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Specialization: Transport Engineering and Logistics

Report number: 2014.TEL.7853

Title: **Concept design of a modular heavy lift beam**

Author: B.B. de Keyzer

Title (in Dutch) Conceptontwerp van een modulaire zwaartransport balk

Assignment: Master thesis

Confidential: No

Initiator (university): prof.dr.ir. G. Lodewijks

Initiator (company): P. Könst (RollDock Shipping B.V.)

Supervisor: ir. W. van den Bos

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Supervisor (RollDock):	P. Könst	Specialization:	TEL
		Report number:	2014.TEL.7853
		Confidential:	No

Subject: **Design of a multipurpose skid beam for BigRoll**

BigRoll shipping (joint venture between BigLift and RollDock) is in the process of designing a heavy lift vessel with the capabilities of loading/discharging 15.000 ton cargo by means of Ro/Ro or Skidding. The vessel will be operated by BigRoll shipping as a service provider for the transport of heavy cargo. Loading and discharging of the cargo will be either scope of the charter party or will be subcontracted by BigRoll. Part of the Roll-company entities is Roll-Lift. Roll-Lift is a heavy lift and transport company which provides onshore service.

Roll-Lift will be the main contact party for the Ro/Ro, Lo/Lo and skidding operations for BigRoll.
For the above services Roll-Lift has a broad range of lifting (cranes up to 1350t) and transport (SPMT and conventional trailers) equipment. To provide the total scope of work for the BigRoll vessels Roll-lift is in the process of buying skidding equipment.

The skidding equipment will consist of several parts, being amongst all a skid beams, skid shoes and a jacking system. Skid shoes and jacking systems can be purchased modular up to 400t of the shelf. Skid beams are often tailor made for the skidding project as these require load introduction in either quays of the vessel structure.
For the above purpose a new skid beam design will have to be made which assesses the following design boundaries;

- A lightweight design (max 40ft/20ft weight)
- Modular transportable (20ft of 40ft)
- Matches the BigRoll vessel deck structure (Longitudinal and transverse)
- Multipurpose (for example also as gantry beam usable)
- Capable of facilitating different skid shoes
- Easy connectable in length (by pin/bolt, etc.. connection)

The report should comply with the guidelines of the section. Details can be found on the website.

The professor,

Prof. dr. ir. G. Lodewijks

Concept design of a modular heavy lift beam

by

B. B. de Keyzer

in partial fulfillment of the requirements for the degree of

Master of Science
in Mechanical Engineering

at the Delft University of Technology,

Report number:	2014.TEL.7853	
Student number:	1263773	
Professor:	Prof. dr. ir. G. Lodewijks	
Daily supervisors:	ir. W. van den Bos,	TU Delft
	P. Könst,	RollDock Shipping B.V.

Preface

This master thesis describes the research I conducted as final part of the Transport Engineering and Logistics program at the Delft University of Technology, for the Master of Science degree in Mechanical Engineering.

This thesis is the result of 10 months research at Roll-Lift in Capelle aan den IJssel, on the concept design of a modular heavy lift beam. At first the design of a steel beam seemed a manageable subject, but during the course of the project it proved to be quite a challenge.

The graduation project is conducted under the watchful eye of the following supervisors:

prof. dr. ir. G. Lodewijks	Delft University of Technology
ir. W. van den Bos	Delft University of Technology
P. Könst	Roll Group

B. B. de Keyzer
Capelle aan den IJssel, April 2013

Acknowledgements

I would like to express my gratitude to the persons who supported me along the way of writing my thesis.

First of all, I would like to thank prof. dr. ir. G. Lodewijks and ir. W. van den Bos from the Delft University of Technology for their supervision and support during my graduation project. Their suggestions and remarks helped me keeping focussed and critical about my own work.

Secondly I would like to thank P. Könst, for making my graduation project at Roll-Lift possible and for all the support he provided during the course of my project. He shared his practical knowledge on the subject and had a critical view on every chapter I wrote in this thesis.

Furthermore I would like to thank everyone at RollDock and Roll-Lift, especially at the engineering department, for sharing their knowledge and their encouragement to keep going when I seemed to get stuck.

Finally I would like to thank my parents for their support, without them I would not have come to where I am now.

Summary

The heavy lift market is characterized by ultra heavy and/or odd sized loads. Heavy transports are unique projects, which are engineered for that one time job. Standard heavy lift and transport equipment is used to the extent possible for each project, but often specialized structures are required. A structure that is often used for this purpose is a heavy lift beam. Heavy lift beams are used for load spreading, as well as applications in horizontal and vertical transport.

RollDock is a shipping company that provides worldwide heavy lift services over sea. RollDock is currently in the process of designing a new wide deck heavy lift vessel for loads up to 15000 tons, the Module Carrier (MC). This type of vessel will be put into service in 2015. When loading ultra heavy loads, a support structure is required to prevent damage to the deck due to concentrated loads. Current practice is that this support structure is engineered and manufactured individually for each project. To reduce on engineering and material cost, RollDock has requested to design a standardized steel beam that can be re-used for several projects. For increased productivity of this standardized beam, it must be multi functional to be used for other heavy lift purposes than solely as supports on the MC. These purposes are also found in RollDock's sister company Roll-Lift, which facilitates heavy lift projects on land.

The functions of the beam are determined using past RollDock and Roll-Lift projects, as well as functions that may be useful in future projects. The functions for the modular beam are skid support beam, grillage, different parts of a heavy lift gantry, a vertical spacer and a temporary bridge.

The goal of this research is to develop a concept design of a modular, containerized, multi functional beam for heavy lift purposes, with the main purpose to be used for loading the BigRoll MC using platform trailers or skidding.

An analysis is performed of all the functions and the equipment involved to fulfill the functions. The equipment comprises different types of skid systems, platform trailers and strand jacks. The MC is analyzed as well. From the functions, equipment and MC, along with requirements imposed by RollDock and Roll-Lift, a set of criteria is composed that forms the basis of the concept design. Moreover, the steel calculations on the concept design are in compliance with the standard Eurocode 3 (EC3).

For the basic design, the beam is decomposed into four sub-systems: the shape of the cross section, the length of beam sections, the design of the connections between beam sections and auxiliary components.

In the design of the cross section of the beam, the choice is made between an H-profile, a box beam and a box beam with offset webs. Based on the bending moment in the flanges due to the functions, and therefore the amount of material required, the choice is made to use the box beam with offset flanges.

For the determination of the section length, the fit in transverse direction of the deck of the MC is used to select a set of possible section lengths. Using *Pugh's method*, which compares the options relative to each other according to a set of criteria, the choice is made to use section lengths of 5400mm and 11400mm.

The decision for the connection between sections is also made according to *Pugh's method*. The options for connecting two beam sections are bolts, pin-hole, clamps, container twist locks and a shape fit connection. The pin-hole connection came out most favourable relative to the other options.

The auxiliary components have been devised to aid the beam sections in their performance in the different functions. These auxiliary components are not engineered into detail in this research. The auxiliary components proposed are a T-connector and hinges.

The T-connector is dimensioned to provide the same spacing between beams if connected sideways as the wheel bogey spacing of platform trailers for the use as temporary bridge. If connected head to head, the ideal spacing is created for a gantry tower foundation.

The hinge is used with a beam section of 5400mm to create a link beam, a hinged part of the skid support beam to create a bridge between quay and vessel. Due to the hinges, this bridge allows for a small amount of movement between quay and vessel while still providing a continuous support beam along the entire length of skid track.

The basic design is tested for feasibility regarding structural and economical aspects. For the structural analysis, the plate thicknesses of the parts are first calculated with hand calculations for S355 steel. The calculations revealed that High Strength Steel (HSS) is required for the connections. Then the load cases following from the functions are evaluated using a Finite Element Analysis (FEA) in RFEM. Load combinations are made according to EC3.

For the economic analysis an estimate is made of the implementation cost and recurring cost. With a depreciation period of 15 years, the annual cost are determined. These annual cost are translated to a required revenue to cover the cost, which is approximately €61.- per ton per week. Compared to the prices of beams of competitors, which are €80.- to €100.- per ton per week, this is a reasonable value.

From this research, several conclusions can be drawn regarding the modular, containerized, multi functional design, the fit for the MC, the materials used and the cost of the concept.

For the multi functional design, it has been shown that none of the functions have extreme load requirements compared to the other functions, due to the choice of equipment used. The functions skid support beam and grillage posed requirements on the width and height respectively. These requirements have shown not to interfere with the requirements and performance of other functions. The conclusion about multi functional design is that all functions mentioned can be united in one multi functional heavy lift beam.

The modular design encompasses several components, such as long beam sections, short beam sections, T-connectors and link beam attachments. These components allow for a multitude of arrangements, from a long beam assembly to a frame structure. Due to the design of the of the connections, the components can always be connected regardless of their orientation.

The basic design proves that dimensions and mass of the beam elements are well within the limits for containerized transport, and that multiple elements can be transported in one container.

The concept beam fits the deck structure of the MC, with regard to the length and stiffeners. The length of beam sections is determined using the transverse direction on deck of the MC to ensure a proper fit on deck. To cope with high bearing loads at intersections with the vessels bulkheads, stiffeners are placed at these locations in the beam.

The material used for the overall design of the beam is S355 steel. It is proven through hand calculations and FEA that sufficient strength is achieved with the proposed beam design constructed from S355 steel. The connection between beam elements however, cannot be constructed from S355 steel because this would require a connection that is wider than the beam. Therefore HSS is used for the connection between beam sections.

A minimum revenue per week per ton is determined to cover the cost of the beam concept. It is concluded that the minimum revenue is lower than the rent price of comparable beams of other companies, so the beam concept is profitable.

Samenvatting

De zwaartransportmarkt wordt gekenmerkt door zeer zware en/of overmaatse ladingen. Vaak zijn zwaartransporten eenmalige projecten, die speciaal voor een specifieke klus op maat zijn gemaakt. Waar mogelijk wordt standaard zwaartransport materieel ingezet, maar vaak blijkt dat passende oplossingen bedacht moeten worden. Een type materieel dat vaak gebruikt wordt, zijn zwaartransport balken. Deze balken worden gebruikt voor het uitspreiden van een last en voor verschillende toepassingen in horizontaal en verticaal transport.

RollDock is een transportbedrijf dat gespecialiseerd is in wereldwijd zwaartransport over zee. Tijdens dit onderzoek is RollDock bezig met het ontwerpen van een nieuw open dek zwaartransport schip, de Module Carrier (MC), die ladingstukken tot 15000 ton moet gaan vervoeren. Dit schip zal in 2015 in de vaart worden genomen. Om ultra zware ladingstukken te laden, is een staalstructuur aan boord nodig om te voorkomen dat de lading door het dek zakt. Tot nu toe is het gebruikelijk om deze staalstructuur voor ieder project afzonderlijk te ontwerpen en produceren. Om te besparen op kosten voor het ontwerp en de productie, heeft RollDock de opdracht gegeven om een gestandaardiseerde balk te ontwerpen die hergebruikt kan worden voor meerdere projecten. Om de productiviteit van deze balk te verhogen, moet het een multi-functioneel ontwerp worden. Deze functies komen niet alleen van RollDock, maar ook van haar zusterbedrijf Roll-Lift dat zwaartransport op land verzorgt.

De functies worden bepaald aan de hand van RollDock en Roll-Lift projecten uit het verleden, maar ook functies die in de toekomst van pas kunnen komen. De functies van de balk worden een ondersteuningsbalk voor sleesystemen, lastspreider, verschillende toepassingen in een portaalkraan, een vertikale afstandshouder en een tijdelijke brug.

Het doel van dit onderzoek is om een concept te ontwikkelen van een modulaire, multi-functionele zwaartransportbalk die in containers vervoerd kan worden, met de belangrijkste functie om de BigRoll MC te laden via zwaartransport trailers of sleesystemen.

Als basis voor het ontwerp is er een achtergrondonderzoek gedaan naar het materieel wat rond de balk gebruikt gaat worden. Dit zijn verschillende typen sleesystemen, zwaartransport trailers, hijsmaterieel en de BigRoll MC. Vanuit deze functies, samen met de eisen en wensen van RollDock en Roll-Lift, is een set criteria samengesteld die gebruikt worden voor het concept ontwerp. Staalberekeningen aan het ontwerp zijn uitgevoerd volgens de eisen van Eurocode 3 (EC3).

Het concept ontwerp is onderverdeeld in vier onderdelen: de vorm van de doorsnede van de balk, de lengte van de balkdelen, de verbinding tussen de delen en hulpmiddelen.

Voor de vorm van de balk is een keuze gemaakt tussen een H-balk, een box balk en een box balk met naar binnen geplaatste lijven. Om het buigend moment door de functies in de flenzen zo laag mogelijk te houden, en daarmee de hoeveelheid materiaal, is gekozen voor de box balk met naar binnen geplaatste lijven.

De breedte van het dek van de MC is als maat genomen voor de lengte van de balkdelen. Vier alternatieven zijn met behulp van *Pugh's method*, welke de opties relatief met elkaar vergelijkt, afgewogen waarna gekozen is voor balk lengtes van 5400mm en 11400mm.

