet, in the FOCUS crane. However, this climbing frame was at the bottom, not at the top. Therefore, it will be similar but not the same. Lifting the upper structure and skidding it in place are known techniques for Mammoet. So, this concept is scoring 4.

New material
Two climbing frames need to be designed that can:

- Lift both the mast sections and the upper structure.
- Slide the mast sections and the upper structure in place.

The latter will make that the climbing frame must be open on two sides. This complicates the force transfer and therefore requires a stronger frame. This makes the score 2.

Power
The heaviest lift will be the total upper structure. This weight is shared between the two climbing frames, so each has to lift 450 / 2 = 225 tons. Scoring 3.

(Not) Working at height
Each mast section needs to be fastened when it is in place, with every new section this becomes higher. Also, when the upper structure is slid in place it needs to be fastened to secure a clamed connection between the towers and the upper structure. This is a bit better than concept I, as not the whole upper structure needs to be assembled. Making that the score is 2.

Foundation
The foundation does not need considerable adjustments because of this design. Merely, the weight of the climbing frame will have a small impact. Here the whole upper structure will be lifted eccentric, this is worse than concept II. Making that it scores 3.

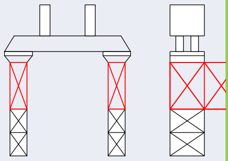
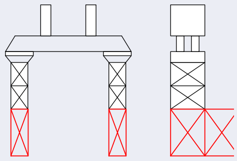
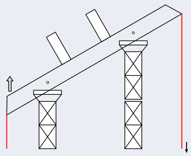
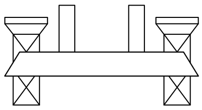
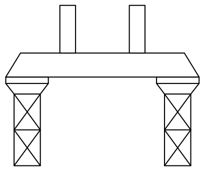
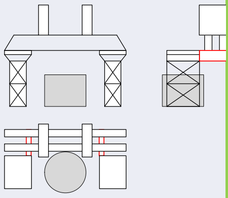
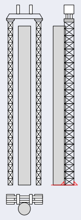
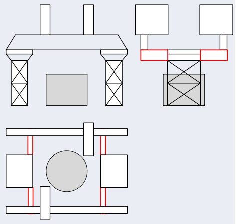
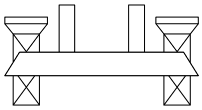
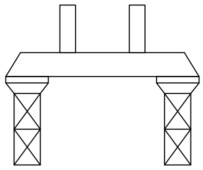
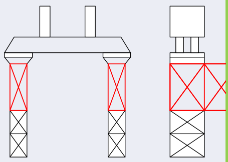
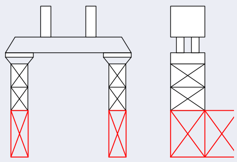
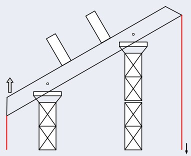
Score

Table B-27 Score concept II

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	4	0.18	0.71	0.17	0.66
New components	4	0.06	0.24	0.10	0.41
New material	2	0.12	0.24	0.21	0.41
Power	3	0.12	0.35	0.15	0.45
(Not) Working at height	2	0.29	0.59	0.17	0.34
Foundation	3	0.24	0.71	0.21	0.62
		Total score	**2.82**	**Total score**	**2.89**

B.3. CONCEPT III CFT SKID

Table B-28 Morphological chart concept III

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls

The motions that occur in this concept are: pushing of the climbing frame, lifting of the new mast section, getting the new mast section in place, and skidding the whole gantry. In total three per frame and one skidding system per tower, so in total eight motion controls. Scoring 3.

New technique
A climbing frame is being used in Mammoet, in the FOCUS crane. However, this climbing frame was at the bottom, not at the top. Therefore, it will be similar but not the same. Skidding large and heavy objects is also known within Mammoet. Scoring 4.

New material
Two climbing frames need to be designed that can:
- Lift the mast sections.
- Slide the mast sections in place.

This climbing frame only requires one opening, making that it can be slenderer than the frame in concept II. Also, two skidding systems that can slide the whole gantry. Scoring 4.

Power
Both climbing frames have to push the upper structure up when a new mast section needs to be added. So, both have to push 225 tons and their (unknown) self-weight.

Which is similar to concept II. However, the whole gantry plus the climbing frame needs to be skidded. This is 1380 tons plus the unknown self-weight. That requires more power. Scoring 2.

(Not) working at height
Each mast section needs to be fastened when it is in place, with every new section this becomes higher. The upper structure is in place, so that does not require working at height. Scoring 3.

Foundation
The total weight of the gantry and the climbing frames needs to be translated to a new location. A bigger foundation needs to be designed for this. Scoring 1.

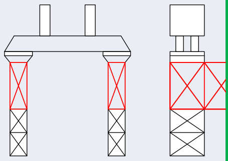
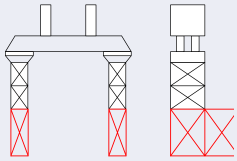
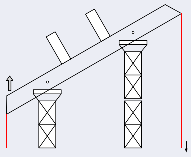
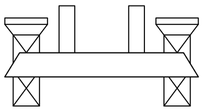
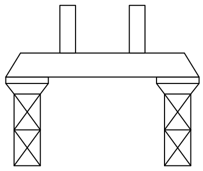
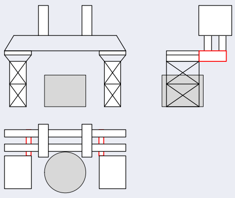
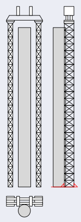
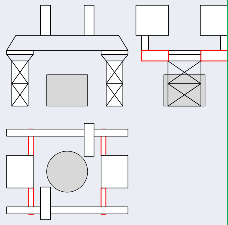
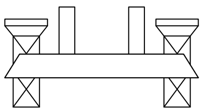
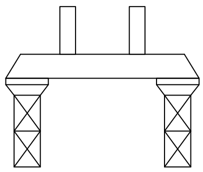
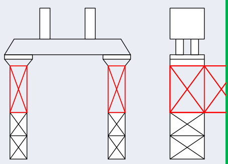
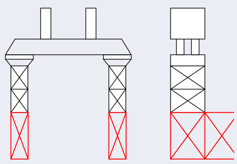
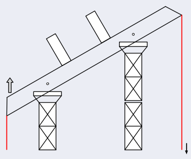
Score

Table B-29 Score concept III

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	3	0.18	0.53	0.17	0.50
New components	4	0.06	0.24	0.10	0.41
New material	4	0.12	0.47	0.21	0.82
Power	2	0.12	0.24	0.15	0.30
(Not) Working at height	3	0.29	0.88	0.17	0.51
Foundation	1	0.24	0.24	0.21	0.21
		Total score	**2.59**	**Total score**	**2.74**

B.4. CONCEPT IV CFT SPLIT

Table B-30 Morphological design concept IV

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls

The motions that occur in this concept are: pushing of the climbing frame, lifting of the new mast section, getting the new mast section in place, and skidding the upper structure. In total three per frame and one per tower per upper structure part. So, with two frames, two towers and two upper structure parts this becomes ten motion controls. Scoring 2.

New technique

A climbing frame is being used in Mammoet, in the FOCUS crane. However, this climbing frame was at the bottom, not at the top. Therefore, it will be similar but not the same. Skidding the upper structure to opposite sides is known to Mammoet. Scoring 4.

New material

Two climbing frames need to be designed that can:

- Lift the mast sections.
- Slide the mast sections in place.

This climbing frame only requires one opening, making that it can be slenderer than the frame in concept II. Also, four skidding systems that can slide both upper structure parts away are needed. This requires four outriggers. All in all, this is more than concept III, so scoring 3.

Power

Both climbing frames have to push the upper structure up when a new mast section needs to be added. So, both have to push 225 tons.

This is also the weight that each skidding system needs to skid to the side. Therefore, it scores 4.

(Not) working at height

Each mast section needs to be fastened when it is in place, with every new section this becomes higher. The upper structure needs to be loosened in order to slide it away. This requires work at height. Scoring 2.

Foundation

The foundation does not need considerable adjustments because of this design. Merely, the weight of the climbing frame will have a small impact. Plus, the eccentricity of lifting the mast sections. Scoring 4.

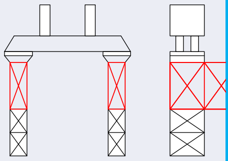
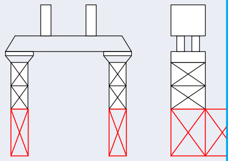
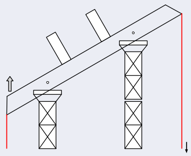
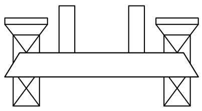
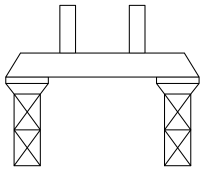
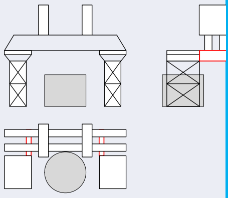
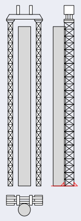
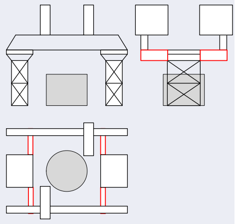
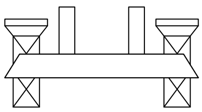
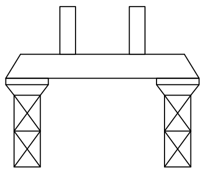
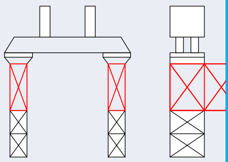
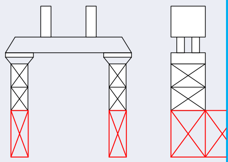
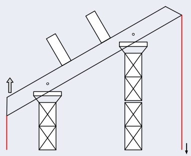
Score

Table B-31 Score concept IV

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	2	0.18	0.35	0.17	0.33
New components	4	0.06	0.24	0.10	0.41
New material	3	0.12	0.35	0.21	0.62
Power	4	0.12	0.47	0.15	0.60
(Not) Working at height	2	0.29	0.59	0.17	0.34
Foundation	4	0.24	0.94	0.21	0.82
		Total score	**2.94**	**Total score**	**3.12**

B.5. CONCEPT V CFB SKID

Table B-32 Morphological chart concept V

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls

The motions that need to be performed are pushing the gantry, getting the new mast sections in place, and skidding the whole gantry. So, three per frame, so six in total. Scoring 4.

New technique

Both a climbing frame at the bottom and skidding heavy objects are known techniques within Mammoet. Scoring 5.

New material

Two climbing frames need to be designed that can:

- Push the whole gantry.
- Slide the mast sections in place.

In order to push and stabilize the whole gantry a heavier climbing frame will be needed than concepts II, III and IV. And two skidding systems are needed for the towers. Scoring 3.

Power

Each time a new mast section is put under the gantry the climbing frame needs to jack more weight. The critical jack is before the last mast section. Then in total the upper structure and two times 11 mast sections need to be jacked, by two climbing frames. This is approximately 650 tons per frame.

To get the upper structure out of the way the whole gantry is skid. Thus, the total gantry weight plus the climbing frames, so 1380 tons plus the unknown self-weight.

This is both the heaviest push and the heaviest skid. Scoring 1.

(Not) working at height

The upper structure is in place during assembly, and it must stay put when the gantry is translated. Therefore, all work can be done on the ground. Scoring 5.

Foundation

The total weight of the gantry and the climbing frames needs to be translated to a new location. A bigger foundation needs to be designed for this. Scoring 1.

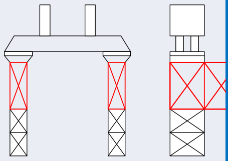
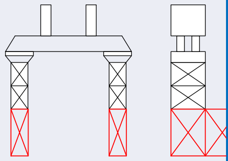
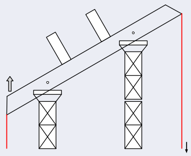
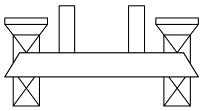
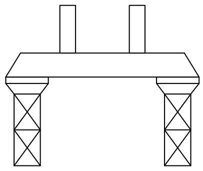
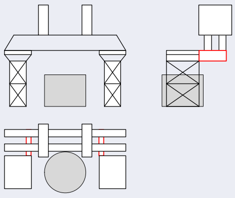
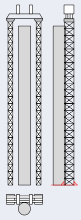
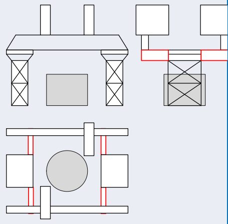
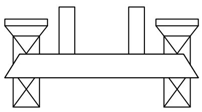
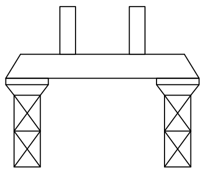
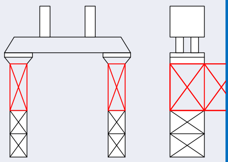
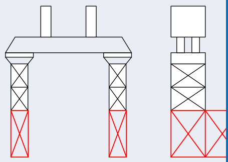
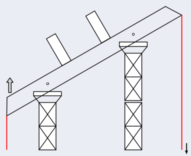
Score

Table B-33 Score concept V

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	4	0.18	0.71	0.17	0.66
New components	5	0.06	0.29	0.10	0.51
New material	3	0.12	0.35	0.21	0.62
Power	1	0.12	0.12	0.15	0.15
(Not) Working at height	5	0.29	1.47	0.17	0.85
Foundation	1	0.24	0.24	0.21	0.21
		Total score	**3.18**	**Total score**	**2.99**

B.6. CONCEPT VI CFB SPLIT

Table B-34 Morphological chart concept VI

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls
The motions that need to be performed are pushing the gantry, getting the new mast sections in place, and skidding the upper structure to opposite sides. So, two per frame and two per tower per upper structure part. So, with two frames and two towers eight motion controls. Scoring 3.

New technique
Both a climbing frame at the bottom and skidding heavy objects are known techniques within Mammoet. Scoring 5.

New material

Two heavy climbing frames are needed and four skidding systems over four outriggers. Scoring 2.

Power

Also, here the critical jack per frame is 650 tons.

To get the upper structure out of the way the upper structure is skidded. The upper structure is split, so each side has to skid 225 tons.

This is a bit better than concept V, so scoring 2.

(Not) working at height

Splitting the upper structure requires workers to disconnect it, this has to be done at height. This is the only work that has to be done at height. Scoring 3.

Foundation

The foundation does not need considerable adjustments because of this design. Merely, the weight of the climbing frame will have a small impact. Scoring 4.5.

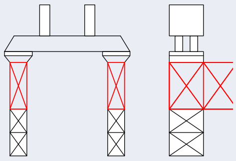
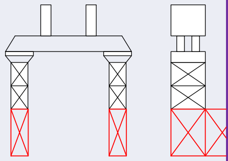
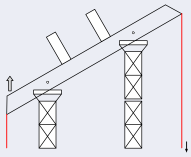
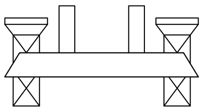
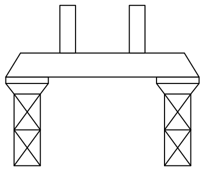
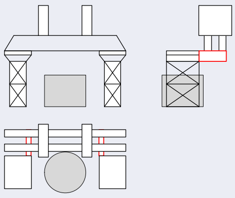
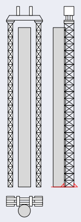
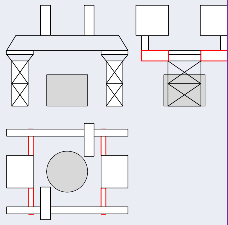
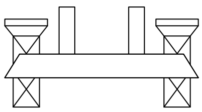
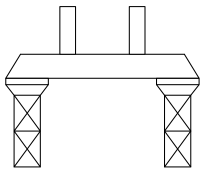
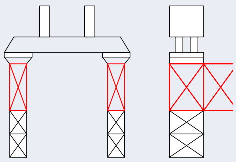
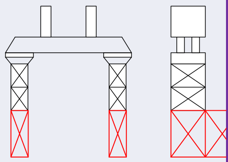
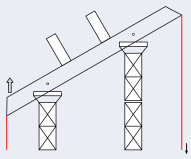
Score

Table B-35 Score concept VI

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	3	0.18	0.53	0.17	0.50
New components	5	0.06	0.29	0.10	0.51
New material	2	0.12	0.24	0.21	0.41
Power	2	0.12	0.24	0.15	0.30
(Not) Working at height	3	0.29	0.88	0.17	0.51
Foundation	4.5	0.24	1.06	0.21	0.93
		Total score	**3.24**	**Total score**	**3.16**

B.7. CONCEPT VII WIPWAP

Table B-36 Morphological chart concept VII

OPTIONS STAGES	1	2	3	4
Tower erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 
Raise upper structure raise	*Additional crane* 	*System along tower* 	*N/A* 	
Upper structure out of the way	*Additional crane* 	*Translation Y* 	*Translation Y* 	*Translation Y* 
Lower upper structure	*Additional crane* 	*System along tower* 	*N/A* 	
Tower de-erection	*Additional crane* 	*Climbing frame top* 	*Climbing frame bottom* 	*Smart use of structure* 

Motion controls

Wipwapping requires two rotations between the gantry beam and the towers, however, these are no operating systems. But, two pulling systems are needed and a system that skids the new mast sections in place. Lastly, a skidding system per tower is needed. In total this comes to six motion controls. Scoring 4.

New technique

All systems in the concept are known techniques to Mammoet. However, combined and at this scale it is never performed. Therefore, it scores a 2.

New material

Two frames that allow for a hinged connection between the upper structure and the towers are needed. This is because one does not want to make a hole in the gantry beam. The gantry beam needs extension parts. Two pulling systems are needed. Lastly, a skidding system per tower is needed. In total this would score 2.

Power

The most critical action is when the last mast section is to be inserted. The weight of the whole gantry acts on one tower. Besides that, the tensile force that is needed to elevate the gantry acts on the gantry. When the gantry beam is 47 meters long, see requirement R4.3. And the pulling system is attached half a meter from the edge of the beam. This all comes to the following mechanical scheme:

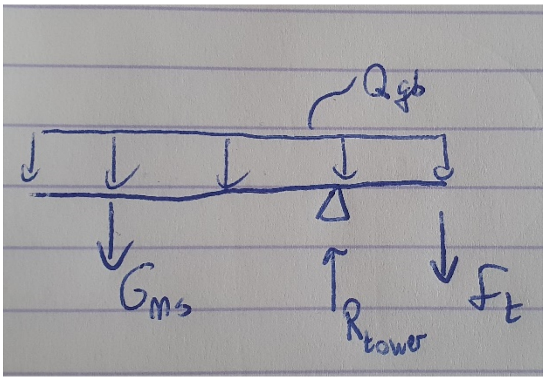


Figure B-61 Mechanical scheme wipwap

The weight of eleven mast sections is:

$$G_{mast\ sections} = 11 * 38.7 = 425.7\ ton = 4257\ kN$$

The upper structure minus the (original) gantry beams weighs:

$$G_{upper\ structure} = 456.97 - 2 * 123.30 = 210.37\ ton = 2064\ kN$$

The weight of one new gantry beam, so plus the two new extensions, is:

$$G_{gb+ext} = G_{gb} + 2 * G_{ext}$$
$$G_{gb+ext} = 75.700 + 2 * 46.873 = 169.446\ ton = 1662\ kN$$

Resulting in a distributed load of:

$$q_{gb} = \frac{1662}{47} = 35\ kN/m$$

Moment equilibrium around the tower results in a tensile force of:

$$F_t = \frac{4257 * 23 + \frac{2064}{2} * 23 + 35 * 47 * 11.5}{11.5} = 12{,}240\ kN = 1247.7\ ton$$

Making that the total load on one tower is:

$$R_{tower} = G_{mast\ sections} + G_{upper\ stucture} + G_{gb+ext} + F_t$$
$$R_{tower} = 4257 + 2064 + 1662 + 12{,}240 = 20{,}223\ kN = 2061.5\ ton$$

A strong pulling system is needed to create a tensile force of 1250 tons. Besides that, the whole gantry needs to be skidded. In total this concept scores 1.

(Not) working at height
During the tower erection all work can be done at ground level. However, the clamping of the upper structure with the towers should be done at height. Making for a score of 3.

Foundation
The foundation needs to withstand the tensile force coming and the massive compressive force of the towers. Besides that, it needs to be bigger because of the skidding. All in all, this scores 0.

Score

Table B-37 Score concept VII

	SCORE	WEIGHT FACTOR AUTHOR	WF * SCORE	WEIGHT FACTOR MEAN	WF * SCORE
Motion controls	4	0.18	0.71	0.17	0.66
New components	2	0.06	0.12	0.10	0.20
New material	2	0.12	0.24	0.21	0.41
Power	1	0.12	0.12	0.15	0.15
(Not) Working at height	3	0.29	0.88	0.17	0.51
Foundation	0	0.24	0.00	0.21	0.00
		Total score	**2.06**	**Total score**	**1.94**

ANNEX C.

C.1. CLIMBING FRAME BOTTOM VS. CLIMBING FRAME TOP

C.1.1. Climbing frame general

C.1.1.1. Jack-up block weld calculation

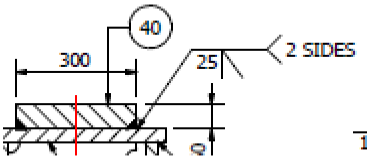


Figure C-62 Jack-up block butt weld

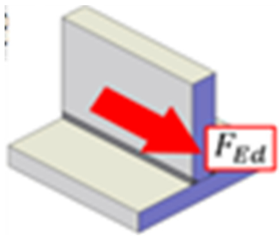


Figure C-63 Jack-up block butt weld load case

The butt welds are subjected to a shear force, for which the unity check holds:

$$U.C. = \frac{\sum \tau_{\parallel}}{\left(\frac{f_u}{\sqrt{3} * \beta * \gamma_{M2}}\right)} \tag{27}$$

For maximum capacity the Unity Check is 1. Rewriting formula (27) results in a shear stress resistance of:

$$\sum \tau_{\parallel} = \frac{f_u}{\sqrt{3} * \beta * \gamma_{M2}} \tag{28}$$

Where $\tau_{//}$ is:

$$\tau_{\parallel} = \frac{\gamma * F_{Ed}}{n * a * l_{eff}} \tag{29}$$

Resulting in a shear force resistance of:

$$F_{Ed} = \frac{f_u * n * a * l_{eff}}{\sqrt{3} * \beta * \gamma_{M2} * \gamma} \tag{30}$$

Where:

- F_E is the design load.
- f_u is the ultimate strength of the (weakest) parental material.
- n is the number of welds.
- a is the weld throat thickness.
- l_{eff} is the effective weld length, which is calculated via:

$$l_{eff} = l - 2 * a \tag{31}$$

  Where:
  - l is the physical length of the weld.

- β
- γ_{M2} is the partial factor for connections.
- γ is the safety factor for the load.

For the jack-up blocks hold:
- $f_u = 770\ N/mm^2$
- n = 2
- a = 25 mm
- l = 620 mm
- $l_{eff} = 570$ mm
- β = 1
- $\gamma_{M2} = 1.25$
- γ = 1.5

So:

$$F_{Ed} = \frac{770 * 2 * 25 * 570}{\sqrt{3} * 1 * 1.25 * 1.5} * 10^{-3} = 6757\ kN = 689\ Te$$

C.1.2. Climbing frame bottom

C.1.2.1. MLS or MSG base

The initial thought was to use MLS mast sections as a base for the climbing frame, see Figure C-64. Using MLS mast sections proved to be able to withstand all loads on the gantry. However, when inserting the mast sections in lateral direction there is no room for the to-be-lifted vessel. Therefore, the base needs to be made out MSG mast sections, see Figure C-64.

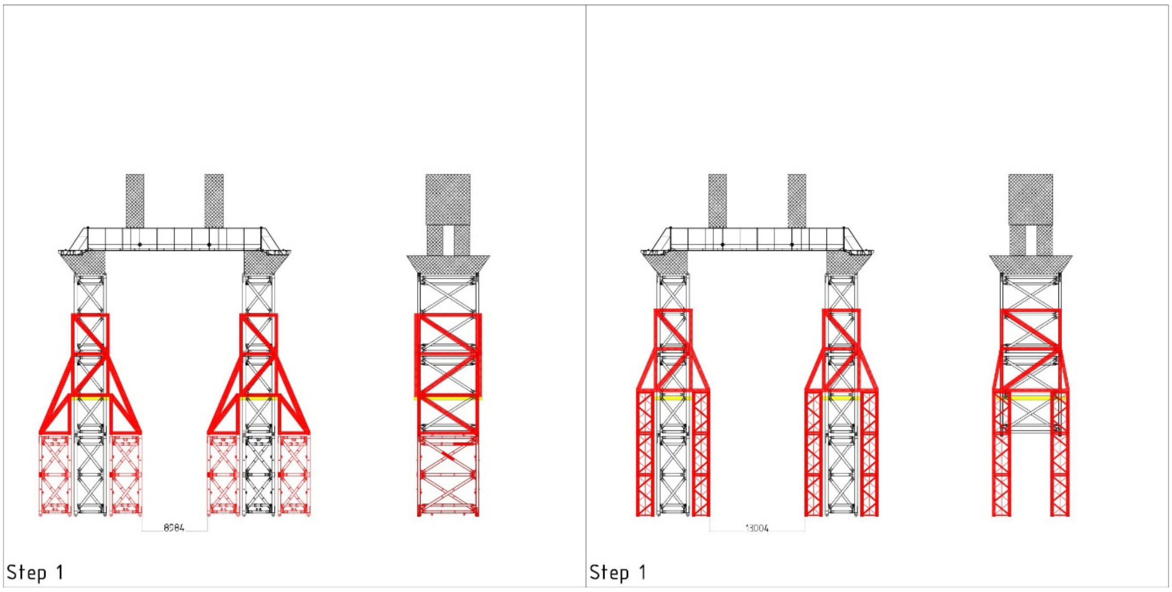


Figure C-64 Climbing frame bottom, MLS base (left), MSG base (right)

C.1.2.2. Building sequence

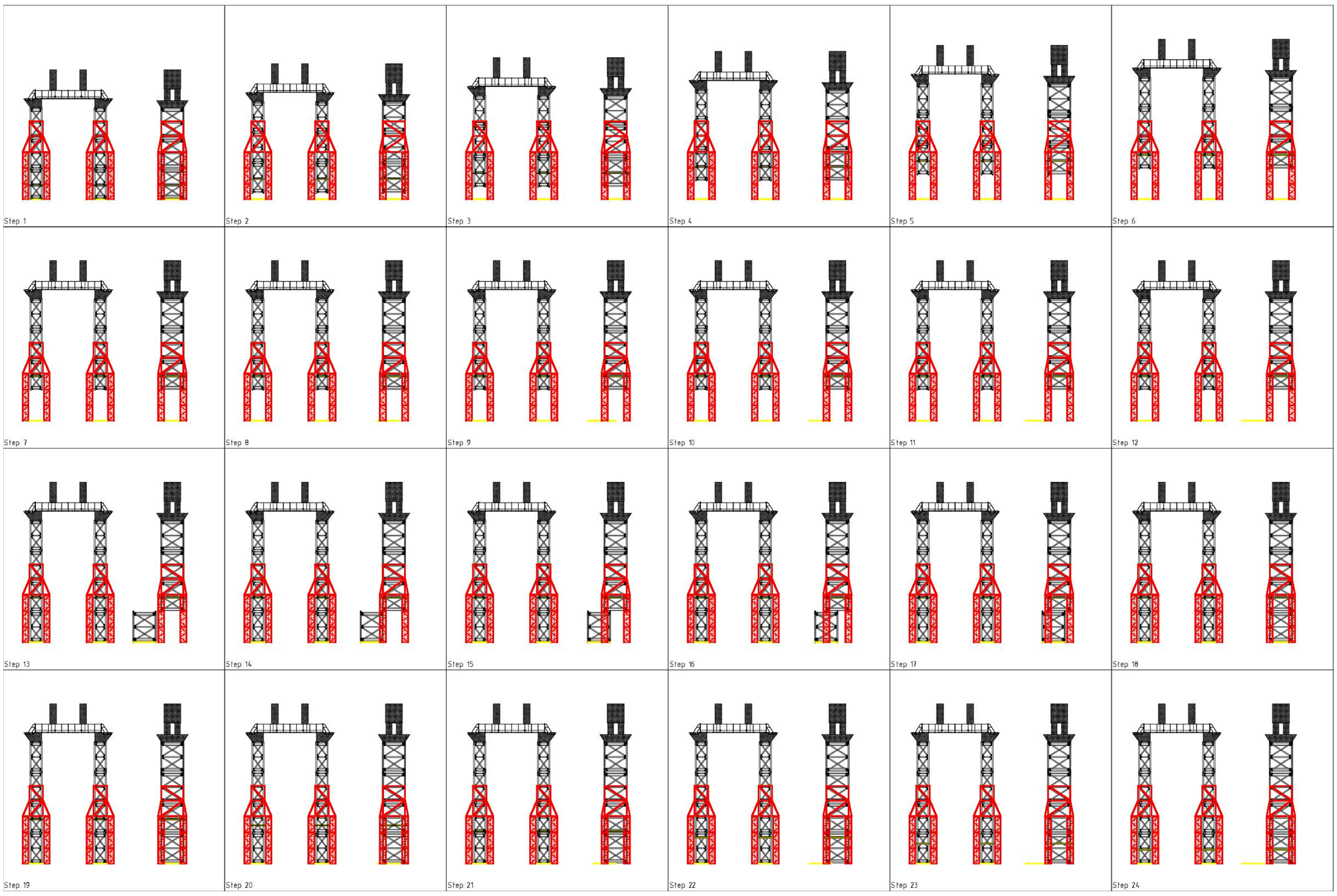


Figure C-65 Climbing frame bottom building sequence

C.1.3. CLIMBING FRAME TOP

C.1.3.1. No tension

This Maple script calculates the length between the tower and the to-be-lifted mast section that is needed for tension in the connection. This will not be larger than 20.15 meters.

```
> restart;
```

b is the width of the mast section, G is the assumed weight of the upper structure plus the top climbing frames divided by two (two towers).
h is the height of the climbing frame, L is the distance between the tower and the lifting of the new mast section, Y is the side load.
RA is the reactional force of the left chord, RB of the right chord, X is the load of the lifting mast seciton plus the hoisting mechanism.

```
> b:=8; G:=3000; X:=500; h:=11; Y:=0.05*(G+X);
```

$$b := 8$$
$$G := 3000$$
$$X := 500$$
$$h := 11$$
$$Y := 175.00$$

```
> eq1:=R__A=1/2*b*G-b*R__B+(b+L)*X+h*Y;
```

$$eq1 := R_A = 17925.00 - 8 R_B + 500 L$$

```
>
> eq2:=R__A+R__B=G+X;
```

$$eq2 := R_A + R_B = 3500$$

```
> S:=solve({eq1,eq2},{R__A,R__B});
```

$$S := \{R_A = 1439.285714 - 71.42857143 L, R_B = 2060.714286 + 71.42857143 L\}$$

```
> assign(S);
> eq3:=R__A=0;
```

$$eq3 := 1439.285714 - 71.42857143 L = 0$$

```
> S2:=solve({eq3},{L});
```

$$S2 := \{L = 20.15000000\}$$

C.1.3.2. Building sequence

Figure C-66 Climbing frame top building sequence

C.2. SKIDDING BOTTOM VS. SPLITTING TOP

C.2.1. SKIDDING BOTTOM

C.2.1.1. Stability

For stationary gantries the side loads are: 1% standard, 1% wind, 1% settlement, 0.5% misalignment and 1.5% unforeseen [6]. Where 1% means 1% of the vertical load. Here only the self-weight of the gantry applies. Meaning that the side load for a stationary gantry is:

$$F_{H,stat} = \frac{1.0 + 1.0 + 1.0 + 0.5 + 1.5}{100} * G_{gantry} = 0.05 * 1489 = 74.5\ Te$$

When the gantry is being skidded another 3% needs to be added [6]. Meaning that the total horizontal load during skidding will be:

$$F_{H,tot,skid} = F_{H,stat} + 0.03 * G_{gantry} = 74.5 + 0.03 * 1489 = 119.1\ Te$$

According to the Engineering Handbook Transport 1.01 the acceleration/ deceleration force that acts on the structure can be calculated using Figure C-67.

Speed (km/h)	Acceleration	Application example
0.0 – 0.5	5%	RoRo operations
0.5 – 3.0	10%	Site moves
3.0 – 8.0	15%	Long-distance transports

Figure C-67 Acceleration forces [41]

Assuming a low speed of 0.0 – 0.5 km/h, the acceleration force is 5% of the weight. This has to be added to the side load for a stationary gantry. Meaning a total horizontal load of:

$$F_{H,tot,SPMT} = F_{H,stat} + 0.05 * G_{gantry} = 74.5 + 0.05 * 1489 = 148.9\ Te$$

These are horizontal forces that act on the center of gravity (C.o.G.) of the gantry. Which can be calculated as:

$$C.o.G. = \frac{\sum_{i=0}^{n} G_i z_i}{\sum_{i=0}^{n} G_i} \tag{32}$$

Where:

- G_i is the weight of part i.
- z_i is the height of part i, measured from the ground.

The upper structure, weighing 456.97 tons, is assumed to have its C.o.G. in the center of the gantry beam. This is 138.5 meters from the ground. Two towers weigh two times 12 times 38.7 tons, so 928.8 tons. Their C.o.G. is assumed to be halfway the height, so 6 times 11 meters plus the base frame height. This is 67.2 meters from the ground. This makes:

$$C.o.G. = \frac{456.97 * 138.5 + 928.8 * 67.2}{1385.8} = 90.8\ meter$$

See Figure C-68 for a visual interpretation.

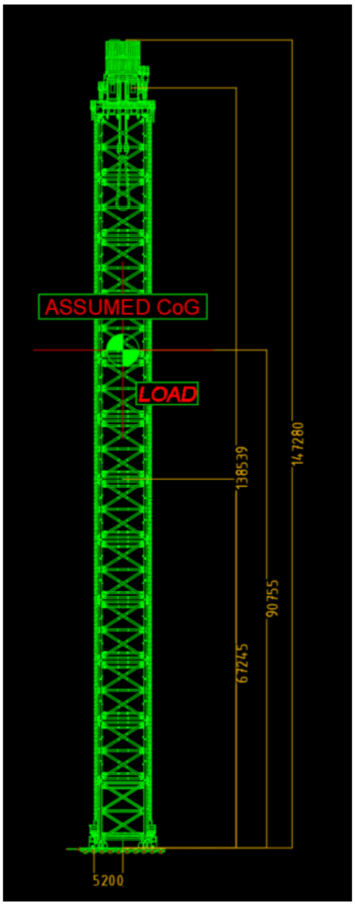


Figure C-68 CoG gantry

This leads to the structural scheme with accompanying reaction forces in Figure C-69.

MAMMOET

Client

Project

Subject

SAP nr.

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Page

Date

Rev.

Skidding

$F_V = 1490$ Te

$F_H = 120$ Te

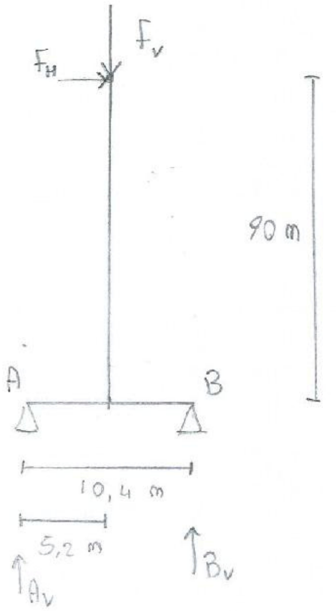
$\Sigma T|A = 0: \; 5{,}2 \cdot 1380 + 90 \cdot 120 - 10{,}4 \cdot B_V = 0$

$10{,}4\, B_V = 5{,}2 \cdot 1490 + 90 \cdot 120$

$10{,}4\, B_V = 7748 + 10800$

$10{,}4\, B_V = 18548$

$B_V = 1783$ Te



Vertical force eq.

$A_V = F_V - B_V$

$= 1490 - 1783$

$= -293$ Te

Negative, so tensile force!

SPMT

$F_V = 1490$ Te

$F_H = 150$ Te

Bigger than 120, so also tensile force!

Client

Project

MAMMOET THE BIGGEST THING WE MOVE IS TIME

Figure C-69 Hand calculation translating gantry reaction forces

MAMMOET

Client

Project

Subject

SAP nr.

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Ref.

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Date

Rev.

Widening base skidding

$F_V = 1490$ Te

$F_H = 120$ Te

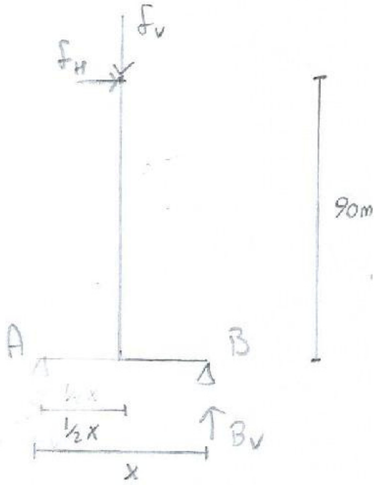
$A_V = 0$ Te   No tension

vertical force eq.

$B_V = F_V - A_V$

$= 1490 - 0$

$= 1490$ Te



$\Sigma T|B = 0: \frac{1}{2}x \cdot F_V - 90 \cdot F_H = 0$

$\frac{1}{2}x \cdot F_V = 90 F_H$

$x = 2 \cdot 90 F_H / F_V$

$= 2 \cdot 90 \cdot 120 / 1490$

$= 14,5$ m

Figure C-70 Hand calculation widening base, translating gantry

C.2.1.2. Skidding systems

No tensile forces in the rear means that the front four skid systems carry all the weight, leading to a vertical force capacity of:

$$F_{V\ skid\ shoe} = \frac{G_{gantry}}{4} = \frac{1490}{4} = 372.5\ Te$$

Teflon pads are used to reduce the friction between the skid beam or shoe and the skid track. The stress on the Teflon pad determines the friction coefficient. Assuming one skid beam or shoe rests on the biggest Teflon pad used within Mammoet, the stress will be:

$$\sigma_{teflon\ pad} = \frac{F_{skid\ shoe} * g}{l_{tp} * w_{tp}} \tag{33}$$

$$\sigma_{teflon\ pad} = \frac{372.5 * 9.81 * 10^3}{400 * 250} = 36.5\ N/mm^2$$

This is more than the 12.5 N/mm² capacity of Teflon pads. Meaning that more than one pad is needed per skid beam or shoe. Looking at their lengths, this should be possible. Assuming that the 12.5 N/mm² stress can be achieved, the friction coefficient can be found using Figure C-71.

Pressure	Coefficient of friction μ
5 N/mm²	0.047
10 N/mm²	0.040
15 N/mm²	0.033
Linear interpolation is possible for intermediate values.	

Figure C-71 Friction coefficient teflon pads [6]

Linear interpolation results in:

$$\mu = \frac{0.040 + 0.033}{2} = 0.0365$$

The push/ pull capacity of each skid beam or shoe should be:

$$F_{H\ skid\ shoe} = 372.5 * 0.0365 = 13.6\ Te$$

C.2.2. SPLITTING TOP

C.2.2.1. Skidding systems

Four skidding systems will pick up the total upper structure. Meaning that each system needs a vertical capacity of:

$$F_{V\ skid\ shoe} = \frac{G_{upper\ structure}}{4} = \frac{450}{4} = 112.5\ Te$$

The stress on the Teflon pad will be:

$$\sigma_{teflon\ pad} = \frac{112.5 * 9.81 * 10^3}{400 * 250} = 11.04\ N/mm^2$$

Which is less than the capacity. Assuming this stress, and linear interpolation on the friction coefficient results in:

$$\mu = (11.04 - 10) * \frac{0.033 - 0.040}{15 - 10} + 0.040 = 0.0385$$

The push/ pull capacity of each skid beam or shoe should be:

$$F_{H\ skid\ shoe} = 112.5 * 0.0385 = 4.34\ Te$$

C.2.2.2. Guidance beam

The minimal section modulus of the guidance beam is calculated in this section.

The load from the upper structure on one cantilevering beam is:

$$\frac{1}{4} * G_{upper\ structure} = \frac{1}{4} * 450 = 112.5\ Te = 1103.6\ kN$$

Because it is conservatively taken that the guidance beams carry the total upper structure. A part of the top frame will not be skidded. Meaning that its weight does not have to be considered. However, the necessary skidding system does add new weight to the equation. Two guidance beams carry one gantry beam (half the upper structure), meaning that one guidance beam carries one fourth.

The length of one cantilevering beam is approximately 9 meters.

To ensure the stability of the total system, both gantry beams must skid simultaneously. At their ultimate position the maximum bending moment will be:

$$M_{guidance\ beam} = 1103.6 * 9 = 9932.4\ kNm$$

When using S355, the section modulus will be at least:

$$W_{guidance\ beam} = \frac{9932.4 * 10^6}{355} = 2.80 * 10^7\ mm^3$$

Note that solely the weight of the upper structure is considered. The self-weight and load factors are not yet considered.

C.2.2.3. Strand jack beam

Table C-38 Gantry beam properties

STRAND JACK BEAM GB-TUB-L800-1		
Property		**Unit**
A_{gb}	$1.056*10^2$	mm^2
h_{gb}	1400	mm
b_{gb}	1600	mm
$t_{f,gb}$	40	mm
$t_{w,gb}$	20	mm
$I_{y,gb}$	$59.7*10^9$	mm^4
$I_{z,gb}$	$7.5*10^9$	mm^4
$W_{el,y,gb}$	$85.3*10^6$	mm^3
$W_{el,z,gb}$	$25.1*10^6$	mm^3
E	210,000	N/mm^2
L_{chord}	8000	mm
f_y	355	N/mm^2
γ_{M0}	1	-
γ_{M1}	1	-
γ_{M2}	1,25	-

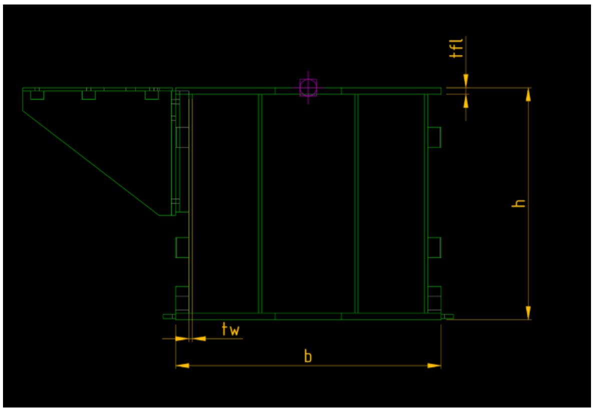


Figure C-72 Strand jack beam dimensions

The load on the strand jack beam is:

$$F_{sjb} = 31{,}808 \text{ kg} = 31.8 \text{ te} = 312.0 \text{ kN}$$

All weights are assumed to be distributed loads.

$$q_{sjb} = \frac{312.0}{8.00} = 39.0\, kN/m$$

Total load (including self-weight) on the strand jack beam:

$$q_{sjb\ tot} = q_{G,sjb} + q_{sjb}$$

$$q_{sjb\ tot} \frac{16.5 * 9.81}{8.00} + 39.0 = 59.23\ \ kN/m$$

The bending moment that will occur:

$$M_{sjb} = \frac{1}{2} * q_{sjb\ tot} * L_{sjb}^2$$

$$M_{sjb} = \frac{1}{2} * 59.23 * 8.00^2 = 1895\, kNm$$

U.C.:

$$U.C. = \frac{\left(\frac{1895}{85.3}\right)}{355} = 0.06$$

ANNEX D.

D.1. GENERAL DESIGN

D.1.1. MEMBERS

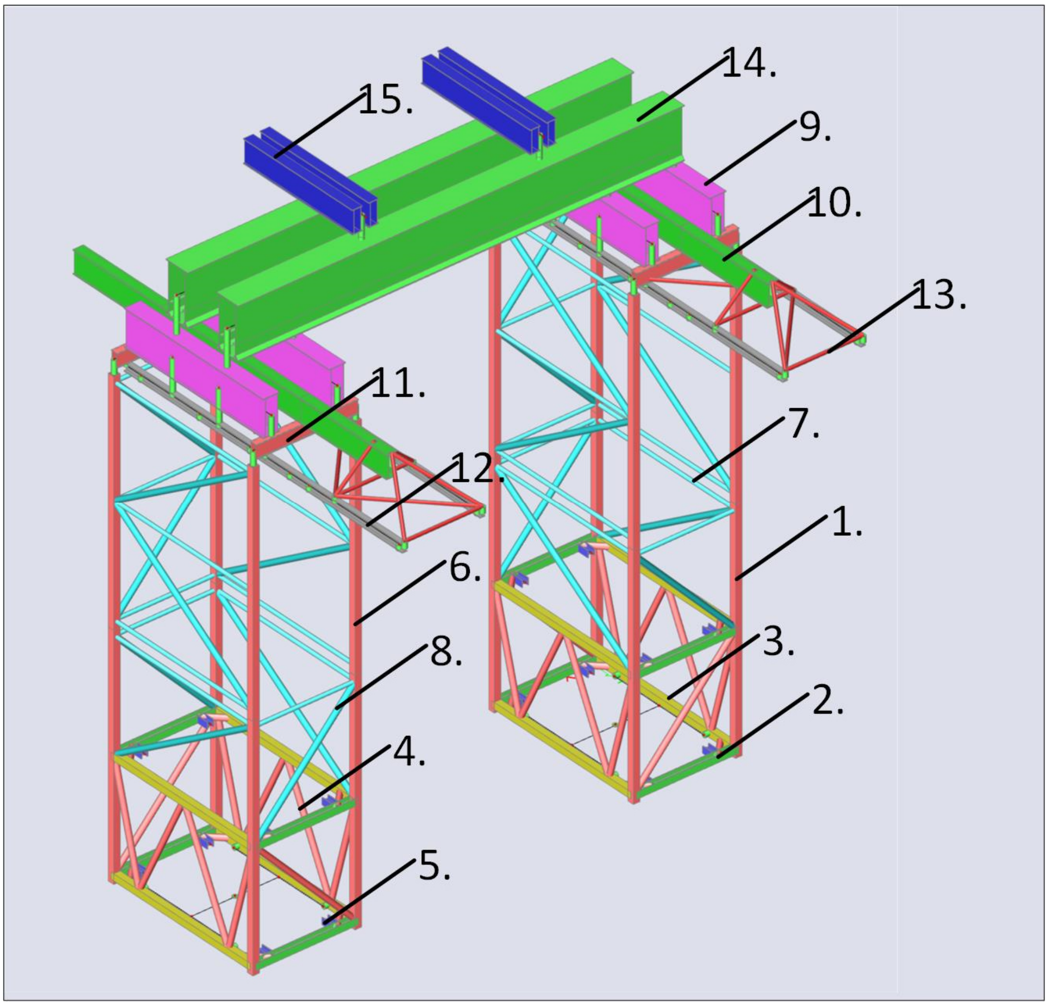


Figure D-73 Members with corresponding numbers

1. Bottom chords
2. Bottom horizontal X brace
3. Bottom horizontal Y brace
4. Bottom diagonal brace
5. Stability beam
6. Top chord
7. Top horizontal brace
8. Top diagonal brace
9. MLS top frame beams
10. Guidance beam
11. Top frame Y
12. Rails
13. Rails diagonal
14. Gantry beam
15. Strand jack beam (tuban beam)

D.1.2. UPPER STRUCTURE SKID

See Annex C.2.1.2 for the explanation of the calculation, a similar problem is done there.

$$F_{V\ skid\ shoe} = 1088\ kN = 110.9\ Te$$

The stress on the Teflon pad will be:

$$\sigma_{teflon\ pad} = \frac{110.9 * 9.81 * 10^3}{400 * 250} = 10.88\ N/mm^2$$

Which is less than the capacity. Assuming this stress, and linear interpolation on the friction coefficient results in:

$$\mu = (10.88 - 10) * \frac{0.033 - 0.040}{15 - 10} + 0.040 = 0.0388$$

The push/ pull capacity of each skid beam or shoe should be:

$$F_{H\ id\ shoe} = 110.9 * 0.0388 = 4.30\ Te$$

D.2. VALIDATION

D.2.1. GLOBAL STABILITY TOTAL GANTRY

The reaction forces, see Annex D.2.7.2, on the governing tower in configuration two imply:

N_{Ed} = 6813 kN
$V_{y,Ed}$ = 342 kN
$V_{z,Ed}$ = 367 kN
$M_{y,Ed}$ = 15,518 kNm
$M_{z,Ed}$ = 7562 kNm

Equations (23) and (24) do not suffice because the shear forces will result in a different bending moment distribution. The following Maple script is used to calculate the Unity Checks for the stability during combined compression and bending moment. The Unity Checks are:

U.C.1 = 0.626
U.C.2 = 0.939

Input

```
> gamma__M1:=1;
> A:=177600; f__y:=690; E:=210000; I__y:=2.8*10^12; I__z:=7.1*10^11; L:=132000; L__buck:=2*L; alpha:=0.49; G__tower:=12*379.65*1000;
  zeta:=3.17;
> N__Ed:=6813000; V__yEd:=342000; V__zEd:=367000; M__ySCIA:=15518*10^6; M__zSCIA:=7562*10^6;
> M__yEd:=V__yEd*L+M__ySCIA; M__zEd:=V__zEd*L+M__zSCIA;
> psi__z:=M__zSCIA/M__zEd; psi__y:=M__ySCIA/M__yEd;
> N__Rk:=4*23805*10^3; M__yRk:=380880*10^6; M__zRk:=190440*10^6;
```

Output

Buckling

```
> N__cry:=Pi^2*E*I__y/L__buck^2; N__crz:=Pi^2*E*I__z/L__buck^2;
> lambda__y:=sqrt(A*f__y/N__cry);Phi__y:=0.5*(1+alpha*(lambda__y-0.2)+lambda__y^2); chi__y:=1/(Phi__y+sqrt(Phi__y^2-lambda__y^2));
> lambda__z:=sqrt(A*f__y/N__crz);Phi__z:=0.5*(1+alpha*(lambda__z-0.2)+lambda__z^2); chi__z:=1/(Phi__z+sqrt(Phi__z^2-lambda__z^2));
> N__cr:=min(N__cry,N__crz);
>
```

Interaction factors

```
> C__my:=max(0.4,0.6+0.4*psi__y); C__mz:=max(0.4,0.6+0.4*psi__z); #C__mLT:=C__my;
> k__yy1:=C__my*(1+0.6*lambda__y*N__Ed/(chi__y*N__Rk/gamma__M1)); k__yy2:=C__my*(1+0.6*N__Ed/(chi__y*N__Rk/gamma__M1));
> k__zz1:=C__mz*(1+0.6*lambda__z*N__Ed/(chi__z*N__Rk/gamma__M1)); k__zz2:=C__mz*(1+0.6*N__Ed/(chi__z*N__Rk/gamma__M1));
```

See which one is bigger, the biggest should be used!

```
> k__yy:=k__yy1; k__zz:=k__zz1;
> k__yz:=k__zz; k__zy:=0.8*k__yy;
```

Bending moment and compression check:

```
> gamma__M1:=1; deltaM__yEd:=0; deltaM__zEd:=0; chi__LT:=1;
> UC1:=N__Ed/((chi__y*N__Rk)/gamma__M1)+k__yy*(M__yEd+deltaM__yEd)/(chi__LT*(M__yRk/gamma__M1))+k__yz*(M__zEd+deltaM__zEd)/(chi__LT*
  (M__zRk/gamma__M1));
> UC2:=N__Ed/((chi__z*N__Rk)/gamma__M1)+k__zy*(M__yEd+deltaM__yEd)/(chi__LT*(M__yRk/gamma__M1))+k__zz*(M__zEd+deltaM__zEd)/(chi__LT*
  (M__zRk/gamma__M1));
>
```

Plug in the compression force and the bending moment, in Newtons and milimeters

```
> #N__Ed:=6749000; V__yEd:=396000; V__zEd:=355000; M__ySCIA:=17375*10^6; M__zSCIA:=6647*10^6;
> #M__yEd:=V__yEd*L+M__ySCIA; M__zEd:=V__zEd*L+M__zSCIA;
> UC1;UC2;
```

D.2.2. STABILITY CLIMBING FRAME

The tolerance between the stability beams and the MLS chords is assumed to be 5 mm. Meaning that an inclination of two times 5 mm over a length of 6.68 m is possible. This is a ratio of 1/668. An imperfection of the climbing frame of 1/500 needs to be considered. Lastly, an imperfection of the tower of 1/500 needs to be considered. The total imperfection is: 1/668 + 1/500 + 1/500 = 1/182. Over a length of 22.04 m (the length from upper stability beam to the top of the climbing cage. This is a horizontal displacement of 121 mm.

The 121 is put in SCIA as a scaling factor for the imperfection shape. But this is the displacement at the top of the climbing cage. The rails deform most, so the actual input for SCIA is 165 and 180 mm.

D.2.3. CLIMBING CAGE BOTTOM

The governing configuration SCIA report will be shown, this applies to all elements.

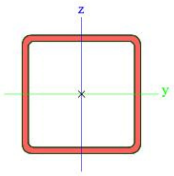
Table D-39 Unity Checks climbing cage bottom

CLIMBING CAGE BOTTOM	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Bottom chords	0.62	0.60	0.62
Bottom horizontal X brace	0.98	0.97	0.98
Bottom horizontal Y brace	0.37	0.37	0.33
Bottom diagonal brace	0.67	0.66	0.67
Stability beams	0.85	0.85	0.80

D.2.3.1. Bottom chords

Cross-sections

Name	Chords
Type	SHS400/400/20.0
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	3,000e+04	
A y, z [mm²]	1,498e+04	1,498e+04
I y, z [mm⁴]	7,154e+08	7,154e+08
I w [mm⁶], t [mm⁴]	1,707e+13	1,125e+09
W_{el} y, z [mm³]	3,577e+06	3,577e+06
W_{pl} y, z [mm³]	4,247e+06	4,247e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	200	200
α [deg]	0,00	
A L, D [m²/m]	1,5500e+00	2,9538e+00
M_{ply} +, - [Nmm]	1,51e+09	1,51e+09
M_{plz} +, - [Nmm]	1,51e+09	1,51e+09

Figure D-74 Bottom chords, properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B742, B747..B749, B923, B928..B930

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B748	7,080 / 14,160 m	SHS400/400/20.0	S 355	All ULS	0,62 -

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 7,080 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-2842	kN
Shear force	$V_{y,Ed}$	-7	kN
Shear force	$V_{z,Ed}$	-27	kN
Torsion	T_{Ed}	-2	kNm
Bending moment	$M_{y,Ed}$	-10	kNm
Bending moment	$M_{z,Ed}$	110	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	340	20	72	124	0,6		1,0	17,0	26,8	30,9	39,7	1
3	I	340	20	126	121	1,0		1,0	17,0	26,8	30,9	34,6	1
5	I	340	20	118	66	0,6		1,0	17,0	26,8	30,9	40,0	1
7	I	340	20	63	68	0,9		1,0	17,0	26,8	30,9	35,0	1

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	3,000e+04	mm^2
Compression resistance	$N_{c,Rd}$	10650	kN
Unity check		0,27	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	4,247e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	1508	kNm
Unity check		0,01	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	4,247e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	1508	kNm
Unity check		0,07	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,500e+04	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3074	kN
Unity check		0,00	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,500e+04	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	3074	kN
Unity check		0,01	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Design plastic moment resistance reduced due to N_{Ed}	$M_{N,y,Rd}$	1442	kNm
Exponent of bending ratio y	α	1,81	
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,z,Rd}$	1442	kNm
Exponent of bending ratio z	β	1,81	

Unity check (6.41) = 0,00 + 0,01 = 0,01 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 12,379 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	340	20	121	140	0,9		1,0	17,0	26,8	30,9	35,8	1
3	I	340	20	137	72	0,5		1,0	17,0	26,8	30,9	40,6	1
5	I	340	20	67	48	0,7		1,0	17,0	26,8	30,9	37,7	1
7	I	340	20	50	116	0,4		1,0	17,0	26,8	30,9	42,0	1

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6,623	6,623	m
Buckling factor	k	1,98	0,69	
Buckling length	l_{cr}	13,124	4,572	m
Critical Euler load	N_{cr}	8609	70933	kN
Slenderness	λ	84,98	29,61	
Relative slenderness	λ_{rel}	1,11	0,39	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	a	
Imperfection	α	0,21	0,21	
Reduction factor	χ	0,59	0,96	
Buckling resistance	$N_{b,Rd}$	6259	10182	kN

Flexural Buckling verification			
Cross-section area	A	3,000e+04	mm^2
Buckling resistance	$N_{b,Rd}$	6259	kN
Unity check		0,45	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	3,000e+04	mm^2
Plastic section modulus	$W_{pl,y}$	4,247e+06	mm^3
Plastic section modulus	$W_{pl,z}$	4,247e+06	mm^3
Design compression force	N_{Ed}	2842	kN

Bending and axial compression check parameters			
Design bending moment (maximum)	$M_{y,Ed}$	-158	kNm
Design bending moment (maximum)	$M_{z,Ed}$	110	kNm
Characteristic compression resistance	N_{Rk}	10650	kN
Characteristic moment resistance	$M_{y,Rk}$	1508	kNm
Characteristic moment resistance	$M_{z,Rk}$	1508	kNm
Reduction factor	χ_y	0,59	
Reduction factor	χ_z	0,96	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,23	
Interaction factor	k_{yz}	0,47	
Interaction factor	k_{zy}	0,74	
Interaction factor	k_{zz}	0,78	

Maximum moment $M_{y,Ed}$ is derived from beam B748 position 13,703 m.
Maximum moment $M_{z,Ed}$ is derived from beam B748 position 7,080 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		line load q	
End moment	$M_{h,z}$	110	kNm
Field moment	$M_{s,z}$	74	kNm
Factor	$\alpha_{s,z}$	0,68	
Ratio of end moments	ψ_z	0,12	
Equivalent moment factor	C_{mz}	0,74	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	-158	kNm
Field moment	$M_{s,LT}$	-112	kNm
Factor	$\alpha_{s,LT}$	0,71	
Ratio of end moments	ψ_{LT}	0,07	
Equivalent moment factor	C_{mLT}	0,77	

Unity check (6.61) = 0,45 + 0,13 + 0,03 = 0,62 -
Unity check (6.62) = 0,28 + 0,08 + 0,06 = 0,41 -

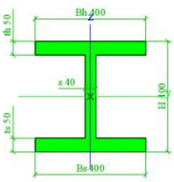
The member satisfies the stability check.

Figure D-75 Bottom chord, SCIA report

D.2.3.2. Bottom horizontal X brace

Cross-sections

Name	Bottom horizontal X brace
Type	I ng
Detailed	400; 400; 400; 50; 50; 40
Item material	S 355
Fabrication	general
Flexural buckling y-y	d
Flexural buckling z-z	d
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm²]	5,200e+04	
A y, z [mm²]	3,891e+04	1,599e+04
I y, z [mm⁴]	1,323e+09	5,349e+08
I w [mm⁶], t [mm⁴]	1,613e+13	3,740e+07
W_{el} y, z [mm³]	6,617e+06	2,675e+06
W_{pl} y, z [mm³]	7,900e+06	4,120e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	200	200
α [deg]	0,00	
A L, D [m²/m]	2,3200e+00	2,3200e+00
M_{ply} +, - [Nmm]	2,80e+09	2,80e+09
M_{plz} +, - [Nmm]	1,46e+09	1,46e+09

Figure D-76 Bottom horizontal X brace properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B743, B745, B754, B756, B924, B926, B935, B937

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B937	**1,100 / 6,200 m**	**I ng (400; 400; 400; 50; 50; 40)**	**S 355**	**All ULS**	**0,98 -**

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		General	

...::SECTION CHECK::...

The critical check is on position 1,100 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-2111	kN
Shear force	$V_{y,Ed}$	41	kN
Shear force	$V_{z,Ed}$	-1558	kN
Torsion	T_{Ed}	4	kNm
Bending moment	$M_{y,Ed}$	-1722	kNm
Bending moment	$M_{z,Ed}$	52	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	200	50	239	258	0,9	0,5	1,0	4,0	7,3	8,1	11,5	1
2	UO	200	50	258	277	0,9	0,4	1,0	4,0	7,3	8,1	11,3	1
3	I	350	40	258	-180	-0,7		0,6	8,8	48,4	55,7	77,7	1
4	UO	200	50	-199	-180								
5	UO	200	50	-180	-160								

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	5,200e+04	mm^2
Compression resistance	$N_{c,Rd}$	18460	kN
Unity check		0,11	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	7,900e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	2805	kNm
Unity check		0,61	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	4,120e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	1463	kNm
Unity check		0,04	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	2	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,01	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.
Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	116	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,57	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	8	
Total torsional moment	τ_{Ed}	5	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,02	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		1	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	41	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	260	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	19	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	320	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	0	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	320	N/mm^2
Unity check		0,90	-

Note: For this section no plastic shear resistance and corresponding Rho value can be determined. Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 1,100 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	200	50	239	258	0,9	0,5	1,0	4,0	7,3	8,1	11,5	1
2	UO	200	50	258	277	0,9	0,4	1,0	4,0	7,3	8,1	11,3	1
3	I	350	40	258	-180	-0,7		0,6	8,8	48,4	55,7	77,7	1
4	UO	200	50	-199	-180								
5	UO	200	50	-180	-160								

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	1,100	6,200	m
Buckling factor	k	2,03	1,00	
Buckling length	l_{cr}	2,232	6,200	m
Critical Euler load	N_{cr}	550326	28844	kN
Slenderness	λ	13,99	61,13	
Relative slenderness	λ_{rel}	0,18	0,80	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		d	d	
Imperfection	α	0,76	0,76	
Reduction factor	χ	1,00	0,58	
Buckling resistance	$N_{b,Rd}$	18460	10702	kN

Flexural Buckling verification			
Cross-section area	A	5,200e+04	mm^2
Buckling resistance	$N_{b,Rd}$	10702	kN
Unity check		0,20	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	6,200	m
Elastic critical load	$N_{cr,T}$	108861	kN
Elastic critical load	$N_{cr,TF}$	28844	kN
Relative slenderness	$\lambda_{rel,T}$	0,80	
Limit slenderness	$\lambda_{rel,0}$	0,20	
Buckling curve		d	
Imperfection	α	0,76	
Reduction factor	χ	0,58	
Cross-section area	A	5,200e+04	mm^2
Buckling resistance	$N_{b,Rd}$	10702	kN
Unity check		0,20	-

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Plastic section modulus	$W_{pl,y}$	7,900e+06	mm^3
Elastic critical moment	M_{cr}	14279	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,44	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	
LTB curve		d	
Imperfection	α_{LT}	0,76	
Reduction factor	χ_{LT}	0,82	
Design buckling resistance	$M_{b,Rd}$	2298	kNm
Unity check		0,75	-

Mcr parameters			
LTB length	L_{LT}	6,200	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,35	
LTB moment factor	C_2	0,63	
LTB moment factor	C_3	0,41	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Warning: Not all conditions of the Dutch NEN-EN NA (Art. NB.NB.1) are fulfilled, therefore the standard EC-EN approach is used.
Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	5,200e+04	mm^2
Plastic section modulus	$W_{pl,y}$	7,900e+06	mm^3
Plastic section modulus	$W_{pl,z}$	4,120e+06	mm^3
Design compression force	N_{Ed}	2111	kN
Design bending moment (maximum)	$M_{y,Ed}$	-1722	kNm
Design bending moment (maximum)	$M_{z,Ed}$	60	kNm
Characteristic compression resistance	N_{Rk}	18460	kN
Characteristic moment resistance	$M_{y,Rk}$	2805	kNm
Characteristic moment resistance	$M_{z,Rk}$	1463	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	0,58	
Reduction factor	χ_{LT}	0,82	
Interaction factor	k_{yy}	0,90	
Interaction factor	k_{yz}	0,68	
Interaction factor	k_{zy}	0,98	
Interaction factor	k_{zz}	1,14	

Maximum moment $M_{y,Ed}$ is derived from beam B937 position 1,100 m.
Maximum moment $M_{z,Ed}$ is derived from beam B937 position 3,100 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.2	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		line load q	
End moment	$M_{h,z}$	0	kNm
Field moment	$M_{s,z}$	60	kNm
Factor	$\alpha_{h,z}$	0,00	
Ratio of end moments	ψ_z	-0,22	
Equivalent moment factor	C_{mz}	0,95	
Resulting load type LT		point load F	
End moment	$M_{h,LT}$	0	kNm
Field moment	$M_{s,LT}$	-1550	kNm
Factor	$\alpha_{h,LT}$	0,00	
Ratio of end moments	ψ_{LT}	0,40	
Equivalent moment factor	C_{mLT}	0,90	

Unity check (6.61) = 0,11 + 0,67 + 0,03 = 0,82 -
Unity check (6.62) = 0,20 + 0,73 + 0,05 = 0,98 -

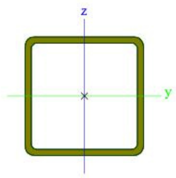
The member satisfies the stability check.

Figure D-77 Bottom horizontal X brace, SCIA report

D.2.3.3. Bottom horizontal Y brace

Cross-sections

Name	Bottom horizontal Y brace
Type	SHS400/400/20.0
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	3,000e+04	
A y, z [mm²]	1,498e+04	1,498e+04
I y, z [mm⁴]	7,154e+08	7,154e+08
I w [mm⁶], t [mm⁴]	1,707e+13	1,125e+09
W_{el} y, z [mm³]	3,577e+06	3,577e+06
W_{pl} y, z [mm³]	4,247e+06	4,247e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	200	200
α [deg]	0,00	
A L, D [m²/m]	1,5500e+00	2,9538e+00
M_{ply} +, - [Nmm]	1,51e+09	1,51e+09
M_{plz} +, - [Nmm]	1,51e+09	1,51e+09

Figure D-78 Bottom horizontal Y brace properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B3, B5, B14, B16, B575, B577, B586, B588

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B14	1,100 / 10,200 m	SHS400/400/20.0	S 355	All ULS	0,37 -

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 1,100 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	1482	kN
Shear force	$V_{y,Ed}$	517	kN
Shear force	$V_{z,Ed}$	-7	kN
Torsion	T_{Ed}	-30	kNm
Bending moment	$M_{y,Ed}$	-3	kNm
Bending moment	$M_{z,Ed}$	565	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	340	20	-183	86	-2,1		0,3	17,0	91,8	105,8	230,8	1
3	I	340	20	101	100	1,0		1,0	17,0	26,8	30,9	34,3	1
5	I	340	20	84	-185	-2,2		0,3	17,0	93,7	108,0	239,3	1
7	I	340	20	-200	-199								

The cross-section is classified as Class 1

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

Cross-section area	A	3,000e+04	mm^2
Plastic tension resistance	$N_{pl,Rd}$	10650	kN
Ultimate tension resistance	$N_{u,Rd}$	11016	kN
Tension resistance	$N_{t,Rd}$	10650	kN
Unity check		0,14	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	4,247e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	1508	kNm
Unity check		0,00	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	4,247e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	1508	kNm
Unity check		0,37	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,500e+04	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3074	kN
Unity check		0,17	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,500e+04	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	3074	kN
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	5	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,03	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Design plastic moment resistance reduced due to N_{Ed}	$M_{N,y,Rd}$	1508	kNm
Exponent of bending ratio y	α	1,70	
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,z,Rd}$	1508	kNm
Exponent of bending ratio z	β	1,70	

Unity check (6.41) = 0,00 + 0,19 = 0,19 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 1,100 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	340	20	-183	86	-2,1		0,3	17,0	91,8	105,8	230,8	1
3	I	340	20	101	100	1,0		1,0	17,0	26,8	30,9	34,3	1
5	I	340	20	84	-185	-2,2		0,3	17,0	93,7	108,0	239,3	1
7	I	340	20	-200	-199								

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

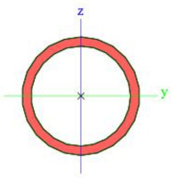
The member satisfies the stability check.

Figure D-79 Bottom horizontal y brace, SCIA report

D.2.3.4. Bottom diagonal brace

Cross-sections

Name	Bottom diagonal brace
Type	CHS323.9/25.0
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	2,350e+04	
A y, z [mm²]	1,495e+04	1,495e+04
I y, z [mm⁴]	2,640e+08	2,640e+08
I w [mm⁶], t [mm⁴]	4,466e-21	5,280e+08
W_{el} y, z [mm³]	1,630e+06	1,630e+06
W_{pl} y, z [mm³]	2,203e+06	2,203e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	162	162
α [deg]	0,00	
A L, D [m²/m]	1,0200e+00	1,8779e+00
M_{ply} +, - [Nmm]	7,95e+08	7,95e+08
M_{plz} +, - [Nmm]	7,95e+08	7,95e+08

Figure D-80 Bottom diagonal brace properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B758, B759, B834..B837, B839..B842, B847..B854, B939, B940, B1015..B1018, B1020..B1023, B1028..B1035

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B834	6,923 / 6,923 m	CHS323.9/25.0	S 355	All ULS	0,67 -

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 6,923 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-2849	kN
Shear force	$V_{y,Ed}$	15	kN
Shear force	$V_{z,Ed}$	-7	kN
Torsion	T_{Ed}	4	kNm
Bending moment	$M_{y,Ed}$	0	kNm
Bending moment	$M_{z,Ed}$	226	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
324	25	13,0	33,1	46,3	59,6	1

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	2,350e+04	mm^2
Compression resistance	$N_{c,Rd}$	8343	kN
Unity check		0,34	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	2,203e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	782	kNm
Unity check		0,00	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	2,203e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	782	kNm
Unity check		0,29	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,496e+04	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3066	kN
Unity check		0,00	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,496e+04	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	3066	kN
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	1	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Resultant bending moment	$M_{resultant}$	226	kNm
Resultant shear force	$V_{resultant}$	16	kN
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,Rd}$	656	kNm
Unity check		0,34	-

Note: The resultant internal forces are used for CHS sections.
Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
324	25	13,0	33,1	46,3	59,6	1

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6,923	6,923	m
Buckling factor	k	1,00	0,77	
Buckling length	l_{cr}	6,923	5,315	m
Critical Euler load	N_{cr}	11415	19371	kN
Slenderness	λ	65,32	50,14	
Relative slenderness	λ_{rel}	0,85	0,66	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	a	
Imperfection	α	0,21	0,21	
Reduction factor	χ	0,76	0,87	
Buckling resistance	$N_{b,Rd}$	6364	7236	kN

Flexural Buckling verification			
Cross-section area	A	2,350e+04	mm^2
Buckling resistance	$N_{b,Rd}$	6364	kN
Unity check		0,45	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a CHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns a CHS section which is not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	2,350e+04	mm^2
Plastic section modulus	$W_{pl,y}$	2,203e+06	mm^3
Plastic section modulus	$W_{pl,z}$	2,203e+06	mm^3
Design compression force	N_{Ed}	2849	kN
Design bending moment (maximum)	$M_{y,Ed}$	12	kNm
Design bending moment (maximum)	$M_{z,Ed}$	226	kNm
Characteristic compression resistance	N_{Rk}	8343	kN
Characteristic moment resistance	$M_{y,Rk}$	782	kNm
Characteristic moment resistance	$M_{z,Rk}$	782	kNm
Reduction factor	χ_y	0,76	

Bending and axial compression check parameters			
Reduction factor	χ_z	0,87	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,16	
Interaction factor	k_{yz}	0,55	
Interaction factor	k_{zy}	0,70	
Interaction factor	k_{zz}	0,92	

Maximum moment $M_{y,Ed}$ is derived from beam B834 position 3,462 m.
Maximum moment $M_{z,Ed}$ is derived from beam B834 position 6,923 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		line load q	
End moment	$M_{h,z}$	226	kNm
Field moment	$M_{s,z}$	146	kNm
Factor	$\alpha_{s,z}$	0,64	
Ratio of end moments	ψ_z	-0,27	
Equivalent moment factor	C_{mz}	0,72	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	0	kNm
Field moment	$M_{s,LT}$	12	kNm
Factor	$\alpha_{h,LT}$	0,00	
Ratio of end moments	ψ_{LT}	0,03	
Equivalent moment factor	C_{mLT}	0,95	

Unity check (6.61) = 0,45 + 0,02 + 0,16 = 0,62 -
Unity check (6.62) = 0,39 + 0,01 + 0,26 = 0,67 -

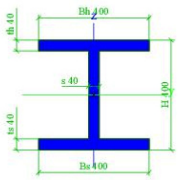
The member satisfies the stability check.

Figure D-81 Bottom diagonal brace, SCIA report

D.2.3.5. Stability beams

Cross-sections

Name	Stability beams
Type	I ng
Detailed	400; 400; 400; 40; 40; 40
Item material	S 355
Fabrication	general
Flexural buckling y-y	d
Flexural buckling z-z	d
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm²]	4,480e+04	
A y, z [mm²]	3,153e+04	1,579e+04
I y, z [mm⁴]	1,150e+09	4,284e+08
I w [mm⁶], t [mm⁴]	1,362e+13	2,309e+07
W_{el} y, z [mm³]	5,751e+06	2,142e+06
W_{pl} y, z [mm³]	6,784e+06	3,328e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	200	200
α [deg]	0,00	
A L, D [m²/m]	2,3200e+00	2,3200e+00
M_{ply} +, - [Nmm]	2,41e+09	2,41e+09
M_{plz} +, - [Nmm]	1,18e+09	1,18e+09

Figure D-82 Stability beam properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B25, B39, B379, B381, B383, B387, B389, B391, B595, B596, B729, B731, B733, B737, B739, B741

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B387	**0,000 / 0,700 m**	**I ng (400; 400; 400; 40; 40; 40)**	**S 355**	**All ULS**	**0,85 -**

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		General	

...::SECTION CHECK::...