De beslissing van de verbinding tussen balkdelen is ook gemaakt op basis van *Pugh's method*, waar de pen-gat verbinding als beste uitkwam, vergeleken met verbindingen met bouten, klemmen, container twist locks en een vormgesloten verbinding.

De hulpmiddelen zijn bedacht als toevoeging aan de balkdelen om de functionaliteit te vergroten of vergemakkelijken. De hulpmiddelen worden in dit onderzoek niet uitgewerkt. De voorgestelde hulpmiddelen zijn een T-verbinding en een scharnier.

De breedte van de T-verbinding is zo bedacht dat dezelfde tussenruimte ontstaat als de wielsets van zwaartransport trailers voor het gebruik in een tijdelijke brug. De lengte van twee gekoppelde T-verbindingen is optimaal voor de fundering van portaal torens.

Het scharnier wordt in samenstelling met een balkdeel van 5400mm gebruikt om een brug tussen wal en schip te maken in de sleebalk. Door de scharnieren ontstaat een stuk balk die ruimte biedt voor beweging van het schip ten opzichte van de kade.

Het concept is geëvalueerd op sterkte en op economisch vlak. Voor de evaluatie van de sterkte zijn eerst de plaatdiktes bepaald met handberekeningen, gebaseerd op S355 staal. Uit de berekeningen is gebleken dat hogesterkte staal (HSS) een vereiste is voor de verbinding tussen balkdelen. Vervolgens zijn de handberekeningen gecontroleerd met een eindige elementen analyse (EEA) in RFEM.

Voor de economische analyse is een schatting gemaakt van de implementatiekosten en de jaarlijkse kosten. Met een afschrijvingstermijn van 15 jaar zijn de implementatiekosten vertaald naar jaarlijkse bedragen. Met de totale jaarlijkse kosten zijn de minimale opbrengsten bepaald, wat neerkomt op ongeveer €61,- per ton per week. Vergeleken met balken van concurrenten, die tussen de €80,- en €100,- per ton per week liggen, is dit een goede uitkomst.

Uit dit onderzoek kunnen een aantal conclusies getrokken worden, aangaande het modulaire, multifunctionele ontwerp binnen container restricties, de geschiktheid voor de MC, de gebruikte materialen en de kosten van het concept.

Wat betreft het multi-functionele ontwerp is aangetoond dat meerdere functies verenigd kunnen worden in één zwaartransport balk. Door de keuze van het materieel zijn de belastingen van de verschillende functies op de balk vergelijkbaar. Eisen opgelegd door de ene functie blijken niet nadelig voor de andere functies.

Door het gebruik van verschillende onderdelen, zoals lange balkdelen, korte balkdelen, T-verbindingen en scharnieren, allen met dezelfde eindverbindingen ontstaat een modulaire balkenset. Met deze set kan een breed scala aan samenstellingen gecreëerd worden, afhankelijk van de situatie.

Met de sterkteberekeningen is aangetoond dat de onderdelen van de balk qua afmetingen en gewicht binnen de eisen voor container transport blijven, en dat zelfs meerdere elementen in een container getransporteerd kunnen worden.

Wanneer de balk op het dek van de MC gebruikt wordt, komen de locaties van sterke punten in de balk overeen met de sterke punten van het schip. Ook qua lengte past de balk netjes op het dek.

Het gebruikte materiaal voor de balk is S355 staal. Door middel van handberekeningen en EEA is aangetoond dat het voorgestelde balk ontwerp voldoende sterkte heeft om de beschreven functies uit te kunnen voeren. Voor de verbindingen tussen balkdelen is gebleken dat S355 staal niet voldoende sterkte bezit, daarom zullen de verbindingen worden uitgevoerd in HSS.

Er is een basis balkenset samengesteld om iedere functie uit te kunnen voeren. Voor deze balkenset is de minimum opbrengst per week per ton bepaald om de kosten van het concept te dekken. Er kan geconcludeerd worden dat de balk winstgevend kan zijn, gezien de kostprijs van deze balkenset beduidend lager ligt dan de huurprijs van balken van andere bedrijven.

List of symbols

A	Accidental actions
A	Area
$A_{Eff,flange}$	Cross section area of the effective width of the flange
$A_{flanges}$	Cross sectoin area of the flanges
A_{LS}	Cross section area of the longitudinal stiffeners
A_{OS}	Cross section area of the outstand stiffeners
A_v	Shear area
$A_{v,LS}$	Shear area of the longitudinal stiffener
$A_{v,OS}$	Shear area of the outstand stiffener
$A_{v,TS}$	Shear area of the transverse stiffener
$A_{v,webs}$	Shear area of the webs
A_{TS}	Cross section area of the transverse stiffeners
A_{webs}	Cross section area of the webs
c	Plate width
E	Young's modulus
E_d	Design value of the effect of actions
$E_{d,dst}$	Design value of the effect of destabilising actions
$E_{d,stb}$	Design value of the effect of stabilising actions
F	Concentrated load
F_d	Design value of an action
F_k	Characteristic value of an action
F_{rep}	Representative value of an action
f_u	Ultimate strength
f_y	Yield strength
G	Permanent actions
G	Shear modulus
G_z	Self weight
H_{beam}	Heigth of the beam
h_{LS}	Height of the longitudinal stiffeners
h_{OS}	Height of the outstand stiffeners
h_{sup}	Height of the supports
h_{TS}	Height of the transverse stiffeners
h_{web}	Heigth of the web
I	Second moment of inertia
$I_{Eff,flange}$	Second moment of inertia of the effective width of the flange
$I_{flanges}$	Second moment of inertia of the flanges
I_{LS}	Second moment of inertia of the longitudinal stiffeners
I_{OS}	Second moment of inertia of the outstand stiffeners

I_{TS}	Second moment of inertia of the transverse stiffeners
I_{webs}	Second moment of inertia of the webs
CO	Load combination
l_{rep}	Unsupported length
l_{out}	Length of outstand
M	Bending moment
M_{Ed}	Design bending moment
M_{Rd}	Design bending resistance
N_{Ed}	Design normal force
$N_{c,Rd}$	Design compression resistance
$N_{t,Rd}$	Design tension resistance
q	Distributed load
Q	Variable actions
q_{allow}	Allowable line load on modular beam
Q_k	Characteristic value of a variable action
Q_{load}	Value of the leading variable action
q_{sup}	Maximum line load on deck structure
Q_{wind}	Value of the variable action due to wind
R	Reaction force
C_d	Design value of the effect of actions specified in the serviceability criterion
R_d	Design value of the resistance to the corresponding action
S	First moment of inertia
γ_{SF}	Safety factor
t	Plate thickness
T_{ed}	Design torsional moment
T	Temperature
T_{Ed}	Reference temperature
t_{flange}	Thickness of the flanges
t_{LS}	Thickness of the longitudinal stiffeners
t_{OS}	Thickness of the outstand stiffeners
T_{Rd}	Design torsional resistance
$T_{t,Ed}$	Internal St. Vernant torsion
t_{TS}	Thickness of the transverse stiffeners
t_{web}	Thickness of the webs
$T_{w,Ed}$	Internal warping torsion
v	Poisson's ratio in elastic stage
V	Shear force
V_{Ed}	Design shear force
V_{Rd}	Design shear resistance
W_{beam}	Width of the beam
w_{eff}	Effective width of the flange from centerline of the stiffener
W_{el}	Design elastic section modulus
w_{flange}	Width of the flange

w_{sup}	Width of the supports
w_{tot}	Total vertical deflection
x	Distance
y	Distance from neutral line to extreme fibre
$\overline{y}$	Location of the neutral axis in z-direction
$\overline{y}_{flanges}$	Location of the neutral axis of the flanges in z-direction
$\overline{y}_{LS}$	Location of the neutral axis of the longitudinal stiffeners in z-direction
$\overline{y}_{OS}$	Location of the neutral axis of the outstand stiffeners in z-direction
$\overline{y}_{TS}$	Location of the neutral axis of the transverse stiffeners in z-direction
$\overline{y}_{webs}$	Location of the neutral axis of the webs in z-direction
Z_{Ed}	Required Z-value resulting form the magnitude of strains from restrained metal shrinkage under the weld beads
α	Load multiplier
α	Coefficient of linear thermal expansion
δ_{max}	Maximum deflection
ϵ	Strain
ϵ_u	Strain at ultimate strength
ϵ_y	Strain at yield strength
γ_G	Partial safety factor for permanent actions
γ_f	Partial factor for an action
γ_M	Partial factor for resistance of cross sections
γ_Q	Partial safety factor for variable actions
ψ_0	Factor for combination value of a variable action
ψ_1	Factor for frequent value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action
σ	Normal stress
σ_{allow}	Allowable stress
σ_{Ed}	Reference stress
σ_y	Yield stress
τ_{allow}	Allowable shear stress
τ_{Ed}	Design shear stress
τ_{max}	Maximum shear stress
ξ	Reduction factor

List of abbreviations

AISC	American Institute of Steel Construction
COG	Center of Gravity
EC0	Eurocode 0
EC1	Eurocode 1
EC3	Eurocode 3
FEA	Finite Element Analysis
FEM	Finite Element Method
HSS	High Strength Steel
ISO	International Organisation for Standardization
LB	Longitudinal Bulkhead
MC	Module Carrier
SLS	Serviceability Limit States
SPMT	Self Propelled Modular Tranporter
SPT	Self Propelled Trailer
TB	Transverse Bulkhead
ULS	Ultimate Limit States
WF	Web Frames

Contents

1
Introduction

The heavy lift market is characterized by ultra heavy and/or odd sized loads. Often heavy transports are unique projects, which are engineered for that one time job. Standard heavy lift and transport equipment is used to the extent possible for each project, but often specialized structures are required to provide enough load spreading or a precise fit of the load onto the equipment. Structures that are often used for this purpose are load spreader mats and heavy lift beams. The first offers primarily load spreading while the second is also used to span an unsupported length between supports. This research will focus on the heavy lift beams.

1.1. Modular beam for heavy lift purposes

Heavy lift beams are used for load spreading, as well as applications in horizontal and vertical transport. In the heavy lift market, each company has its own heavy lift beams and often multiple beams for different purposes. For some applications, heavy lift beams are made specially for one project and scrapped afterwards. Simple heavy lift beams are steel boxes that can only be used individually (Figure 1.1(a)), while more sophisticated beams have couplings to connect to each other and form longer beams (Figure 1.1(b)). The similarity in all heavy lift beams is that they can be transported in or as standard shipping containers, to provide easy and relatively cheap shipping all around the globe to the location of the projects.

(a) Heavy lift beam in its simplest form (b) More sophisticated heavy lift beam with couplings

Figure 1.1: Examples of heavy lift beams

1.2. Problem definition

RollDock is a shipping company that provides worldwide heavy lift services, currently with two vessels and two other being manufactured. RollDock is furthermore in the process of designing a new wide deck heavy lift vessel for loads up to 15000 tons, the Module Carrier (MC). This type of vessel will be put into service in 2015. When loading ultra heavy loads, a support structure is required to prevent damage to the deck due to concentrated loads. Current practice is that this support structure is engineered and manufactured individually for each project.
To reduce on engineering and material cost, RollDock has requested to design a standardized steel beam that can be re-used for several projects. For increased productivity of this standardized beam, it must be multi functional to be used for other heavy lift purposes than solely as supports on the MC. These purposes are also found in RollDock's sister company Roll-Lift, which facilitates heavy lift projects on land.
The focus of this research is the concept design of a modular, multi functional heavy lift beam. The beam has to be modular for easy and cost effective transport. The beam must be multi functional to be applicable in a wide range of heavy lift purposes, to support RollDock and Roll-Lift in every phase of a project "from factory to foundation".

1.3. Functions of the beam

The functions of the beam are determined using past RollDock and Roll-Lift projects, as well as functions that may be useful in future projects. The primary functions of the beam are based on the loading methods of the MC, via skidding or platform trailers. The corresponding functions for the modular beam are skid support beam (see Figure 1.2(a)) and grillage (see Figure 1.2(b)). The other functions are different parts of a heavy lift gantry, a vertical spacer and a temporary bridge (see Figure 1.2(c) to 1.2(f) respectively).