The critical check is on position 0,000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-2449	kN
Shear force	$V_{y,Ed}$	25	kN
Shear force	$V_{z,Ed}$	1781	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	-1251	kNm
Bending moment	$M_{z,Ed}$	-15	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	200	40	248	241	1,0	0,4	1,0	5,0	7,3	8,1	11,2	1
2	UO	200	40	241	234	1,0	0,4	1,0	5,0	7,3	8,1	11,3	1
3	I	360	40	241	-135	-0,6		0,6	9,0	44,0	50,7	70,5	1
4	UO	200	40	-128	-135								
5	UO	200	40	-135	-143								

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	4,480e+04	mm^2
Compression resistance	$N_{c,Rd}$	15904	kN
Unity check		0,15	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	6,784e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	2408	kNm
Unity check		0,52	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	3,328e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	1181	kNm
Unity check		0,01	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	1	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,01	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.
Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	131	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,64	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	8	
Total torsional moment	τ_{Ed}	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		13	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	55	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	174	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	1	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	229	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	1	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	113	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	114	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	302	N/mm^2
Unity check		0,85	-

Note: For this section no plastic shear resistance and corresponding Rho value can be determined. Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 $[N/mm^2]$	σ_2 $[N/mm^2]$	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	200	40	248	241	1,0	0,4	1,0	5,0	7,3	8,1	11,2	1
2	UO	200	40	241	234	1,0	0,4	1,0	5,0	7,3	8,1	11,3	1
3	I	360	40	241	-135	-0,6		0,6	9,0	44,0	50,7	70,5	1
4	UO	200	40	-128	-135								
5	UO	200	40	-135	-143								

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	0,700	0,700	m
Buckling factor	k	3,18	1,00	
Buckling length	l_{cr}	2,225	0,700	m
Critical Euler load	N_{cr}	481373	1812043	kN
Slenderness	λ	13,89	7,16	
Relative slenderness	λ_{rel}	0,18	0,09	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	0,700	m
Elastic critical load	$N_{cr,T}$	1687813	kN
Elastic critical load	$N_{cr,TF}$	481373	kN
Relative slenderness	$\lambda_{rel,T}$	0,18	
Limit slenderness	$\lambda_{rel,0}$	0,20	

Note: The slenderness or compression force is such that Torsional(-Flexural) Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Plastic section modulus	$W_{pl,y}$	6,784e+06	mm^3
Elastic critical moment	M_{cr}	581051	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,06	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	L_{LT}	0,700	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,77	
LTB moment factor	C_2	0,00	
LTB moment factor	C_3	1,00	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Warning: Not all conditions of the Dutch NEN-EN NA (Art. NB.NB.1) are fulfilled, therefore the standard EC-EN approach is used.
Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	4,480e+04	mm^2
Plastic section modulus	$W_{pl,y}$	6,784e+06	mm^3
Plastic section modulus	$W_{pl,z}$	3,328e+06	mm^3
Design compression force	N_{Ed}	2449	kN
Design bending moment (maximum)	$M_{y,Ed}$	-1251	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-15	kNm
Characteristic compression resistance	N_{Rk}	15904	kN
Characteristic moment resistance	$M_{y,Rk}$	2408	kNm
Characteristic moment resistance	$M_{z,Rk}$	1181	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	1,00	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	0,90	
Interaction factor	k_{yz}	0,31	
Interaction factor	k_{zy}	0,69	
Interaction factor	k_{zz}	0,51	

Maximum moment $M_{y,Ed}$ is derived from beam B387 position 0,000 m.
Maximum moment $M_{z,Ed}$ is derived from beam B387 position 0,000 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.2	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		line load q	
End moment	$M_{h,z}$	-15	kNm
Field moment	$M_{s,z}$	-6	kNm
Factor	$\alpha_{s,z}$	0,39	
Ratio of end moments	ψ_z	0,00	
Equivalent moment factor	C_{mz}	0,51	
Resulting load type LT		linear moment M	
Ratio of end moments	ψ_{LT}	0,00	
Equivalent moment factor	C_{mLT}	0,60	

Unity check (6.61) = 0,15 + 0,47 + 0,00 = 0,63 -
Unity check (6.62) = 0,15 + 0,36 + 0,01 = 0,52 -

The member satisfies the stability check.

Figure D-83 Stability beam, SCIA report

D.2.4. CLIMBING CAGE TOP

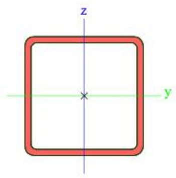
Table D-40 Unity Checks climbing cage top

CLIMBING CAGE TOP	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Top chords	0.81	0.73	0.81
Top horizontal brace	0.39	0.34	0.39
Top diagonal brace	0.58	0.43	0.58

D.2.4.1. Top chords

Cross-sections

Name	Chords
Type	SHS400/400/20.0
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	3,000e+04	
A y, z [mm²]	1,498e+04	1,498e+04
I y, z [mm⁴]	7,154e+08	7,154e+08
I w [mm⁶], t [mm⁴]	1,707e+13	1,125e+09
W_{el} y, z [mm³]	3,577e+06	3,577e+06
W_{pl} y, z [mm³]	4,247e+06	4,247e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	200	200
α [deg]	0,00	
A L, D [m²/m]	1,5500e+00	2,9538e+00
M_{ply} +, - [Nmm]	1,51e+09	1,51e+09
M_{plz} +, - [Nmm]	1,51e+09	1,51e+09

Figure D-84 Top chord properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B766..B769, B947..B950

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B950	0,400 / 14,160 m	SHS400/400/20.0	S 355	All ULS	0,81 -

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm²
Ultimate strength	f_u	510	N/mm²
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 0,400 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-2408	kN
Shear force	$V_{y,Ed}$	2	kN
Shear force	$V_{z,Ed}$	1	kN
Torsion	T_{Ed}	26	kNm
Bending moment	$M_{y,Ed}$	-46	kNm
Bending moment	$M_{z,Ed}$	6	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	340	20	91	94	1,0		1,0	17,0	26,8	30,9	34,5	1
3	I	340	20	93	71	0,8		1,0	17,0	26,8	30,9	37,1	1
5	I	340	20	70	67	1,0		1,0	17,0	26,8	30,9	34,7	1
7	I	340	20	68	90	0,8		1,0	17,0	26,8	30,9	37,2	1

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	3,000e+04	mm²
Compression resistance	$N_{c,Rd}$	10650	kN
Unity check		0,23	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	4,247e+06	mm³
Plastic bending moment	$M_{pl,y,Rd}$	1508	kNm
Unity check		0,03	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	4,247e+06	mm³
Plastic bending moment	$M_{pl,z,Rd}$	1508	kNm
Unity check		0,00	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,500e+04	mm²
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3074	kN
Unity check		0,00	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	1,500e+04	mm²
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	3074	kN
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	4	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,02	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Design plastic moment resistance reduced due to N_{Ed}	$M_{N,y,Rd}$	1508	kNm
Exponent of bending ratio y	α	1,76	
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,z,Rd}$	1508	kNm
Exponent of bending ratio z	β	1,76	

Unity check (6.41) = 0,00 + 0,00 = 0,00 -

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,457 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	340	20	91	94	1,0		1,0	17,0	26,8	30,9	34,5	1
3	I	340	20	93	71	0,8		1,0	17,0	26,8	30,9	37,1	1
5	I	340	20	70	67	1,0		1,0	17,0	26,8	30,9	34,7	1
7	I	340	20	68	90	0,8		1,0	17,0	26,8	30,9	37,2	1

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6,680	14,160	m
Buckling factor	k	2,96	1,00	
Buckling length	l_{cr}	19,744	14,160	m
Critical Euler load	N_{cr}	3804	7395	kN
Slenderness	λ	127,86	91,69	
Relative slenderness	λ_{rel}	1,67	1,20	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	a	
Imperfection	α	0,21	0,21	
Reduction factor	χ	0,31	0,53	
Buckling resistance	$N_{b,Rd}$	3280	5644	kN

Flexural Buckling verification			
Cross-section area	A	3,000e+04	mm²
Buckling resistance	$N_{b,Rd}$	3280	kN
Unity check		0,73	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	3,000e+04	mm²
Plastic section modulus	$W_{pl,y}$	4,247e+06	mm³
Plastic section modulus	$W_{pl,z}$	4,247e+06	mm³
Design compression force	N_{Ed}	2408	kN

Bending and axial compression check parameters			
Design bending moment (maximum)	$M_{y,Ed}$	-46	kNm
Design bending moment (maximum)	$M_{z,Ed}$	57	kNm
Characteristic compression resistance	N_{Rk}	10650	kN
Characteristic moment resistance	$M_{y,Rk}$	1508	kNm
Characteristic moment resistance	$M_{z,Rk}$	1508	kNm
Reduction factor	χ_y	0,31	
Reduction factor	χ_z	0,53	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,43	
Interaction factor	k_{yz}	0,72	
Interaction factor	k_{zy}	0,86	
Interaction factor	k_{zz}	1,21	

Maximum moment $M_{y,Ed}$ is derived from beam B950 position 0,400 m.
Maximum moment $M_{z,Ed}$ is derived from beam B950 position 7,742 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	0	kNm
Field moment	$M_{s,z}$	57	kNm
Factor	$\alpha_{h,z}$	0,00	
Ratio of end moments	ψ_z	1,00	
Equivalent moment factor	C_{mz}	0,90	
Resulting load type LT		point load F	
End moment	$M_{h,LT}$	0	kNm
Field moment	$M_{s,LT}$	106	kNm
Factor	$\alpha_{h,LT}$	0,00	
Ratio of end moments	ψ_{LT}	1,00	
Equivalent moment factor	C_{mLT}	0,90	

Unity check (6.61) = 0,73 + 0,04 + 0,03 = 0,81 -
Unity check (6.62) = 0,43 + 0,03 + 0,05 = 0,50 -

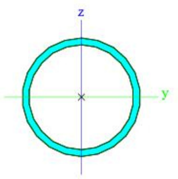
The member satisfies the stability check.

Figure D-85 Top chord, SCIA report

D.2.4.2. Top horizontal brace

Cross-sections

Name	Top horizontal brace
Type	CHS219.1/12.5
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm^2]	8,110e+03	
A y, z [mm^2]	5,165e+03	5,165e+03
I y, z [mm^4]	4,345e+07	4,345e+07
I w [mm^6], t [mm^4]	8,878e-22	8,689e+07
W_{el} y, z [mm^3]	3,970e+05	3,970e+05
W_{pl} y, z [mm^3]	5,258e+05	5,258e+05
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	110	110
α [deg]	0,00	
A L, D [m^2/m]	6,8800e-01	1,2980e+00
M_{ply} +, - [Nmm]	1,90e+08	1,90e+08
M_{plz} +, - [Nmm]	1,90e+08	1,90e+08

Figure D-86 Top horizontal brace properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B750..B753, B770..B778, B931..B934, B951..B959

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B770	**10,200 / 10,200 m**	**CHS219.1/12.5**	**S 355**	**All ULS**	**0,39 -**

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 10,200 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-144	kN
Shear force	$V_{y,Ed}$	3	kN
Shear force	$V_{z,Ed}$	-10	kN
Torsion	T_{Ed}	-8	kNm
Bending moment	$M_{y,Ed}$	0	kNm
Bending moment	$M_{z,Ed}$	26	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
219	13	17,5	33,1	46,3	59,6	1

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	8,110e+03	mm^2
Compression resistance	$N_{c,Rd}$	2879	kN
Unity check		0,05	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	5,258e+05	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	187	kNm
Unity check		0,00	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	5,258e+05	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	187	kNm
Unity check		0,14	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	5,163e+03	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1058	kN
Unity check		0,00	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	5,163e+03	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1058	kN
Unity check		0,01	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	10	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,05	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Resultant bending moment	$M_{resultant}$	26	kNm
Resultant shear force	$V_{resultant}$	11	kN
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,Rd}$	186	kNm
Unity check		0,14	-

Note: The resultant internal forces are used for CHS sections.
Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
219	13	17,5	33,1	46,3	59,6	1

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	10,200	10,200	m
Buckling factor	k	1,00	0,68	
Buckling length	l_{cr}	10,200	6,969	m
Critical Euler load	N_{cr}	866	1854	kN
Slenderness	λ	139,35	95,21	
Relative slenderness	λ_{rel}	1,82	1,25	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	a	
Imperfection	α	0,21	0,21	
Reduction factor	χ	0,26	0,50	
Buckling resistance	$N_{b,Rd}$	760	1444	kN

Flexural Buckling verification			
Cross-section area	A	8,110e+03	mm^2
Buckling resistance	$N_{b,Rd}$	760	kN
Unity check		0,19	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a CHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns a CHS section which is not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	8,110e+03	mm^2
Plastic section modulus	$W_{pl,y}$	5,258e+05	mm^3
Plastic section modulus	$W_{pl,z}$	5,258e+05	mm^3
Design compression force	N_{Ed}	144	kN
Design bending moment (maximum)	$M_{y,Ed}$	26	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-27	kNm
Characteristic compression resistance	N_{Rk}	2879	kN
Characteristic moment resistance	$M_{y,Rk}$	187	kNm
Characteristic moment resistance	$M_{z,Rk}$	187	kNm
Reduction factor	χ_y	0,26	

Bending and axial compression check parameters			
Reduction factor	χ_z	0,50	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,04	
Interaction factor	k_{yz}	0,40	
Interaction factor	k_{zy}	0,62	
Interaction factor	k_{zz}	0,66	

Maximum moment $M_{y,Ed}$ is derived from beam B770 position 5,100 m.
Maximum moment $M_{z,Ed}$ is derived from beam B770 position 0,000 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	-27	kNm
Field moment	$M_{s,z}$	13	kNm
Factor	$\alpha_{s,z}$	-0,48	
Ratio of end moments	ψ_z	-0,97	
Equivalent moment factor	C_{mz}	0,58	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	0	kNm
Field moment	$M_{s,LT}$	26	kNm
Factor	$\alpha_{h,LT}$	0,00	
Ratio of end moments	ψ_{LT}	0,14	
Equivalent moment factor	C_{mLT}	0,95	

Unity check (6.61) = 0,19 + 0,15 + 0,06 = 0,39 -
Unity check (6.62) = 0,10 + 0,09 + 0,09 = 0,28 -

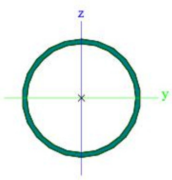
The member satisfies the stability check.

Figure D-87 Top horizontal brace, SCIA report

D.2.4.3. Top diagonal brace

Cross-sections

Name	Top diagonal brace
Type	CHS323.9/12.5
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm²]	1,220e+04	
A y, z [mm²]	8,152e+03	8,152e+03
I y, z [mm⁴]	1,485e+08	1,485e+08
I w [mm⁶], t [mm⁴]	1,171e-02	2,907e+08
W_{el} y, z [mm³]	9,170e+05	9,170e+05
W_{pl} y, z [mm³]	1,194e+06	1,194e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	162	162
α [deg]	0,00	
A L, D [m²/m]	1,0200e+00	1,9565e+00
M_{ply} +, - [Nmm]	4,30e+08	4,30e+08
M_{plz} +, - [Nmm]	4,30e+08	4,30e+08

Figure D-88 Top diagonal brace properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B760..B763, B779..B784, B941..B944, B960..B965

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B783	0,000 / 12,193 m	CHS323.9/12.5	S 355	All ULS	0,58 -

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 0,000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-600	kN
Shear force	$V_{y,Ed}$	-8	kN
Shear force	$V_{z,Ed}$	17	kN
Torsion	T_{Ed}	-12	kNm
Bending moment	$M_{y,Ed}$	0	kNm
Bending moment	$M_{z,Ed}$	13	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
324	13	25,9	33,1	46,3	59,6	1

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1,220e+04	mm^2
Compression resistance	$N_{c,Rd}$	4331	kN
Unity check		0,14	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	1,194e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	424	kNm
Unity check		0,03	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	7,767e+03	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1592	kN
Unity check		0,01	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	7,767e+03	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1592	kN
Unity check		0,01	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	15	
Total torsional moment	τ_{Ed}	7	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,03	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Resultant bending moment	$M_{resultant}$	13	kNm
Resultant shear force	$V_{resultant}$	19	kN
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,Rd}$	409	kNm
Unity check		0,03	-

Note: The resultant internal forces are used for CHS sections.
Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
324	13	25,9	33,1	46,3	59,6	1

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	12,193	12,193	m
Buckling factor	k	1,00	0,78	
Buckling length	l_{cr}	12,193	9,521	m
Critical Euler load	N_{cr}	2070	3395	kN
Slenderness	λ	110,51	86,30	
Relative slenderness	λ_{rel}	1,45	1,13	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	a	
Imperfection	α	0,21	0,21	
Reduction factor	χ	0,40	0,58	
Buckling resistance	$N_{b,Rd}$	1715	2495	kN

Flexural Buckling verification			
Cross-section area	A	1,220e+04	mm^2
Buckling resistance	$N_{b,Rd}$	1715	kN
Unity check		0,35	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a CHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns a CHS section which is not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	1,220e+04	mm^2
Plastic section modulus	$W_{pl,y}$	1,194e+06	mm^3
Plastic section modulus	$W_{pl,z}$	1,194e+06	mm^3
Design compression force	N_{Ed}	600	kN
Design bending moment (maximum)	$M_{y,Ed}$	55	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-43	kNm
Characteristic compression resistance	N_{Rk}	4331	kN
Characteristic moment resistance	$M_{y,Rk}$	424	kNm
Characteristic moment resistance	$M_{z,Rk}$	424	kNm
Reduction factor	χ_y	0,40	
Reduction factor	χ_z	0,58	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,15	
Interaction factor	k_{yz}	0,77	
Interaction factor	k_{zy}	0,69	
Interaction factor	k_{zz}	1,29	

Maximum moment $M_{y,Ed}$ is derived from beam B783 position 6,096 m.
Maximum moment $M_{z,Ed}$ is derived from beam B783 position 9,754 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		line load q	
End moment	$M_{h,z}$	-39	kNm
Field moment	$M_{s,z}$	-37	kNm
Factor	$\alpha_{s,z}$	0,95	
Ratio of end moments	ψ_z	-0,33	
Equivalent moment factor	C_{mz}	0,96	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	0	kNm
Field moment	$M_{s,LT}$	55	kNm
Factor	$\alpha_{h,LT}$	0,00	
Ratio of end moments	ψ_{LT}	0,43	
Equivalent moment factor	C_{mLT}	0,95	

Unity check (6.61) = 0,35 + 0,15 + 0,08 = 0,58 -
Unity check (6.62) = 0,24 + 0,09 + 0,13 = 0,46 -

The member satisfies the stability check.

Figure D-89 Top diagonal brace, SCIA report

D.2.5. TOP FRAME

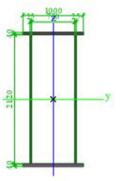
Table D-41 Unity Checks top frame

TOP FRAME	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
MLS top frame beam	0.06	0.06	0.02
Guidance beam	0.73	0.46	0.73
Top frame Y	0.91	0.81	0.91

D.2.5.1. MLS top frame beam

Cross-sections

Name	MLS Topframe beam
Type	General cross-section
Item material	S 690
Fabrication	general
Flexural buckling y-y	d
Flexural buckling z-z	d
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm^2]	1,860e+05	
A y, z [mm^2]	6,432e+04	1,068e+05
I y, z [mm^4]	1,330e+11	2,138e+10
I w [mm^6], t [mm^4]	5,974e+15	4,781e+10
W_{el} y, z [mm^3]	1,209e+08	4,276e+07
W_{pl} y, z [mm^3]	1,426e+08	5,949e+07
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	0	0
α [deg]	0,00	
A L, D [m^2/m]	6,8600e+00	1,2540e+01
M_{ply} +, - [Nmm]	9,84e+10	9,84e+10
M_{plz} +, - [Nmm]	4,10e+10	4,10e+10

Figure D-90 MLS top frame properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B75, B76, B622, B623

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B623	**6,950 / 10,200 m**	**General cross-section**	**S 690**	**All ULS**	**0,06 -**

Combination key
All ULS / NC3 Operation pos 2

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	690	N/mm^2
Ultimate strength	f_u	770	N/mm^2
Fabrication		General	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

The critical check is on position 6,950 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-181	kN
Shear force	$V_{y,Ed}$	58	kN
Shear force	$V_{z,Ed}$	-1426	kN
Torsion	T_{Ed}	125	kNm
Bending moment	$M_{y,Ed}$	4653	kNm
Bending moment	$M_{z,Ed}$	-53	kNm

Classification for cross-section design
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1,860e+05	mm^2
Compression resistance	$N_{c,Rd}$	128340	kN
Unity check		0,00	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	1,209e+08	mm^3
Elastic bending moment	$M_{el,y,Rd}$	83442	kNm
Unity check		0,06	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	4,276e+07	mm^3
Elastic bending moment	$M_{el,z,Rd}$	29505	kNm
Unity check		0,00	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	1	N/mm^2
Elastic shear resistance	τ_{Rd}	398	N/mm^2
Unity check		0,00	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.
Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	15	N/mm^2
Elastic shear resistance	τ_{Rd}	398	N/mm^2
Unity check		0,04	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	22	
Total torsional moment	τ_{Ed}	2	N/mm²
Elastic shear resistance	τ_{Rd}	398	N/mm²
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		12	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	1	N/mm²
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	37	N/mm²
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	1	N/mm²
Total longitudinal stress	$\sigma_{tot,Ed}$	39	N/mm²
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm²
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	9	N/mm²
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm²
Total shear stress	$\tau_{tot,Ed}$	9	N/mm²
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	42	N/mm²
Unity check		0,06	-

The member satisfies the section check.

...::STABILITY CHECK::...

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	2,150	3,400	m
Buckling factor	k	6,64	1,00	
Buckling length	l_{cr}	14,271	3,393	m
Critical Euler load	N_{cr}	1353840	3848717	kN
Slenderness	λ	16,87	10,01	
Relative slenderness	λ_{rel}	0,31	0,18	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	3,400	m
Elastic critical load	$N_{cr,T}$	5942526	kN
Elastic critical load	$N_{cr,TF}$	1353840	kN
Relative slenderness	$\lambda_{rel,T}$	0,31	
Limit slenderness	$\lambda_{rel,0}$	0,20	

Note: The slenderness or compression force is such that Torsional(-Flexural) Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Elastic section modulus	$W_{el,y}$	1,209e+08	mm³
Elastic critical moment	M_{cr}	7244247	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,11	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	L_{LT}	3,400	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,67	
LTB moment factor	C_2	0,05	

Mcr parameters			
LTB moment factor	C_3	1,00	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Warning: Not all conditions of the Dutch NEN-EN NA (Art. NB.NB.1) are fulfilled, therefore the standard EC-EN approach is used.
Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	1,860e+05	mm^2
Elastic section modulus	$W_{el,y}$	1,209e+08	mm^3
Elastic section modulus	$W_{el,z}$	4,276e+07	mm^3
Design compression force	N_{Ed}	181	kN
Design bending moment (maximum)	$M_{y,Ed}$	4653	kNm
Design bending moment (maximum)	$M_{z,Ed}$	145	kNm
Characteristic compression resistance	N_{Rk}	128340	kN
Characteristic moment resistance	$M_{y,Rk}$	83442	kNm
Characteristic moment resistance	$M_{z,Rk}$	29505	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	1,00	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	0,90	
Interaction factor	k_{yz}	0,40	
Interaction factor	k_{zy}	1,00	
Interaction factor	k_{zz}	0,40	

Maximum moment $M_{y,Ed}$ is derived from beam B623 position 6,950 m.
Maximum moment $M_{z,Ed}$ is derived from beam B623 position 10,200 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.2	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	145	kNm
Field moment	$M_{s,z}$	-40	kNm
Factor	$\alpha_{s,z}$	-0,28	
Ratio of end moments	ψ_z	0,34	
Equivalent moment factor	C_{mz}	0,40	
Resulting load type LT		point load F	
End moment	$M_{h,LT}$	4472	kNm
Field moment	$M_{s,LT}$	4345	kNm
Factor	$\alpha_{s,LT}$	0,97	
Ratio of end moments	ψ_{LT}	-0,04	
Equivalent moment factor	C_{mLT}	0,98	

Unity check (6.61) = 0,00 + 0,05 + 0,00 = 0,05 -
Unity check (6.62) = 0,00 + 0,06 + 0,00 = 0,06 -

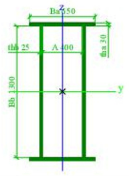
The member satisfies the stability check.

Figure D-91 MLS top frame, SCIA report

D.2.5.2. Guidance beam

Cross-sections

Name	Guidance beam
Type	Box fl
Detailed	650; 30; 1300; 25; 400
Item material	S 355
Fabrication	welded
Flexural buckling y-y	b
Flexural buckling z-z	b
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	1,040e+05	
A y, z [mm²]	3,022e+04	6,571e+04
I y, z [mm⁴]	2,640e+10	4,312e+09
I w [mm⁶], t [mm⁴]	4,040e+14	9,511e+09
W_{el} y, z [mm³]	3,883e+07	1,327e+07
W_{pl} y, z [mm³]	4,706e+07	2,015e+07
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	325	680
α [deg]	0,00	
A L, D [m²/m]	4,4200e+00	7,8200e+00
M_{ply} +, - [Nmm]	1,67e+10	1,67e+10
M_{plz} +, - [Nmm]	7,15e+09	7,15e+09

Figure D-92 Guidance beam properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B818, B819, B821, B999, B1000, B1002

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B999	**10,200 / 10,200 m**	**Box fl (650; 30; 1300; 25; 400)**	**S 355**	**All ULS**	**0,73 -**

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Welded	

...::SECTION CHECK::...

The critical check is on position 10,200 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	960	kN
Shear force	$V_{y,Ed}$	132	kN
Shear force	$V_{z,Ed}$	-293	kN
Torsion	T_{Ed}	198	kNm
Bending moment	$M_{y,Ed}$	-7766	kNm
Bending moment	$M_{z,Ed}$	670	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	112	30	-150	-168								
2	I	1300	25	-228	145	-1,6		0,4	52,0	75,4	86,9	162,9	1
3	I	1300	25	-163	210	-0,8		0,6	52,0	51,0	58,7	82,6	2
4	UO	112	30	232	214	0,9	0,4	1,0	3,7	7,3	8,1	11,3	1
5	UO	112	30	-233	-250								
6	I	15	25	-228	-233								
7	UO	112	30	149	132	0,9	0,5	1,0	3,7	7,3	8,1	11,7	1
8	I	15	25	145	149	1,0		1,0	0,6	26,8	30,9	34,5	1
9	I	425	30	-168	-233								
10	I	15	25	-163	-168								
11	I	425	30	214	149	0,7		1,0	14,2	26,8	30,9	38,0	1
12	I	15	25	210	214	1,0		1,0	0,6	26,8	30,9	34,4	1

The cross-section is classified as Class 2

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

Cross-section area	A	1,040e+05	mm^2
Plastic tension resistance	$N_{pl,Rd}$	36920	kN
Ultimate tension resistance	$N_{u,Rd}$	38189	kN
Tension resistance	$N_{t,Rd}$	36920	kN
Unity check		0,03	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	4,706e+07	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	16706	kNm
Unity check		0,46	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	2,015e+07	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	7153	kNm
Unity check		0,09	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	5	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,03	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	5	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,03	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	13	
Total torsional moment	τ_{Ed}	8	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,04	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		4	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	-9	N/mm²
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	-200	N/mm²
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	-50	N/mm²
Total longitudinal stress	$\sigma_{tot,Ed}$	-260	N/mm²
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm²
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm²
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	1	N/mm²
Total shear stress	$\tau_{tot,Ed}$	1	N/mm²
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	260	N/mm²
Unity check		0,73	-

Note: For this section no plastic shear resistance and corresponding Rho value can be determined. Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design

Decisive position for stability classification: 10,200 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	112	30	-150	-168								
2	I	1300	25	-228	145	-1,6		0,4	52,0	75,4	86,9	162,9	1
3	I	1300	25	-163	210	-0,8		0,6	52,0	51,0	58,7	82,6	2
4	UO	112	30	232	214	0,9	0,4	1,0	3,7	7,3	8,1	11,3	1
5	UO	112	30	-233	-250								
6	I	15	25	-228	-233								
7	UO	112	30	149	132	0,9	0,5	1,0	3,7	7,3	8,1	11,7	1
8	I	15	25	145	149	1,0		1,0	0,6	26,8	30,9	34,5	1
9	I	425	30	-168	-233								
10	I	15	25	-163	-168								
11	I	425	30	214	149	0,7		1,0	14,2	26,8	30,9	38,0	1
12	I	15	25	210	214	1,0		1,0	0,6	26,8	30,9	34,4	1

The cross-section is classified as Class 2
Note: The stability classification is based on the maximum section classification along the member.

Lateral Torsional Buckling check

According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Plastic section modulus	$W_{pl,y}$	4,706e+07	mm^3
Elastic critical moment	M_{cr}	128148	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,36	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	
LTB curve		d	
Imperfection	α_{LT}	0,76	
Reduction factor	χ_{LT}	0,88	
Design buckling resistance	$M_{b,Rd}$	14677	kNm
Unity check		0,53	-

Mcr parameters			
LTB length	L_{LT}	23,000	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,12	
LTB moment factor	C_2	0,43	
LTB moment factor	C_3	0,53	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Warning: Not all conditions of the Dutch NEN-EN NA (Art. NB.NB.1) are fulfilled, therefore the standard EC-EN approach is used.
Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial tension check
According to EN 1993-1-3 article 6.3

Normal force	N_{Ed}	960	kN
Bending moment	$M_{y,Ed}$	-7766	kNm
Bending moment	$M_{z,Ed}$	670	kNm
Tension resistance	$N_{t,Rd}$	36920	kN
Bending resistance	$M_{b,y,Rd}$	14677	kNm
Bending resistance	$M_{c,z,Rd,com}$	7153	kNm

Unity check = 0,53 + 0,09 - 0,03 = 0,60 -

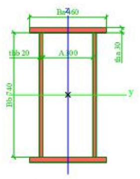
The member satisfies the stability check.

Figure D-93 Guidance beam, SCIA report

D.2.5.3. Top frame Y

Cross-sections

Name	Top frame Y
Type	Box fl
Detailed	460; 30; 740; 20; 300
Item material	S 355
Fabrication	welded
Flexural buckling y-y	b
Flexural buckling z-z	b
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm^2]	5,720e+04	
A y, z [mm^2]	2,202e+04	3,157e+04
I y, z [mm^4]	5,444e+09	1,245e+09
I w [mm^6], t [mm^4]	4,041e+13	2,482e+09
W_{el} y, z [mm^3]	1,361e+07	5,415e+06
W_{pl} y, z [mm^3]	1,610e+07	7,910e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	230	400
α [deg]	0,00	
A L, D [m^2/m]	2,7600e+00	4,8400e+00
M_{ply} +, - [Nmm]	5,72e+09	5,72e+09
M_{plz} +, - [Nmm]	2,81e+09	2,81e+09

Figure D-94 Top frame Y properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B785, B786, B966, B967

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B967	**3,100 / 6,200 m**	**Box fl (460; 30; 740; 20; 300)**	**S 355**	**All ULS**	**0,91 -**

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Welded	

...::SECTION CHECK::...

The critical check is on position 3,100 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-206	kN
Shear force	$V_{y,Ed}$	-570	kN
Shear force	$V_{z,Ed}$	544	kN
Torsion	T_{Ed}	-35	kNm
Bending moment	$M_{y,Ed}$	1759	kNm
Bending moment	$M_{z,Ed}$	-1023	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	70	30	-60	-4								
2	I	740	20	248	16	0,1		1,0	37,0	26,8	30,9	49,4	3
3	I	740	20	-9	-241								
4	UO	70	30	-301	-245								
5	UO	70	30	252	308	0,8	0,4	1,0	2,3	7,3	8,1	11,4	1
6	I	15	20	248	252	1,0		1,0	0,8	26,8	30,9	34,4	1
7	UO	70	30	11	67	0,2	0,5	1,0	2,3	7,3	8,1	12,5	1
8	I	15	20	16	11	0,7		1,0	0,8	26,8	30,9	37,9	1
9	I	320	30	-4	252	0,0		1,0	10,7	27,3	31,5	51,4	1
10	I	15	20	-9	-4								
11	I	320	30	-245	11	-21,9		0,0	10,7	672,0	774,7	5421,3	1
12	I	15	20	-241	-245								

The cross-section is classified as Class 3

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	5,720e+04	mm^2
Compression resistance	$N_{c,Rd}$	20306	kN
Unity check		0,01	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	1,361e+07	mm^3
Elastic bending moment	$M_{el,y,Rd}$	4831	kNm
Unity check		0,36	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	5,415e+06	mm^3
Elastic bending moment	$M_{el,z,Rd}$	1922	kNm
Unity check		0,53	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	30	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,15	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	20	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,10	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	13	
Total torsional moment	τ_{Ed}	4	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,02	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		4	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	4	N/mm²
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	129	N/mm²
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	189	N/mm²
Total longitudinal stress	$\sigma_{tot,Ed}$	322	N/mm²
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm²
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm²
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm²
Total shear stress	$\tau_{tot,Ed}$	0	N/mm²
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	322	N/mm²
Unity check		0,91	-

Note: For this section no plastic shear resistance and corresponding Rho value can be determined. Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design

Decisive position for stability classification: 5,675 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	70	30	19	15	0,8	0,4	1,0	2,3	7,3	8,1	11,4	1
2	I	740	20	-1	0								
3	I	740	20	15	16	0,9		1,0	37,0	26,8	30,9	34,8	4
4	UO	70	30	20	16	0,8	0,4	1,0	2,3	7,3	8,1	11,4	1
5	UO	70	30	-1	-5								
6	I	15	20	-1	-1								
7	UO	70	30	0	-4								
8	I	15	20	0	0								
9	I	320	30	15	-1	-0,1		0,9	10,7	28,9	33,3	52,8	1
10	I	15	20	15	15	1,0		1,0	0,8	26,8	30,9	34,2	1
11	I	320	30	16	0	0,0		1,0	10,7	27,0	31,1	51,2	1
12	I	15	20	16	16	1,0		1,0	0,8	26,8	30,9	34,2	1

The cross-section is classified as Class 4
Note: The stability classification is based on the maximum section classification along the member.

Effective section N-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	UO	70	355	355	1,0	0,4	0,2	1,0	70		
2	I	740	355	355	1,0	4,0	0,8	0,9	670	335	335
3	I	740	355	355	1,0	4,0	0,8	0,9	670	335	335
4	UO	70	355	355	1,0	0,4	0,2	1,0	70		
5	UO	70	355	355	1,0	0,4	0,2	1,0	70		
6	I	15	355	355	1,0	4,0	0,0	1,0	15	8	8
7	UO	70	355	355	1,0	0,4	0,2	1,0	70		
8	I	15	355	355	1,0	4,0	0,0	1,0	15	8	8
9	I	320	355	355	1,0	4,0	0,2	1,0	320	160	160
10	I	15	355	355	1,0	4,0	0,0	1,0	15	8	8
11	I	320	355	355	1,0	4,0	0,2	1,0	320	160	160
12	I	15	355	355	1,0	4,0	0,0	1,0	15	8	8

Effective section My+
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	UO	70	355	355	1,0	0,4	0,2	1,0	70		
2	I	740	341	-341	-1,0	23,9	0,3	1,0	370	148	222
3	I	740	341	-341	-1,0	23,9	0,3	1,0	370	148	222
4	UO	70	-355	-355							
5	UO	70	355	355	1,0	0,4	0,2	1,0	70		
6	I	15	355	341	1,0	4,1	0,0	1,0	15	7	8
7	UO	70	-355	-355							
8	I	15	-341	-355							
9	I	320	355	355	1,0	4,0	0,2	1,0	320	160	160
10	I	15	355	341	1,0	4,1	0,0	1,0	15	7	8
11	I	320	-355	-355							
12	I	15	-341	-355							

Effective section Mz-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	UO	70	-237	-343							
2	I	740	249	249	1,0	4,0	0,8	0,9	670	335	335
3	I	740	-237	-237							
4	UO	70	-237	-343							
5	UO	70	355	249	0,7	0,5	0,1	1,0	70		
6	I	15	249	249	1,0	4,0	0,0	1,0	15	8	8
7	UO	70	355	249	0,7	0,5	0,1	1,0	70		
8	I	15	249	249	1,0	4,0	0,0	1,0	15	8	8
9	I	320	249	-237	-1,0	22,7	0,1	1,0	164	66	98
10	I	15	-237	-237							
11	I	320	249	-237	-1,0	22,7	0,1	1,0	164	66	98
12	I	15	-237	-237							

Effective properties						
Effective area	A_{eff}	5,561e+04	mm²			
Effective second moment of area	$I_{eff,y}$	5,615e+09	mm⁴	$I_{eff,z}$	1,240e+09	mm⁴
Effective section modulus	$W_{eff,y}$	1,404e+07	mm³	$W_{eff,z}$	5,299e+06	mm³
Shift of the centroid	$e_{N,y}$	0	mm	$e_{N,z}$	0	mm

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	2,000	6,200	m
Buckling factor	k	10,00	1,00	
Buckling length	l_{cr}	20,000	6,200	m
Critical Euler load	N_{cr}	28207	67155	kN
Slenderness	λ	64,83	42,02	
Relative slenderness	λ_{rel}	0,84	0,54	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	6,200	m
Elastic critical load	$N_{cr,T}$	1732895	kN
Elastic critical load	$N_{cr,TF}$	28207	kN
Relative slenderness	$\lambda_{rel,T}$	0,84	
Limit slenderness	$\lambda_{rel,0}$	0,20	

Note: The slenderness or compression force is such that Torsional(-Flexural) Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Effective section modulus	$W_{eff,y}$	1,404e+07	mm^3
Elastic critical moment	M_{cr}	162808	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,17	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	L_{LT}	6,200	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,40	
LTB moment factor	C_2	0,74	
LTB moment factor	C_3	0,41	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Warning: Not all conditions of the Dutch NEN-EN NA (Art. NB.NB.1) are fulfilled, therefore the standard EC-EN approach is used.
Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section effective area	A_{eff}	5,561e+04	mm^2
Effective section modulus	$W_{eff,y}$	1,404e+07	mm^3
Effective section modulus	$W_{eff,z}$	5,299e+06	mm^3
Design compression force	N_{Ed}	206	kN
Design bending moment (maximum)	$M_{y,Ed}$	1759	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-985	kNm
Additional moment	$\Delta M_{y,Ed}$	0	kNm
Additional moment	$\Delta M_{z,Ed}$	0	kNm
Characteristic compression resistance	N_{Rk}	19742	kN
Characteristic moment resistance	$M_{y,Rk}$	4983	kNm
Characteristic moment resistance	$M_{z,Rk}$	1881	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	1,00	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	0,90	
Interaction factor	k_{yz}	0,90	
Interaction factor	k_{zy}	1,00	
Interaction factor	k_{zz}	0,90	

Maximum moment $M_{y,Ed}$ is derived from beam B967 position 3,100 m.
Maximum moment $M_{z,Ed}$ is derived from beam B967 position 3,100 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.2	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	0	kNm
Field moment	$M_{s,z}$	-985	kNm
Factor	$\alpha_{h,z}$	0,00	
Ratio of end moments	ψ_z	-0,36	
Equivalent moment factor	C_{mz}	0,90	
Resulting load type LT		point load F	
End moment	$M_{h,LT}$	-356	kNm
Field moment	$M_{s,LT}$	1808	kNm
Factor	$\alpha_{h,LT}$	-0,20	
Ratio of end moments	ψ_{LT}	0,26	
Equivalent moment factor	C_{mLT}	0,88	

Unity check (6.61) = 0,01 + 0,32 + 0,47 = 0,80 -
Unity check (6.62) = 0,01 + 0,35 + 0,47 = 0,84 -

The member satisfies the stability check.

Figure D-95 Top frame Y, SCIA report

D.2.6. MLS LIFT

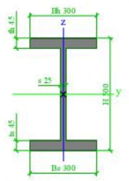
Table D-42 Unity Checks MLS lift

MLS LIFT	GOVERNING U.C.	CONFIGURATION 1 U.C.	CONFIGURATION 2 U.C.
Rails	0.97	0.68	0.97
Rails diagonal	0.33	0.23	0.33

D.2.6.1. Rails

Cross-sections

Name	Rails
Type	I ng
Detailed	500; 300; 300; 45; 45; 25
Item material	S 355
Fabrication	general
Flexural buckling y-y	d
Flexural buckling z-z	d
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm^2]	3,725e+04	
A y, z [mm^2]	2,605e+04	1,238e+04
I y, z [mm^4]	1,546e+09	2,030e+08
I w [mm^6], t [mm^4]	1,044e+13	1,866e+07
W_{el} y, z [mm^3]	6,182e+06	1,354e+06
W_{pl} y, z [mm^3]	7,193e+06	2,089e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	150	250
α [deg]	0,00	
A L, D [m^2/m]	2,1500e+00	2,1500e+00
M_{ply} +, - [Nmm]	2,55e+09	2,55e+09
M_{plz} +, - [Nmm]	7,42e+08	7,42e+08

Figure D-96 Rails properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B801..B804, B982..B985

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B984	**7,700 / 10,200 m**	**I ng (500; 300; 300; 45; 45; 25)**	**S 355**	**All ULS**	**0,97 -**

Combination key
All ULS / NC2 Operation pos 1

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		General	

...::SECTION CHECK::...

The critical check is on position 7,700 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-737	kN
Shear force	$V_{y,Ed}$	-11	kN
Shear force	$V_{z,Ed}$	-286	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	-396	kNm
Bending moment	$M_{z,Ed}$	-24	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	150	45	93	76	0,8	0,4	1,0	3,3	7,3	8,1	11,4	1
2	UO	150	45	76	58	0,8	0,5	1,0	3,3	7,3	8,1	12,4	1
3	I	455	25	76	-37	-0,5		0,7	18,2	41,8	48,1	67,3	1
4	UO	150	45	-19	-37								
5	UO	150	45	-37	-55								

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	3,725e+04	mm^2
Compression resistance	$N_{c,Rd}$	13224	kN
Unity check		0,06	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	7,193e+06	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	2554	kNm
Unity check		0,16	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	2,089e+06	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	742	kNm
Unity check		0,03	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	1	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.
Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	27	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,13	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	16	
Total torsional moment	τ_{Ed}	1	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		15	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	20	N/mm²
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	64	N/mm²
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	18	N/mm²
Total longitudinal stress	$\sigma_{tot,Ed}$	102	N/mm²
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm²
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm²
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm²
Total shear stress	$\tau_{tot,Ed}$	0	N/mm²
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	102	N/mm²
Unity check		0,29	-

Note: For this section no plastic shear resistance and corresponding Rho value can be determined. Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 10,200 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	150	45	269	250	0,9	0,4	1,0	3,3	7,3	8,1	11,3	1
2	UO	150	45	250	231	0,9	0,5	1,0	3,3	7,3	8,1	11,6	1
3	I	455	25	250	-212	-0,8		0,5	18,2	53,4	61,5	87,7	1
4	UO	150	45	-193	-212								
5	UO	150	45	-212	-231								

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	5,100	10,200	m
Buckling factor	k	2,18	0,97	
Buckling length	l_{cr}	11,130	9,925	m
Critical Euler load	N_{cr}	25861	4272	kN
Slenderness	λ	54,64	134,44	
Relative slenderness	λ_{rel}	0,72	1,76	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		d	d	
Imperfection	α	0,76	0,76	
Reduction factor	χ	0,63	0,22	
Buckling resistance	$N_{b,Rd}$	8375	2869	kN

Flexural Buckling verification			
Cross-section area	A	3,725e+04	mm^2
Buckling resistance	$N_{b,Rd}$	2869	kN
Unity check		0,26	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	10,200	m
Elastic critical load	$N_{cr,T}$	36528	kN
Elastic critical load	$N_{cr,TF}$	4272	kN
Relative slenderness	$\lambda_{rel,T}$	1,76	
Limit slenderness	$\lambda_{rel,0}$	0,20	
Buckling curve		d	
Imperfection	α	0,76	
Reduction factor	χ	0,22	
Cross-section area	A	3,725e+04	mm^2
Buckling resistance	$N_{b,Rd}$	2869	kN
Unity check		0,26	-

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Plastic section modulus	$W_{pl,y}$	7,193e+06	mm^3
Elastic critical moment	M_{cr}	9804	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,51	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	
LTB curve		d	
Imperfection	α_{LT}	0,76	
Reduction factor	χ_{LT}	0,77	
Design buckling resistance	$M_{b,Rd}$	1971	kNm
Unity check		0,20	-

Mcr parameters			
LTB length	l_{LT}	10,200	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	3,72	
LTB moment factor	C_2	0,88	
LTB moment factor	C_3	0,41	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Warning: Not all conditions of the Dutch NEN-EN NA (Art. NB.NB.1) are fulfilled, therefore the standard EC-EN approach is used.
Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	3,725e+04	mm^2
Plastic section modulus	$W_{pl,y}$	7,193e+06	mm^3
Plastic section modulus	$W_{pl,z}$	2,089e+06	mm^3
Design compression force	N_{Ed}	737	kN
Design bending moment (maximum)	$M_{y,Ed}$	-1624	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-32	kNm
Characteristic compression resistance	N_{Rk}	13224	kN
Characteristic moment resistance	$M_{y,Rk}$	2554	kNm
Characteristic moment resistance	$M_{z,Rk}$	742	kNm
Reduction factor	χ_y	0,63	
Reduction factor	χ_z	0,22	
Reduction factor	χ_{LT}	0,77	
Interaction factor	k_{yy}	0,94	
Interaction factor	k_{yz}	0,42	
Interaction factor	k_{zy}	0,83	
Interaction factor	k_{zz}	0,70	

Maximum moment $M_{y,Ed}$ is derived from beam B984 position 10,200 m.
Maximum moment $M_{z,Ed}$ is derived from beam B984 position 8,400 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.2	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	-26	kNm
Field moment	$M_{s,z}$	17	kNm
Factor	$\alpha_{s,z}$	-0,64	
Ratio of end moments	ψ_z	0,17	
Equivalent moment factor	C_{mz}	0,51	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	-1624	kNm
Field moment	$M_{s,LT}$	273	kNm
Factor	$\alpha_{s,LT}$	-0,17	
Ratio of end moments	ψ_{LT}	-0,12	
Equivalent moment factor	C_{mLT}	0,40	

Unity check (6.61) = 0,09 + 0,77 + 0,02 = 0,88 -
Unity check (6.62) = 0,26 + 0,68 + 0,03 = 0,97 -

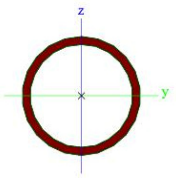
The member satisfies the stability check.

Figure D-97 Rails, SCIA report

D.2.6.2. Rails diagonal

Cross-sections

Name	Rails diagonal
Type	CHS193.7/12.5
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	7,120e+03	
A y, z [mm²]	4,530e+03	4,530e+03
I y, z [mm⁴]	2,934e+07	2,934e+07
I w [mm⁶], t [mm⁴]	4,853e-22	5,869e+07
W_{el} y, z [mm³]	3,030e+05	3,030e+05
W_{pl} y, z [mm³]	4,046e+05	4,046e+05
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	97	97
α [deg]	0,00	
A L, D [m²/m]	6,0900e-01	1,1385e+00
M_{ply} +, - [Nmm]	1,46e+08	1,46e+08
M_{plz} +, - [Nmm]	1,46e+08	1,46e+08

Figure D-98 Rails diagonal properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: B855, B856, B859, B860, B866, B867, B891, B892, B1036, B1037, B1040, B1041, B1047, B1048, B1072, B1073

EN 1993-1-1 Code Check
National annex: Dutch NEN-EN NA

Member B1047	3,136 / 3,136 m	CHS193.7/12.5	S 355	All ULS	0,33 -

Combination key
All ULS / NC4 Operation pos 3

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 3,136 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-403	kN
Shear force	$V_{y,Ed}$	-16	kN
Shear force	$V_{z,Ed}$	0	kN
Torsion	T_{Ed}	1	kNm
Bending moment	$M_{y,Ed}$	0	kNm
Bending moment	$M_{z,Ed}$	-27	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
194	13	15,5	33,1	46,3	59,6	1

The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	7,120e+03	mm^2
Compression resistance	$N_{c,Rd}$	2528	kN
Unity check		0,16	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	4,046e+05	mm^3
Plastic bending moment	$M_{pl,y,Rd}$	144	kNm
Unity check		0,00	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	4,046e+05	mm^3
Plastic bending moment	$M_{pl,z,Rd}$	144	kNm
Unity check		0,19	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	4,533e+03	mm^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	929	kN
Unity check		0,02	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1,20	
Shear area	A_v	4,533e+03	mm^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	929	kN
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	2	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,01	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Resultant bending moment	$M_{resultant}$	27	kNm
Resultant shear force	$V_{resultant}$	16	kN
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,Rd}$	137	kNm
Unity check		0,20	-

Note: The resultant internal forces are used for CHS sections.
Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
194	13	15,5	33,1	46,3	59,6	1

The cross-section is classified as Class 1
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	3,136	3,136	m
Buckling factor	k	1,00	0,99	
Buckling length	l_{cr}	3,136	3,116	m
Critical Euler load	N_{cr}	6182	6263	kN
Slenderness	λ	48,86	48,54	
Relative slenderness	λ_{rel}	0,64	0,64	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	a	
Imperfection	α	0,21	0,21	
Reduction factor	χ	0,87	0,88	
Buckling resistance	$N_{b,Rd}$	2210	2214	kN

Flexural Buckling verification			
Cross-section area	A	7,120e+03	mm^2
Buckling resistance	$N_{b,Rd}$	2210	kN
Unity check		0,18	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a CHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns a CHS section which is not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	7,120e+03	mm^2
Plastic section modulus	$W_{pl,y}$	4,046e+05	mm^3
Plastic section modulus	$W_{pl,z}$	4,046e+05	mm^3
Design compression force	N_{Ed}	403	kN
Design bending moment (maximum)	$M_{y,Ed}$	0	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-27	kNm
Characteristic compression resistance	N_{Rk}	2528	kN
Characteristic moment resistance	$M_{y,Rk}$	144	kNm
Characteristic moment resistance	$M_{z,Rk}$	144	kNm
Reduction factor	χ_y	0,87	

Bending and axial compression check parameters			
Reduction factor	χ_z	0,88	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	0,97	
Interaction factor	k_{yz}	0,47	
Interaction factor	k_{zy}	0,58	
Interaction factor	k_{zz}	0,78	

Maximum moment $M_{y,Ed}$ is derived from beam B1047 position 1,882 m.
Maximum moment $M_{z,Ed}$ is derived from beam B1047 position 3,136 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0,90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	-27	kNm
Field moment	$M_{s,z}$	-17	kNm
Factor	$\alpha_{s,z}$	0,62	
Ratio of end moments	ψ_z	-0,96	
Equivalent moment factor	C_{mz}	0,70	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	0	kNm
Field moment	$M_{s,LT}$	0	kNm
Factor	$\alpha_{h,LT}$	-0,04	
Ratio of end moments	ψ_{LT}	-0,94	
Equivalent moment factor	C_{mLT}	0,95	

Unity check (6.61) = 0,18 + 0,00 + 0,09 = 0,27 -
Unity check (6.62) = 0,18 + 0,00 + 0,15 = 0,33 -

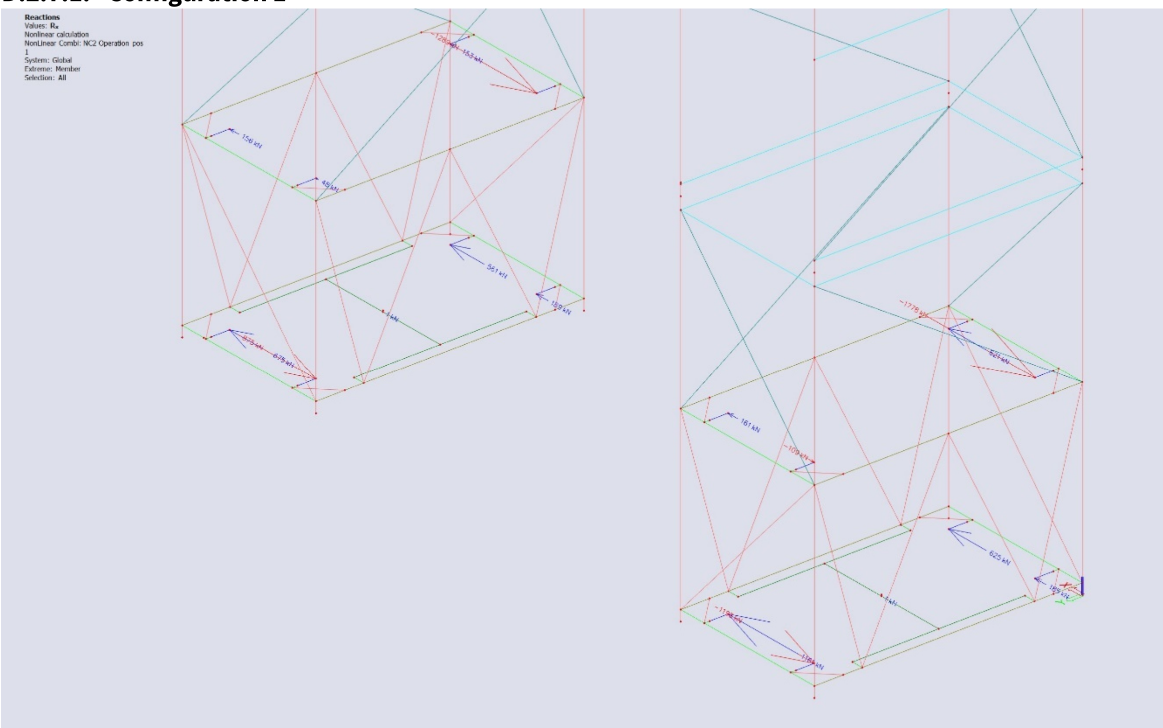
The member satisfies the stability check.

Figure D-99 Rails diagonal, SCIA report

D.2.7. Reaction forces

The critical reaction forces occurred during the lifting of the MLS mast section in the operational load case.

D.2.7.1. Configuration 1



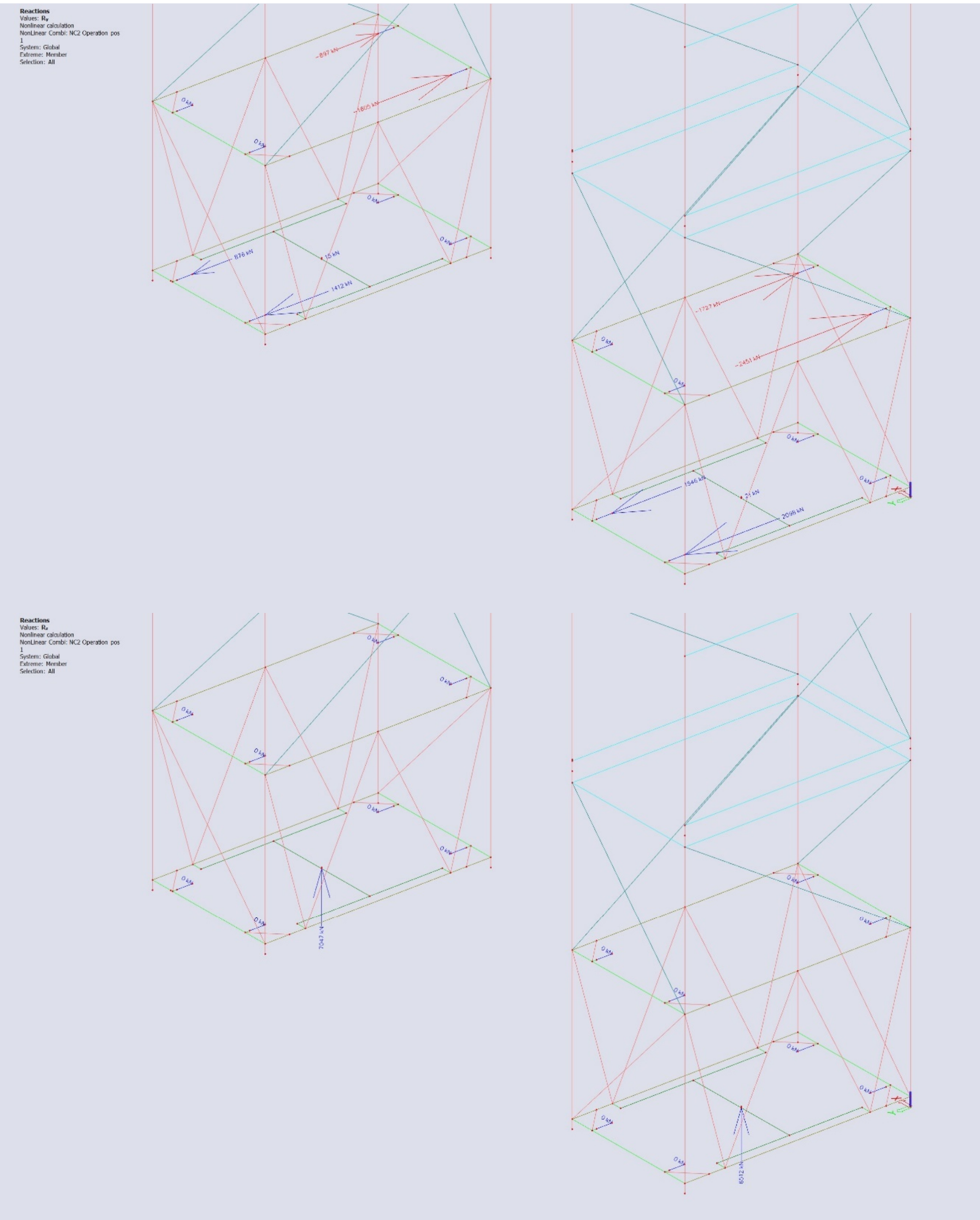
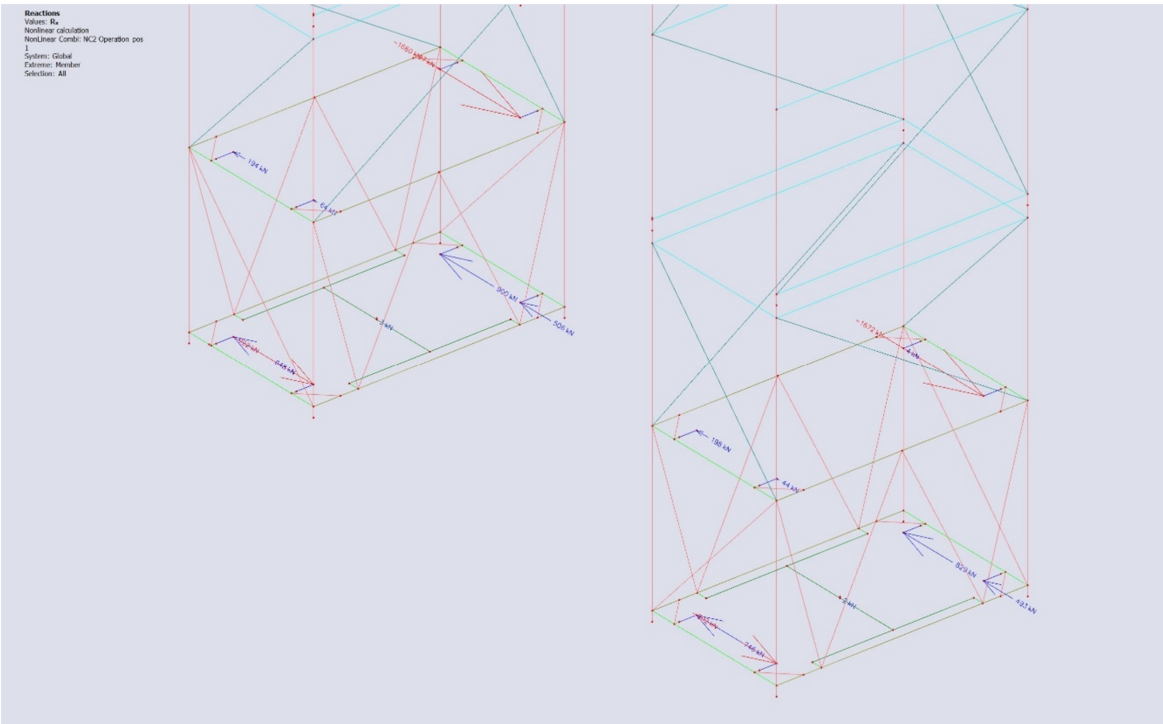
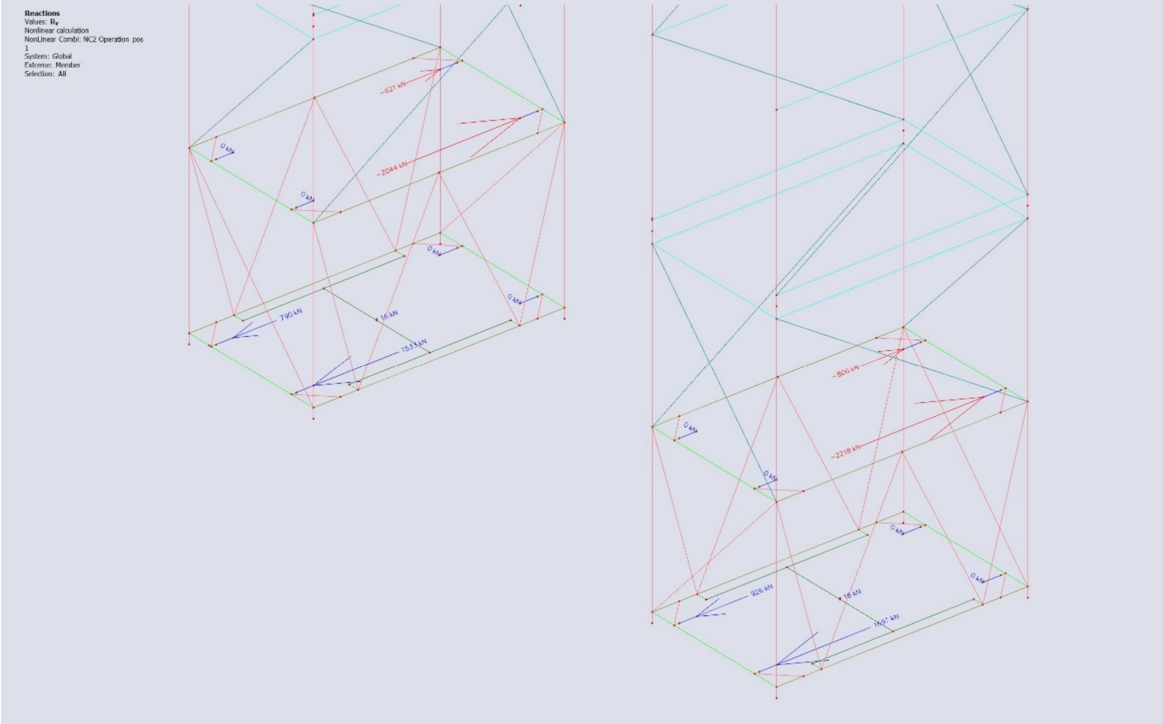


Figure D-100 Critical reaction forces configuration 1 in X, Y, and Z direction

D.2.7.2. Configuration 2





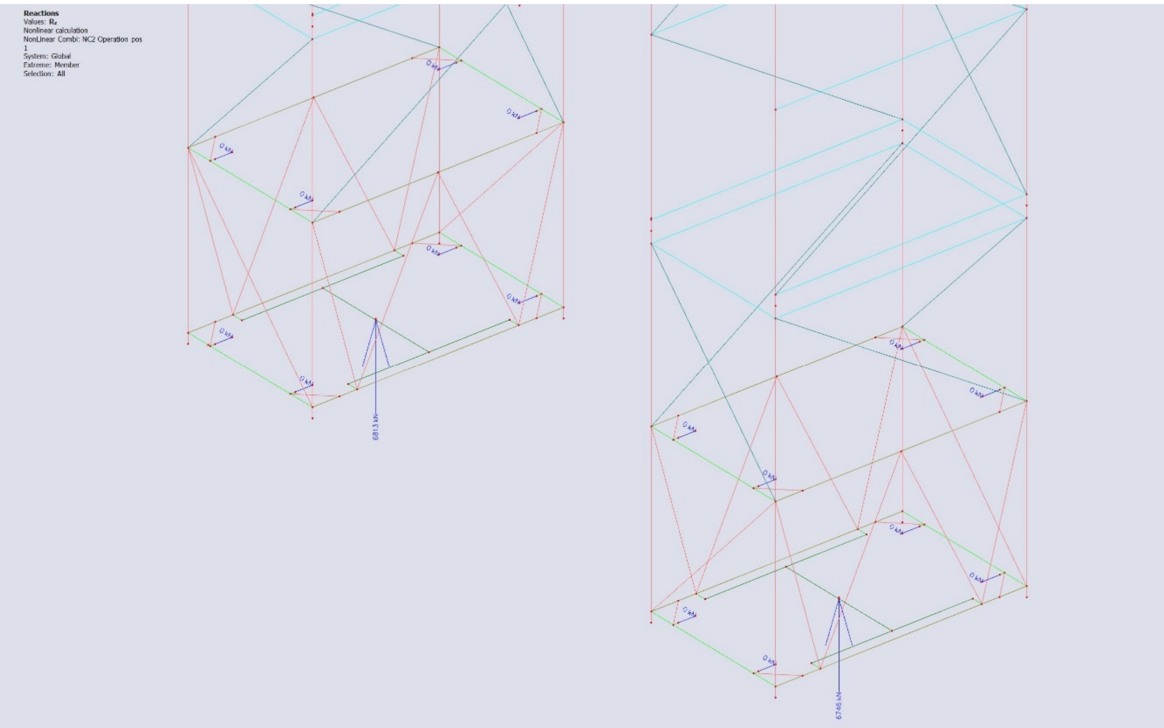


Figure D-101 Critical reaction forces configuration 2 in X, Y, and Z direction

D.3. CONNECTIONS

D.3.1.1. MLS chord global

Four load cases need to be verified. The stationary case and the three operational cases.

Case I generates the following loads on the MLS chord:

- 703 kN in X direction
- 736 kN in Y direction
- 1430 kN in Z direction

Case II generates the following loads on the MLS chord:

- 1778 kN in X direction
- 2451 kN in Y direction
- 1760 kN in Z direction

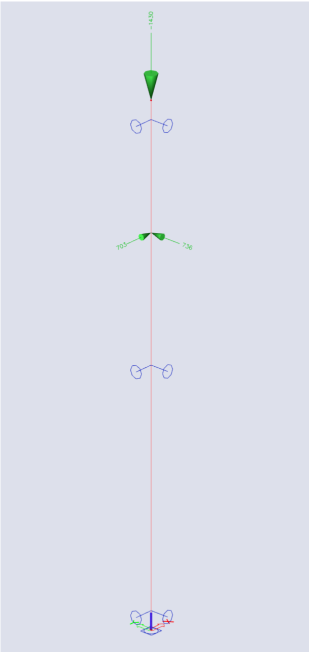
Case III generates the following loads on the MLS chord:

- 1371 kN in X direction
- 1774 kN in Y direction
- 1760 kN in Z direction

Case IV generates the following loads on the MLS chord:

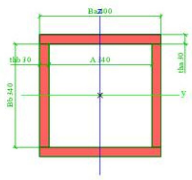
- 1105 kN in X direction
- 1334 kN in Y direction
- 1760 kN in Z direction

The horizontal loads can be present along the whole length of the MLS column. Figure D-102 shows the critical position. Then the member does suffice for cases I and IV but not for cases II and III. However, the loads in cases II and III occur during the lifting of the MLS section. Then the horizontal forces can be introduced in a stronger place of the chord. If the loads are introduced 1.55 meters above or below the horizontal brace the member suffices.



Cross-sections

Name	MLS main cord
Type	Box fl
Detailed	400; 30; 340; 30; 340
Item material	S 690
Fabrication	welded
Flexural buckling y-y	b
Flexural buckling z-z	b
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm^2]	4,440e+04	
A y, z [mm^2]	2,311e+04	2,311e+04
I y, z [mm^4]	1,020e+09	1,020e+09
I w [mm^6], t [mm^4]	4,620e+09	1,533e+09
W_{el} y, z [mm^3]	5,099e+06	5,099e+06
W_{pl} y, z [mm^3]	6,174e+06	6,174e+06
d y, z [mm]	0	0
c YUCS, ZUCS [mm]	200	200
α [deg]	0,00	
A L, D [m^2/m]	1,6000e+00	2,9600e+00
M_{ply} +, - [Nmm]	4,26e+09	4,26e+09
M_{plz} +, - [Nmm]	4,26e+09	4,26e+09

Figure D-102 MLS chord model and properties

The horizontal supports in the middle resemble the horizonal braces of the MLS mast. Their compressive buckling resistances are larger than these reaction forces, see Annex A.2.1. Meaning that the members will suffice.

EC-EN 1993 Steel check ULS

Linear calculation
Class: All ULS
Coordinate system: Principal
Extreme 1D: Global
Selection: All

EN 1993-1-1 Code Check
National annex: Standard EN

Member B1	**7,150 / 11,000 m**	**Box fl (400; 30; 340; 30; 340)**	**S 690**	**All ULS**	**0,97 -**

Combination key
All ULS / LC 2

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	690	N/mm^2
Ultimate strength	f_u	770	N/mm^2
Fabrication		Welded	

...::SECTION CHECK::...

The critical check is on position 7,150 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-1300	kN
Shear force	$V_{y,Ed}$	-1814	kN
Shear force	$V_{z,Ed}$	1285	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	1347	kNm
Bending moment	$M_{z,Ed}$	-1901	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	15	30	-92	-66								
2	I	340	30	566	141	0,2		1,0	11,3	19,3	22,2	32,6	1
3	I	340	30	-84	-510								
4	UO	15	30	-555	-529								
5	UO	15	30	585	611	1,0	0,4	1,0	0,5	5,3	5,8	8,1	1
6	I	15	30	566	585	1,0		1,0	0,5	19,3	22,2	24,8	1
7	UO	15	30	122	148	0,8	0,4	1,0	0,5	5,3	5,8	8,2	1
8	I	15	30	141	122	0,9		1,0	0,5	19,3	22,2	25,6	1
9	I	370	30	-66	585	-0,1		0,9	12,3	21,6	24,9	38,7	1
10	I	15	30	-84	-66								
11	I	370	30	-529	122	-4,3		0,2	12,3	112,2	129,3	402,5	1
12	I	15	30	-510	-529								

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	4,440e+04	mm^2
Compression resistance	$N_{c,Rd}$	30636	kN
Unity check		0,04	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	5,099e+06	mm^3
Elastic bending moment	$M_{el,y,Rd}$	3518	kNm
Unity check		0,38	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	5,099e+06	mm^3
Elastic bending moment	$M_{el,z,Rd}$	3518	kNm
Unity check		0,54	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	91	N/mm²
Elastic shear resistance	τ_{Rd}	398	N/mm²
Unity check		0,23	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	65	N/mm²
Elastic shear resistance	τ_{Rd}	398	N/mm²
Unity check		0,16	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		4	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	29	N/mm²
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	264	N/mm²
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	373	N/mm²
Total longitudinal stress	$\sigma_{tot,Ed}$	666	N/mm²
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm²
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm²
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm²
Total shear stress	$\tau_{tot,Ed}$	0	N/mm²
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	666	N/mm²
Unity check		0,97	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 7,150 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	15	30	-92	-66								
2	I	340	30	566	141	0,2		1,0	11,3	19,3	22,2	32,6	1
3	I	340	30	-84	-510								
4	UO	15	30	-555	-529								
5	UO	15	30	585	611	1,0	0,4	1,0	0,5	5,3	5,8	8,1	1
6	I	15	30	566	585	1,0		1,0	0,5	19,3	22,2	24,8	1
7	UO	15	30	122	148	0,8	0,4	1,0	0,5	5,3	5,8	8,2	1
8	I	15	30	141	122	0,9		1,0	0,5	19,3	22,2	25,6	1
9	I	370	30	-66	585	-0,1		0,9	12,3	21,6	24,9	38,7	1
10	I	15	30	-84	-66								
11	I	370	30	-529	122	-4,3		0,2	12,3	112,2	129,3	402,5	1
12	I	15	30	-510	-529								

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		non-sway	non-sway	
System length	L	5,100	5,100	m
Buckling factor	k	0,70	0,70	
Buckling length	l_{cr}	3,570	3,570	m
Critical Euler load	N_{cr}	165830	165830	kN
Slenderness	λ	23,56	23,56	
Relative slenderness	λ_{rel}	0,43	0,43	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	5,100	m
Elastic critical load	$N_{cr,T}$	2696428	kN
Elastic critical load	$N_{cr,TF}$	165830	kN
Relative slenderness	$\lambda_{rel,T}$	0,43	
Limit slenderness	$\lambda_{rel,0}$	0,20	

Note: The slenderness or compression force is such that Torsional(-Flexural) Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Elastic section modulus	$W_{el,y}$	5,099e+06	mm^3
Elastic critical moment	M_{cr}	116637	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,17	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	5,100	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,16	
LTB moment factor	C_2	0,58	
LTB moment factor	C_3	0,53	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 1	
Cross-section area	A	4,440e+04	mm^2
Elastic section modulus	$W_{el,y}$	5,099e+06	mm^3
Elastic section modulus	$W_{el,z}$	5,099e+06	mm^3
Design compression force	N_{Ed}	1300	kN
Design bending moment (maximum)	$M_{y,Ed}$	1347	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-1901	kNm
Characteristic compression resistance	N_{Rk}	30636	kN
Characteristic moment resistance	$M_{y,Rk}$	3518	kNm
Characteristic moment resistance	$M_{z,Rk}$	3518	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	1,00	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,01	
Interaction factor	k_{yz}	1,01	
Interaction factor	k_{zy}	1,01	
Interaction factor	k_{zz}	1,01	

Maximum moment $M_{y,Ed}$ is derived from beam B1 position 7,150 m.
Maximum moment $M_{z,Ed}$ is derived from beam B1 position 7,150 m.