1.4. Research goal

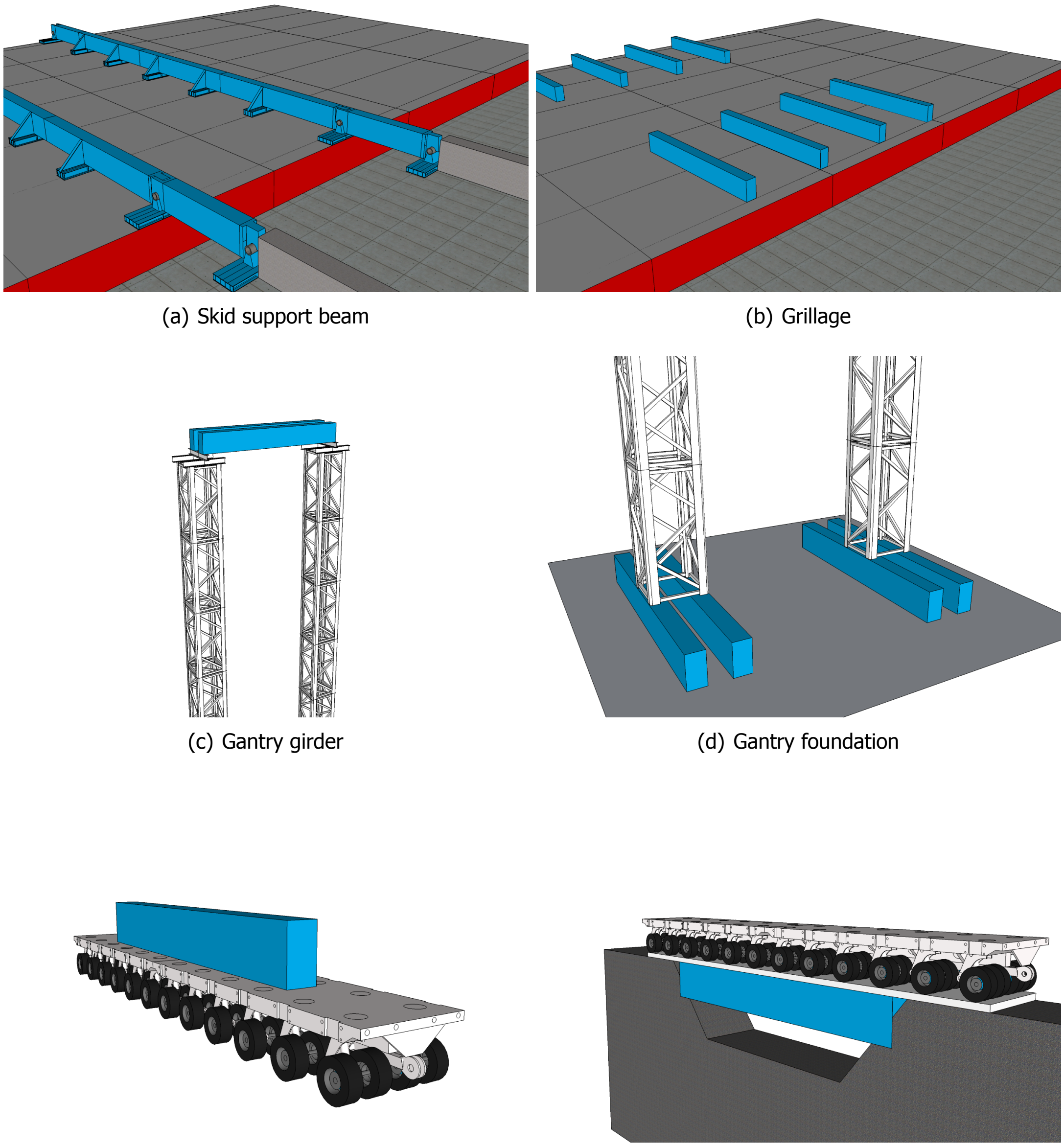
The objective of this research is: The aim of this research is to develop a concept design of a modular, containerized, multi functional beam for heavy lift purposes, with the main purpose to be used for loading the BigRoll MC using platform trailers or skidding.

1.5. Research method

This research will be divided in three phases:

1. Determination of the functions and literature research
2. Concept generation
3. Concept evaluation

In the first phase, all requirements and boundaries will be determined. The second phase will start with a set of design criteria. This set of design criteria will be used to develop the concept. In the third phase, this concept will be evaluated for economic feasibility and structural requirements.



(a) Skid support beam

(b) Grillage

(c) Gantry girder

(d) Gantry foundation

(e) Vertical spacer

(f) Temporary bridge

Figure 1.2: Functions of the modular beam, the beam elements are indicated in blue

1.6. Report overview

The structure of this report is shown schematically in Figure 1.3. The connections between the different subjects in the figure show multiple design cycles exist in the form of feedback to previous blocks.

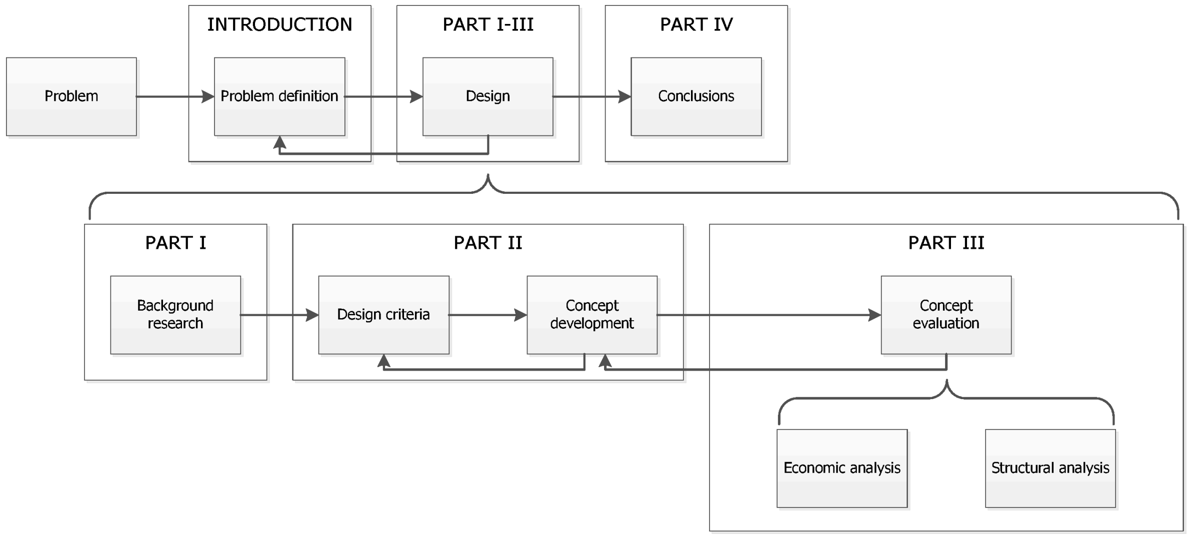


Figure 1.3: Schematic overview of the contents of this report

PART I

Background research

2

Functions of the modular beam

In this chapter, a short description per beam function will be presented along with the equipment used to fulfill the function. In section 2.1 the function skid support beam will be discussed. Section 2.2 elaborates the function grillage. In Section 2.3 the functions gantry girder and gantry foundation will be illustrated. Section 2.4 discusses the function vertical spacer and Section 2.5 presents the function of temporary bridge.

2.1. Skid support beam

2.1.1. General

Skidding [1–3] is the process of sliding a load in order to transport it. Skidding is often used for heavy loads with a small footprint, too small for other transport methods. Skidding is only used for loads that need to be transported over short distances. A range of skid systems is available in the heavy lift industry, but they all consist of the same elements to support and move a load. These elements are the support beams, skid tracks, skid shoes, push-pull units and lubrication (see Figure 2.1).

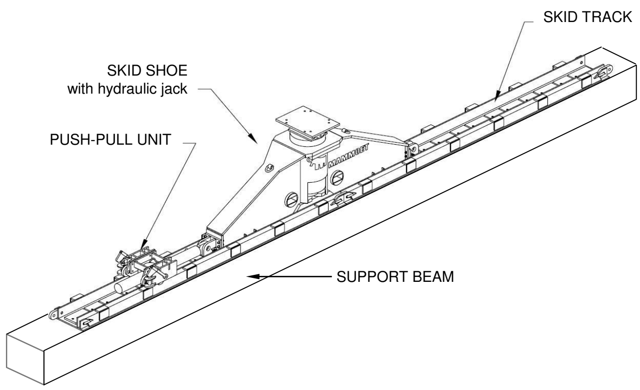


Figure 2.1: Example of a skid system [2]

2.1.2. Purpose of a skid support beam

When using skidding to transfer a heavy load onto a vessel, a support beam is required to prevent damage to the deck of the vessel due to the high ground pressure of the skid system. In short, the purposes of a skid support beam are:

- spread the load over a larger deck area
- transfer concentrated loads to strong points in the deck
- create a bridge between quay and vessel

2.1.3. Equipment used with a skid support beam

Skid systems are available in several capacities up to 600t as standard systems, but for exceptional loads a specially designed skid structure may be better. These special skid structures do not use push-pull units for movement, but strand jacks which will be described in Section 2.3. However, for the design in this research only the standard systems will be considered. The design choice is made to use skid systems of approximately 600t. These systems can support a very heavy load while still offering a favorable load spreading. Table 2.1 gives the specifications of two 600t skid systems that are often used for heavy lift projects, the Mammoet heavy skid system which is used for many years and the more recent Enerpac HSK6000 skid system. The dimensions are illustrated in Figure 2.2. The difference between the average and maximum line loads is elaborated in Appendix B. A graphical presentation of the skid systems mentioned in the table is given in Figure 2.3.

Table 2.1: Specifications of two 600t skid systems that are commonly used for heavy lifting [2, 4]

	Unit	**Enerpac**	**Mammoet**
Type		HSK6000	Heavy system
Capacity	t	600	600
Skid shoe length	mm	4490	4500
Supported length	mm	3000	4000
Offset of supports	mm	220	0
Skid shoe height	mm	1400-1700	1515-2115
Skid track width	mm	880	680
Push-pull unit length	mm	1200	3500
Skid shoes/push-pull unit		1	2
Average line load	t/m	105	96
Maximum line load	t/m	200	150

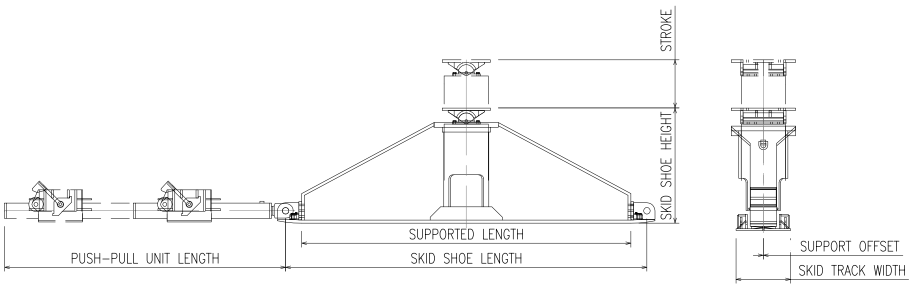
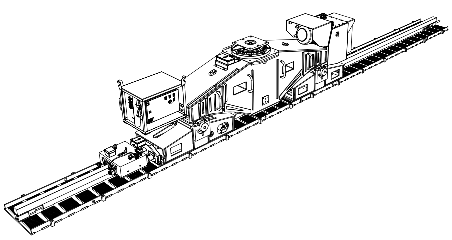
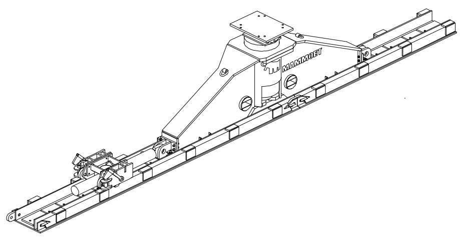


Figure 2.2: Illustration of the dimensions of skid systems [2]



(a) Enerpac HSK6000, capacity 600t [4]



(b) Mammoet heavy system, capacity 600t [2]

Figure 2.3: Skid systems with 600t capacity

2.2. Grillage

2.2.1. General

Grillage is the description of the support structure for loads on the deck of a vessel. The grillage is installed on the deck before the load is brought on board with platform trailers. Once in the right place, the load is lowered onto the grillage using the suspension stroke of the trialers. An example of a grillage is shown in Figure 2.4.

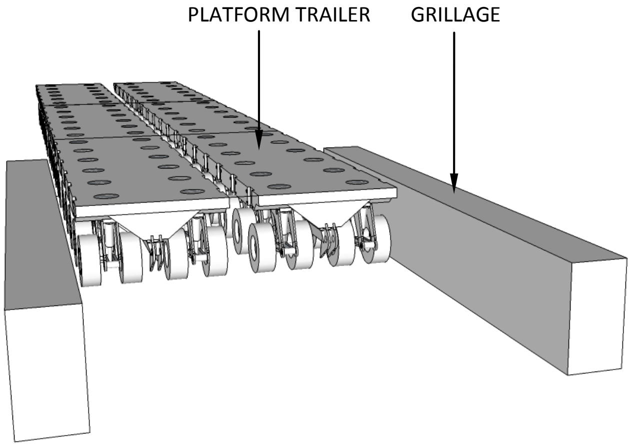


Figure 2.4: Example of the use of a grillage for unloading platform trailers

2.2.2. Purpose of a grillage

The grillage is used to support the load and prevent damage to the deck due to concentrated loads. In short, the purposes of a grillage are largely the same as the skid support beam:

- spread the load over a larger deck area
- transfer concentrated loads to strong points in the deck
- create height between the deck and the load for platform trailers

2.2.3. Equipment used with a grillage

Platform trailers [1, 5–7] are in general sets of axles connected with a rigid trailer body. The trailers are most common as sets of 4 or 6 axle lines, but other configurations with 3, 5 or 8 axle lines are also available. The trailers are modular, which means they can be connected to form one large trailer setup. This connection can be lengthwise to form a long trailer setup, sideways to form a wide trailer setup, or a combination of both to form one big platform. The suspension system of platform trailers is hydraulic, with hydraulic piping between the suspension cylinders to equalize the pressure on each axle. With a suspension stroke of approximately 0.6m, the trailers can compensate for unevenness in the road surface and keep the trailer level while negotiating a slope or camber in the road. This suspension stroke also allows the trailers to put down their load onto supports. Three types of trailers are available, the conventional trailer, Self Propelled Trailer (SPT) and Self Propelled Modular Tranporter (SPMT). These types differ in their specifications, see Table 2.2. The SPT is in fact hybrid form, it has the dimensions of a conventional trailer, but it can be moved by a truck or with its own propulsion. The dimensions are illustrated in Figure 2.5. Also the available steering modes are different, as is shown in Table 2.3. A multi functional grillage must operate with all types of platform trailers, for which especially the height of the grillage is important. Table 2.4 gives an overview of trailer heights of conventional trailers and SPMTs from different manufacturers, as well as the range of the ideal grillage height. The detailed specifications can be found in Appendix B.