Interaction method 1 parameters			
Critical Euler load	$N_{cr,y}$	165830	kN
Critical Euler load	$N_{cr,z}$	165830	kN
Elastic critical load	$N_{cr,T}$	2696428	kN
Elastic section modulus	$W_{el,y}$	5,099e+06	mm^3
Second moment of area	I_y	1,020e+09	mm^4
Second moment of area	I_z	1,020e+09	mm^4
Torsional constant	I_t	1,533e+09	mm^4
Method for equivalent moment factor $C_{my,0}$		Table A.2 Line 2 (General)	
Design bending moment (maximum)	$M_{y,Ed}$	1347	kNm
Maximum relative deflection	δ_z	-13,0	mm
Equivalent moment factor	$C_{my,0}$	1,00	
Method for equivalent moment factor $C_{mz,0}$		Table A.2 Line 2 (General)	
Design bending moment (maximum)	$M_{z,Ed}$	-1901	kNm

Interaction method 1 parameters			
Maximum relative deflection	δ_y	18,4	mm
Equivalent moment factor	$C_{mz,0}$	1,00	
Factor	μ_y	1,00	
Factor	μ_z	1,00	
Factor	ε_y	9,02	
Factor	a_{LT}	0,00	
Critical moment for uniform bending	$M_{cr,0}$	100320	kNm
Relative slenderness	$\lambda_{rel,0}$	0,19	
Limit relative slenderness	$\lambda_{rel,0,lim}$	0,22	
Equivalent moment factor	C_{my}	1,00	
Equivalent moment factor	C_{mz}	1,00	
Equivalent moment factor	C_{mLT}	1,00	

Unity check (6.61) = 0,04 + 0,39 + 0,54 = 0,97 -
Unity check (6.62) = 0,04 + 0,39 + 0,54 = 0,97 -

The member satisfies the stability check.

Figure D-103 MLS chord, SCIA report

D.3.1.2. MLS chord local

!NOTE: CALCULATIONS ARE BASED ON OLDER SCIA MODEL. AN UPDATE IS REQUIRED!

Locally the chord will be loaded in X and Y direction. Figure D-104 shows the chord and its dimensions.

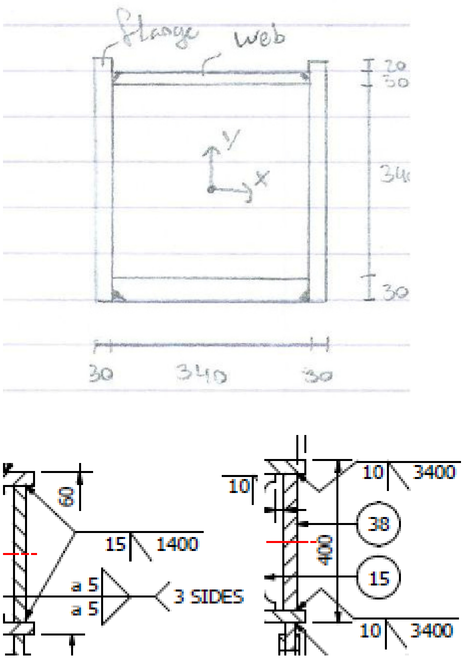


Figure D-104 MLS chord welds

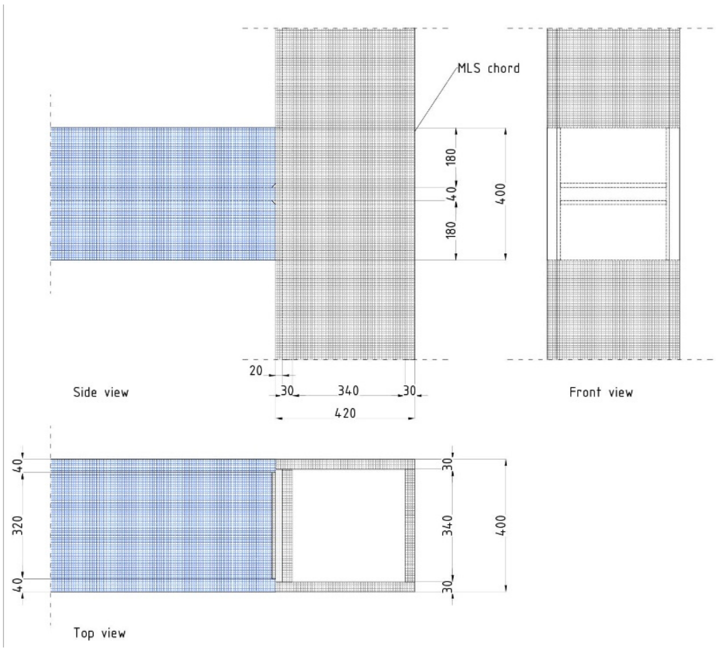
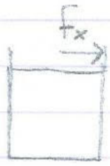


Figure D-105 Stability beam to MLS chord connection

X direction

The shear force and the bending moment resistances are calculated in Figure D-106.

X - Direction



$F_x = 1676$ kN

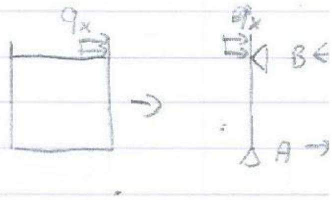
$A = 20 \times 400$

$= 8000 \text{ mm}^2$

$Q_x = F_x / A$

$= 1676 / 8000 \cdot 10^6$

$= 209500 \text{ kN/m}^2$



$q_x = 83{,}800$ kN/m

$\Sigma T | A = 0: \; B_H \cdot 0{,}4 - q_x \cdot 0{,}02 \cdot 0{,}41 = 0$

$0{,}4 \, B_H = 0{,}0082 \cdot 83.800$

$0{,}4 \, B_H = 687{,}16$

$B_H = 1717{,}9$ kN

Horizontal force eq.

$A_H = B_H - F_x$

$= 1717{,}9 - 1676$

$= 41{,}9$ kN

Shear check: $V_{Rd} = f_y \frac{1}{\sqrt{3}} A$

$= 690 \cdot \frac{1}{\sqrt{3}} \cdot 0{,}4 \cdot 0{,}03 \cdot 10^3$

$= 4780$ kN

$U.C. = 1676 / 4780 = 0{,}35 < 1{,}0$ Ok

Bending Moment check: $M_{Ed} = F_x \cdot 0{,}01 = 1676 \cdot 0{,}01 = 16{,}76$ kNm

$W_x = \frac{1}{6} b h^2 = \frac{1}{6} \cdot 0{,}40 \cdot 0{,}03^2 = 6 \cdot 10^{-5} \text{ m}^3$

$\sigma = 16{,}76 / (6 \cdot 10^{-5}) \cdot 10^{-3} = 279{,}3 \text{ N/mm}^2$

$U.C. = 279{,}3 / 690 = 0{,}40 < 1{,}0$ ok

Figure D-106 Shear force and bending moment resistance X direction MLS chord hand calculation

The welds are calculated in Figure D-107 to Figure D-109. By the way the load is introduced in the member, the bending moment will be taken by both the weld and the parent material. It will not occur that the bending moment will act the other way around. Therefore, it is chosen to let n be 2. This has a consequence for the calculation for the load in the loading case 1. Therefore, the load is doubled to compensate. Figure D-107 deems the connection not strong enough. However, this can be omitted by lifting in a stronger position of the mast, like Annex D.3.1.1. The weld will suffice when it is 15 mm, see Figure D-108. Moreover, with the lower load from load case IV (the minimum it must withstand) it suffices as well. The shear force in that case is 1023 kN (so 2046 kN in the sheet) and the bending moment is 0.01*1023 = 10.23 kNm.

MAMMOET CALCULATION

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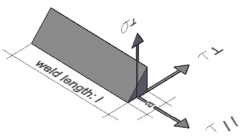
Calculation for PJP welds - Acc. EN-1993-1-8 Chp. 4

Calculation sheet 050900012

Detail



Input parameters

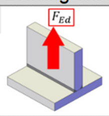
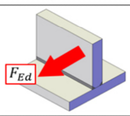
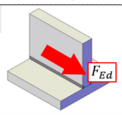
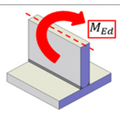
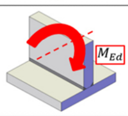
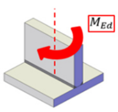
Steel material		S690		
Weld throat	*a*	10	mm	
Plate width	*b*	30	mm	
Fysical length weld	*l*	400	mm	
Eff. weld length	l_{eff}	380	mm	=l-2*a
Welded sides	*n*	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	2.00		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1.

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 3352 kN	$\sigma_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$ $\tau_{//} = 0 , \tau_\perp = 0$	$\sigma_\perp = \frac{3352e3}{2*10*380} =$	441 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = 0, \tau_{//} = 0$ $\tau_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$	$\tau_\perp = \frac{0e3}{2*10*380} =$	0 N/mm²
3 	F_{Ed} 0 kN	$\tau_{//} = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$ $\sigma_\perp = \tau_\perp = 0$	$\tau_{//} = \frac{0e3}{2*10*380} =$	0 N/mm²
4 	M_{Ed} 17 kNm	$n = 2$: $\sigma_\perp = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$ $n = 1$ and σ⊥ OK: $\sigma_\perp = \frac{M_{Ed}}{1/6 \cdot a^2 \cdot l_{eff}}$ $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{16.7e6}{(30-10)*10*380} =$	220 N/mm²
5 	M_{Ed} 0 kNm	$\sigma_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$ $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{0e6}{2*1/6*10*380^2} =$	0 N/mm²
6 	M_{Ed} 0 kNm	$\sigma_\perp = 0$ $l_{eff} \leq 2 \cdot b$ and $n = 2$: $\tau_{//} = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$ $l_{eff} > 2 \cdot b$ or $l_{eff} \leq 2 \cdot b$ and $n = 1$: $\tau_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$	$\tau_{//} = \frac{0e6}{2*1/6*10*380^2} =$	0 N/mm²

Material information - Steel grade S690

Yield strength	f_y	=	690	N/mm²	
(Nominal) tensile strength	f_u	=	770	N/mm²	
Correlation factor	β	=	1		

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	356	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	554	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	616	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	0	N/mm²	= 0
Total actual normal stress	$\Sigma\tau_\perp$	=	0	N/mm²	= 0+0
Total actual normal stress	$\Sigma\sigma_\perp$	=	661	N/mm²	= 0+220+441
Actual combined stress	$\sigma_{w,u,d}$	=	661	N/mm²	= √(661+3·(0²+0²))

Unity checks welds

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3} \cdot \beta \cdot \gamma_{M2}}\right)} \leq 1 = \frac{0}{356} = 0.00 \leq 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9 \cdot f_u}{\gamma_{M2}}\right)} \leq 1 = \frac{661}{554} = 1.19 > 1$ Not ok

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta \cdot \gamma_{M2}}\right)} \leq 1 = \frac{661}{616} = 1.07 > 1$ Not ok

Unity Checks base material

Shear stress: $\frac{\Sigma\tau_{//}}{f_{w,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_y}{\sqrt{3}}\right)} \leq 1 = \frac{0}{398} = 0.00 \leq 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{f_y} \leq 1 = \frac{661}{690} = 0.96 \leq 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{f_y} \leq 1 = \frac{661}{690} = 0.96 \leq 1$ OK

Figure D-107 Weld load case II weak posistion

MAMMOET CALCULATION

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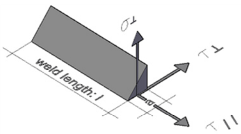
Calculation for PJP welds - Acc. EN-1993-1-8 Chp. 4 Calculation sheet 050900012

Detail



Input parameters

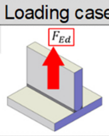
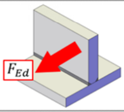
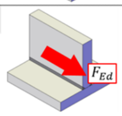
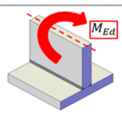
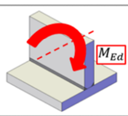
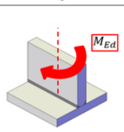
Steel material		S690	
Weld throat	a	15	mm
Plate width	b	30	mm
Fysical length weld	l	400	mm
Eff. weld length	l_{eff}	370	mm =l-2*a
Welded sides	n	2	
Partial factor	γ_{M2}	1.25	
Input forces		ULS	
Load factor	γ	2.00	

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1.

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 3352 kN	$\sigma_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{3352e3}{2*15*370} =$	302 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = 0, \tau_{//} = 0$; $\tau_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$	$\tau_\perp = \frac{0e3}{2*15*370} =$	0 N/mm²
3 	F_{Ed} 0 kN	$\tau_{//} = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$; $\sigma_\perp = \tau_\perp = 0$	$\tau_{//} = \frac{0e3}{2*15*370} =$	0 N/mm²
4 	M_{Ed} 17 kNm	$n = 2$: $\sigma_\perp = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$; $n = 1$ and σ⊥ OK: $\sigma_\perp = \frac{M_{Ed}}{1/6 \cdot a^2 \cdot l_{eff}}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{16.8e6}{(30-15)*15*370} =$	201 N/mm²
5 	M_{Ed} 0 kNm	$\sigma_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{0e6}{2*1/6*15*370^2} =$	0 N/mm²
6 	M_{Ed} 0 kNm	$\sigma_\perp = 0$; $l_{eff} \le 2 \cdot b$ and $n = 2$: $\tau_{//} = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$; $l_{eff} > 2 \cdot b$ or $l_{eff} \le 2 \cdot b$ and $n = 1$: $\tau_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$	$\tau_{//} = \frac{0e6}{2*1/6*15*370^2} =$	0 N/mm²

Material information - Steel grade ***S690***

Yield strength	f_y	=	690	N/mm²	
(Nominal) tensile strength	f_u	=	770	N/mm²	
Correlation factor	β	=	1		

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	356	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	554	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	616	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	0	N/mm²	= 0
Total actual normal stress	$\Sigma\tau_\perp$	=	0	N/mm²	= 0+0
Total actual normal stress	$\Sigma\sigma_\perp$	=	503	N/mm²	= 0+201+302
Actual combined stress	$\sigma_{w,u,d}$	=	503	N/mm²	= √(503+3·(0²+0²))

Unity checks welds

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3} \cdot \beta \cdot \gamma_{M2}}\right)} \le 1 \quad = \frac{0}{356} = 0.00 \le 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9 \cdot f_u}{\gamma_{M2}}\right)} \le 1 \quad = \frac{503}{554} = 0.91 \le 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta \cdot \gamma_{M2}}\right)} \le 1 \quad = \frac{503}{616} = 0.82 \le 1$ OK

Unity Checks base material

Shear stress: $\frac{\Sigma\tau_{//}}{f_{w,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_y}{\sqrt{3}}\right)} \le 1 \quad = \frac{0}{398} = 0.00 \le 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{f_y} \le 1 \quad = \frac{503}{690} = 0.73 \le 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{f_y} \le 1 \quad = \frac{503}{690} = 0.73 \le 1$ OK

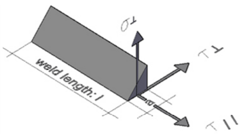
Figure D-108 Weld load case II strong position

MAMMOET — CALCULATION

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for PJP welds - Acc. EN-1993-1-8 Chp. 4 — Calculation sheet 050900012

Detail



Input parameters

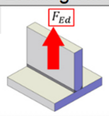
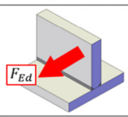
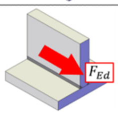
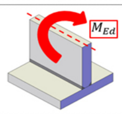
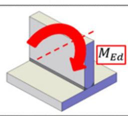
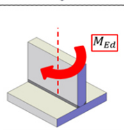
Steel material		S690		
Weld throat	a	15	mm	
Plate width	b	30	mm	
Fysical length weld	l	400	mm	
Eff. weld length	l_{eff}	370	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	2.00		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1.

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 2046 kN	$\sigma_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{2046e3}{2*15*370}$ =	184 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = 0, \tau_{//} = 0$; $\tau_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$	$\tau_\perp = \frac{0e3}{2*15*370}$ =	0 N/mm²
3 	F_{Ed} 0 kN	$\tau_{//} = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$; $\sigma_\perp = \tau_\perp = 0$	$\tau_{//} = \frac{0e3}{2*15*370}$ =	0 N/mm²
4 	M_{Ed} 10 kNm	$n = 2$: $\sigma_\perp = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$; $n = 1$ and $\sigma\perp$ OK: $\sigma_\perp = \frac{M_{Ed}}{1/6 \cdot a^2 \cdot l_{eff}}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{10.2e6}{(30-15)*15*370}$ =	123 N/mm²
5 	M_{Ed} 0 kNm	$\sigma_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{0e6}{2*1/6*15*370^2}$ =	0 N/mm²
6 	M_{Ed} 0 kNm	$\sigma_\perp = 0$; $l_{eff} \le 2 \cdot b$ and $n = 2$: $\tau_{//} = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$; $l_{eff} > 2 \cdot b$ or $l_{eff} \le 2 \cdot b$ and $n = 1$: $\tau_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$	$\tau_{//} = \frac{0e6}{2*1/6*15*370^2}$ =	0 N/mm²

Material information - Steel grade S690

Yield strength	f_y =	690	N/mm²	
(Nominal) tensile strength	f_u =	770	N/mm²	
Correlation factor	β =	1		

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$ =	356	N/mm²	
Normal stress resistance	$f_{\perp,d}$ =	554	N/mm²	
Combined stress resistance	$f_{w,u,d}$ =	616	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$ =	0	N/mm²	= 0
Total actual normal stress	$\Sigma\tau_\perp$ =	0	N/mm²	= 0+0
Total actual normal stress	$\Sigma\sigma_\perp$ =	307	N/mm²	= 0+123+184
Actual combined stress	$\sigma_{w,u,d}$ =	307	N/mm²	= √(307+3·(0²+0²))

Unity checks welds

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3} \cdot \beta \cdot \gamma_{M2}}\right)} \le 1 = \frac{0}{356} = 0.00 \le 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9 \cdot f_u}{\gamma_{M2}}\right)} \le 1 = \frac{307}{554} = 0.55 \le 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta \cdot \gamma_{M2}}\right)} \le 1 = \frac{307}{616} = 0.50 \le 1$ OK

Unity Checks base material

Shear stress: $\frac{\Sigma\tau_{//}}{f_{w,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_y}{\sqrt{3}}\right)} \le 1 = \frac{0}{398} = 0.00 \le 1$ OK

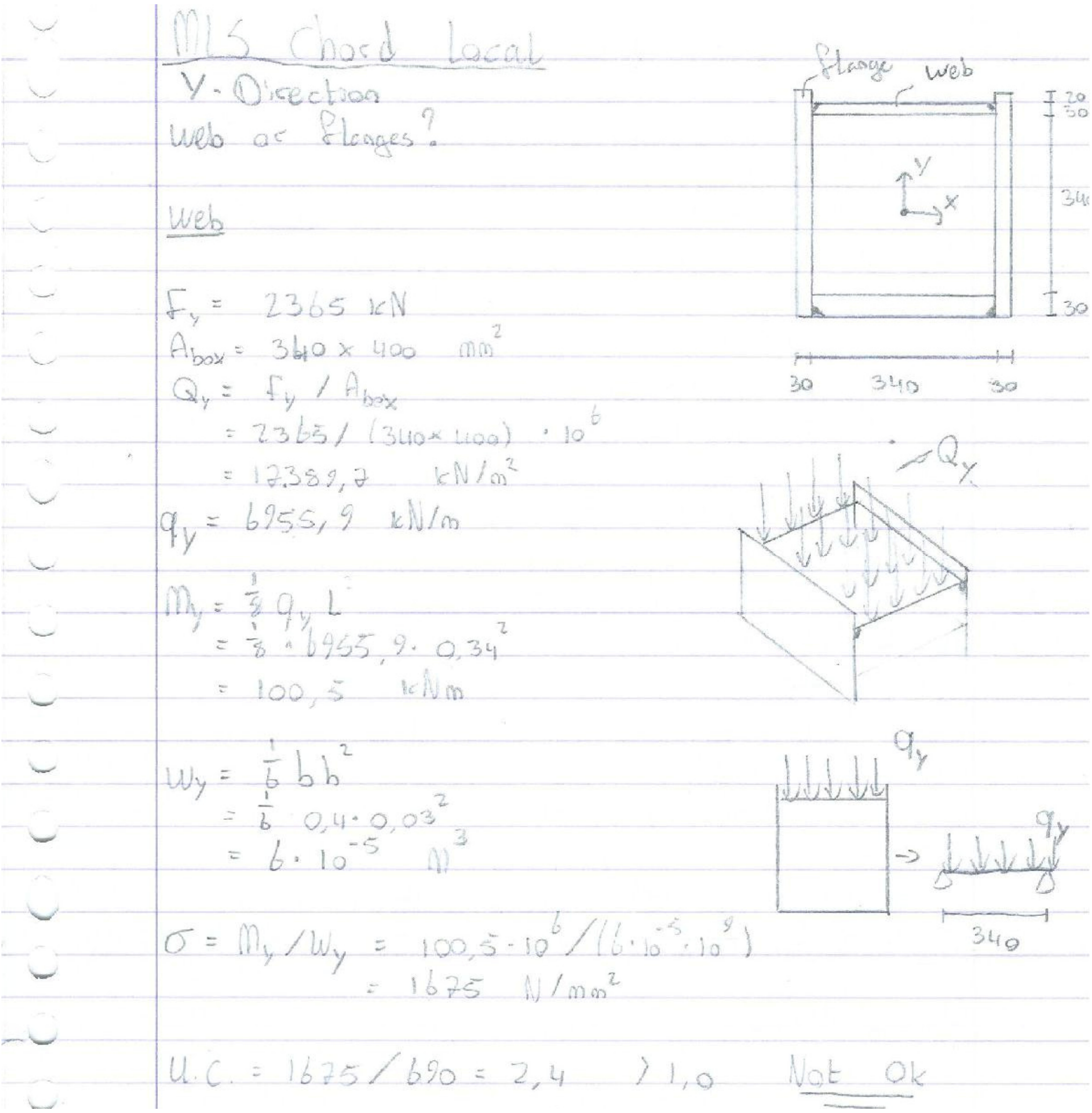
Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{f_y} \le 1 = \frac{307}{690} = 0.45 \le 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{f_y} \le 1 = \frac{307}{690} = 0.45 \le 1$ OK

Figure D-109 Weld load case IV weak position

Y direction

First, it will be checked if the box load can be taken by the web. See Figure D-110 for the hand calculation. This proved not strong enough. Therefore, Figure D-110 calculated the strength of the flanges. Figure D-111 shows the calculation of the welds. Both proved to be strong enough.



MLS Chord load

Y-Direction

Web or flanges?

Web

$F_y = 2365$ kN

$A_{box} = 340 \times 400$ mm^2

$Q_y = F_y / A_{box}$

$= 2365 / (340 \times 400) \cdot 10^6$

$= 17389,7$ kN/m^2

$q_y = 6955,9$ kN/m

$M_y = \frac{1}{8} q_y L^2$

$= \frac{1}{8} \cdot 6955,9 \cdot 0,34^2$

$= 100,5$ kNm

$W_y = \frac{1}{6} b h^2$

$= \frac{1}{6} \, 0,4 \cdot 0,03^2$

$= 6 \cdot 10^{-5}$ m^3

$\sigma = M_y / W_y = 100,5 \cdot 10^6 / (6 \cdot 10^{-5} \cdot 10^9)$

$= 1675$ N/mm^2

U.C. $= 1675 / 690 = 2,4 > 1,0$ Not Ok

Flanges

Compression $A_{fl} = 30 \times 400$

$= 12{,}000 \ mm^2$

$F_y = 2365 \ kN$

$\sigma = \frac{1}{2} f_y / A_{fl}$

$= \frac{1}{2} \cdot 2365 \cdot 10^3 / 12000$

$= 98{,}5 \ N/mm^2$

U.C. = 98,5 / 690 = 0,14 < 1,0 OK

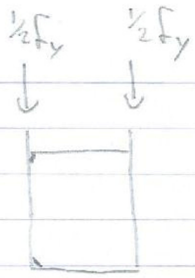


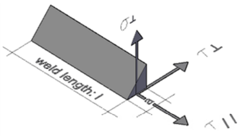
Figure D-110 Horizontal force resistance Y direction MLS chord hand calculation

MAMMOET CALCULATION

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for PJP welds - Acc. EN-1993-1-8 Chp. 4 Calculation sheet 050900012

Detail



Input parameters

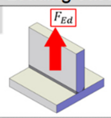
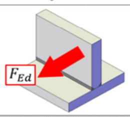
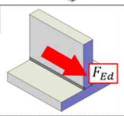
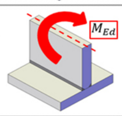
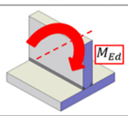
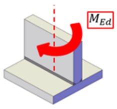
Steel material		S690		
Weld throat	a	10	mm	
Plate width	b	30	mm	
Fysical length weld	l	400	mm	
Eff. weld length	l_{eff}	380	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	2.00		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1.

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 0 kN	$\sigma_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$; $\tau_{//} = 0$, $\tau_\perp = 0$	$\sigma_\perp = \frac{0e3}{2*10*380} =$	0 N/mm²
2 	F_{Ed} 1183 kN	$\sigma_\perp = 0, \tau_{//} = 0$; $\tau_\perp = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$	$\tau_\perp = \frac{1182.5e3}{2*10*380} =$	156 N/mm²
3 	F_{Ed} 0 kN	$\tau_{//} = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$; $\sigma_\perp = \tau_\perp = 0$	$\tau_{//} = \frac{0e3}{2*10*380} =$	0 N/mm²
4 	M_{Ed} 0 kNm	$n = 2$: $\sigma_\perp = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$; $n = 1$ and σ⊥ OK: $\sigma_\perp = \frac{M_{Ed}}{1/6 \cdot a^2 \cdot l_{eff}}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{0e6}{(30-10)*10*380} =$	0 N/mm²
5 	M_{Ed} 0 kNm	$\sigma_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$; $\tau_{//} = 0, \tau_\perp = 0$	$\sigma_\perp = \frac{0e6}{2*1/6*10*380^2} =$	0 N/mm²
6 	M_{Ed} 0 kNm	$\sigma_\perp = 0$; $l_{eff} \le 2 \cdot b$ and $n = 2$: $\tau_{//} = \frac{M_{Ed}}{(b-a) \cdot a \cdot l_{eff}}$; $l_{eff} > 2 \cdot b$ or $l_{eff} \le 2 \cdot b$ and $n = 1$: $\tau_\perp = \frac{M_{Ed}}{n \cdot \frac{1}{6} \cdot a \cdot l_{eff}^2}$	$\tau_{//} = \frac{0e6}{2*1/6*10*380^2} =$	0 N/mm²

Material information - Steel grade ***S690***

Yield strength	f_y	=	690	N/mm²	
(Nominal) tensile strength	f_u	=	770	N/mm²	
Correlation factor	β	=	1		

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	356	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	554	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	616	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	0	N/mm²	= 0
Total actual normal stress	$\Sigma\tau_\perp$	=	156	N/mm²	= 156+0
Total actual normal stress	$\Sigma\sigma_\perp$	=	0	N/mm²	= 0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	269	N/mm²	= √(0+3·(0²+156²))

Unity checks welds

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3} \cdot \beta \cdot \gamma_{M2}}\right)} \le 1 = \frac{0}{356} =$ 0.00 ≤ 1 OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9 \cdot f_u}{\gamma_{M2}}\right)} \le 1 = \frac{0}{554} =$ 0.00 ≤ 1 OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta \cdot \gamma_{M2}}\right)} \le 1 \quad \frac{269}{616} =$ 0.44 ≤ 1 OK

Unity Checks base material

Shear stress: $\frac{\Sigma\tau_{//}}{f_{w,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_y}{\sqrt{3}}\right)} \le 1 = \frac{0}{398} =$ 0.00 ≤ 1 OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{f_y} \le 1 = \frac{0}{690} =$ 0.00 ≤ 1 OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{f_y} \le 1 = \frac{269}{690} =$ 0.39 ≤ 1 OK

Figure D-111 MLS chord weld check

D.3.1.3. Stability beams (blue beams)

!NOTE: CALCULATIONS ARE BASED ON OLDER SCIA MODEL. AN UPDATE IS REQUIRED!

It has to be checked that all shear will be taken by the webs and all compression will be taken by the flanges as that is where these loads are imposed. The following checks are performed.

Stability Beams (Blue beams)

$V_{Ed} = 1676 \text{ kN}$

$\tau = 355/\sqrt{3}$

$= 205 \text{ N/mm}^2$

$A_{webs} = 2 \cdot 20 \cdot 320$

$= 12{,}800 \text{ mm}^2$

$V_{Rd} = \tau \cdot A_{webs}$

$= 205 \cdot 12{,}800 \cdot 10^{-3}$

$= 2623 \text{ kN}$

$U.C. = V_{Ed} / V_{Rd} = 1676/2623$

$= 0{,}64 < 1{,}0$ Ok

$N_{Ed} = 2365 \text{ kN}$

$f_y = 355 \text{ N/mm}^2$

$A_{flanges} = 2 \cdot 30 \cdot 400$ *force introduction

$= 24{,}000 \text{ mm}^2$

$N_{Rd} = f_y \cdot A_{flanges}$

$= 355 \cdot 24{,}000 \cdot 10^{-3}$

$= 8520 \text{ kN}$

$U.C. = N_{Ed} / N_{Rd} = 2365/8520$

$= 0{,}28 < 1{,}0$ Ok

Figure D-112 Shear force and compression check stability beams hand calculation

D.3.1.4. End plate to stability beam

!NOTE: CALCULATIONS ARE BASED ON OLDER SCIA MODEL. AN UPDATE IS REQUIRED!

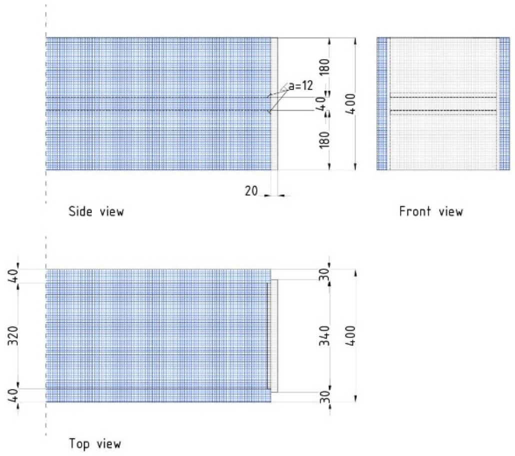


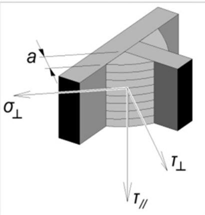
Figure D-113 End plate to stability beam connection

MAMMOET CALCULATION

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4 Calculation sheet 050900011

Detail



Input parameters

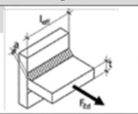
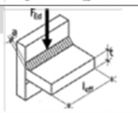
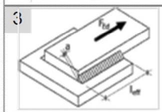
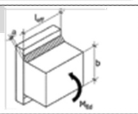
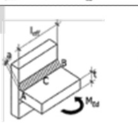
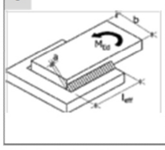
Steel material		S355		
Weld throat	a	12	mm	
Plate width	b	40	mm	
Fysical length weld	l	320	mm	
Eff. weld length	l_{eff}	296	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 0 kN	$\sigma_{\perp}=\tau_{\perp}=\frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$ $\tau_{//}=0$	= 0e3*2^0.5 / 2*2*12*296 = = 0 =	0 N/mm²
2 	F_{Ed} 0 kN	$\sigma_{\perp}=\tau_{\perp}=\frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$ $\tau_{//}=0$	= 0e3*2^0.5 / 2*2*12*296 = = 0 =	0 N/mm²
3 	F_{Ed} 1676 kN	$\sigma_{\perp}=\tau_{\perp}=0$ $\tau_{//}=\frac{F_{Ed}}{n\cdot a\cdot l_{eff}}$	= 0 = = 1676e3 / 2*12*296 =	236 N/mm²
4 	M_{Ed} 0 kNm	$\sigma_{\perp}=\tau_{\perp}=\frac{M_{Ed}}{\sqrt{2}\cdot a\cdot l_{eff}\cdot\left(b+\frac{2}{3}a\sqrt{2}\right)}$ $\tau_{//}=0$	= 0e6 / 2^0.5*12*296*[40+(2/3)*12*2^0.5] = = 0 =	0 N/mm²
5 	M_{Ed} 0 kNm	In A,B $\sigma_{\perp}=\tau_{\perp}=\frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$ In C $\sigma_{\perp}=\tau_{\perp}=0$ In A,B,C $\tau_{//}=0$	= 6*0e6*2^0.5 / 2*2*12*296^2 = = 0 = = 0 =	0 N/mm²
6 	M_{Ed} 0 kNm	l_{eff}≤2b and n=2: $\tau_{//}=\frac{M_{Ed}}{a\cdot l_{eff}\cdot\left(b+\frac{2}{3}\cdot a\cdot\sqrt{2}\right)}$ $\sigma_{\perp}=\tau_{\perp}=0$ l_{eff}>2b or l_{eff}≤2b and n=1 $\sigma_{\perp}=\tau_{\perp}=\frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$ $\tau_{//}=0$	= -- / -- = = -- = = 6*0e6*2^0.5 / 2*2*12*296^2 = = 0 =	0 N/mm² 0 N/mm²

Material information - Steel grade ***S355***

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method *(EN 1993-1-8: 2005, article 4.5.3.2)*

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	236	N/mm²	236
Total actual normal stress	$\Sigma\sigma_{\perp}$	=	0	N/mm²	0+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	409	N/mm²	=[0^2+3*(0^2+236^2)]^0.5

Unity checks

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}}=\frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3}\cdot\beta\cdot\gamma_{M2}}\right)}\leq 1=\frac{236}{262}=$ 0.90 ≤ 1 OK

Normal stress: $\frac{\Sigma\sigma_{\perp}}{f_{\perp,d}}=\frac{\Sigma\sigma_{\perp}}{\left(\frac{0.9\cdot f_u}{\gamma_{M2}}\right)}\leq 1=\frac{0}{367}=$ 0.00 ≤ 1 OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}}=\frac{\sqrt{\sigma_{\perp}^2+3\cdot(\tau_{\perp}^2+\tau_{//}^2)}}{\left(\frac{f_u}{\beta\cdot\gamma_{M2}}\right)}\leq 1=\frac{409}{453}=$ 0.90 ≤ 1 OK

Figure D-114 End plate to stability beam weld check

D.3.1.5. Stability beam to bottom horizontal X brace

!NOTE: CALCULATIONS ARE BASED ON OLDER SCIA MODEL. AN UPDATE IS REQUIRED!

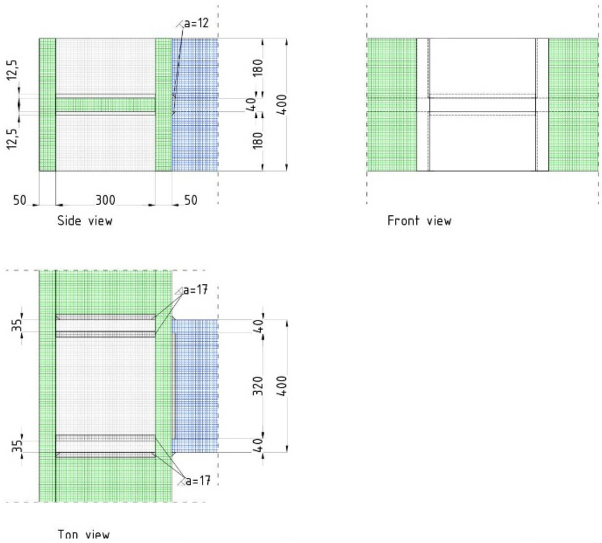
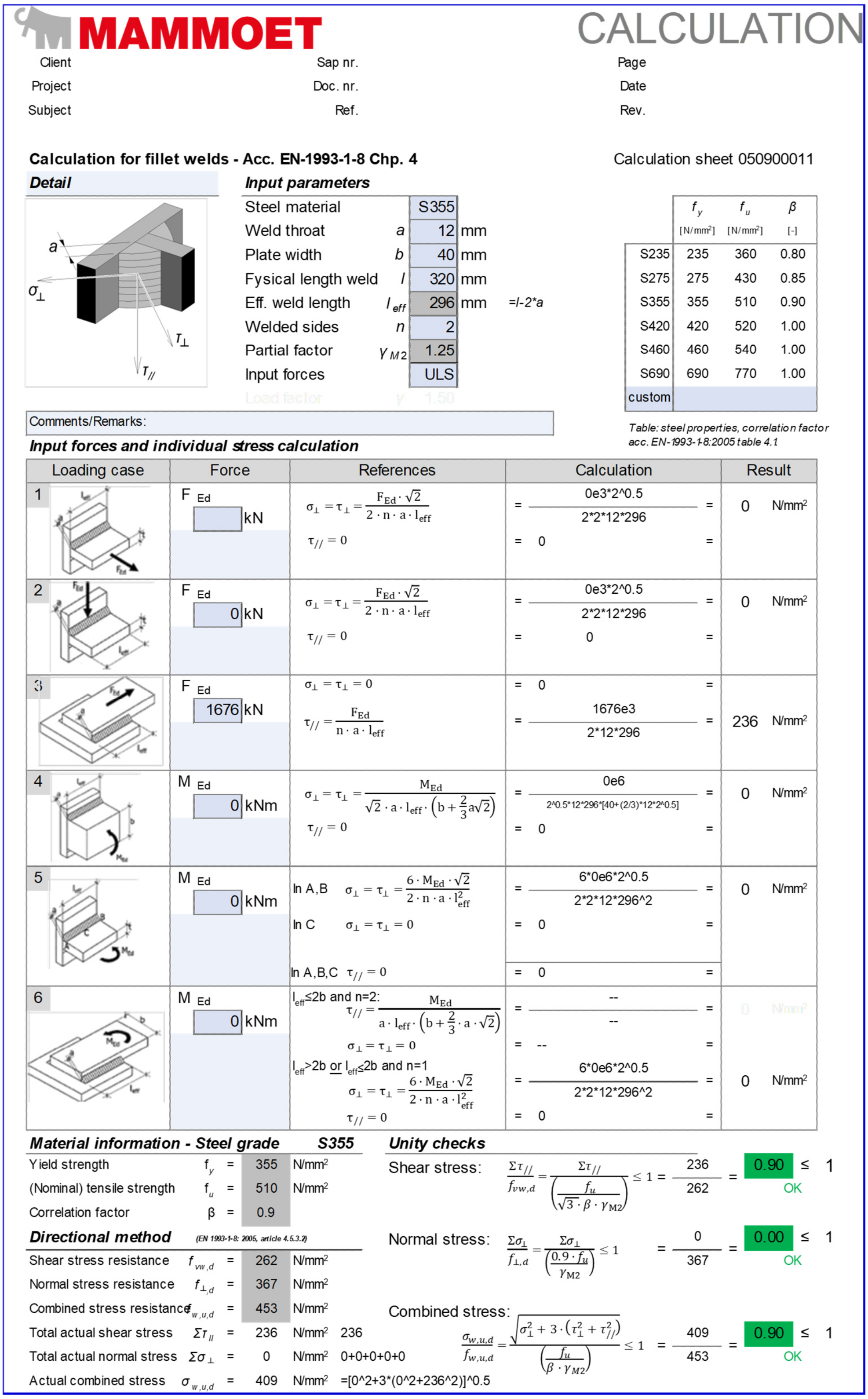


Figure D-115 Stability beam to bottom horizontal X brace connection



MAMMOET CALCULATION

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4 — Calculation sheet 050900011

Detail

Input parameters

Steel material		S355		
Weld throat	a	12	mm	
Plate width	b	40	mm	
Fysical length weld	l	320	mm	
Eff. weld length	l_{eff}	296	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1	F_{Ed} kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}}$; $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*12*296 = ; = 0 =	0 N/mm²
2	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}}$; $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*12*296 = ; = 0 =	0 N/mm²
3	F_{Ed} 1676 kN	$\sigma_\perp = \tau_\perp = 0$; $\tau_{//} = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$	= 0 = ; = 1676e3 / 2*12*296 =	236 N/mm²
4	M_{Ed} 0 kNm	$\sigma_\perp = \tau_\perp = \frac{M_{Ed}}{\sqrt{2} \cdot a \cdot l_{eff} \cdot \left(b + \frac{2}{3} a\sqrt{2}\right)}$; $\tau_{//} = 0$	= 0e6 / 2^0.5*12*296*[40+(2/3)*12*2^0.5] = ; = 0 =	0 N/mm²
5	M_{Ed} 0 kNm	In A,B $\sigma_\perp = \tau_\perp = \frac{6 \cdot M_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}^2}$; In C $\sigma_\perp = \tau_\perp = 0$; In A,B,C $\tau_{//} = 0$	= 6*0e6*2^0.5 / 2*2*12*296^2 = ; = 0 = ; = 0 =	0 N/mm²
6	M_{Ed} 0 kNm	l_{eff}≤2b and n=2: $\tau_{//} = \frac{M_{Ed}}{a \cdot l_{eff} \cdot \left(b + \frac{2}{3} \cdot a \cdot \sqrt{2}\right)}$; $\sigma_\perp = \tau_\perp = 0$; l_{eff}>2b or l_{eff}≤2b and n=1: $\sigma_\perp = \tau_\perp = \frac{6 \cdot M_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}^2}$; $\tau_{//} = 0$	= -- / -- = ; = -- = ; = 6*0e6*2^0.5 / 2*2*12*296^2 = ; = 0 =	0 N/mm²; 0 N/mm²

Material information - Steel grade S355

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	236	N/mm²	236
Total actual normal stress	$\Sigma\sigma_\perp$	=	0	N/mm²	0+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	409	N/mm²	=[0^2+3*(0^2+236^2)]^0.5

Unity checks

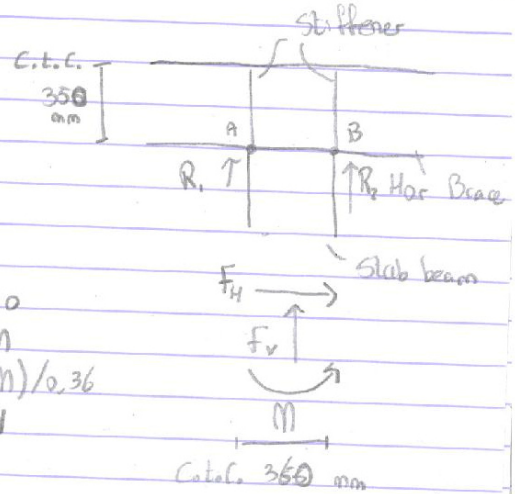
Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3} \cdot \beta \cdot \gamma_{M2}}\right)} \le 1$ = 236 / 262 = 0.90 ≤ 1 OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9 \cdot f_u}{\gamma_{M2}}\right)} \le 1$ = 0 / 367 = 0.00 ≤ 1 OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta \cdot \gamma_{M2}}\right)} \le 1$ = 409 / 453 = 0.90 ≤ 1 OK

Figure D-116 Weld stability beam to bottom horizontal X brace check

Shear Stress Web Horizontal Brace X



$F_H = 1676$ kN
$F_V = 2365$ kN
$M = F_H \cdot 0,7$
$= 1676 \cdot 0,7 = 1173$ kNm

$\Sigma T|A = 0 : 0,18 \cdot F_V + 0,36 R_2 + M = 0$
$0,36 R_2 = -0,18 F_V - M$
$R_2 = (-0,18 F_V - M)/0,36$
$= -4441$ kN

$R_1 = -F_V - R_2$
$= -2365 + 4441$
$= 2076$ kN

$\tau_{web} = |R_2 \cdot 10^3| / (350 \cdot t_{web})$
$= 4441 \cdot 10^3 / (350 \cdot 40)$
$= 317,2$ N/mm²

$\sigma_{web} = \tau_{web} \cdot \sqrt{3}$
$= 549,4$ N/mm²

U.C. $= 549,4/355$
$= 1,55 > 1,0$ Not ok

$t_{web} \rightarrow 65$ mm $\rightarrow$ U.C. $= 0,95 < 1,0$ ok

Figure D-117 Shear stress web horizontal X brace check hand calculation

Stiffener thickness

$N = 4441$ KN
$\sigma = 355$ N/mm^2

$A = 4441 \cdot 10^3 / 355$
$= 12{,}509$ mm^2

$h_{stiff} = 400 - 40$
$= 360$ mm

$t_{stiff} = 12{,}509 / 360$
$= 34{,}7$ mm $\rightarrow 35$ mm

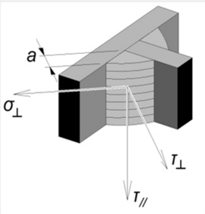
Figure D-118 Stiffener thickness hand calculation

MAMMOET CALCULATION

Client	Sap nr.	Page
Project	Doc. nr.	Date
Subject	Ref.	Rev.

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4 Calculation sheet 050900011

Detail



Input parameters

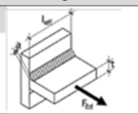
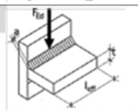
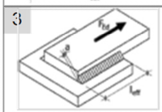
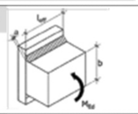
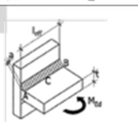
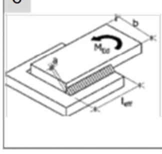
Steel material		S355		
Weld throat	a	17	mm	
Plate width	b	35	mm	
Fysical length weld	l	300	mm	
Eff. weld length	l_{eff}	266	mm	$=l-2*a$
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$ $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*17*266 = = 0 =	0 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$ $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*17*266 = = 0 =	0 N/mm²
3 	F_{Ed} 2221 kN	$\sigma_\perp = \tau_\perp = 0$ $\tau_{//} = \frac{F_{Ed}}{n\cdot a\cdot l_{eff}}$	= 0 = = 2220.5e3 / 2*17*266 =	246 N/mm²
4 	M_{Ed} 0 kNm	$\sigma_\perp = \tau_\perp = \frac{M_{Ed}}{\sqrt{2}\cdot a\cdot l_{eff}\cdot\left(b+\frac{2}{3}a\sqrt{2}\right)}$ $\tau_{//} = 0$	= 0e6 / 2^0.5*17*266*[35+(2/3)*17*2^0.5] = = 0 =	0 N/mm²
5 	M_{Ed} 0 kNm	In A,B $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$ In C $\sigma_\perp = \tau_\perp = 0$ In A,B,C $\tau_{//} = 0$	= 6*0e6*2^0.5 / 2*2*17*266^2 = = 0 = = 0 =	0 N/mm²
6 	M_{Ed} 0 kNm	l_{eff}≤2b and n=2: $\tau_{//} = \frac{M_{Ed}}{a\cdot l_{eff}\cdot\left(b+\frac{2}{3}\cdot a\cdot\sqrt{2}\right)}$ $\sigma_\perp = \tau_\perp = 0$ l_{eff}>2b or l_{eff}≤2b and n=1 $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$ $\tau_{//} = 0$	= -- / -- = = -- = = 6*0e6*2^0.5 / 2*2*17*266^2 = = 0 =	0 N/mm² 0 N/mm²

Material information - Steel grade S355

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	246	N/mm²	246
Total actual normal stress	$\Sigma\sigma_\perp$	=	0	N/mm²	0+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	425	N/mm²	=[0^2+3*(0^2+246^2)]^0.5

Unity checks

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3}\cdot\beta\cdot\gamma_{M2}}\right)} \le 1 = \frac{246}{262} = 0.94 \le 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9\cdot f_u}{\gamma_{M2}}\right)} \le 1 = \frac{0}{367} = 0.00 \le 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3\cdot(\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta\cdot\gamma_{M2}}\right)} \le 1 = \frac{425}{453} = 0.94 \le 1$ OK

Figure D-119 Stiffening plates welds

D.3.1.6. Bottom horizontal X brace to bottom chord

!NOTE: CALCULATIONS ARE BASED ON OLDER SCIA MODEL. AN UPDATE IS REQUIRED!

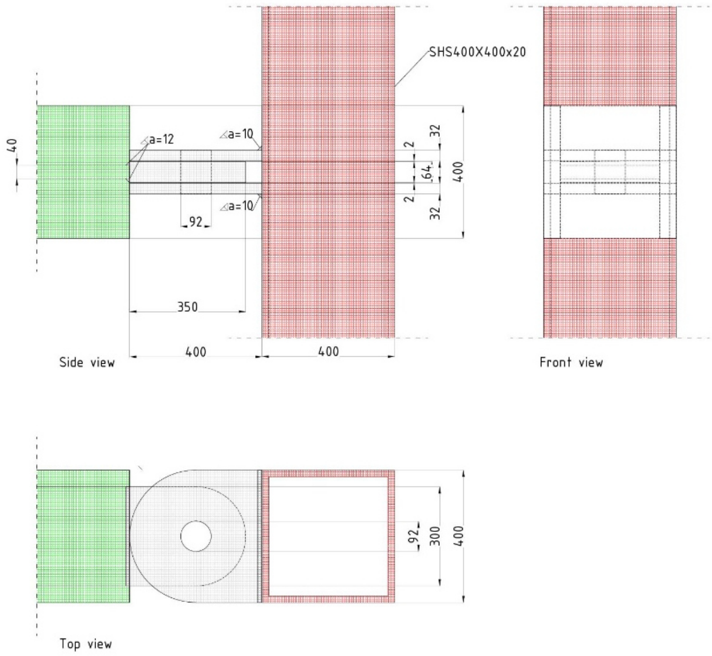


Figure D-120 Bottom horizontal X brace to bottom chord connection

Forces on connection:
N = 1429 kN
V = 940 kN

Combined = sqrt(1429^2+940^2)=1710 kN

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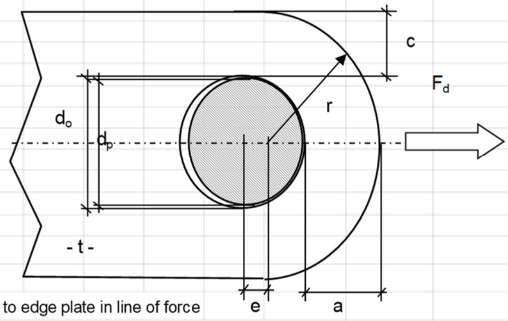
STANDARD CALCULATION 3.6A

Client:	MAMMOET Solutions	Sap nr.:		Page:	1 of 1
Project:	Rekenboek benchmark	Doc. nr.:	0010012345-C00-01	Date:	19/10/2024
Subject:	Pin connection	Ref.:		Rev.:	01

EN 1993-1-8:2005 art. 3.13 Pin connections - Check plate geometry



Input cells are marked yellow

Pin replaceable **Yes**

Geometry

r	=	200	mm	radius
e	=	0	mm	eccentricity
d_o	=	92	mm	diameter pin hole
d	=	90	mm	diameter pin
a	=	154	mm	distance edge pin hole to edge plate in line of force
c	=	154	mm	distance edge pin hole to edge plate perpendicular to line of force

Material

t	=	64	mm	thickness plate
f_y	=	355	N/mm^2	yield strength plate material
γ_{M0}	=	1.00	[-]	partial factor, by default; γ_{M0} = 1.00 EN 1993-1-8:2005
$\gamma_{M6,ser}$	=	1.00	[-]	Serviceability factor for replaceable pins ; $\gamma_{M6,ser}$ = 1.00 acc. EN 1993-1-8:2005

Load

	F_d	=	1710	kN	actual load (without load factors)
	γ_q	=	1.00	[-]	load factor
	γ_{dyn}	=	1.00	[-]	dynamic amplification factor (if not applicable γ_{dyn} = 1.00)
	$\gamma_{overall}$	=	1.00	[-]	overall safety = γ_q x γ_{dyn}
[ULS]	F_{Ed}	=	1710	kN	design load = F_d x γ_q x γ_{dyn}
[SLS]	$F_{b,Ed;ser}$	=	1710	kN	design load = F_d x γ_{dyn}

Decisive check: 0.93 Bearing strength for serviceability

Check strength in line of force

a	≥	$F_{t,Rd}$ / (2 x t x f_y) + 2/3 x d_o	Formula table 3.9
$F_{t,Rd}$	=	(a -2/3 x d_o) x 2 x t x f_y	$F_{t,Rd}$ = plate strength based on dimension a
$F_{t,Rd}$	=	(154 - 2/3 x 92) x 2 x 64 x 355	
$F_{t,Rd}$	=	4211 kN	

F_{Ed}		1710				
$F_{t,Rd}$	=	4211	=	0.41	< 1.00	OK

Check strength perpendicular to line of force

c	≥	$F_{t,Rd}$ / (2 x t x f_y) + 1/3 x d_o	Formula table 3.9
$F_{t,Rd}$	=	(c -1/3 x d_o) x 2 x t x f_y	$F_{t,Rd}$ = plate strength based on dimension c
$F_{t,Rd}$	=	(154 - 1/3 x 92) x 2 x 64 x 355	
$F_{t,Rd}$	=	5604 kN	

F_{Ed}		1710				
$F_{t,Rd}$	=	5604	=	0.31	< 1.00	OK

Check ULS bearing strength

$F_{b;Rd}$	=	1.5 x d x t x f_y	Formula table 3.10
$F_{b;Rd}$	=	1.5 x 90 x 64 x 355	
$F_{b;Rd}$	=	3067 kN	

F_{Ed}		1710				
$F_{b,Rd}$	=	3067	=	0.56	< 1.00	OK

Check bearing strength for serviceability (only applicable for replaceable pin)

$F_{b;Rd,ser}$	=	0.9 x t x dp x fy
$F_{b;Rd,ser}$	=	0.9 x 64x90x355
$F_{b;Rd,ser}$	=	1840 kN

$F_{b,Ed,ser}$		1710				
$F_{b;Rd,ser}$	=	1840	=	0.93	< 1.00	OK

Figure D-121 Caluclation plates welded to column

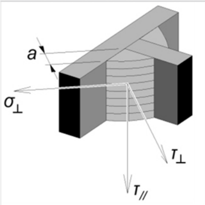
MAMMOET CALCULATION

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4

Calculation sheet 050900011

Detail



Input parameters

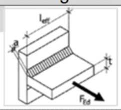
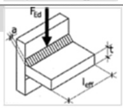
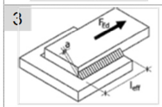
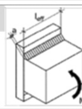
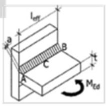
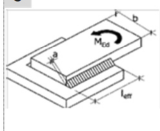
Steel material		S355		
Weld throat	a	10	mm	
Plate width	b	64	mm	
Fysical length weld	l	400	mm	
Eff. weld length	l_{eff}	380	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1.

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 1429 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$; $\tau_{//} = 0$	= 1429e3*2^0.5 / 2*2*10*380 = ; = 0 =	133 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$; $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*10*380 = ; = 0 =	0 N/mm²
3 	F_{Ed} 940 kN	$\sigma_\perp = \tau_\perp = 0$; $\tau_{//} = \frac{F_{Ed}}{n\cdot a\cdot l_{eff}}$	= 0 = ; = 940e3 / 2*10*380 =	124 N/mm²
4 	M_{Ed} 0 kNm	$\sigma_\perp = \tau_\perp = \frac{M_{Ed}}{\sqrt{2}\cdot a\cdot l_{eff}\cdot\left(b+\frac{2}{3}a\sqrt{2}\right)}$; $\tau_{//} = 0$	= 0e6 / 2^0.5*10*380*[64+(2/3)*10*2^0.5] = ; = 0 =	0 N/mm²
5 	M_{Ed} 0 kNm	In A,B $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$; In C $\sigma_\perp = \tau_\perp = 0$; In A,B,C $\tau_{//} = 0$	= 6*0e6*2^0.5 / 2*2*10*380^2 = ; = 0 = ; = 0 =	0 N/mm²
6 	M_{Ed} 0 kNm	$l_{eff}\leq 2b$ and n=2: $\tau_{//} = \frac{M_{Ed}}{a\cdot l_{eff}\cdot\left(b+\frac{2}{3}\cdot a\cdot\sqrt{2}\right)}$; $\sigma_\perp = \tau_\perp = 0$; $l_{eff}>2b$ or $l_{eff}\leq 2b$ and n=1 $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$; $\tau_{//} = 0$	= – / – = ; = – = ; = 6*0e6*2^0.5 / 2*2*10*380^2 = ; = 0 =	0 N/mm² ; 0 N/mm²

Material information - Steel grade ***S355***

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method *(EN 1993-1-8: 2005, article 4.5.3.2)*

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	124	N/mm²	124
Total actual normal stress	$\Sigma\sigma_\perp$	=	133	N/mm²	133+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	341	N/mm²	=[133^2+3*(133^2+124^2)]^0.5

Unity checks

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3}\cdot\beta\cdot\gamma_{M2}}\right)} \leq 1 = \frac{124}{262} = 0.47 \leq 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9\cdot f_u}{\gamma_{M2}}\right)} \leq 1 = \frac{133}{367} = 0.36 \leq 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2+3\cdot\left(\tau_\perp^2+\tau_{//}^2\right)}}{\left(\frac{f_u}{\beta\cdot\gamma_{M2}}\right)} \leq 1 = \frac{341}{453} = 0.75 \leq 1$ OK

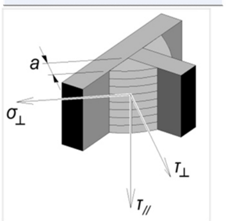
Figure D-122 Weld check plate to bottom column

MAMMOET **CALCULATION**

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4 Calculation sheet 050900011

Detail



Input parameters

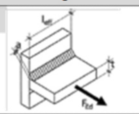
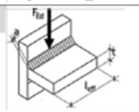
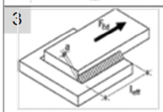
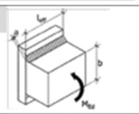
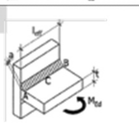
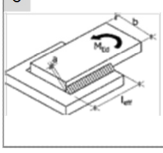
Steel material		S355		
Weld throat	a	12	mm	
Plate width	b	64	mm	
Fysical length weld	l	300	mm	
Eff. weld length	l_{eff}	276	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 1429 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$; $\tau_{//} = 0$	= 1429e3*2^0.5 / 2*2*12*276 = ; = 0 =	153 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$; $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*12*276 = ; = 0 =	0 N/mm²
3 	F_{Ed} 940 kN	$\sigma_\perp = \tau_\perp = 0$; $\tau_{//} = \frac{F_{Ed}}{n\cdot a\cdot l_{eff}}$	= 0 = ; = 940e3 / 2*12*276 =	142 N/mm²
4 	M_{Ed} 0 kNm	$\sigma_\perp = \tau_\perp = \frac{M_{Ed}}{\sqrt{2}\cdot a\cdot l_{eff}\cdot\left(b+\frac{2}{3}a\sqrt{2}\right)}$; $\tau_{//} = 0$	= 0e6 / 2^0.5*12*276*[64+(2/3)*12*2^0.5] = ; = 0 =	0 N/mm²
5 	M_{Ed} 0 kNm	In A,B $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$; In C $\sigma_\perp = \tau_\perp = 0$; In A,B,C $\tau_{//} = 0$	= 6*0e6*2^0.5 / 2*2*12*276^2 = ; = 0 = ; = 0 =	0 N/mm²
6 	M_{Ed} 0 kNm	l_{eff}≤2b and n=2: $\tau_{//} = \frac{M_{Ed}}{a\cdot l_{eff}\cdot\left(b+\frac{2}{3}\cdot a\cdot\sqrt{2}\right)}$; $\sigma_\perp = \tau_\perp = 0$; l_{eff}>2b or l_{eff}≤2b and n=1: $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$; $\tau_{//} = 0$	= -- / -- = ; = -- = ; = 6*0e6*2^0.5 / 2*2*12*276^2 = ; = 0 =	0 N/mm²; 0 N/mm²

Material information - Steel grade S355

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	142	N/mm²	142
Total actual normal stress	$\Sigma\sigma_\perp$	=	153	N/mm²	153+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	392	N/mm²	=[153^2+3*(153^2+142^2)]^0.5

Unity checks

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3}\cdot\beta\cdot\gamma_{M2}}\right)} \le 1 = \frac{142}{262} = 0.54 \le 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9\cdot f_u}{\gamma_{M2}}\right)} \le 1 = \frac{153}{367} = 0.42 \le 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3\cdot\left(\tau_\perp^2+\tau_{//}^2\right)}}{\left(\frac{f_u}{\beta\cdot\gamma_{M2}}\right)} \le 1 = \frac{392}{453} = 0.86 \le 1$ OK

Figure D-123 Weld end plate to bottom horizontal X brace

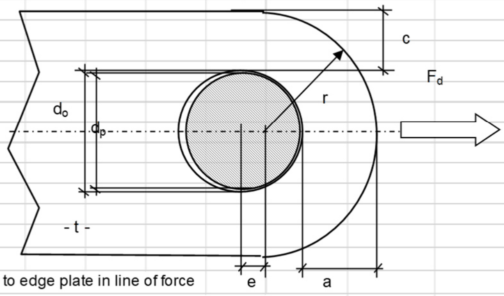
Worldwide specialists in heavy lifting and transport

MAMMOET

STANDARD CALCULATION 3.6A

Client:	MAMMOET Solutions	Sap nr.:		Page:	1 of 1
Project:	Rekenboek benchmark	Doc. nr.:	0010012345-C00-01	Date:	23/10/2024
Subject:	Pin connection	Ref.:		Rev.:	01

EN 1993-1-8:2005 art. 3.13 Pin connections - Check plate geometry



Input cells are marked yellow

Pin replaceable: **Yes**

Geometry

r	=	150	mm	radius
e	=	0	mm	eccentricity
d_o	=	92	mm	diameter pin hole
d	=	90	mm	diameter pin
a	=	104	mm	distance edge pin hole to edge plate in line of force
c	=	104	mm	distance edge pin hole to edge plate perpendicular to line of force

Material

t	=	64	mm	thickness plate
f_y	=	355	N/mm^2	yield strength plate material
γ_{M0}	=	1.00	[-]	partial factor, by default; γ_{M0} = 1.00 EN 1993-1-8:2005
$\gamma_{M6,ser}$	=	1.00	[-]	Serviceability factor for replaceable pins ; $\gamma_{M6,ser}$ = 1.00 acc. EN 1993-1-8:2005

Load

	F_d	=	1710	kN	actual load (without load factors)
	γ_q	=	1.00	[-]	load factor
	γ_{dyn}	=	1.00	[-]	dynamic amplification factor (if not applicable γ_{dyn} = 1.00)
	$\gamma_{overall}$	=	1.00	[-]	overall safety = $\gamma_q \times \gamma_{dyn}$
[ULS]	F_{Ed}	=	1710	kN	design load = $F_d \times \gamma_q \times \gamma_{dyn}$
[SLS]	$F_{b,Ed;ser}$	=	1710	kN	design load = $F_d \times \gamma_{dyn}$

Decisive check: 0.93 Bearing strength for serviceability

Check strength in line of force

a	≥	$F_{t,Rd}$ / (2 x t x f_y) + 2/3 x d_o	Formula table 3.9
$F_{t,Rd}$	=	(a -2/3 x d_o) x 2 x t x f_y	$F_{t,Rd}$ = plate strength based on dimension a
$F_{t,Rd}$	=	(104 - 2/3 x 92) x 2 x 64 x 355	
$F_{t,Rd}$	=	1939 kN	F_{Ed} / $F_{t,Rd}$ = 1710 / 1939 = 0.88 < 1.00 OK

Check strength perpendicular to line of force

c	≥	$F_{t,Rd}$ / (2 x t x f_y) + 1/3 x d_o	Formula table 3.9
$F_{t,Rd}$	=	(c -1/3 x d_o) x 2 x t x f_y	$F_{t,Rd}$ = plate strength based on dimension c
$F_{t,Rd}$	=	(104 - 1/3 x 92) x 2 x 64 x 355	
$F_{t,Rd}$	=	3332 kN	F_{Ed} / $F_{t,Rd}$ = 1710 / 3332 = 0.51 < 1.00 OK

Check ULS bearing strength

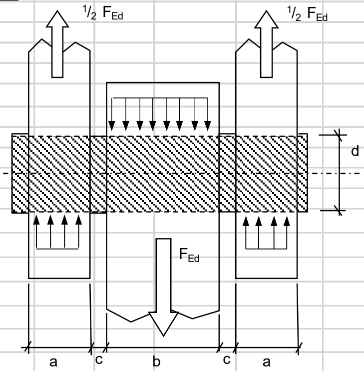
$F_{b;Rd}$	=	1.5 x d x t x f_y	Formula table 3.10
$F_{b;Rd}$	=	1.5 x 90 x 64 x 355	
$F_{b;Rd}$	=	3067 kN	F_{Ed} / $F_{b,Rd}$ = 1710 / 3067 = 0.56 < 1.00 OK

Check bearing strength for serviceability (only applicable for replaceable pin)

$F_{b;Rd,ser}$	=	0.9 x t x dp x fy	
$F_{b;Rd,ser}$	=	0.9 x 64x90x355	
$F_{b;Rd,ser}$	=	1840 kN	$F_{b,Ed,ser}$ / $F_{b;Rd,ser}$ = 1710 / 1840 = 0.93 < 1.00 OK

Figure D-124 End plates from bottom horizontal X brace

EN 1993-1-8:2005 art. 3.13 Pin connections - Check pin



Input cells are marked yellow

Pin replaceable		Yes		
Geometry				
d_p	=	90	mm	pin diameter
d	=	92	mm	diameter pin hole
a	=	32	mm	thickness outer plates
b	=	64	mm	thickness inner plate
c	=	2	mm	gap
Material				
$f_{y(b)}$	=	355	N/mm^2	yield stress material inner plate (b)
$f_{y(a)}$	=	355	N/mm^2	yield stress material outer plates (a)
f_{yp}	=	640	N/mm^2	yield stress pin material
f_{up}	=	800	N/mm^2	tensile strength pin material
γ_{M0}	=	1.00	[-]	partial factor
γ_{M2}	=	1.25	[-]	partial factor; always 1.25
k_M	=	1.00	[-]	calculation factor M_{Rd} (Km = by default 1.0)
$\gamma_{M6,ser}$	=	1.00	[-]	partial factor for replaceable pins

Static values pin

A_p	=	6362	mm^2
W_{el}	=	71569	mm^3

Bending moments pin

Max. moment

M_{Ed} = 0.125 x F_{Ed} x (b + 4 x c + (2 x a))

Bending moment at maximum shear forces

$M_{Ed,v,max}$ = 0.25 x F_{Ed} x (a + 2 x c)

	Load				
	F_d	=	1710	kN	actual load (without load factors)
	γ_q	=	1.00	[-]	load factor
	γ_{dyn}	=	1.00	[-]	dynamic amplification factor
	$\gamma_{overall}$	=	1.00	[-]	overall safety = γ_q x γ_{dyn}
[ULS]	F_{Ed}	=	1710	kN	design load = F_d x $\gamma_{overall}$
[SLS]	$F_{b,Ed,ser}$	=	1710	kN	Serviceability design load = F_d x γ_{dyn}
	M_{Ed}	=	29	kNm	design load maximum bending moment
	$M_{Ed,v,max}$	=	15	kNm	design load bending moment at maximum shear force

Decisive check	0.93	Bearing strength for serviceability plate a

Check shear of the pin (two shear planes)

$F_{v,Rd}$	=	2 x 0.6 x A_p x f_{up} / γ_{M2}							
$F_{v,Rd}$	=	2 x 0.6 x 6362 x 800 / 1.25		$F_{v,Ed}$		1710			
$F_{v,Rd}$	=	4886	kN	$F_{v,Rd}$	=	4886	=	0.35	< 1.00 OK

Check bending of the pin

M_{Rd}	=	k_M x W_{el} x f_{yp} / γ_{M0}							
M_{Rd}	=	1 x 71569 x 640		M_{Ed}		29			
M_{Rd}	=	46	kNm	M_{Rd}	=	46	=	0.63	< 1.00 OK

Check combined shear and bending of the pin

Worst case interaction between shear and bending occurs at the edge of mid plate b.

$F_{v,Ed}$			$M_{Ed,v,max}$		15.4		
$F_{v,Rd}$	=	0.35	M_{Rd}	=	45.8	=	0.34

(0.35)^2 + (0.34)^2 = 0.24 < 1.00 OK

Check bearing of the plate and pin

$F_{b;Rd}$	=	1.5 x b x d x fy(b)							
$F_{b;Rd}$	=	1.5 x 64 x 90 x 355		$F_{b,Ed}$		1710			
$F_{b;Rd}$	=	3067	kN	$F_{b;Rd}$	=	3067	=	0.56	< 1.00 OK

Check bearing strength for serviceability (only applicable for replaceable pins)

$F_{b;Rd,ser}$	=	0.9 x 2 x a x d x fy(a)							
$F_{b;Rd,ser}$	=	0.9 x 2 x32x90x355		$F_{b,Ed,ser}$		1710			
$F_{b;Rd,ser}$	=	1840	kN	$F_{b;Rd,ser}$	=	1840	=	0.93	< 1.00 OK

Check contact bearing stress pin (only applicable for replaceable pins)

contact bearing stress following Hertz — Formulas 3.15 and 3.16

$\sigma_{h;Ed}$	=	0.591 x sqrt((E x 0.5 * FEd:ser x (d0 - d))/(d^2 x a))							
$\sigma_{h;Ed}$	=	696	N/mm^2	$\sigma_{h;Ed}$		696			
				2.5 x f_y / $\gamma_{M6,ser}$	=	888	=	0.78	< 1.00 OK

Figure D-125 Pin check bottom horizontal X brace to bottom chord

D.3.1.7. Horizontal diagonal brace to stability beam and horizontal X brace

!NOTE: CALCULATIONS ARE BASED ON OLDER SCIA MODEL. AN UPDATE IS REQUIRED!

Solely a normal force is to be transferred through this connection. The magnitude is 943 kN.