Table 2.2: Specifications of conventional trailers, SPTs and SPMTs [1]

	Conventional trailer	**SPT**	**SPMT**
Capacity [t/axle line]	30	30	36
Propulsion	Pulled by truck	Hybrid	Drive in axles
Width [mm]	3000	300	2500
Height [mm]	900-1400	900-1400	1200-1800
Axle line spacing [mm]	1500	1500	1400
Wheels/axle line [-]	8	8	4
Maximum speed [km/h]	80	80	5

Table 2.3: Available steering modes for conventional trailers and SPMTs

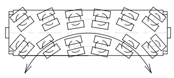
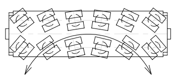
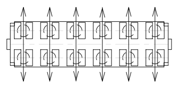
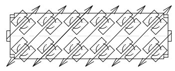
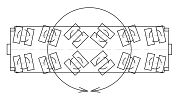
Steering modes:	**Normal**	**Transverse**	**Crawl**	**Carousel**
Conventional trailer				
SPMT				

Table 2.4: Height range of conventional trailers and SPMTs from several manufacturers [2, 6, 8]

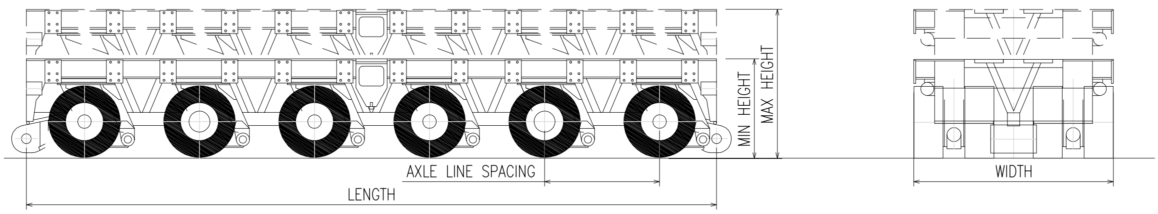


Figure 2.5: Illustration of the dimensions of platform trailers

2.3. Gantry

2.3.1. General

A gantry is a portal type of crane that is capable of lifting very heavy loads. In fact, a gantry is a bridge structure with a lifting device on top. For heavy gantries, the lifting devices of choice are strand jacks [1]. A gantry for heavy lift purposes consists from top to bottom of strand jacks, strand jack supports, gantry girders, gantry girder supports, towers, a foundation and for high gantries guy wires, see Figure 2.6. The strand jacks do the lifting, the rest of the components is the steel structure supporting the strand jacks. The beam in this research will be used as gantry girder and as gantry foundation.

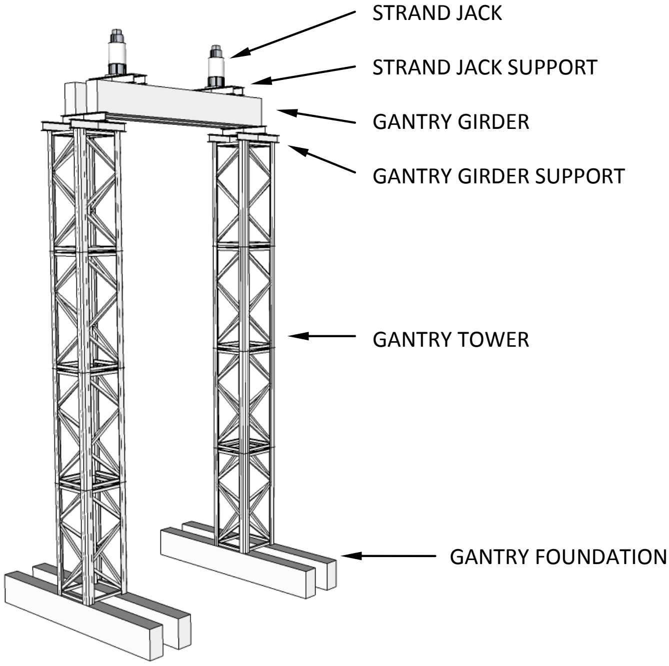


Figure 2.6: Example of a gantry and its components

2.3.2. Purpose of a gantry

A gantry in heavy lifting is used to erect tall and heavy loads and to lift heavy loads to great heights. A gantry is the preferred method for upending tall reactor vessels for the oil and gas industry [1]. If horizontal movement is required as well, the strand jack assembly on top or even the entire gantry can be placed on a skid system as described in Section 2.1.

2.3.3. Equipment used with a gantry

A gantry consists of a steel structure and lifting equipment. The steel structure is basically built up of a load spreading foundation, towers and two gantry girders on top. The actual lifting of the load is performed using strand jacks, placed on top of the gantry girders. The lifting cables of the strand jacks are hanging between the two gantry girders. The towers are often assembled with steel lattice elements, which have high resistance to bending moments and axial loads. In a strand jack, a bundle of steel cables or strands are guided through a hydraulic cylinder. Above and below the cylinder are anchor systems with wedges that grip the strand bundle. By stroking the cylinder in and out while the grips are engaged in the anchors, a lifting or lowering movement is achieved. Table 2.5 shows the specifications of several 600t strand jacks. The dimensions are illustrated in Figure 2.7. The detailed specifications can be found in Appendix B.

Table 2.5: Specifications of several strand jacks with a capacity of approximately 600t [2, 4, 9, 10]

Type	**Dorman Long** DL-S588	**Enerpac** HSL6500	**Fagioli** L600	**Mammoet** SSL550
Capacity [t]	588	650	573	600
Base length [mm]	730	850	900	790
Base width [mm]	730	850	800	790
Height [mm]	2140	2237	2400	1845
Stroke [mm]	500	480	450	400
Mass [kg]	4420	3950	4520	2390

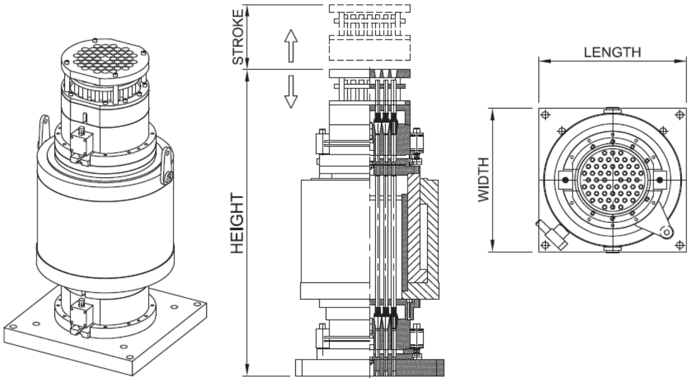


Figure 2.7: Illustration of the dimensions of strand jacks [2]

2.4. Vertical spacer

2.4.1. General

A vertical spacer for heavy lift purposes is in general any wooden or steel structure that is strong enough to carry the load placed on top. Vertical spacers are often used in combination with platform trailers, as can be seen in Figure 2.8.

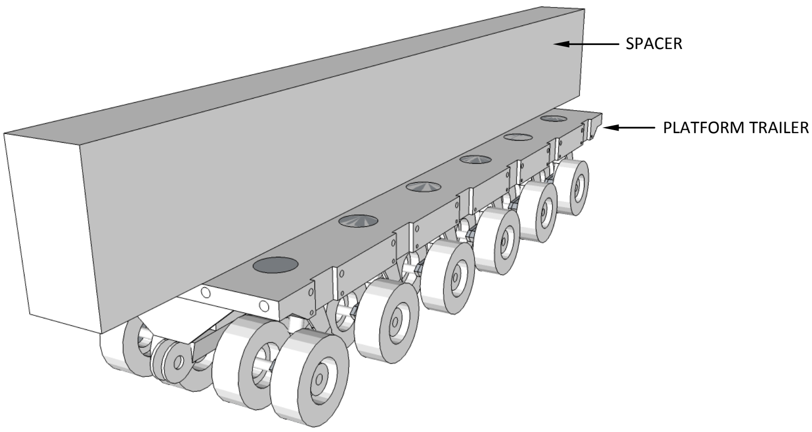


Figure 2.8: Example of the application of a vertical spacer

2.4.2. Purpose of a vertical spacer

A vertical spacer is primarily used when extra elevation of the load is required, in the case that the load is transported on platform trailers. An example is shown in Figure 2.9. Besides this, the vertical spacer brings more advantages. The purposes of a vertical spacer are:

- create extra height on top of a platform trailer
- provide better loadspreading on the platform trailer
- provide added resistance to bending to the platform trailer

2.4.3. Equipment used with a vertical spacer

A vertical spacer is used on top of platform trailers to create extra height. For the detailed description of platform trailers see Section 2.2 and Appendix B.

Figure 2.9: Application where a vertical spacer is required [3]

2.5. Temporary bridge

2.5.1. General

A temporary bridge is a temporary structure, often made of steel. For a small span, steel mats provide enough strength to support the trailer. Larger spans are crossed with beams and H-profiles or steel mats on top, to create a stronger bridge (see Figure 2.10).

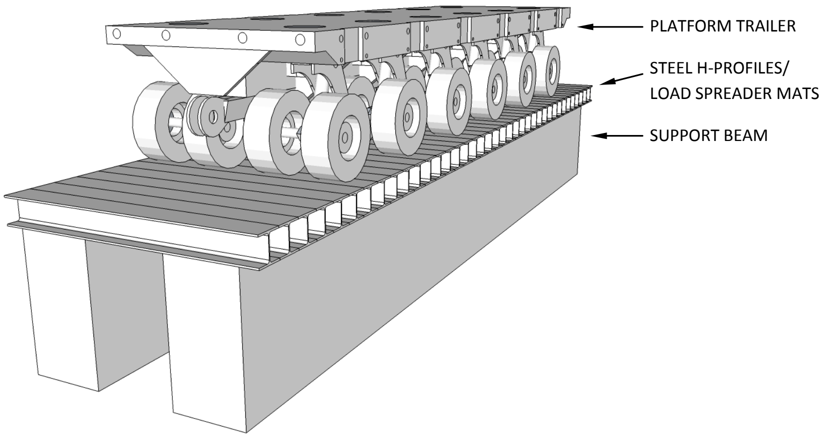


Figure 2.10: Example of a temporary bridge and its components

2.5.2. Purpose of a temporary bridge

It occurs that heavy loads cannot be transported via existing roads, because of obstacles on the route such as low overpasses or poorly maintained bridges. If such an obstacle is encountered, a small detour is created over a temporary bridge.

2.5.3. Equipment used with a temporary bridge

The temporary bridge will be used to create a small detour for heavy transport in case of an obstacle on the normal route. Therefore the bridge will be loaded with trailers used for heavy transport: platform trailers. The description of these trailers can be found in Section 2.2. The bridge deck will be formed from steel H-profiles or steel mats, which is standard load spreading equipment used for heavy lifting.

3
The Module Carrier

The modular beam can be used as skid support beam and grillage on board the to be built MC. This chapter presents the properties of the MC. Section 3.1 gives an overview of the most important specifications of the MC. Section 3.2 elaborates on the loading methods of the MC and Section 3.4 presents details on the strength of the deck.

Figure 3.1: The BigRoll Module Carrier [11]

3.1. Vessel specifications

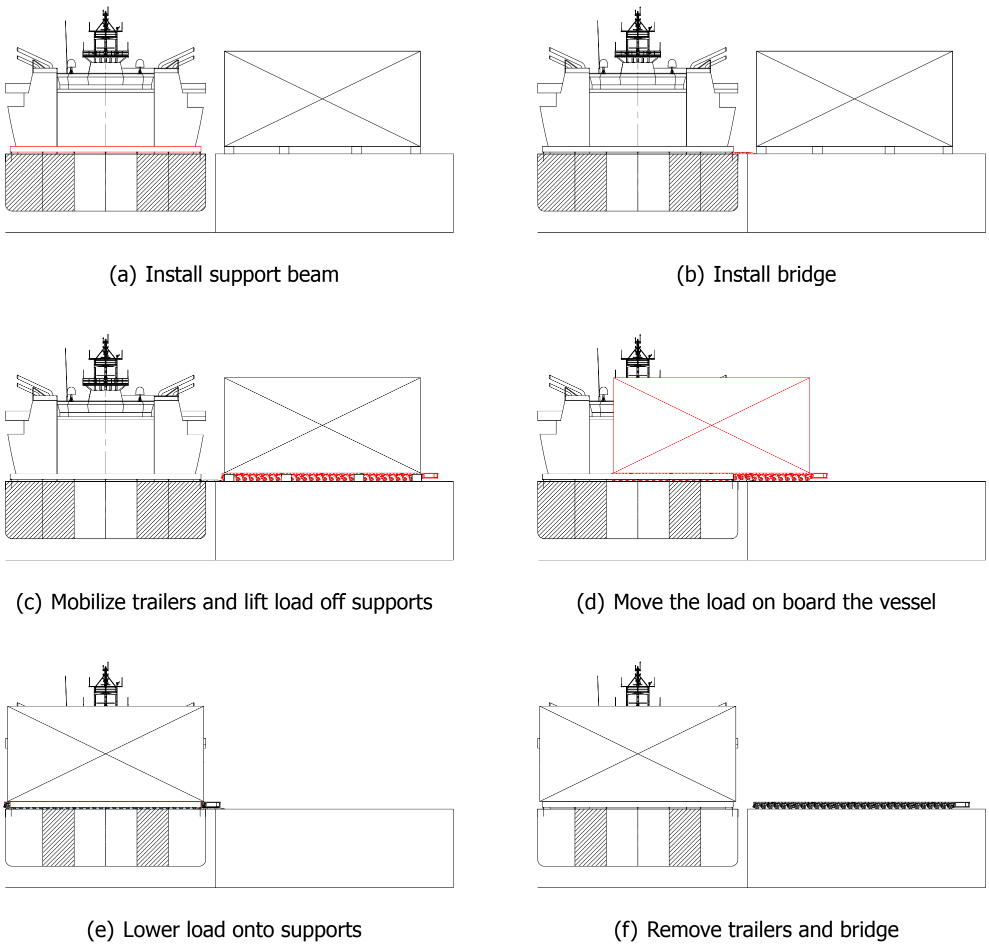
The MCs are designed to transport ultra large and heavy module cargoes by sea for the oil and gas industry, renewables market, power generation, container cranes and shipyard industries. The MC will have Finnish Swedish 1A Ice Class and optional DP2 notations. The Ice Class notation enables it to serve remote areas such as the arctic, where other heavy lift vessel cannot sail. The notation DP is short for dynamic positioning, this will enable direct offshore delivery of modules [11]. The vessel particulars are given in Table 3.1.

3.2. Loading methods

The MC is designed to be loaded via platform trailers or skidding. The equipment used for these methods is already discussed in Chapter 2. In this section, the process of loading via these two methods will be presented. Figure 3.2 illustrates the simplified process of loading via platform trailers. Figure 3.3 gives the simplified process of loading via skidding. Although the representations are simplified, all essential steps in the process of loading are illustrated.

Table 3.1: Particulars of the MC [11]

Property	Sub-property	Value
Dimensions	Length over all	173m
	Beam	42m
	Draft	5.5m
Deck dimensions	Length	125m
	Width	42m
Loading capacity	Transverse	15000t
	Longitudinal	10000t
Ballast capacity		12000m^3/hr
Service speed		13 knots



(a) Install support beam

(b) Install bridge

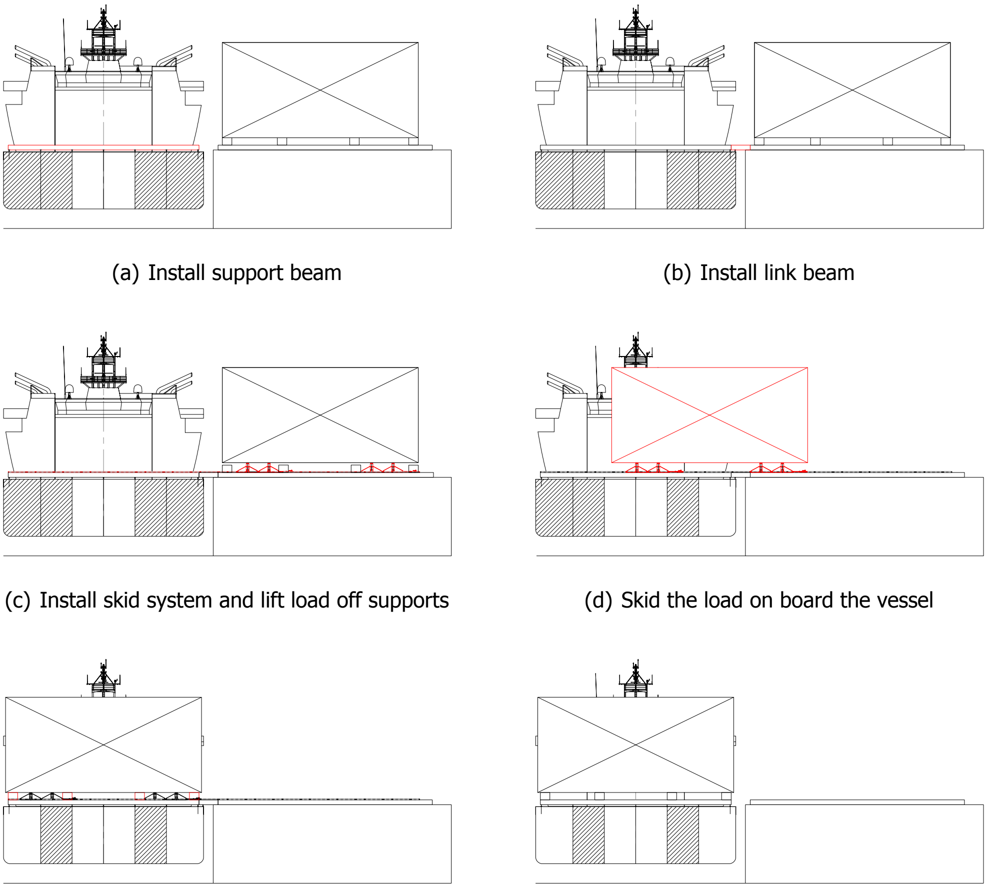
(c) Mobilize trailers and lift load off supports

(d) Move the load on board the vessel

(e) Lower load onto supports

(f) Remove trailers and bridge

Figure 3.2: Simplified process of loading using platform trailers, the hatched areas indicate the basic ballast procedure to maintain stability



(a) Install support beam

(b) Install link beam

(c) Install skid system and lift load off supports

(d) Skid the load on board the vessel

(e) Lower load onto supports

(f) Remove skid system and link beam

Figure 3.3: Simplified process of loading using a skid system, the hatched areas indicate the basic ballast procedure to maintain stability

3.3. Stability of the vessel

To prevent capsizing of the vessel, stability is a major issue. In general, three stability states exist:

- Stable: After a disturbance, the system returns to its original equilibrium state
- Indifferent: After a disturbance, the system assumes a new equilibrium state
- Unstable: After a disturbance, the system will move further and further away from its original equilibrium state

For a vessel, only the first stability state prevents the vessel from cape sizing. Stability of a vessel is closely related to the height and position of the combined Center of Gravity (COG) of vessel and cargo. This COG can be influenced by filling or emptying the ballast tanks. During loading and unloading, ballasting is performed to compensate for the load being moved on or off the vessel and tide as well. As mentioned in Section 3.1, the MC has a ballast capacity of 12000m^3/hr, which is assumed to be sufficient to maintain stability during (un)loading operations. In Figure 3.2 and 3.3 the basic ballasting procedure to compensate for a load moved onto the MC is illustrated. In practice, the combination of transferring the load onto the vessel and compensation with ballasting is performed step by step according to a predetermined ballasting plan.

3.4. Deck details

The deck surface of the MC can support 20t/m^2 anywhere on deck. However loads that can be transferred directly into the deck support structure may be much higher and with proper supports on deck the maximum line loads as shown in Figure 3.4 are allowable. Detailed deck calculations can be found in Appendix C.

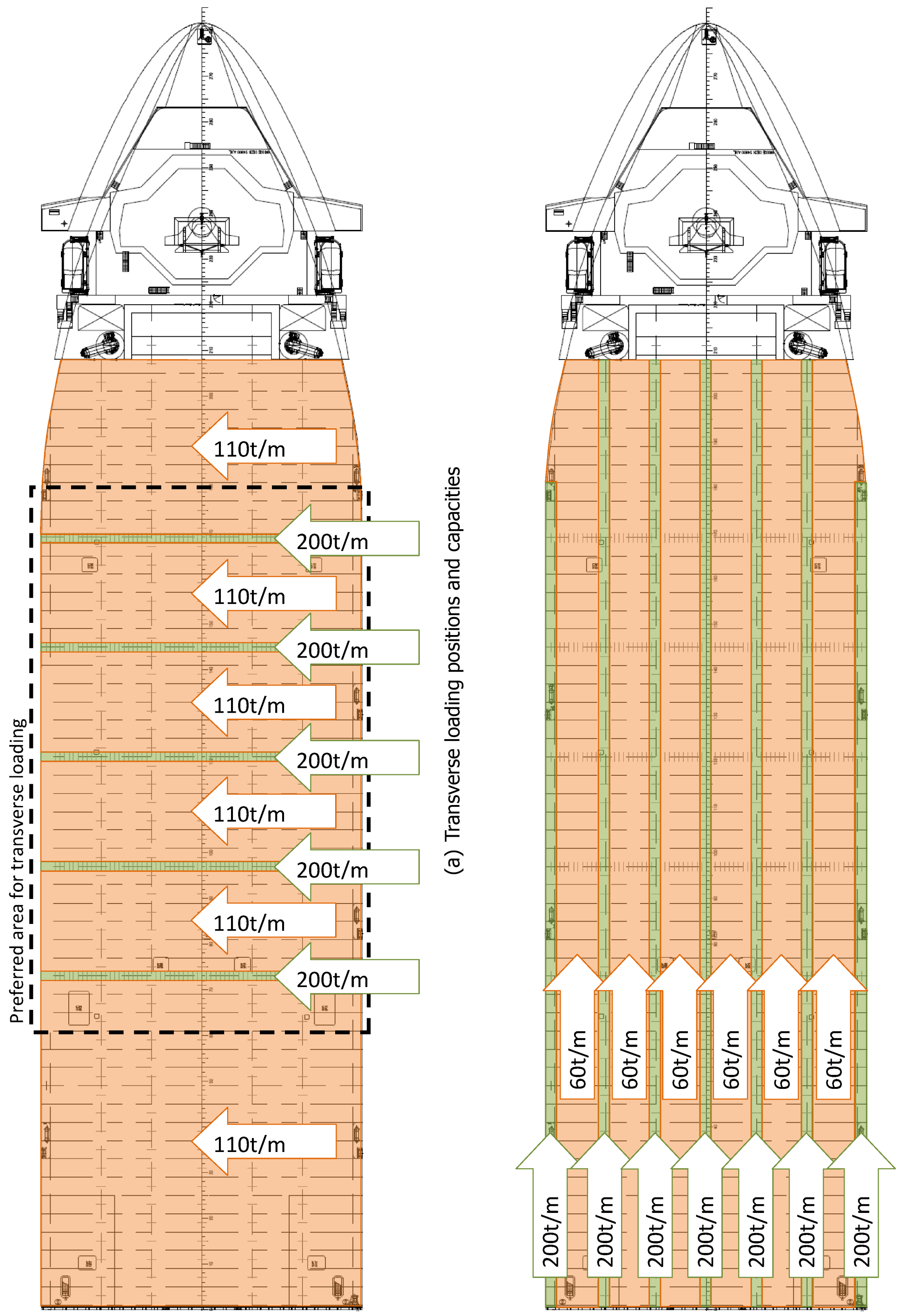


Figure 3.4: Loading positions and capacities of the MC, in the green areas no supports are required, in the orange areas appropriate supports are required for load spreading

PART II

Concept development

4
Design criteria

The design of the beam is based on several requirements and assumptions regarding the equipment used, RollDock and Roll-Lift and the applicable standards. This chapter will give an overview of requirements that are applicable for the concept design of the modular beam. Section 4.1 gives an overview of requirements and preferences of RollDock and Roll-Lift. In Section 4.2 a short description per function will be presented, along with equipment specifications and resulting load cases. In Section 4.3, the choice for applicable design codes will be substantiated and the requirements that follow from the applicable standards will be presented.

4.1. Requirements of RollDock and Roll-Lift

RollDock and Roll-Lift have requested the design of a modular, re-usable, multi-functional beam for heavy lifting. They have formulated a set of operational requirements to simplify implementation in practice.

- For easy transport, the beam must be able to be transported either in standardized shipping containers or in the form of standardized shipping containers. The latter means that the beam in transport form meets the minimum requirements to be qualified as container, i.e. the dimensions, mass and presence of corner castings, as defined by the International Organisation for Standardization (ISO) [12], see Table 4.1.
- Installation of the beam is performed with a crane, therefore the beam must be able to be reached from the top at any time.
- To be flexible in loads on the MC, the beam must be able to be located in any position on deck, both in transverse and longitudinal direction. This gives requirements for the unsupported length (l_{rep}) and the total length of the beam. For a visual representation of the supports on different positions on deck of the MC, see Figure 4.1.
- During installation of the beam the vessel stays in the harbor, which is quite expensive. Therefore the installation and removal must take as little time as possible.
- To achieve short installation and removal times and reduce the complexity of the beam system, the number of parts of the beam must be minimal.
- The beam is always used in outside environments, which means that the beam is exposed to all weather influences. Therefore the beam must be a closed structure and conserved properly to prevent corrosion.
- For securing the beam or to attach structures to the beam, no welding is allowed. For those purposes, special interfaces are required.