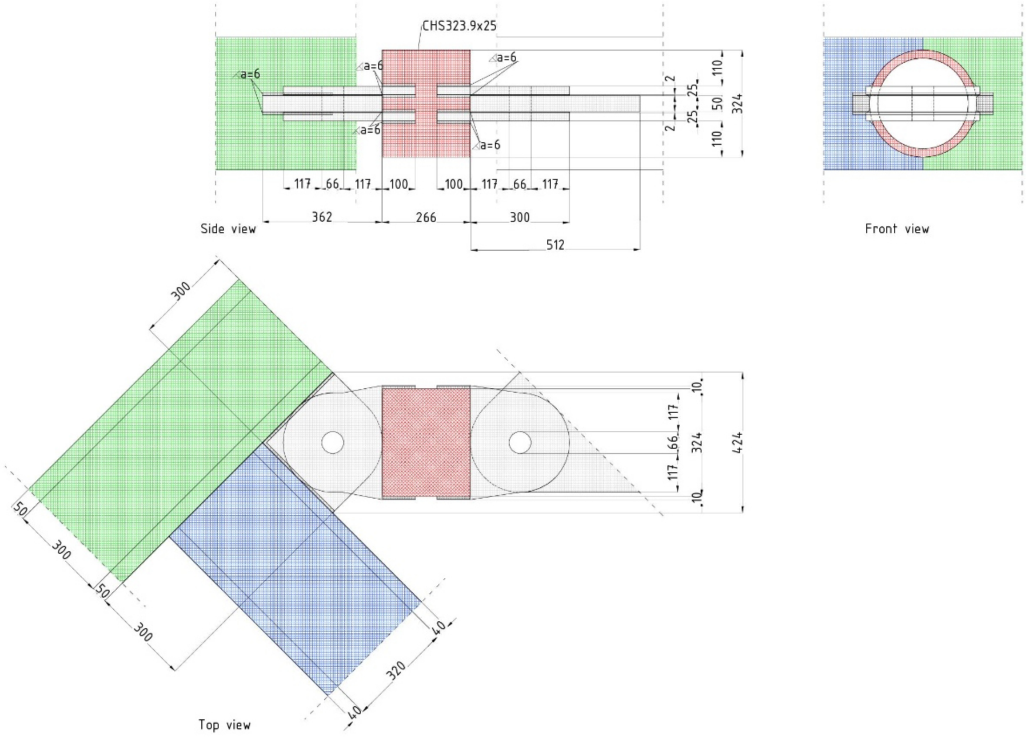


Figure D-126 Horizontal diagonal brace to stability beam and horizontal X brace connection

Worldwide specialists in heavy lifting and transport

MAMMOET

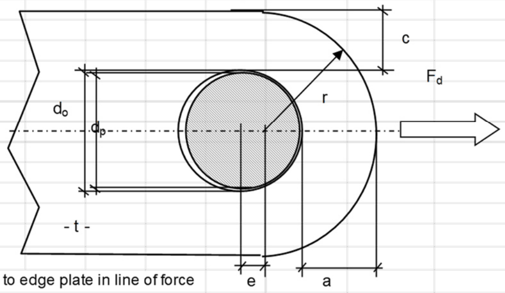
STANDARD CALCULATION 3.6A

Client:	MAMMOET Solutions	Sap nr.:		Page:	1 of 1
Project:	Rekenboek benchmark	Doc. nr.:	0010012345-C00-01	Date:	23/10/2024
Subject:	Pin connection	Ref.:		Rev.:	01

EN 1993-1-8:2005 art. 3.13 Pin connections - Check plate geometry

Input cells are marked yellow

Pin replaceable **Yes**



Geometry

r	=	150	mm	radius
e	=	0	mm	eccentricity
d_o	=	66	mm	diameter pin hole
d	=	64	mm	diameter pin
a	=	117	mm	distance edge pin hole to edge plate in line of force
c	=	117	mm	distance edge pin hole to edge plate perpendicular to line of force

Material

t	=	50	mm	thickness plate
f_y	=	355	N/mm^2	yield strength plate material
γ_{M0}	=	1.00	[-]	partial factor, by default; γ_{M0} = 1.00 EN 1993-1-8:2005
$\gamma_{M6,ser}$	=	1.00	[-]	Serviceability factor for replaceable pins ; $\gamma_{M6,ser}$ = 1.00 acc. EN 1993-1-8:2005

Load

	F_d	=	943	kN	actual load (without load factors)
	γ_q	=	1.00	[-]	load factor
	γ_{dyn}	=	1.00	[-]	dynamic amplification factor (if not applicable γ_{dyn} = 1.00)
	$\gamma_{overall}$	=	1.00	[-]	overall safety = $\gamma_q \times \gamma_{dyn}$
[ULS]	F_{Ed}	=	943	kN	design load = $F_d \times \gamma_q \times \gamma_{dyn}$
[SLS]	$F_{b,Ed,ser}$	=	943	kN	design load = $F_d \times \gamma_{dyn}$

Decisive check: 0.92 Bearing strength for serviceability

Check strength in line of force

a ≥ $F_{t,Rd}$ / (2 x t x f_y) + 2/3 x d_o — Formula table 3.9

$F_{t,Rd}$ = (a -2/3 x d_o) x 2 x t x f_y — $F_{t,Rd}$ = plate strength based on dimension a

$F_{t,Rd}$ = (117 - 2/3 x 66) x 2 x 50 x 355

$F_{t,Rd}$ = 2592 kN

$\frac{F_{Ed}}{F_{t,Rd}} = \frac{943}{2592}$ = 0.36 < 1.00 OK

Check strength perpendicular to line of force

c ≥ $F_{t,Rd}$ / (2 x t x f_y) + 1/3 x d_o — Formula table 3.9

$F_{t,Rd}$ = (c -1/3 x d_o) x 2 x t x f_y — $F_{t,Rd}$ = plate strength based on dimension c

$F_{t,Rd}$ = (117 - 1/3 x 66) x 2 x 50 x 355

$F_{t,Rd}$ = 3373 kN

$\frac{F_{Ed}}{F_{t,Rd}} = \frac{943}{3373}$ = 0.28 < 1.00 OK

Check ULS bearing strength

$F_{b;Rd}$ = 1.5 x d x t x f_y — Formula table 3.10

$F_{b;Rd}$ = 1.5 x 64 x 50 x 355

$F_{b;Rd}$ = 1704 kN

$\frac{F_{Ed}}{F_{b,Rd}} = \frac{943}{1704}$ = 0.55 < 1.00 OK

Check bearing strength for serviceability (only applicable for replaceable pin)

$F_{b;Rd,ser}$ = 0.9 x t x dp x fy

$F_{b;Rd,ser}$ = 0.9 x 50x64x355

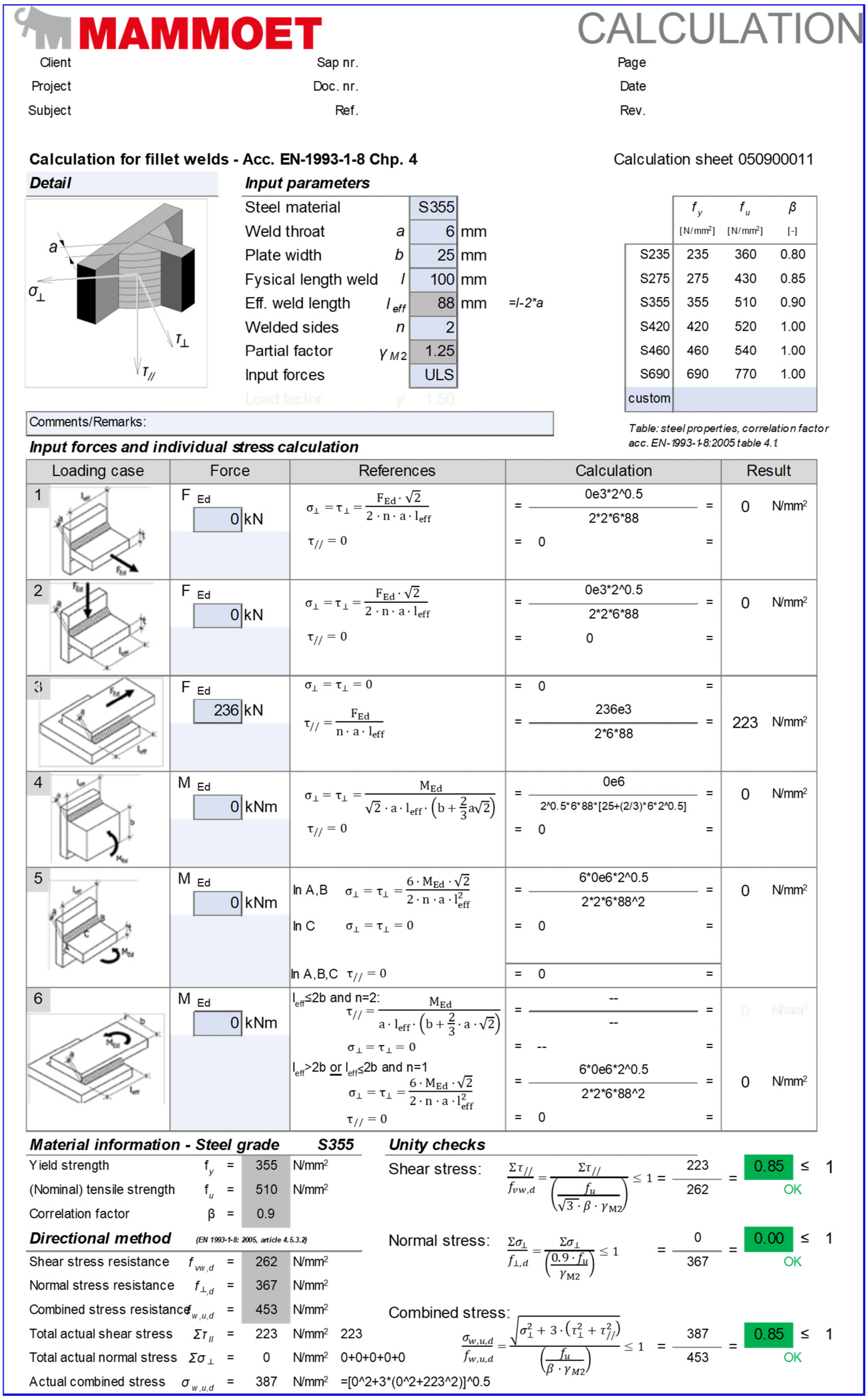
$F_{b;Rd,ser}$ = 1022 kN

$\frac{F_{b,Ed,ser}}{F_{b;Rd,ser}} = \frac{943}{1022}$ = 0.92 < 1.00 OK

Figure D-127 Pin and bearing of the plates

Tongue plate and end plate

Two tongue plates are welded on four sides, so each pair of welds will take 235.75 kN.



MAMMOET CALCULATION

Client	Sap nr.	Page
Project	Doc. nr.	Date
Subject	Ref.	Rev.

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4 Calculation sheet 050900011

Detail ***Input parameters***

Steel material		S355		
Weld throat	a	6	mm	
Plate width	b	25	mm	
Fysical length weld	l	100	mm	
Eff. weld length	l_{eff}	88	mm	$=l-2*a$
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$; $\tau_{//}=0$	= 0e3*2^0.5 / 2*2*6*88 = ; = 0 =	0 N/mm²
2	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}}$; $\tau_{//}=0$	= 0e3*2^0.5 / 2*2*6*88 = ; = 0 =	0 N/mm²
3	F_{Ed} 236 kN	$\sigma_\perp = \tau_\perp = 0$; $\tau_{//} = \frac{F_{Ed}}{n\cdot a\cdot l_{eff}}$	= 0 = ; = 236e3 / 2*6*88 =	223 N/mm²
4	M_{Ed} 0 kNm	$\sigma_\perp = \tau_\perp = \frac{M_{Ed}}{\sqrt{2}\cdot a\cdot l_{eff}\cdot\left(b+\frac{2}{3}a\sqrt{2}\right)}$; $\tau_{//}=0$	= 0e6 / 2^0.5*6*88*[25+(2/3)*6*2^0.5] = ; = 0 =	0 N/mm²
5	M_{Ed} 0 kNm	In A,B $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$; In C $\sigma_\perp = \tau_\perp = 0$; In A,B,C $\tau_{//}=0$	= 6*0e6*2^0.5 / 2*2*6*88^2 = ; = 0 = ; = 0 =	0 N/mm²
6	M_{Ed} 0 kNm	$l_{eff}\leq 2b$ and n=2: $\tau_{//} = \frac{M_{Ed}}{a\cdot l_{eff}\cdot\left(b+\frac{2}{3}\cdot a\cdot\sqrt{2}\right)}$; $\sigma_\perp=\tau_\perp=0$; $l_{eff}>2b$ or $l_{eff}\leq 2b$ and n=1: $\sigma_\perp = \tau_\perp = \frac{6\cdot M_{Ed}\cdot\sqrt{2}}{2\cdot n\cdot a\cdot l_{eff}^2}$; $\tau_{//}=0$	= -- / -- = ; = -- = ; = 6*0e6*2^0.5 / 2*2*6*88^2 = ; = 0 =	0 N/mm²; 0 N/mm²

Material information - Steel grade S355

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	223	N/mm²	223
Total actual normal stress	$\Sigma\sigma_\perp$	=	0	N/mm²	0+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	387	N/mm²	=[0^2+3*(0^2+223^2)]^0.5

Unity checks

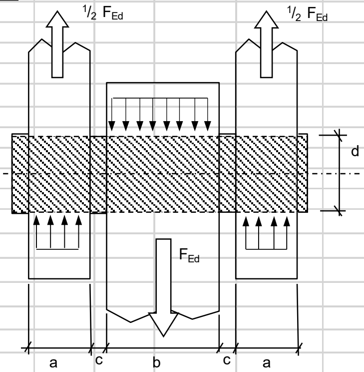
Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3}\cdot\beta\cdot\gamma_{M2}}\right)} \leq 1 = \frac{223}{262} = 0.85 \leq 1$ OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9\cdot f_u}{\gamma_{M2}}\right)} \leq 1 = \frac{0}{367} = 0.00 \leq 1$ OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2+3\cdot(\tau_\perp^2+\tau_{//}^2)}}{\left(\frac{f_u}{\beta\cdot\gamma_{M2}}\right)} \leq 1 = \frac{387}{453} = 0.85 \leq 1$ OK

Figure D-128 Weld tongue plate to horizontal diagonal

EN 1993-1-8:2005 art. 3.13 Pin connections - Check pin



Input cells are marked yellow

Pin replaceable Yes

Geometry

d_p	=	64	mm	pin diameter
d	=	66	mm	diameter pin hole
a	=	25	mm	thickness outer plates
b	=	50	mm	thickness inner plate
c	=	2	mm	gap

Material

$f_{y(b)}$	=	355	N/mm^2	yield stress material inner plate (b)
$f_{y(a)}$	=	355	N/mm^2	yield stress material outer plates (a)
f_{yp}	=	640	N/mm^2	yield stress pin material
f_{up}	=	800	N/mm^2	tensile strength pin material
γ_{M0}	=	1.00	[-]	partial factor
γ_{M2}	=	1.25	[-]	partial factor; always 1.25
k_M	=	1.00	[-]	calculation factor M_{Rd} (Km = by default 1.0)
$\gamma_{M6,ser}$	=	1.00	[-]	partial factor for replaceable pins

Static values pin

A_p	=	3217	mm^2
W_{el}	=	25736	mm^3

Bending moments pin

Max. moment

M_{Ed} = 0.125 x F_{Ed} x (b + 4 x c + (2 x a))

Bending moment at maximum shear forces

$M_{Ed,v,max}$ = 0.25 x F_{Ed} x (a + 2 x c)

Load

	F_d	=	943	kN	actual load (without load factors)
	γ_q	=	1.00	[-]	load factor
	γ_{dyn}	=	1.00	[-]	dynamic amplification factor
	$\gamma_{overall}$	=	1.00	[-]	overall safety = γ_q x γ_{dyn}
[ULS]	F_{Ed}	=	943	kN	design load = F_d x $\gamma_{overall}$
[SLS]	$F_{b,Ed,ser}$	=	943	kN	Serviceability design load = F_d x γ_{dyn}
	M_{Ed}	=	13	kNm	design load maximum bending moment
	$M_{Ed,v,max}$	=	7	kNm	design load bending moment at maximum shear force

Decisive check	0.93	Contact bearing stress at a

Check shear of the pin (two shear planes)

$F_{v,Rd}$	=	2 x 0.6 x A_p x f_{up} / γ_{M2}							
$F_{v,Rd}$	=	2 x 0.6 x 3217 x 800 / 1.25	$F_{v,Ed}$		943				
$F_{v,Rd}$	=	2471 kN	$F_{v,Rd}$	=	2471	=	0.38	< 1.00	OK

Check bending of the pin

M_{Rd}	=	k_M x W_{el} x f_{yp} / γ_{M0}							
M_{Rd}	=	1 x 25736 x 640	M_{Ed}		13				
M_{Rd}	=	16 kNm	M_{Rd}	=	16	=	0.77	< 1.00	OK

Check combined shear and bending of the pin

Worst case interaction between shear and bending occurs at the edge of mid plate b.

$F_{v,Ed}$			$M_{Ed,v,max}$		6.8		
$F_{v,Rd}$	=	0.38	M_{Rd}	=	16.5	=	0.42

(0.38)^2 + (0.42)^2 = 0.32 < 1.00 OK

Check bearing of the plate and pin

$F_{b;Rd}$	=	1.5 x b x d x fy(b)							
$F_{b;Rd}$	=	1.5 x 50 x 64 x 355	$F_{b,Ed}$		943				
$F_{b;Rd}$	=	1704 kN	$F_{b;Rd}$	=	1704	=	0.55	< 1.00	OK

Check bearing strength for serviceability (only applicable for replaceable pins)

$F_{b;Rd,ser}$	=	0.9 x 2 x a x d x fy(a)							
$F_{b;Rd,ser}$	=	0.9 x 2 x25x64x355	$F_{b,Ed,ser}$		943				
$F_{b;Rd,ser}$	=	1022 kN	$F_{b;Rd,ser}$	=	1022	=	0.92	< 1.00	OK

Check contact bearing stress pin (only applicable for replaceable pins)

contact bearing stress following Hertz — Formulas 3.15 and 3.16

$\sigma_{h;Ed}$	=	0.591 x sqrt((E x 0.5 * FEd:ser x (d0 - d))/(d^2 x a))							
$\sigma_{h;Ed}$	=	822 N/mm^2	$\sigma_{h;Ed}$		822				
			2.5 x f_y / $\gamma_{M6,ser}$	=	888	=	0.93	< 1.00	OK

Figure D-129 Tongue and end plate

Weld end plate to stability beam and bottom horizontal X brace

It is chosen to weld the end plate to both beams as it will add bending moment strength to the stability beams. The 943 kN load is a shear force and a normal force, from the perspective of the welds. As it is on an angle of 45° to the welds. Therefore, the two load cases have a load of 667 kN.

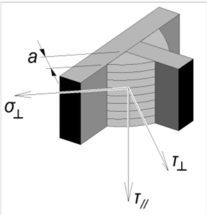
MAMMOET CALCULATION

Client		Sap nr.		Page	
Project		Doc. nr.		Date	
Subject		Ref.		Rev.	

Calculation for fillet welds - Acc. EN-1993-1-8 Chp. 4

Calculation sheet 050900011

Detail



Input parameters

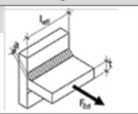
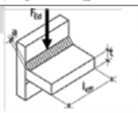
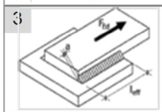
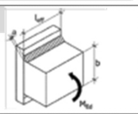
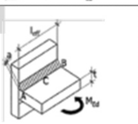
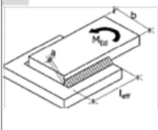
Steel material		S355		
Weld throat	a	6	mm	
Plate width	b	50	mm	
Fysical length weld	l	600	mm	
Eff. weld length	l_{eff}	588	mm	=l-2*a
Welded sides	n	2		
Partial factor	γ_{M2}	1.25		
Input forces		ULS		
Load factor	γ	1.50		

	f_y [N/mm²]	f_u [N/mm²]	β [-]
S235	235	360	0.80
S275	275	430	0.85
S355	355	510	0.90
S420	420	520	1.00
S460	460	540	1.00
S690	690	770	1.00
custom			

Table: steel properties, correlation factor acc. EN-1993-1-8:2005 table 4.1

Comments/Remarks:

Input forces and individual stress calculation

Loading case	Force	References	Calculation	Result
1 	F_{Ed} 667 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}}$ $\tau_{//} = 0$	= 667e3*2^0.5 / 2*2*6*588 = = 0 =	67 N/mm²
2 	F_{Ed} 0 kN	$\sigma_\perp = \tau_\perp = \frac{F_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}}$ $\tau_{//} = 0$	= 0e3*2^0.5 / 2*2*6*588 = = 0 =	0 N/mm²
3 	F_{Ed} 667 kN	$\sigma_\perp = \tau_\perp = 0$ $\tau_{//} = \frac{F_{Ed}}{n \cdot a \cdot l_{eff}}$	= 0 = = 667e3 / 2*6*588 =	95 N/mm²
4 	M_{Ed} 0 kNm	$\sigma_\perp = \tau_\perp = \frac{M_{Ed}}{\sqrt{2} \cdot a \cdot l_{eff} \cdot \left(b + \frac{2}{3} a\sqrt{2}\right)}$ $\tau_{//} = 0$	= 0e6 / 2^0.5*6*588*[50+(2/3)*6*2^0.5] = = 0 =	0 N/mm²
5 	M_{Ed} 0 kNm	In A,B $\sigma_\perp = \tau_\perp = \frac{6 \cdot M_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}^2}$ In C $\sigma_\perp = \tau_\perp = 0$ In A,B,C $\tau_{//} = 0$	= 6*0e6*2^0.5 / 2*2*6*588^2 = = 0 = = 0 =	0 N/mm²
6 	M_{Ed} 0 kNm	l_{eff}≤2b and n=2: $\tau_{//} = \frac{M_{Ed}}{a \cdot l_{eff} \cdot \left(b + \frac{2}{3} \cdot a \cdot \sqrt{2}\right)}$ $\sigma_\perp = \tau_\perp = 0$ l_{eff}>2b or l_{eff}≤2b and n=1 $\sigma_\perp = \tau_\perp = \frac{6 \cdot M_{Ed} \cdot \sqrt{2}}{2 \cdot n \cdot a \cdot l_{eff}^2}$ $\tau_{//} = 0$	= -- / -- = = -- = = 6*0e6*2^0.5 / 2*2*6*588^2 = = 0 =	0 N/mm² 0 N/mm²

Material information - Steel grade S355

Yield strength	f_y	=	355	N/mm²
(Nominal) tensile strength	f_u	=	510	N/mm²
Correlation factor	β	=	0.9	

Directional method (EN 1993-1-8: 2005, article 4.5.3.2)

Shear stress resistance	$f_{vw,d}$	=	262	N/mm²	
Normal stress resistance	$f_{\perp,d}$	=	367	N/mm²	
Combined stress resistance	$f_{w,u,d}$	=	453	N/mm²	
Total actual shear stress	$\Sigma\tau_{//}$	=	95	N/mm²	95
Total actual normal stress	$\Sigma\sigma_\perp$	=	67	N/mm²	67+0+0+0+0
Actual combined stress	$\sigma_{w,u,d}$	=	211	N/mm²	=[67^2+3*(67^2+95^2)]^0.5

Unity checks

Shear stress: $\frac{\Sigma\tau_{//}}{f_{vw,d}} = \frac{\Sigma\tau_{//}}{\left(\frac{f_u}{\sqrt{3} \cdot \beta \cdot \gamma_{M2}}\right)} \leq 1 = \frac{95}{262} =$ 0.36 ≤ 1 OK

Normal stress: $\frac{\Sigma\sigma_\perp}{f_{\perp,d}} = \frac{\Sigma\sigma_\perp}{\left(\frac{0.9 \cdot f_u}{\gamma_{M2}}\right)} \leq 1 = \frac{67}{367} =$ 0.18 ≤ 1 OK

Combined stress: $\frac{\sigma_{w,u,d}}{f_{w,u,d}} = \frac{\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)}}{\left(\frac{f_u}{\beta \cdot \gamma_{M2}}\right)} \leq 1 = \frac{211}{453} =$ 0.47 ≤ 1 OK

Figure D-130 Weld end plates to stability beam and bottom horizontal X brace

D.4. MECHANICAL DETAILS

D.4.1. MLS HOISTING SYSTEM

D.4.1.1. HEB300 Load spreader verification SCIA

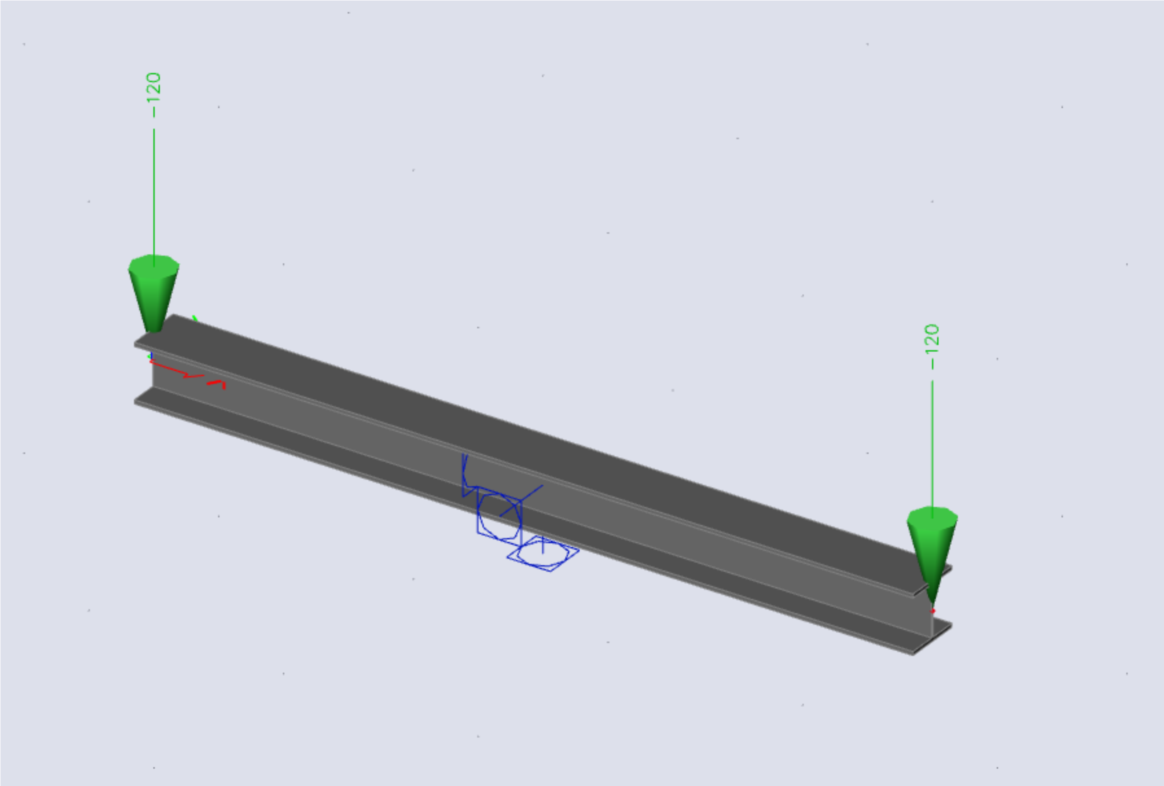


Figure D-131 Load spreader, SCIA render

EC-EN 1993 Steel check ULS

Linear calculation
Combination: CO1
Coordinate system: Principal
Extreme 1D: Global
Selection: All

EN 1993-1-1 Code Check
National annex: Standard EN

Member B1	2,000 / 4,000 m	HEB300	S 355	CO1	0,71 -

Combination key
CO1 / 1.73*LC1 + 1.35*LC2

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm²
Ultimate strength	f_u	510	N/mm²
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 2,000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	0	kN
Shear force	$V_{y,Ed}$	0	kN
Shear force	$V_{z,Ed}$	-212	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	-420	kNm
Bending moment	$M_{z,Ed}$	0	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	117,5	19,0	234	234	1,0	0,4	1,0	6,2	7,3	8,1	11,4	1
3	SO	117,5	19,0	234	234	1,0	0,4	1,0	6,2	7,3	8,1	11,4	1
4	I	208,0	11,0	173	-173	-1,0		0,5	18,9	58,6	67,5	100,9	1
5	SO	117,5	19,0	-234	-234								
7	SO	117,5	19,0	-234	-234								

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	1,678e+06	mm³
Elastic bending moment	$M_{el,y,Rd}$	596	kNm
Unity check		0,70	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	72	N/mm²
Elastic shear resistance	τ_{Rd}	205	N/mm²
Unity check		0,35	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		14	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	0	N/mm²
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	-250	N/mm²
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	0	N/mm²
Total longitudinal stress	$\sigma_{tot,Ed}$	-250	N/mm²
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm²
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	18	N/mm²
Shear stress due to uniform (St.	$\tau_{t,Ed}$	0	N/mm²

Elastic verification			
Venant) torsion			
Total shear stress	$\tau_{tot,Ed}$	18	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	252	N/mm^2
Unity check		0,71	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 2,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	117,5	19,0	234	234	1,0	0,4	1,0	6,2	7,3	8,1	11,4	1
3	SO	117,5	19,0	234	234	1,0	0,4	1,0	6,2	7,3	8,1	11,4	1
4	I	208,0	11,0	173	-173	-1,0		0,5	18,9	58,6	67,5	100,9	1
5	SO	117,5	19,0	-234	-234								
7	SO	117,5	19,0	-234	-234								

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Elastic section modulus	$W_{el,y}$	1,678e+06	mm^3
Elastic critical moment	M_{cr}	11930	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,22	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	2,000	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,77	
LTB moment factor	C_2	0,00	
LTB moment factor	C_3	1,00	
Shear centre distance	d_z	0,0	mm
Distance of load application	z_g	0,0	mm
Mono-symmetry constant	β_y	0,0	mm
Mono-symmetry constant	z_j	0,0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	4,000	m
Web		unstiffened	
Web height	h_w	262,0	mm
Web thickness	t	11,0	mm
Material coefficient	ε	0,81	
Shear correction factor	η	1,20	

Shear Buckling verification		
Web slenderness	h_w/t	23,82
Web slenderness limit		48,82

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

Figure D-132 Load spreader, SCIA report

D.4.1.2. Upper frame

A load of 250 kN (25 tons) needs to be lifted per winch. This means that the wire needs to have a breaking force of (5 * 250 = 1250 kN) 125 tons. Diepa is a manufacturer of wires. For a breaking force of 125 Te, they recommend using a wire of 35 mm in diameter when using the DIEPA B5 wire, with strength 2160 N/mm^2 [42]. A rule of thumb for determining the diameter of the winch drum is:

$$\phi_{winch\ drum} = 20 * \phi_{wire}$$

$$\phi_{winch\ drum} = 20 * 35 = 700\ mm$$

A wire of approximately 150 meters is needed. The number of windings comes to:

$$\#_{windings} = \frac{150 * 10^3}{\pi * 700} = 68.2 \approx 70$$

It is recommended to have approximately five windings above each other and ten to fifteen next to each other. A configuration of five high and fourteen wide makes for 70 windings. Meaning that the width of the winch drum is:

$$b_{winch\ drum} = \phi_{winch\ drum} * 14$$

$$b_{winch\ drum} = 35 * 14 = 490\ mm$$

This is rounded to 500 mm, to be conservative. The maximum angle for (un)winding the wire is 2° (SOURCE). The wire will steadily hang in the middle over a distance of:

$$l_{(un)winding\ wire} = \frac{\frac{b_{winch\ drum}}{2}}{\tan(2°)}$$

$$l_{(un)winding\ wire} = \frac{\frac{500}{2}}{\tan(2°)} = 7160$$

Therefore, it must travel back and forth over a turning disk that can translate in Y direction. This makes for a distance of 3580 mm, which is rounded to 3600 mm between the winch drum and the turning disk.

This has consequences for the load introduction in the frame. The loads that act on the frame can be seen in Figure D-133. The two arrows in the middle of the structure have a magnitude of 500 kN.

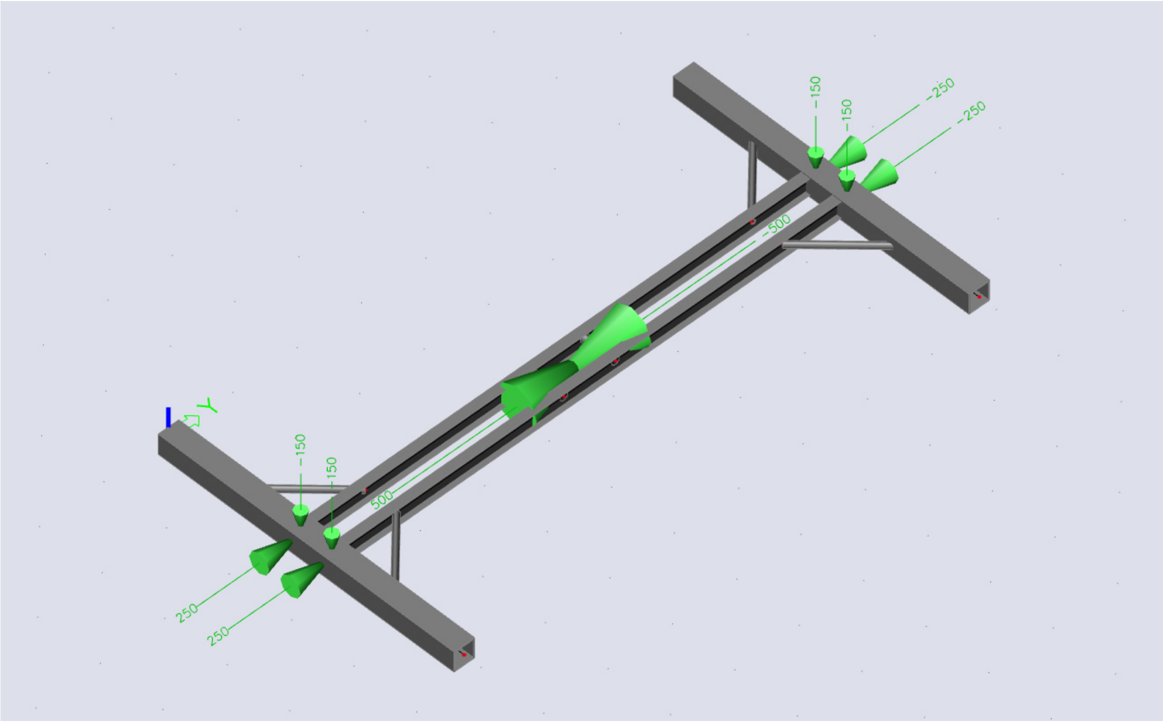
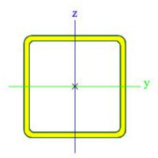


Figure D-133 Upper frame, SCIA render

Cross-sections

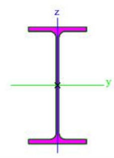
Name	CS2
Type	SHS320/320/16.0
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	1,930e+04	
A y, z [mm²]	9,588e+03	9,588e+03
I y, z [mm⁴]	2,965e+08	2,965e+08
I w [mm⁶], t [mm⁴]	4,474e+12	4,601e+08
W_{el} y, z [mm³]	1,853e+06	1,853e+06
W_{pl} y, z [mm³]	2,174e+06	2,174e+06
d y, z [mm]	0,0	0,0
c YUCS, ZUCS [mm]	160,0	160,0
α [deg]	0,00	
A L, D [m²/m]	1,2386e+00	2,3630e+00
M_{ply} +, - [Nmm]	7,72e+08	7,72e+08
M_{plz} +, - [Nmm]	7,72e+08	7,72e+08

Cross-sections

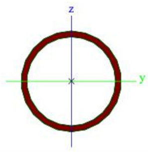
Name	CS1
Type	IPE270
Source description	ArcelorMittal / Sales Programme / Version 2012-1
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	b
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	4,590e+03	
A y, z [mm²]	2,771e+03	1,827e+03
I y, z [mm⁴]	5,790e+07	4,200e+06
I w [mm⁶], t [mm⁴]	7,060e+10	1,590e+05
W_{el} y, z [mm³]	4,290e+05	6,220e+04
W_{pl} y, z [mm³]	4,840e+05	9,700e+04
d y, z [mm]	0,0	0,0
c YUCS, ZUCS [mm]	67,5	135,0
α [deg]	0,00	
A L, D [m²/m]	1,0409e+00	1,0409e+00
M_{ply} +, - [Nmm]	1,72e+08	1,72e+08
M_{plz} +, - [Nmm]	3,44e+07	3,44e+07

Cross-sections

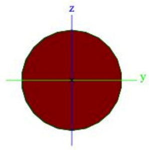
Name	CS4
Type	CHS88.9/5.0
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm²]	1,320e+03	
A y, z [mm²]	8,390e+02	8,390e+02
I y, z [mm⁴]	1,160e+06	1,160e+06
I w [mm⁶], t [mm⁴]	1,626e-24	2,330e+06
W_{el} y, z [mm³]	2,620e+04	2,620e+04
W_{pl} y, z [mm³]	3,468e+04	3,468e+04
d y, z [mm]	0,0	0,0
c YUCS, ZUCS [mm]	44,5	44,5
α [deg]	0,00	
A L, D [m²/m]	2,7900e-01	5,2713e-01
M_{ply} +, - [Nmm]	1,25e+07	1,25e+07
M_{plz} +, - [Nmm]	1,25e+07	1,25e+07

Cross-sections

Name	CS3
Type	RND120
Source description	Stahlbau Zentrum Schweiz / Konstruktionstabellen / 9.Ausgabe 2005
Item material	S 690
Fabrication	rolled
Flexural buckling y-y	c
Flexural buckling z-z	c
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm²]	1,130e+04	
A y, z [mm²]	1,017e+04	1,017e+04
I y, z [mm⁴]	1,020e+07	1,020e+07
I w [mm⁶], t [mm⁴]	1,531e+00	2,040e+07
W_{el} y, z [mm³]	1,700e+05	1,700e+05
W_{pl} y, z [mm³]	2,880e+05	2,880e+05
d y, z [mm]	0,0	0,0
c YUCS, ZUCS [mm]	60,0	60,0
α [deg]	0,00	
A L, D [m²/m]	3,7700e-01	3,7697e-01
M_{ply} +, - [Nmm]	1,99e+08	1,99e+08
M_{plz} +, - [Nmm]	1,99e+08	1,99e+08

Figure D-134 Upper frame, member properties

EC-EN 1993 Steel check ULS

Linear calculation
Combination: CO1
Coordinate system: Principal
Extreme 1D: Member
Selection: B2, B4, B6, B9

EN 1993-1-1 Code Check
National annex: Standard EN

Member B2	4,400 / 8,000 m	IPE270	S 355	CO1	0,75 -

Combination key
CO1 / 1.73*LC2 load + 1.35*LC1 Selft weight

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 4,400 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-411	kN
Shear force	$V_{y,Ed}$	0	kN
Shear force	$V_{z,Ed}$	-1	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	7	kNm
Bending moment	$M_{z,Ed}$	0	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	49,2	10,2	73	74	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1
3	SO	49,2	10,2	73	73	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1
4	I	219,6	6,6	76	103	0,7		1,0	33,3	26,8	30,9	37,5	3
5	SO	49,2	10,2	106	105	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1
7	SO	49,2	10,2	106	106	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1

The cross-section is classified as Class 3
Note: The Elastic verification has been set by the user.