Table 4.1: Specifications of common container types [13]

		Unit	20ft	40ft	40ft high cube	40ft open top	40ft flat rack
Outer dimensions	Length	m	6.096	12.192	12.192	12.192	12.192
	Width	m	2.438	2.438	2.438	2.438	2.438
	Height	m	2.591	2.591	2.895	2.591	2.591
Inner dimensions	Length	m	5.888	12.030	12.030	12.022	12.082
	Width	m	2.350	2.352	2.350	2.346	2.200
	Height	m	2.390	2.383	2.688	2.350	2.002
Max. total weight		kg	30480	30480	30480	30480	45000
Max. load weight		kg	28095	26580	26330	26480	38750

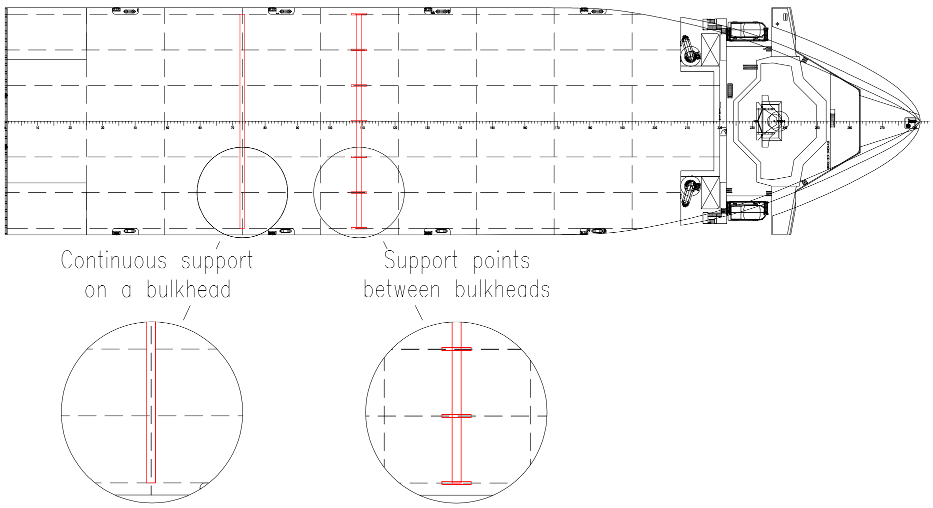


Figure 4.1: Difference in support arrangement and thus unsupported beam length between position on a bulkhead (left detail) and not on a bulkhead (right detail)

- The beam must be constructed of S355 steel, where 'S' denotes structural steel and 355 denotes the yield strength (σ_y) in N/mm^2. The reason for this choice of material is that it offers a reasonable strength to weight ratio, while still being easy weldable. Welding may be required on site when the beam needs repair works.

4.2. Requirements based on functions

This section describes the requirements imposed by the different functions of the beam. Each function comes with equipment that poses demands on load resistance and dimensions of the beam.

4.2.1. Requirements due to equipment

Due to the equipment used for the functions as presented in Chapter 2, the following requirements on the beam design exist:

- The beam must have a completely flat top and bottom, to be put on deck without supports and to accommodate skid systems on top.
- The minimum width of the beam is determined from the dimensions of the skid systems. Due to the flexibility in skid systems on top of the beam, the width of the beam must be sufficient to accommodate all skid systems as presented in Table 2.1. This results in a width of 880mm.
- The height of the beam is determined using the function grillage. For trailers to be able to move a load over the grillage and put it down, the height of the grillage must be 1200mm to 1400mm

4.2.2. Load cases

A schematic overview of the load cases due to the beam functions is presented in in Figure 4.2. The maximum bending moment and shear force due to these load cases are then calculated in Table 4.3. The function skid support beam is represented in load case 1 to 4. Load case 5 to 7 represent the loads due to the function grillage. Load case 7 also represents the load on the beam due to the function vertical spacer. The gantry girder and foundation are represented in load case 8 to 10 and 11 respectively. Load case 12 finally gives the loads on the beam due to the function temporary bridge.

4.3. Limits according to design codes

The beam design has to comply with design codes in order to be accepted for use in practice. Although design codes are also known as standards, no international accepted design codes exist. Two design codes for steel construction, which together cover a substantial part of the world, are the Eurocodes and the American Institute of Steel Construction (AISC) design codes. The Eurocodes are generally accepted throughout Europe, where Eurocode 3 (EC3) forms the basis for steel construction. The AISC design codes are accepted in North America. However, design codes for steel construction in the rest of the world are often derived from either one of these design codes [14]. EC3 and AISC are both based on limit state design. This means that a structure that is designed according to this principle will sustain all actions likely to occur during its lifetime with an appropriate level of reliability for each limit state. EC3 and AISC are comparable in reliability approaches and strength criteria, but they vary in serviceability. In this matter, EC3 offers more accurate, more detailed criteria [14–16]. Therefore Eurocodes will be used as design code in this research for the concept design of a modular heavy lift beam. EC3 describes the codes on design of steel structures, which will be used as a basis for design. EC3 refers to Eurocode 0 (EC0) for basics in design and to Eurocode 1 (EC1) for the actions on structures. Fatigue will not be considered in the design of the beam, due to the limited amount of load cycles during its service life according to the criteria of EC3. Buckling will be assessed to the extent necessary according to the classification of the cross section of the beam and accompanying criteria, as expressed in EC3.

4.3.1. Criteria following from Eurocode 3

In Table 4.2, the most important design values due to application of EC3 are presented. An in-depth investigation into EC3 is given in Appendix D.

Table 4.2: Most important design values resulting from EC3

Criterion		Value	Unit
Safety factors on loads	Permanent (γ_G)	1.35	-
	Variable (γ_Q)	1.5	-
Allowable stress	σ_{allow}	σ_y	N/mm^2
	τ_{allow}	$0.58\sigma_y$	N/mm^2
Maximum deflection	δ_{max}	$1/150 l_{rep}$	mm
Maximum strain	ϵ	0.002	-

4.3.2. Load combinations

The EC3 not only describes the design limits of steel structures, but it also refers to EC0 where the load combinations that should be applied for a thorough structural analysis are stated. The load combinations are generally formed by a combination of self weight, a load case, as described in Section 4.2, wind and other external loads, multiplied with appropriate safety factors.
The load combinations rules that will be considered for each load case are described in this section. EC0 describes two limit states:

- Ultimate Limit States (ULS): limit states that affect the safety of people or the structure
- Serviceability Limit States (SLS): limit states that affect the appearance or effectiveness of the structure

For each load case, the following load combinations are considered:

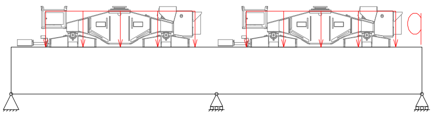
- Characteristic combination, Formula (4.1)
- ULS EQU: Loss of static equilibrium of the structure, Formula (4.2)
- ULS STR: Internal failure or excessive deformation of the structure, Formula (4.3)
- SLS: Characteristic combination, Formula (4.4)

$$CO_{k,1} = G_z" + "Q_{load,k}" + "Q_{wind,k} \quad (4.1)$$
$$CO_{k,2} = \gamma_G G_z" + "\gamma_Q Q_{load,k}" + "\gamma_Q Q_{wind,k} \quad (4.2)$$
$$CO_{k,3} = \xi \gamma_G G_z" + "\gamma_Q Q_{load,k}" + "\gamma_Q \psi_0 Q_{wind,k} \quad (4.3)$$
$$CO_{k,4} = G_z" + "Q_{load,k}" + "\psi_0 Q_{wind,k} \quad (4.4)$$
$$Q_{wind} = 0.10 \times Q_{load} \quad (4.5)$$

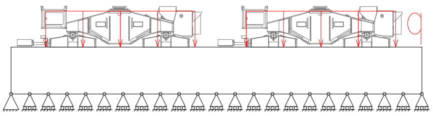
Where $CO_{k,i}$ = Load combination i of load case k
γ_G = Partial safety factor for permanent actions
G_z = Self weight
γ_Q = Partial safety factor for variable actions
Q_{load} = Value of the leading variable action
Q_{wind} = Value of the variable action due to wind
ξ = Reduction factor (=0.85)
ψ_0 = Factor for combination value of a variable action (=1.0)
"+" means *combined with*

Table 4.3: Bending moments and shear force due to load cases

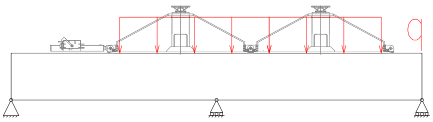
Load case		l_{rep} **[m]**	F **[kN]**	q **[kN/m]**	M **[kNm]**	V **[kN]**
1	Enerpac skid (1)	6.6		1315	5771	3896
2	Enerpac skid (2)	0.6		1315	99	395
3	Mammoet skid (1)	6.6		1305	5740	3871
4	Mammoet skid (2)	0.6		1305	98	392
5	Grillage (1)	6.6	5886		9712	2943
6	Grillage (2)	6.6	5886		0	5886
7	Grillage (3)	0.6	5886		2453	2943
8	Gantry girder (1)	11.4	2943		8388	1472
9	Gantry girder (2)	11.4	2943		5886	2943
10	Gantry girder (3)	22.8	2943		5886	2943
11	Gantry foundation	0	1472		0	1472
12	Temporary bridge	22.8		140	9106	1598



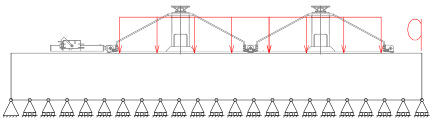
(a) Load case 1: Distributed load due to Enerpac HSK6000 skid shoe



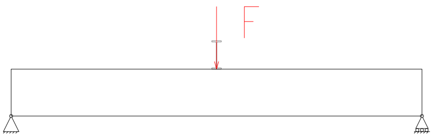
(b) Load case 2: Distributed load due to Enerpac HSK6000 skid shoe on beam above bulkhead



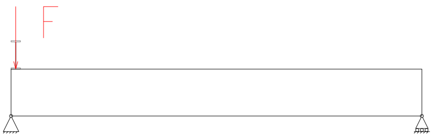
(c) Load case 3: Distributed load due to Mammoet skid shoe



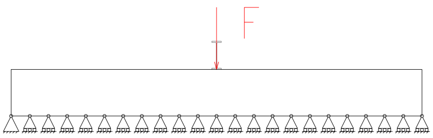
(d) Load case 4: Distributed load due to Mammoet skid shoe on beam above bulkhead



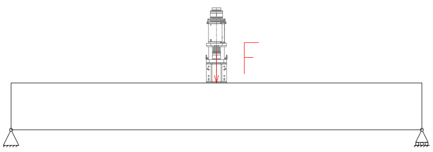
(e) Load case 5: Concentrated load due to load on grillage



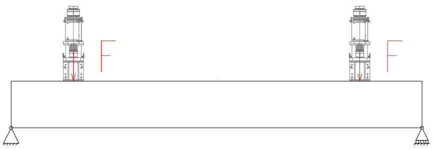
(f) Load case 6: Concentrated load due to load on grillage



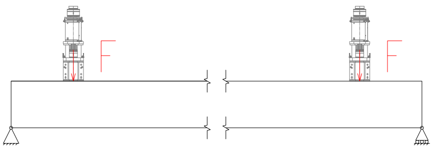
(g) Load case 7: Concentrated load due to load on grillage above bulkhead or load on a spacer on a platform trailer



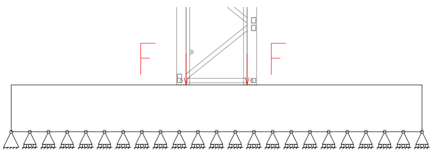
(h) Load case 8: Concentrated load due to gantry girder loaded in the center



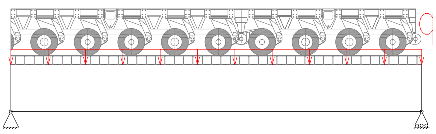
(i) Load case 9: Concentrated loads due to gantry girder loaded near the ends of one beam section



(j) Load case 10: Concentrated loads due to gantry girder loaded near the ends of two beam sections



(k) Load case 11: Concentrated loads due to the legs of a gantry tower



(l) Load case 12: Distributed load due to platform trailer on temporary bridge

Figure 4.2: Graphical overview of load cases

5
Basic design

In this chapter, the concept design of the modular heavy lift beam will be developed. In order to split the problem into several smaller problems, the beam system is decomposed into sub-systems that can be analyzed individually in Section 5.1. These sub-systems are the beam shape, beam section length, connections between sections and the auxiliary components that are used for the different functions, as presented in Section 5.2 to 5.5. The auxiliary components are for skidding the supports on deck of the MC and the link beam and for the gantry a T-section to connect the two parallel gantry girders and/or transverse beam sections. Finally, the sub-systems will be combined to a total concept in Section 5.6.

5.1. Decomposition of the concept into sub-systems

For the design of the total modular beam, the concept will first be decomposed in the following sub-concepts (see also Figure 5.1):

- Shape of the cross section of the beam
- Length of the beam sections
- Connections between the sections
- Auxiliary components used for the different functions

In this section, each sub-concept will be worked out based on the requirements as presented in Chapter 4. For each sub-concept, the options resulting from a brainstorm will be presented. Next, the relevant criteria for the choice of the best option will be mentioned. Finally the best option per sub-concept resulting from these criteria will be presented. For an elaboration of the choice for the best option per sub-concept, see Appendix E. At the end of this chapter, the total concept will be composed from the results of the sub-concepts.

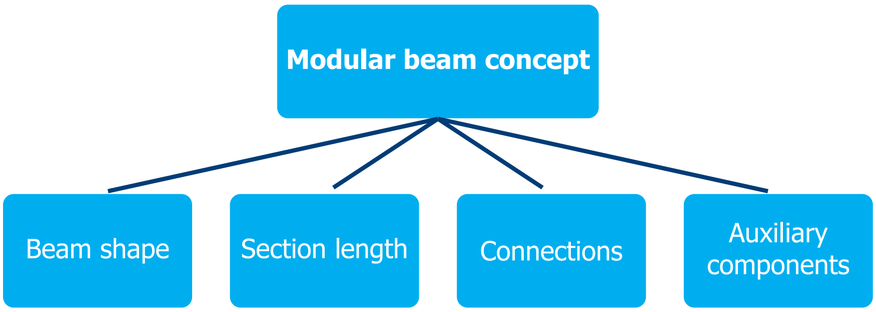
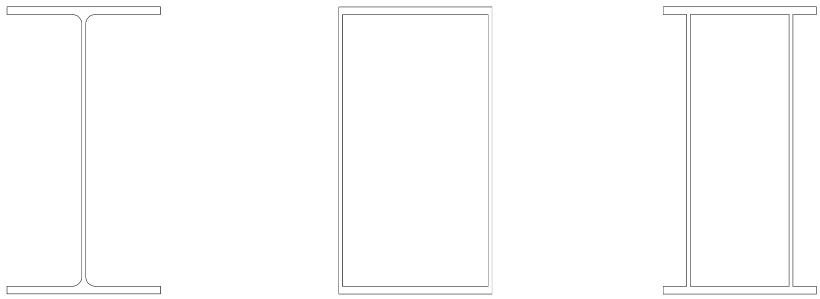


Figure 5.1: Decomposition of the beam concept into sub-concepts

5.2. Determination of the beam shape

5.2.1. Options for the beam shape

Several options for the shape of the beam are considered, without looking at the required plate thicknesses yet. The options are shown in Figure 5.2. The options considered are a standard H-profile and two box girder shapes constructed from steel plates.



(a) H-beam (b) Box beam (c) Box beam with offset of webs

Figure 5.2: Cross sections of several options for the shape of the modular beam

5.2.2. Choice of beam shape

For the selection of the best beam shape, the beam is assessed on criteria for outer dimensions, resistance to loads, stability, resistance to weather influences and if its fit for all functions. However, the decisive criterion, which is non contradictory to the other criteria, for the determination of the beam shape is the local bending moment in the flanges, for the reason that a high bending moment needs a lot of material to resist that moment. An elaborate explanation is given in Appendix E. Local bending moments in the flanges are created due to an offset of the point of impact of the loads of the functions, especially skidding (see Figure 5.3). For the other functions, steel structures or contact plates are used to spread the load over the width of the beam. This results in the local bending moments shown in Figure 5.4. This figure clearly shows that in both cases of an H-beam and a box beam, with web offsets of 0mm and 440mm respectively, the highest bending moments occur. This results in a box beam shape with an offset of the flanges. The value of the offset is determined using the criterion on height (between 1200mm and 1400mm) and a width-to-height ratio of 1:3 for the enclosed part of the beam, which yields a strong but relatively light structure [17]. This yield an offset of the webs between 200mm and 230mm from centerline, which corresponds to the black dashed lines in Figure 5.4. An offset of 220mm is chosen for the design, because the moment due to the Enerpac skid system is zero. With the width-to-height ratio, this results in a beam height $H_{beam} = 1320mm$.

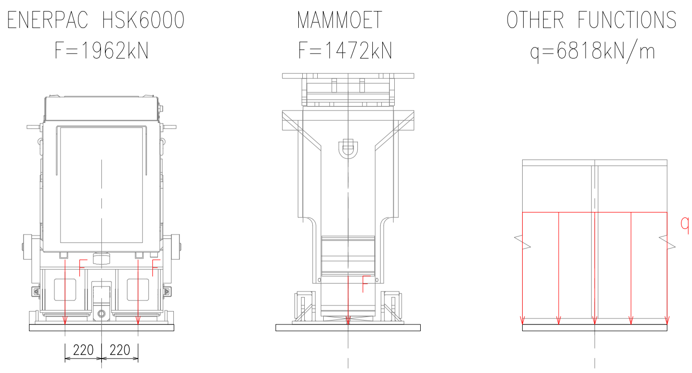


Figure 5.3: Loads introduced on the top flange of the beam

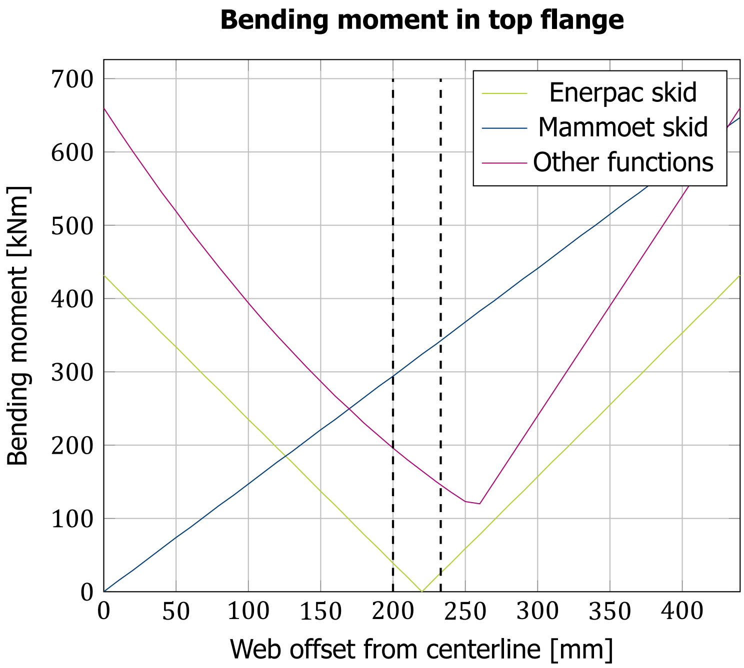


Figure 5.4: Bending moments in the top flange of the beam due the different skid systems and other functions as function of the offset of the webs. Offset 0mm corresponds to an H-beam (Figure 5.2(a)) and offset 440mm corresponds to a box beam (Figure 5.2(b)). Everything in between corresponds to a box beam with offset webs (Figure 5.2(c))

5.3. Determination of the section length

5.3.1. Options for the section length

The deck of the MC is chosen as the critical variable for the determination of the section length, together with the maximum dimensions of shipping containers. Within the boundaries resulting from these variables, four combinations of beam lengths are considered (see also Figure 5.5):

1. Only sections of 9900mm long
2. Only sections of 10200mm long
3. A combination of sections 11400mm long and half sections of 5700mm
4. A combination of sections 11400mm long and 'half' sections of 5400mm

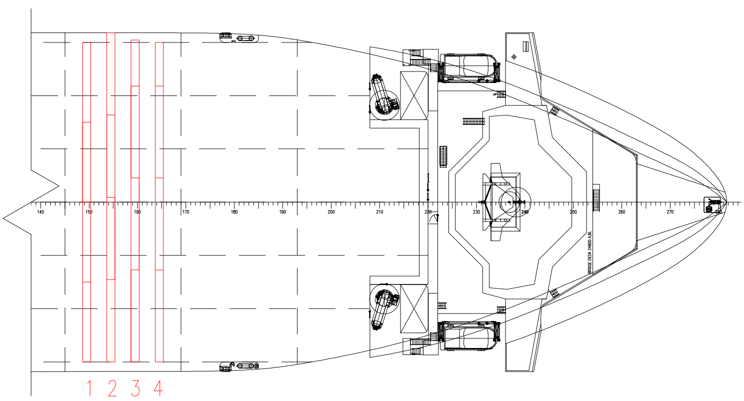


Figure 5.5: Fit of the section length options on the deck of the MC: Sections of 9900mm (No.1), 10200mm (No.2), 11400mm+5700mm (No.3) and 11400mm+5400mm (No.4)

5.3.2. Choice of section length

For the selection of the section length, *Pugh's method* [18] will be used. This is an effective method for comparing alternatives relative to each other in their ability to meet the applicable criteria. At the start a reference concept is chosen to be used as datum, the other concepts are scored compared to this datum. After a scoring round, the best concept is chosen as datum in the next round until all other alternatives return a negative total score. The scoring options are presented in Table 5.1. In this section, only the last iteration is shown in Table 5.2. The combination of section lengths of 5400mm and 11400mm is the best relative to the alternatives, according to *Pugh's method*. These lengths will be used in the further design stages of the modular beam. For the complete selection process of the section length and the elaboration on the scores given, see Appendix E.

Table 5.1: Scoring options for Pugh's method

Score	**Meaning**
-1	Worse than datum
0	Same as datum
1	Better than datum

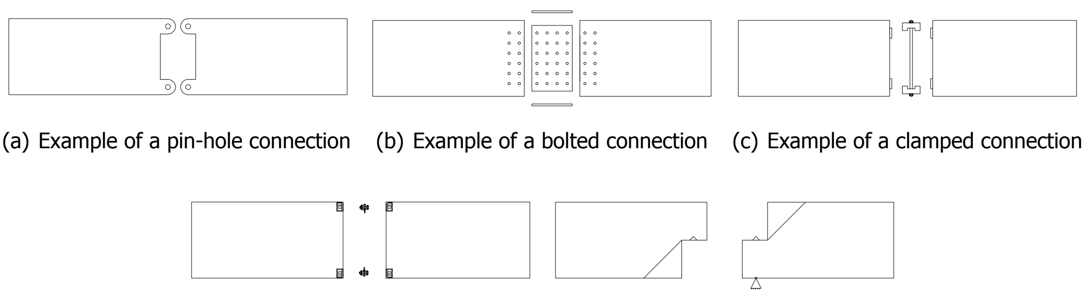
Table 5.2: Pugh's method for the selection of the section length, last iteration: datum is '11400+5400'

Criterion	**9900** Fig. 5.5 No.1	**10200** Fig. 5.5 No.2	**11400+5700** Fig. 5.5 No.3	**11400+5400** Fig. 5.5 No.4
Transverse fit on deck	0	-1	-1	0
Multiples of 600mm	-1	0	-1	0
Maximized within container	-1	-1	0	0
Flexibility in beam length	-1	-1	0	0
TOTAL	-3	-3	-2	0

5.4. Design of the connections between beam sections

5.4.1. Options for connections between beam sections

A large number of options for the connection between beam sections is are reviewed. Among the options are several implementations of a pin-hole connection, bolted connections, connections using fasteners or clamps, container twist locks and shape fit connections. A selection of these options is visualized in Figure 5.6.



(a) Example of a pin-hole connection (b) Example of a bolted connection (c) Example of a clamped connection

(d) Container twist lock connection (e) Example of a shape fit connection

Figure 5.6: Options for connecting the beam sections

5.4.2. Choice of connection between beam sections

For the selection of the best connection type, the same method will be used as for the selection of the section length: *Pugh's method*. In this section, only the last iteration is shown in Table 5.3. The pin-hole connection is the best relative to the alternatives, according to *Pugh's method*. This connection type will be used in the further design stages of the modular beam. The final execution of this connection type will be determined in a later stadium. For the complete selection process of the section length and the elaboration on the scores given, see Appendix E.

Table 5.3: Pugh's method for the selection of the connection type, last iteration: datum is 'pin-hole'

Criterion	**Pin-hole** Fig. 5.6(a)	**Bolts** Fig. 5.6(b)	**Clamps** Fig. 5.6(c)	**Twist locks** Fig. 5.6(d)	**Shape fit** Fig. 5.6(e)
Low number of parts	0	-1	0	0	1
Easy installation	0	-1	-1	0	0
Flat top/bottom	0	-1	0	0	0
Independent of supports	0	0	0	0	-1
Resistance to bending	0	0	0	-1	-1
Resistance to shear	0	0	0	0	0
TOTAL	0	-3	-1	-1	-1

5.5. Auxiliary components for the modular beam

The auxiliary components are additions or attachments to the modular beam, that aid the beam in its ability to perform its functions. The auxiliary components described in this section are supports on deck of the MC, the link beam, T-connectors for connecting two beams as gantry girder. For the attachments, it is an advantage that the connection to the beam sections is equally strong as the beam itself, both in positive and negative direction. If the attachments are also designed to the capacity of the beam, the assembly can be regarded as one beam instead of separate components.