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	4,590e+03	mm^2
Compression resistance	$N_{c,Rd}$	1629	kN
Unity check		0,25	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	4,290e+05	mm^3
Elastic bending moment	$M_{el,y,Rd}$	152	kNm
Unity check		0,05	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	6,220e+04	mm^3
Elastic bending moment	$M_{el,z,Rd}$	22	kNm
Unity check		0,00	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	2	
Total torsional moment	τ_{Ed}	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		15	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	90	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	17	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	1	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	107	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	0	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	107	N/mm^2
Unity check		0,30	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 4,400 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	49,2	10,2	73	74	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1
3	SO	49,2	10,2	73	73	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1
4	I	219,6	6,6	76	103	0,7		1,0	33,3	26,8	30,9	37,5	3
5	SO	49,2	10,2	106	105	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1
7	SO	49,2	10,2	106	106	1,0	0,4	1,0	4,8	7,3	8,1	11,2	1

The cross-section is classified as Class 3
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	sway	
System length	L	8,000	2,600	m
Buckling factor	k	1,00	1,28	
Buckling length	l_{cr}	8,000	3,338	m
Critical Euler load	N_{cr}	1875	781	kN
Slenderness	λ	71,23	110,36	
Relative slenderness	λ_{rel}	0,93	1,44	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		a	b	
Imperfection	α	0,21	0,34	
Reduction factor	χ	0,71	0,36	
Buckling resistance	$N_{b,Rd}$	1161	592	kN

Flexural Buckling verification			
Cross-section area	A	4,590e+03	mm^2
Buckling resistance	$N_{b,Rd}$	592	kN
Unity check		0,69	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Elastic section modulus	$W_{el,y}$	4,290e+05	mm^3
Elastic critical moment	M_{cr}	266	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,76	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	2,600	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,26	
LTB moment factor	C_2	0,04	
LTB moment factor	C_3	1,00	
Shear centre distance	d_z	0,0	mm
Distance of load application	z_g	0,0	mm
Mono-symmetry constant	β_y	0,0	mm
Mono-symmetry constant	z_j	0,0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 1	
Cross-section area	A	4,590e+03	mm^2
Elastic section modulus	$W_{el,y}$	4,290e+05	mm^3
Elastic section modulus	$W_{el,z}$	6,220e+04	mm^3
Design compression force	N_{Ed}	411	kN
Design bending moment (maximum)	$M_{y,Ed}$	7	kNm
Design bending moment (maximum)	$M_{z,Ed}$	0	kNm
Characteristic compression resistance	N_{Rk}	1629	kN
Characteristic moment resistance	$M_{y,Rk}$	152	kNm
Characteristic moment resistance	$M_{z,Rk}$	22	kNm
Reduction factor	χ_y	0,71	
Reduction factor	χ_z	0,36	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,90	
Interaction factor	k_{yz}	0,75	
Interaction factor	k_{zy}	1,20	
Interaction factor	k_{zz}	0,48	

Maximum moment $M_{y,Ed}$ is derived from beam B2 position 4,000 m.
Maximum moment $M_{z,Ed}$ is derived from beam B2 position 7,000 m.

Interaction method 1 parameters			
Critical Euler load	$N_{cr,y}$	1875	kN
Critical Euler load	$N_{cr,z}$	781	kN
Elastic critical load	$N_{cr,T}$	2549	kN
Elastic section modulus	$W_{el,y}$	4,290e+05	mm^3
Second moment of area	I_y	5,790e+07	mm^4
Second moment of area	I_z	4,200e+06	mm^4
Torsional constant	I_t	1,590e+05	mm^4
Method for equivalent moment factor $C_{my,0}$		Table A.2 Line 4 (Line load)	
Equivalent moment factor	$C_{my,0}$	1,01	
Method for equivalent moment factor $C_{mz,0}$		Table A.2 Line 1 (Linear)	
Ratio of end moments	ψ_z	-0,86	
Equivalent moment factor	$C_{mz,0}$	0,38	
Factor	μ_y	0,93	
Factor	μ_z	0,59	
Factor	ε_y	0,19	
Factor	a_{LT}	1,00	
Critical moment for uniform bending	$M_{cr,0}$	211	kNm
Relative slenderness	$\lambda_{rel,0}$	0,85	
Limit relative slenderness	$\lambda_{rel,0,lim}$	0,18	

Interaction method 1 parameters			
Equivalent moment factor	C_{my}	1,00	
Equivalent moment factor	C_{mz}	0,38	
Equivalent moment factor	C_{mLT}	1,60	

Unity check (6.61) = 0,35 + 0,09 + 0,00 = 0,45 -
Unity check (6.62) = 0,69 + 0,06 + 0,00 = 0,75 -

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	8,000	m
Web		unstiffened	
Web height	h_w	249,6	mm
Web thickness	t	6,6	mm
Material coefficient	ε	0,81	
Shear correction factor	η	1,20	

Shear Buckling verification		
Web slenderness	h_w/t	37,82
Web slenderness limit		48,82

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

EN 1993-1-1 Code Check
National annex: Standard EN

Member B4	2,315 / 4,730 m	SHS320/320/16.0	S 355	CO1	0,93 -

Combination key
CO1 / 1.73*LC2 load + 1.35*LC1 Selft weight

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 2,315 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	0	kN
Shear force	$V_{y,Ed}$	0	kN
Shear force	$V_{z,Ed}$	0	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	565	kNm
Bending moment	$M_{z,Ed}$	45	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	272,0	16,0	-314	-272								
3	I	272,0	16,0	-239	286	-0,8		0,5	17,0	53,0	61,0	86,7	1
5	I	272,0	16,0	314	272	0,9		1,0	17,0	26,8	30,9	35,8	1
7	I	272,0	16,0	239	-286	-1,2		0,5	17,0	64,3	74,2	121,2	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1,930e+04	mm^2
Compression resistance	$N_{c,Rd}$	6852	kN
Unity check		0,00	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	1,853e+06	mm^3
Elastic bending moment	$M_{el,y,Rd}$	658	kNm
Unity check		0,86	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	1,853e+06	mm^3
Elastic bending moment	$M_{el,z,Rd}$	658	kNm
Unity check		0,07	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		9	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	0	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	305	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	21	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	325	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	0	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	325	N/mm^2
Unity check		0,92	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 2,315 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	272,0	16,0	-314	-272								
3	I	272,0	16,0	-239	286	-0,8		0,5	17,0	53,0	61,0	86,7	1
5	I	272,0	16,0	314	272	0,9		1,0	17,0	26,8	30,9	35,8	1
7	I	272,0	16,0	239	-286	-1,2		0,5	17,0	64,3	74,2	121,2	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	sway	
System length	L	4,730	0,500	m
Buckling factor	k	1,00	4,53	
Buckling length	l_{cr}	4,730	2,265	m
Critical Euler load	N_{cr}	27468	119818	kN
Slenderness	λ	38,16	18,27	
Relative slenderness	λ_{rel}	0,50	0,24	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 1	
Cross-section area	A	1,930e+04	mm²
Elastic section modulus	$W_{el,y}$	1,853e+06	mm³
Elastic section modulus	$W_{el,z}$	1,853e+06	mm³
Design compression force	N_{Ed}	0	kN
Design bending moment (maximum)	$M_{y,Ed}$	565	kNm
Design bending moment (maximum)	$M_{z,Ed}$	45	kNm
Characteristic compression resistance	N_{Rk}	6852	kN
Characteristic moment resistance	$M_{y,Rk}$	658	kNm
Characteristic moment resistance	$M_{z,Rk}$	658	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	1,00	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,00	
Interaction factor	k_{yz}	1,00	
Interaction factor	k_{zy}	1,00	
Interaction factor	k_{zz}	1,00	

Maximum moment $M_{y,Ed}$ is derived from beam B4 position 2,315 m.
Maximum moment $M_{z,Ed}$ is derived from beam B4 position 2,115 m.

Interaction method 1 parameters			
Critical Euler load	$N_{cr,y}$	27468	kN
Critical Euler load	$N_{cr,z}$	119818	kN
Elastic critical load	$N_{cr,T}$	2416661	kN
Elastic section modulus	$W_{el,y}$	1,853e+06	mm³
Second moment of area	I_y	2,965e+08	mm⁴
Second moment of area	I_z	2,965e+08	mm⁴
Torsional constant	I_t	4,601e+08	mm⁴
Method for equivalent moment factor $C_{my,0}$		Table A.2 Line 3 (Point load)	
Equivalent moment factor	$C_{my,0}$	1,00	
Method for equivalent moment factor $C_{mz,0}$		Table A.2 Line 1 (Linear)	
Ratio of end moments	ψ_z	1,00	
Equivalent moment factor	$C_{mz,0}$	1,00	
Factor	μ_y	1,00	
Factor	μ_z	1,00	
Factor	ε_y	172961,91	
Factor	a_{LT}	0,00	
Critical moment for uniform bending	$M_{cr,0}$	427227	kNm
Relative slenderness	$\lambda_{rel,0}$	0,04	
Limit relative slenderness	$\lambda_{rel,0,lim}$	0,20	
Equivalent moment factor	C_{my}	1,00	
Equivalent moment factor	C_{mz}	1,00	
Equivalent moment factor	C_{mLT}	1,00	

Unity check (6.61) = 0,00 + 0,86 + 0,07 = 0,93 -
Unity check (6.62) = 0,00 + 0,86 + 0,07 = 0,93 -

The member satisfies the stability check.

EN 1993-1-1 Code Check
National annex: Standard EN

Member B6	**0,250 / 0,500 m**	**RND120**	**S 690**	**CO1**	**0,92 -**

Combination key
CO1 / 1.73*LC2 load + 1.35*LC1 Selft weight

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	690	N/mm^2
Ultimate strength	f_u	770	N/mm^2
Fabrication		Rolled	

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

...::SECTION CHECK::...

The critical check is on position 0,250 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	0	kN
Shear force	$V_{y,Ed}$	433	kN
Shear force	$V_{z,Ed}$	0	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	0	kNm
Bending moment	$M_{z,Ed}$	108	kNm

Classification for cross-section design
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.
Note: The Elastic verification has been set by the user.

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

Cross-section area	A	1,130e+04	mm^2
Plastic tension resistance	$N_{pl,Rd}$	7797	kN
Ultimate tension resistance	$N_{u,Rd}$	6265	kN
Tension resistance	$N_{t,Rd}$	6265	kN
Unity check		0,00	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	1,700e+05	mm^3
Elastic bending moment	$M_{el,y,Rd}$	117	kNm
Unity check		0,00	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	1,700e+05	mm^3
Elastic bending moment	$M_{el,z,Rd}$	117	kNm
Unity check		0,92	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	51	N/mm^2
Elastic shear resistance	τ_{Rd}	398	N/mm^2
Unity check		0,13	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	3	
Total torsional moment	τ_{Ed}	0	N/mm^2
Elastic shear resistance	τ_{Rd}	398	N/mm^2
Unity check		0,00	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		16	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	0	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	0	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	-636	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	-636	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	0	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	636	N/mm^2

Elastic verification			
Unity check		0,92	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Warning: Classification is not supported for this type of cross-section.
The section is checked as elastic, class 3.
Note: The Elastic verification has been set by the user.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Elastic section modulus	$W_{el,y}$	1,700e+05	mm^3
Elastic critical moment	M_{cr}	13305	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,09	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	0,500	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,13	
LTB moment factor	C_2	0,45	
LTB moment factor	C_3	0,53	
Shear centre distance	d_z	0,0	mm
Distance of load application	z_g	0,0	mm
Mono-symmetry constant	β_y	0,0	mm
Mono-symmetry constant	z_j	0,0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

The member satisfies the stability check.

EN 1993-1-1 Code Check
National annex: Standard EN

Member B9	0,000 / 1,414 m	CHS88.9/5.0	S 355	CO1	0,05 -

Combination key
CO1 / 1.73*LC2 load + 1.35*LC1 Selft weight

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 0,000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	0	kN
Shear force	$V_{y,Ed}$	0	kN
Shear force	$V_{z,Ed}$	0	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	0	kNm
Bending moment	$M_{z,Ed}$	0	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
88,9	5,0	17,8	33,1	46,3	59,6	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1,320e+03	mm^2
Compression resistance	$N_{c,Rd}$	469	kN
Unity check		0,00	-

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,z,min}$	2,620e+04	mm^3
Elastic bending moment	$M_{el,z,Rd}$	9	kNm
Unity check		0,04	-

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	1	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	2	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,01	-

Note: The unity check for torsion is lower than the limit value of 0,05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		16	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	0	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	0	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	15	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	16	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	2	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	3	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	16	N/mm^2
Unity check		0,05	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Tubular sections according to EN 1993-1-1 Table 5.2 Sheet 3

d [mm]	t [mm]	d/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
88,9	5,0	17,8	33,1	46,3	59,6	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	sway	
System length	L	1,414	1,414	m
Buckling factor	k	1,00	1,00	
Buckling length	l_{cr}	1,414	1,414	m
Critical Euler load	N_{cr}	1202	1202	kN

Buckling parameters		yy	zz	
Slenderness	λ	47,71	47,71	
Relative slenderness	λ_{rel}	0,62	0,62	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a CHS section which is not susceptible to Torsional(-Flexural) Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 1	
Cross-section area	A	1,320e+03	mm^2
Elastic section modulus	$W_{el,y}$	2,620e+04	mm^3
Elastic section modulus	$W_{el,z}$	2,620e+04	mm^3
Design compression force	N_{Ed}	0	kN
Design bending moment (maximum)	$M_{y,Ed}$	0	kNm
Design bending moment (maximum)	$M_{z,Ed}$	0	kNm
Characteristic compression resistance	N_{Rk}	469	kN
Characteristic moment resistance	$M_{y,Rk}$	9	kNm
Characteristic moment resistance	$M_{z,Rk}$	9	kNm
Reduction factor	χ_y	1,00	
Reduction factor	χ_z	1,00	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,00	
Interaction factor	k_{yz}	0,71	
Interaction factor	k_{zy}	1,00	
Interaction factor	k_{zz}	0,71	

Maximum moment $M_{y,Ed}$ is derived from beam B9 position 0,707 m.
Maximum moment $M_{z,Ed}$ is derived from beam B9 position 0,000 m.

Interaction method 1 parameters			
Critical Euler load	$N_{cr,y}$	1202	kN
Critical Euler load	$N_{cr,z}$	1202	kN
Elastic critical load	$N_{cr,T}$	107075	kN
Elastic section modulus	$W_{el,y}$	2,620e+04	mm^3
Second moment of area	I_y	1,160e+06	mm^4
Second moment of area	I_z	1,160e+06	mm^4
Torsional constant	I_t	2,330e+06	mm^4
Method for equivalent moment factor $C_{my,0}$		Table A.2 Line 4 (Line load)	
Equivalent moment factor	$C_{my,0}$	1,00	
Method for equivalent moment factor $C_{mz,0}$		Table A.2 Line 1 (Linear)	
Ratio of end moments	ψ_z	-0,37	
Equivalent moment factor	$C_{mz,0}$	0,71	
Factor	μ_y	1,00	
Factor	μ_z	1,00	
Factor	ε_y	9,79	
Factor	a_{LT}	0,00	
Critical moment for uniform bending	$M_{cr,0}$	476	kNm
Relative slenderness	$\lambda_{rel,0}$	0,14	
Limit relative slenderness	$\lambda_{rel,0,lim}$	0,21	
Equivalent moment factor	C_{my}	1,00	
Equivalent moment factor	C_{mz}	0,71	
Equivalent moment factor	C_{mLT}	1,00	

Unity check (6.61) = 0,00 + 0,01 + 0,03 = 0,04 -
Unity check (6.62) = 0,00 + 0,01 + 0,03 = 0,04 -

The member satisfies the stability check.

Figure D-135 Upper frame, SCIA report

D.4.1. VERTICAL MOVING FRAME

D.4.1.1. Reeving

A load of 1760 kN (176 tons) needs to be lifted per winch. This means that the wire needs to have a breaking force of (5 * 1760 = 8750 kN) 875 tons. Diepa is a manufacturer of wires. For a breaking force of 875 Te, they recommend using a wire of 98 mm in diameter when using the DIEPA B5 wire, with strength 1960 N/mm² [42]. This diameter is considered too thick. A stronger wire (2160 N/mm²) of 35 mm will be used. Meaning that 7 reevings are needed. A rule of thumb for determining the diameter of the winch drum is:

$$\phi_{winch\ drum} = 20 * \phi_{wire}$$

$$\phi_{winch\ drum} = 20 * 35 = 700\ mm$$

A wire of approximately 200 meters is needed. The number of windings comes to:

$$\#_{windings} = \frac{200 * 10^3}{\pi * 700} = 90.9 \approx 90$$

It is recommended to have approximately five windings above each other and ten to fifteen next to each other. A configuration of six high and fifteen wide makes for 90 windings. Meaning that the width of the winch drum is:

$$b_{winch\ drum} = \phi_{winch\ drum} * 15$$

$$b_{winch\ drum} = 35 * 15 = 525\ mm$$

The maximum angle for (un)winding the wire is 2°. The wire will steadily hang in the middle over a distance of:

$$l_{(un)winding\ wire} = \frac{\frac{b_{winch\ drum}}{2}}{\tan(2°)}$$

$$l_{(un)winding\ wire} = \frac{\frac{525}{2}}{\tan(2°)} = 7517\ mm$$

This is almost the distance between the winch and the vertical moving frame, meaning that the first pulley needs to translate.

A horizontal force of 1/5th of the vertical force is present at all force locations too. These are not shown by SCIA.

D.4.1.2. Vertical moving frame validation

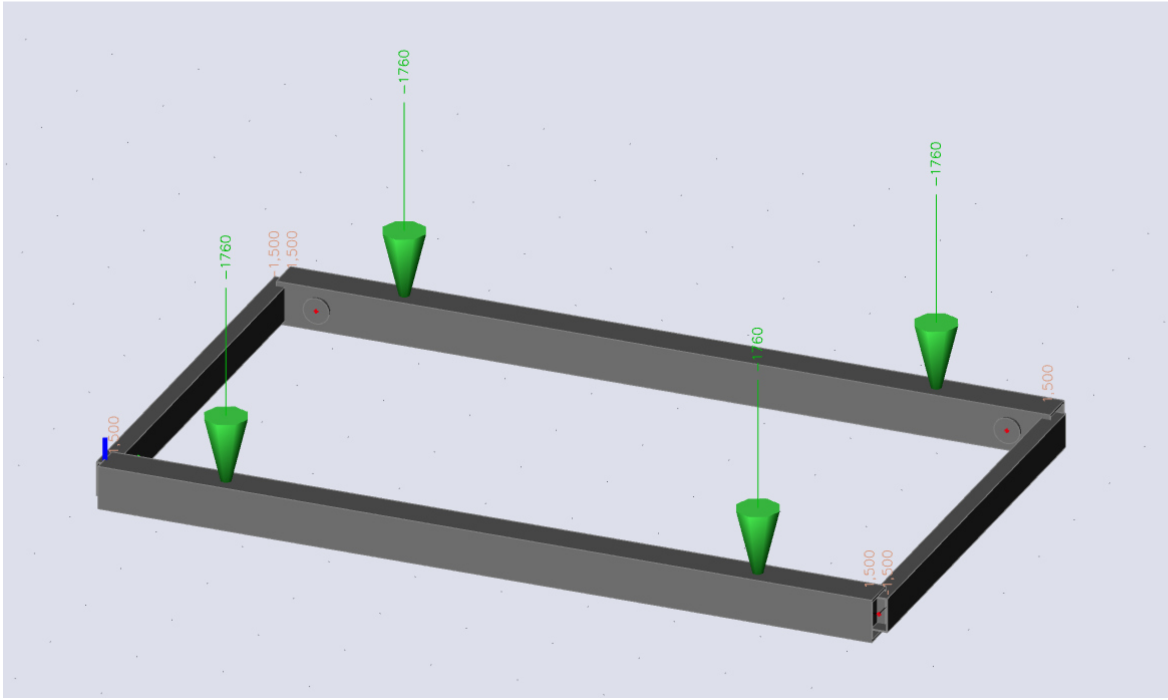
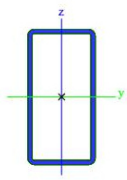


Figure D-136 Vertical moving frame, SCIA render

Cross-sections

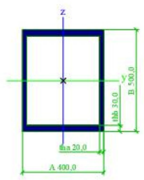
Name	CS2
Type	RHS400/200/12.5
Source description	British Standard / BS 5950 part 1 : 1990 & EN 10210-2
Item material	S 355
Fabrication	rolled
Flexural buckling y-y	a
Flexural buckling z-z	a
Lateral torsional buckling	Default
Use 2D FEM analysis	×



A [mm^2]	1,420e+04	
A y, z [mm^2]	4,701e+03	9,401e+03
I y, z [mm^4]	2,906e+08	9,738e+07
I w [mm^6], t [mm^4]	2,000e+12	2,344e+08
W_{el} y, z [mm^3]	1,453e+06	9,740e+05
W_{pl} y, z [mm^3]	1,794e+06	1,102e+06
d y, z [mm]	0,0	0,0
c YUCS, ZUCS [mm]	100,0	200,0
α [deg]	0,00	
A L, D [m^2/m]	1,1700e+00	2,2569e+00
M_{ply} +, - [Nmm]	6,37e+08	6,37e+08
M_{plz} +, - [Nmm]	3,91e+08	3,91e+08

Cross-sections

Name	CS5
Type	O
Detailed	400,0; 20,0; 500,0; 30,0
Item material	S 355
Fabrication	general
Flexural buckling y-y	d
Flexural buckling z-z	d
Lateral torsional buckling	Default
Use 2D FEM analysis	✓



A [mm^2]	4,160e+04	
A y, z [mm^2]	2,311e+04	1,946e+04
I y, z [mm^4]	1,611e+09	9,559e+08
I w [mm^6], t [mm^4]	2,606e+12	1,682e+09
W_{el} y, z [mm^3]	6,445e+06	4,780e+06
W_{pl} y, z [mm^3]	7,576e+06	5,744e+06
d y, z [mm]	0,0	0,0
c YUCS, ZUCS [mm]	200,0	250,0
α [deg]	0,00	
A L, D [m^2/m]	1,8000e+00	3,4000e+00
M_{ply} +, - [Nmm]	2,69e+09	2,69e+09
M_{plz} +, - [Nmm]	2,04e+09	2,04e+09

Figure D-137 Vertical moving frame member properties

EC-EN 1993 Steel check ULS

Nonlinear calculation
NonLinear Combi: NC1
Coordinate system: Principal
Extreme 1D: Member
Selection: B2, B3

EN 1993-1-1 Code Check
National annex: Standard EN

Member B2	0,000 / 5,060 m	RHS400/200/12.5	S 355	NC1	0,87 -

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm^2
Ultimate strength	f_u	510	N/mm^2
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 0,000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	0	kN
Shear force	$V_{y,Ed}$	0	kN
Shear force	$V_{z,Ed}$	4	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	-446	kNm
Bending moment	$M_{z,Ed}$	0	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	162,5	12,5	301	301	1,0		1,0	13,0	26,8	30,9	34,2	1
3	I	362,5	12,5	282	-282	-1,0		0,5	29,0	58,6	67,5	100,9	1
5	I	162,5	12,5	-301	-301								
7	I	362,5	12,5	-282	282	-1,0		0,5	29,0	58,6	67,5	100,9	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	1,453e+06	mm^3
Elastic bending moment	$M_{el,y,Rd}$	516	kNm
Unity check		0,87	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	1	N/mm^2
Elastic shear resistance	τ_{Rd}	205	N/mm^2
Unity check		0,00	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		9	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	0	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	-307	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	0	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	-307	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	0	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	307	N/mm^2
Unity check		0,87	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0,000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	162,5	12,5	301	301	1,0		1,0	13,0	26,8	30,9	34,2	1
3	I	362,5	12,5	282	-282	-1,0		0,5	29,0	58,6	67,5	100,9	1
5	I	162,5	12,5	-301	-301								
7	I	362,5	12,5	-282	282	-1,0		0,5	29,0	58,6	67,5	100,9	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

The member satisfies the stability check.

EN 1993-1-1 Code Check
National annex: Standard EN

Member B3	**4,480 / 8,960 m**	**O (400,0; 20,0; 500,0; 30,0)**	**S 355**	**NC1**	**0,86 -**

Partial safety factors	
γ_{M0} for resistance of cross-sections	1,00
γ_{M1} for resistance to instability	1,00
γ_{M2} for resistance of net sections	1,25

Material			
Yield strength	f_y	355	N/mm²
Ultimate strength	f_u	510	N/mm²
Fabrication		General	

...::SECTION CHECK::...

The critical check is on position 4,480 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-352	kN
Shear force	$V_{y,Ed}$	0	kN
Shear force	$V_{z,Ed}$	0	kN
Torsion	T_{Ed}	0	kNm
Bending moment	$M_{y,Ed}$	1655	kNm
Bending moment	$M_{z,Ed}$	0	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm²]	σ_2 [N/mm²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	400,0	30,0	-208	-208								
2	I	500,0	20,0	-208	224	-0,9		0,5	25,0	56,1	64,6	94,0	1
3	I	400,0	30,0	224	224	1,0		1,0	13,3	26,8	30,9	34,2	1
4	I	500,0	20,0	224	-208	-0,9		0,5	25,0	56,1	64,6	94,0	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	4,160e+04	mm²
Compression resistance	$N_{c,Rd}$	14768	kN
Unity check		0,02	-

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Elastic section modulus	$W_{el,y,min}$	6,445e+06	mm³
Elastic bending moment	$M_{el,y,Rd}$	2288	kNm
Unity check		0,72	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification			
Fibre		3	
Normal stress due to the normal force N	$\sigma_{N,Ed}$	8	N/mm^2
Normal stress due to the bending moment M_y	$\sigma_{My,Ed}$	257	N/mm^2
Normal stress due to the bending moment M_z	$\sigma_{Mz,Ed}$	0	N/mm^2
Total longitudinal stress	$\sigma_{tot,Ed}$	265	N/mm^2
Shear stress due to the transverse shear force V_y	$\tau_{Vy,Ed}$	0	N/mm^2
Shear stress due to the transverse shear force V_z	$\tau_{Vz,Ed}$	0	N/mm^2
Shear stress due to uniform (St. Venant) torsion	$\tau_{t,Ed}$	0	N/mm^2
Total shear stress	$\tau_{tot,Ed}$	0	N/mm^2
Summation of von Mises stress	$\sigma_{von\ Mises,Ed}$	265	N/mm^2
Unity check		0,75	-

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design

Decisive position for stability classification: 4,480 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [N/mm^2]	σ_2 [N/mm^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	400,0	30,0	-208	-208								
2	I	500,0	20,0	-208	224	-0,9		0,5	25,0	56,1	64,6	94,0	1
3	I	400,0	30,0	224	224	1,0		1,0	13,3	26,8	30,9	34,2	1
4	I	500,0	20,0	224	-208	-0,9		0,5	25,0	56,1	64,6	94,0	1

The cross-section is classified as Class 1
Note: The Elastic verification has been set by the user.
Note: The stability classification is based on the maximum section classification along the member.

Flexural Buckling check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	sway	
System length	L	8,000	8,960	m
Buckling factor	k	2,90	2,76	
Buckling length	l_{cr}	23,229	24,716	m
Critical Euler load	N_{cr}	6189	3243	kN
Slenderness	λ	118,03	163,05	
Relative slenderness	λ_{rel}	1,54	2,13	
Limit slenderness	$\lambda_{rel,0}$	0,20	0,20	
Buckling curve		d	d	
Imperfection	α	0,76	0,76	
Reduction factor	χ	0,26	0,16	
Buckling resistance	$N_{b,Rd}$	3911	2344	kN

Flexural Buckling verification			
Cross-section area	A	4,160e+04	mm^2
Buckling resistance	$N_{b,Rd}$	2344	kN
Unity check		0,15	-

Torsional(-Flexural) Buckling check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Torsional buckling length	l_{cr}	8,960	m
Elastic critical load	$N_{cr,T}$	2202320	kN
Elastic critical load	$N_{cr,TF}$	3243	kN
Relative slenderness	$\lambda_{rel,T}$	2,13	
Limit slenderness	$\lambda_{rel,0}$	0,20	
Buckling curve		d	
Imperfection	α	0,76	
Reduction factor	χ	0,16	
Cross-section area	A	4,160e+04	mm^2
Buckling resistance	$N_{b,Rd}$	2344	kN
Unity check		0,15	-

Lateral Torsional Buckling check

According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters			
Method for LTB curve		General case	
Elastic section modulus	$W_{el,y}$	6,445e+06	mm^3
Elastic critical moment	M_{cr}	65269	kNm
Relative slenderness	$\lambda_{rel,LT}$	0,19	
Limit slenderness	$\lambda_{rel,LT,0}$	0,20	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	8,960	m
Influence of load position		no influence	
Correction factor	k	1,00	
Correction factor	k_w	1,00	
LTB moment factor	C_1	1,13	
LTB moment factor	C_2	0,45	
LTB moment factor	C_3	0,53	
Shear centre distance	d_z	0,0	mm
Distance of load application	z_g	0,0	mm
Mono-symmetry constant	β_y	0,0	mm
Mono-symmetry constant	z_j	0,0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 1	
Cross-section area	A	4,160e+04	mm^2
Elastic section modulus	$W_{el,y}$	6,445e+06	mm^3
Design compression force	N_{Ed}	352	kN
Design bending moment (maximum)	$M_{y,Ed}$	1655	kNm
Design bending moment (maximum)	$M_{z,Ed}$	0	kNm
Characteristic compression resistance	N_{Rk}	14768	kN
Characteristic moment resistance	$M_{y,Rk}$	2288	kNm
Reduction factor	χ_y	0,26	
Reduction factor	χ_z	0,16	
Reduction factor	χ_{LT}	1,00	
Interaction factor	k_{yy}	1,03	
Interaction factor	k_{zy}	0,97	

Maximum moment $M_{y,Ed}$ is derived from beam B3 position 4,480 m.
Maximum moment $M_{z,Ed}$ is derived from beam B3 position 0,000 m.

Interaction method 1 parameters			
Critical Euler load	$N_{cr,y}$	6189	kN
Critical Euler load	$N_{cr,z}$	3243	kN
Elastic critical load	$N_{cr,T}$	2202320	kN
Elastic section modulus	$W_{el,y}$	6,445e+06	mm^3
Second moment of area	I_y	1,611e+09	mm^4
Second moment of area	I_z	9,559e+08	mm^4
Torsional constant	I_t	1,682e+09	mm^4
Method for equivalent moment factor $C_{my,0}$		Table A.2 Line 2 (General)	
Design bending moment (maximum)	$M_{y,Ed}$	1655	kNm
Maximum relative deflection	δ_z	-39,4	mm
Equivalent moment factor	$C_{my,0}$	1,01	
Factor	μ_y	0,96	
Factor	μ_z	0,91	
Factor	ε_y	30,35	
Factor	a_{LT}	0,00	
Critical moment for uniform bending	$M_{cr,0}$	57914	kNm
Relative slenderness	$\lambda_{rel,0}$	0,20	
Limit relative slenderness	$\lambda_{rel,0,lim}$	0,21	
Equivalent moment factor	C_{my}	1,01	
Equivalent moment factor	C_{mLT}	1,00	

Unity check (6.61) = 0,09 + 0,74 + 0,00 = 0,83 -
Unity check (6.62) = 0,15 + 0,71 + 0,00 = 0,86 -

The member satisfies the stability check.

Figure D-138 Vertical moving frame SCIA report

D.5. ASSEMBLY AND DISASSEMBLY

The same script as Annex C.1.3.1 is used. The weights and dimensions are changed to this situation.

```
> restart;
b is the width of the mast section, G is the assumed weight of the upper structure plus the top climbing frames divided by two (two towers).
h is the height of the climbing frame, L is the distance between the tower and the lifting of the new mast section, Y is the side load.
RA is the reactional force of the left chord, RB of the right chord, X is the load of the lifting mast seciton plus the hoisting mechanism.
> b:=10.2; G:=5000; X:=1000; h:=32; Y:=0.05*(G+X);
```

$$b := 10.2$$
$$G := 5000$$
$$X := 1000$$
$$h := 32$$
$$Y := 300.00$$

```
> eq1:=R__A=1/2*b*G-b*R__B+(b+L)*X+h*Y;
```

$$eq1 := R_A = 45300.00000 - 10.2\,R_B + 1000\,L$$

```
>
> eq2:=R__A+R__B=G+X;
```

$$eq2 := R_A + R_B = 6000$$

```
> S:=solve({eq1,eq2},{R__A,R__B});
```

$$S := \{R_A = 1728.260870 - 108.6956522\,L, R_B = 4271.739130 + 108.6956522\,L\}$$

```
> assign(S);
> eq3:=R__A=0;
```

$$eq3 := 1728.260870 - 108.6956522\,L = 0$$

```
> S2:=solve({eq3},{L});
```

$$S2 := \{L = 15.90000000\}$$

D.6. DEMANDS

Figure D-139 (left) shows a top view of all the lifting operations that are needed and the upper structure split that require space. Figure D-139 (right) approximates these areas. Where each area is:

$$A_I = \frac{1}{2} * 31.918 * 11.277 = 179.97\ m^2$$
$$A_{II} = 31.918 * 29.722 = 948.67\ m^2$$
$$A_{III} = \frac{1}{2} * 31.918 * 12.048 = 192.27\ m^2$$
$$A_{tot} = A_I + A_{II} + A_{III} = 1320.19\ m^2$$

The rectangular area, marked with II, is the climbing frame in the second configuration. Compared to the original, the width is unchanged, the depth is increased from 8.6 to 29.7 meters. So, an area of 274.49 m^2 needs to be subtracted. Making that the total area needed for this design is approximately 1050 m^2 is needed.

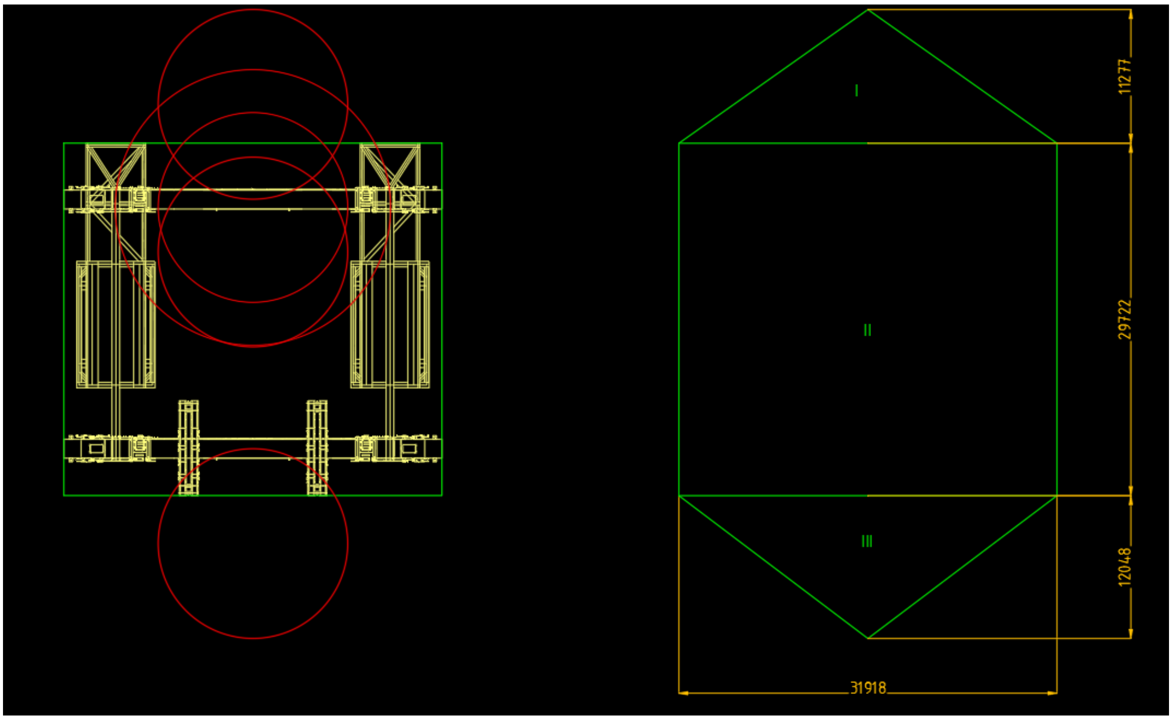


Figure D-139 Area needed for assembly and disassembly of final design