5.5.1. Supports on deck of the MC

Supports on deck are required for load spreading if the beam is placed on deck between bulkheads. In that case, the beam is supported by the bulkheads crossing the line of the beam. On those support locations, high loads must be transferred into the deck structure. By providing extra height between the beam and the deck, the support length can be increased by twice this height, because of a load spread angle of 45 degrees, see Figure 5.7 [15]. The load spreading demands depend on the load and the setup of the beam, but a simple HEB profile is sufficient. For the worst load case, 693t needs to be transferred into the deck per support point. By using a HEB1000 profile, the support length becomes $880mm + 2 \times 1000mm = 2880mm$, which gives a capacity of $2.88m \times 250t/m = 720t$ per support point. If the beam is placed on top of a bulkhead, no supports are required.

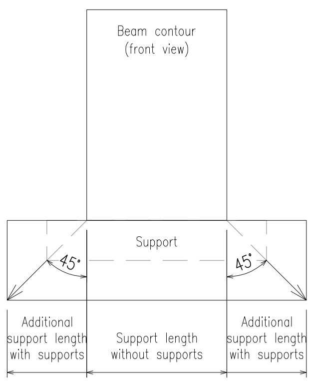


Figure 5.7: Influence of the height of beam supports to the support length

5.5.2. Link beam

The link beam is essential for loading the MC via skidding. It forms the bridge between quay and vessel and creates rotational freedom in the beam to deal with slight height variations between quay and vessel. By providing hinged attachments to both sides of a beam section, an universal link beam is created that is fit for all skid systems that are included in the design of the modular beam, as can be seen in Figure 5.8. For the connection of the link beam on deck, a space of 1200mm is reserved though the choice of beam section length. Assuming that the link beam is supported halfway of this range at 600mm and the same on the shore side, the unsupported length of the link beam becomes $5400mm + 2 \times 600mm = 6600mm$ which is already included in the load cases of the beam for skidding.

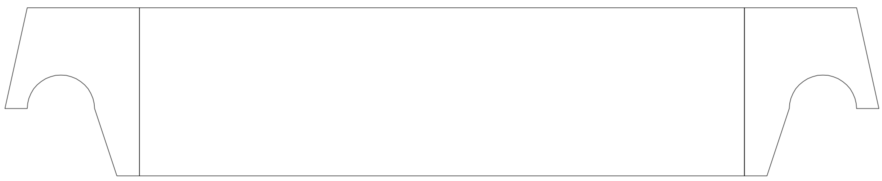


Figure 5.8: Attachments on both sides of a beam section to create a universal link beam (side view)

5.5.3. T-connectors

T-connectors, as shown in Figure 5.9, will be used for the functions gantry and skidding. In a gantry, the T-connector is used for two purposes. The first purpose is to create a rigid connection between to parallel beam sections to create the gantry girder with a gap in the middle, from where the hoist cables hang down, see Figure 5.10(c). Because this provides a rigid gantry girder, this girder can also be placed on top of a skid system to create a gantry with skidding girder, see Figure 5.10(d). The second purpose is to combine two or four T-connectors to form the basis for the gantry foundation, see Figure 5.10(c). Beam sections can be connected to this setup to provide more load spreading. As addition to the use in gantries, the T-connectors can be used to create a sturdy base for a temporary bridge, which provides load spreading as well, see Figure 5.10(e). By giving the T-connector the dimensions given in Figure 5.9, the spacing between parallel beams matches the spacing between trailer wheels, while still being suitable for a strand jack assembly ass well. The length of the base of the T is chosen such that two T-connectors connected by their bases have convenient dimensions for the gantry foundation.

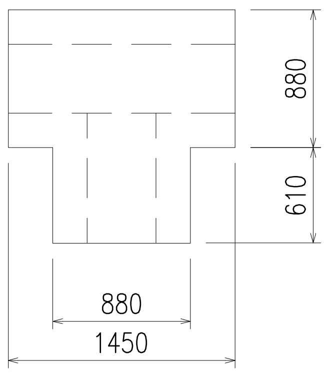
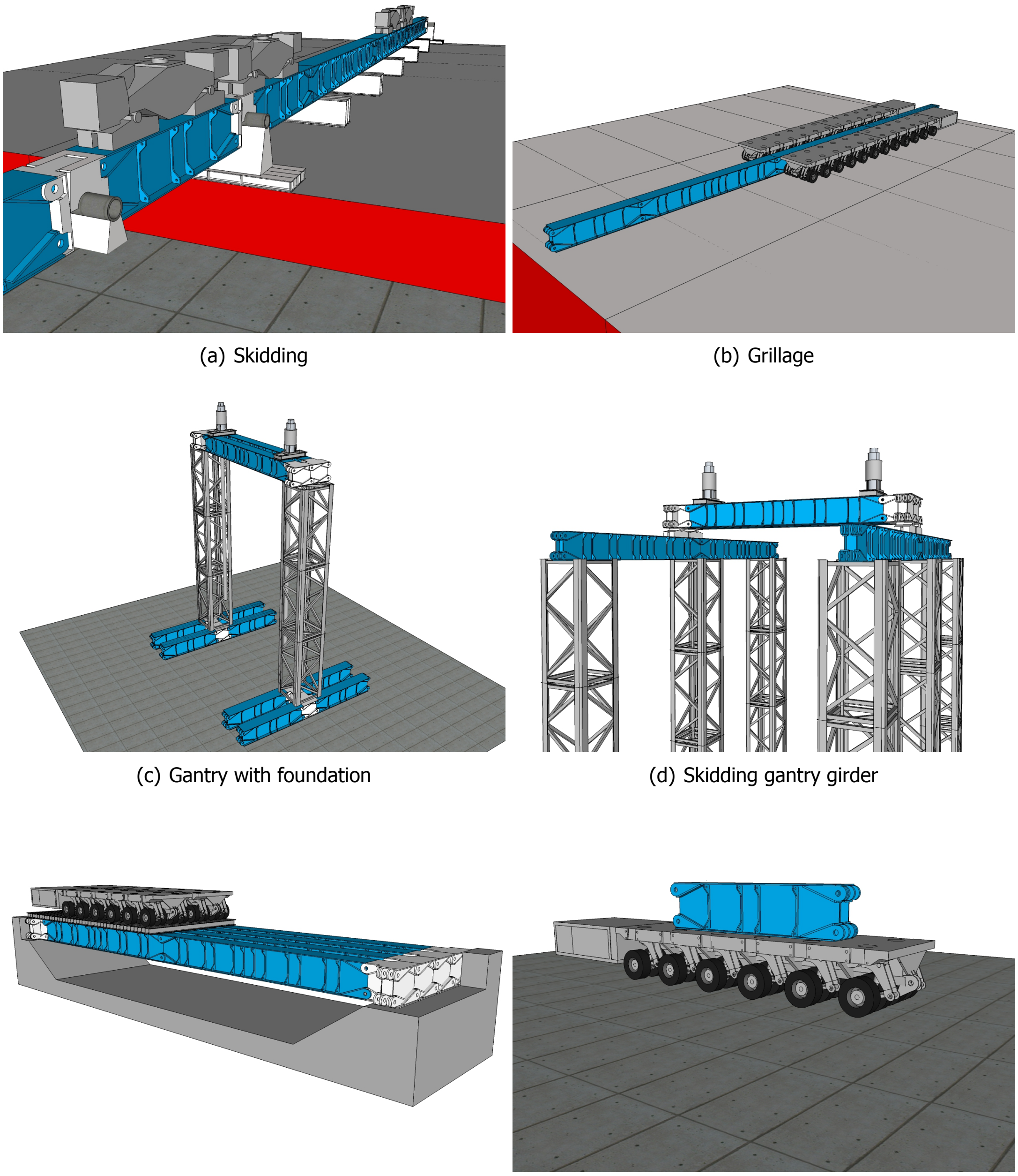


Figure 5.9: Dimensions of the T-connector for multi functional use (top view)



(a) Skidding

(b) Grillage

(c) Gantry with foundation

(d) Skidding gantry girder

(e) Temporary bridge

(f) Spacer on a trailer

Figure 5.10: Visualization of the beam functions and beam components used for these functions

5.6. Composition of total concept

In this section, the sub-systems will be combined to form the total modular beam concept, together with its auxiliary components, as is represented in Figure 5.11. The total modular beam concept will consist

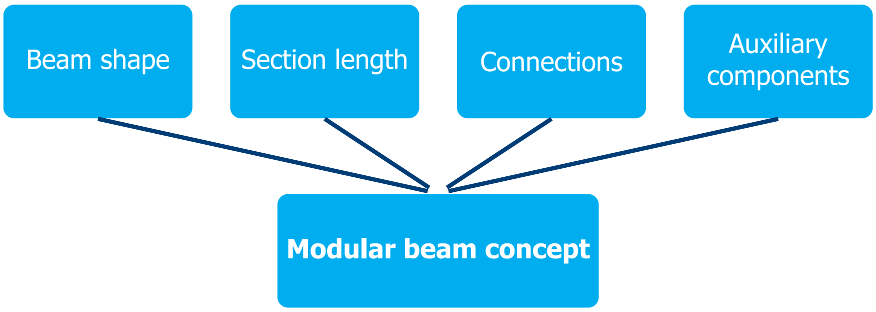


Figure 5.11: Composition of the beam concept out of the sub-concepts

of a box shaped beam with offset flanges, in sections of 11400mm and 5400mm length, connected by pin-hole connections. This can be seen in Figure 5.12. The auxiliary components are HEB-profiles for supports on deck of the MC, attachments to create a link beam of a 5400mm section and a T-connector used for various functions, see Figure 5.13. An overview of the use of the modular beam system in the functions is given in Figure 5.10.

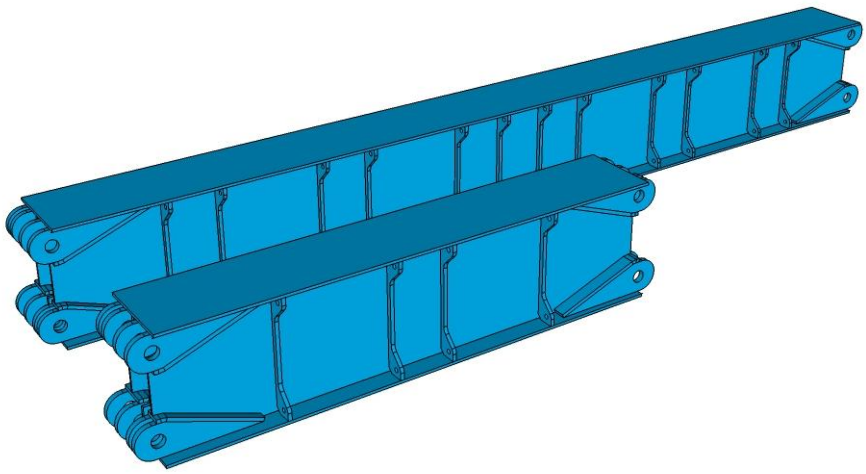


Figure 5.12: Overview of beam sections

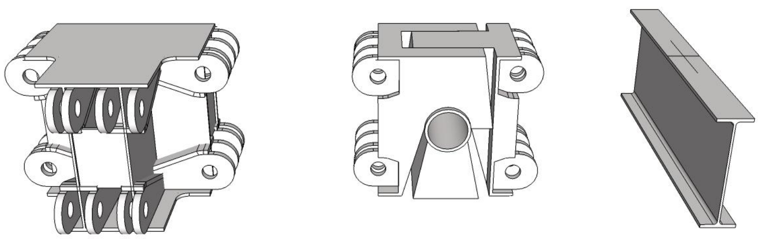


Figure 5.13: Overview of auxiliary components for the modular beam, from left to right: T-connector, link beam attachment, beam support

6
Structural design

In this chapter, the detailed geometry of the beam will be presented. The details can be found in Appendix F. For the structural design, the beam is decomposed into its main components: the flanges, webs, longitudinal stiffeners, transverse stiffeners, outstand stiffeners and the connections. The stiffeners are needed for local load introduction into the beam. In determining the dimensions of the several parts of the beam, the simplified design scheme shown in Figure 6.1 is used. This scheme gives an overview of the interactions between the design of the different components. It also gives an indication about the complexity of the design of the total beam. The combination rules for bending and shear resistance are shown in Formula (F.6) and (F.5) respectively.

6.1. Design loads

6.1.1. Global loads

The global design loads on the beam are the maximum values for bending and shear that follow from the load cases, as are shown in Table 4.3. This gives:

- Global bending: M = 9712kNm
- Global shear: V = 5886kN

This gives the following design loads:

- Design bending moment: M_{Ed} = 1.5×M = 14568 kNm
- Design shear force: V_{Ed} = 1.5×V = 8829 kN

6.1.2. Local loads

Local loads are introduced by the functions and due to the beam used as grillage directly on top of a bulkhead, in which the latter gives the highest values for required local bending and shear. This gives:

- Local bending: M' = 147kNm
- Local shear: V' = 736kN

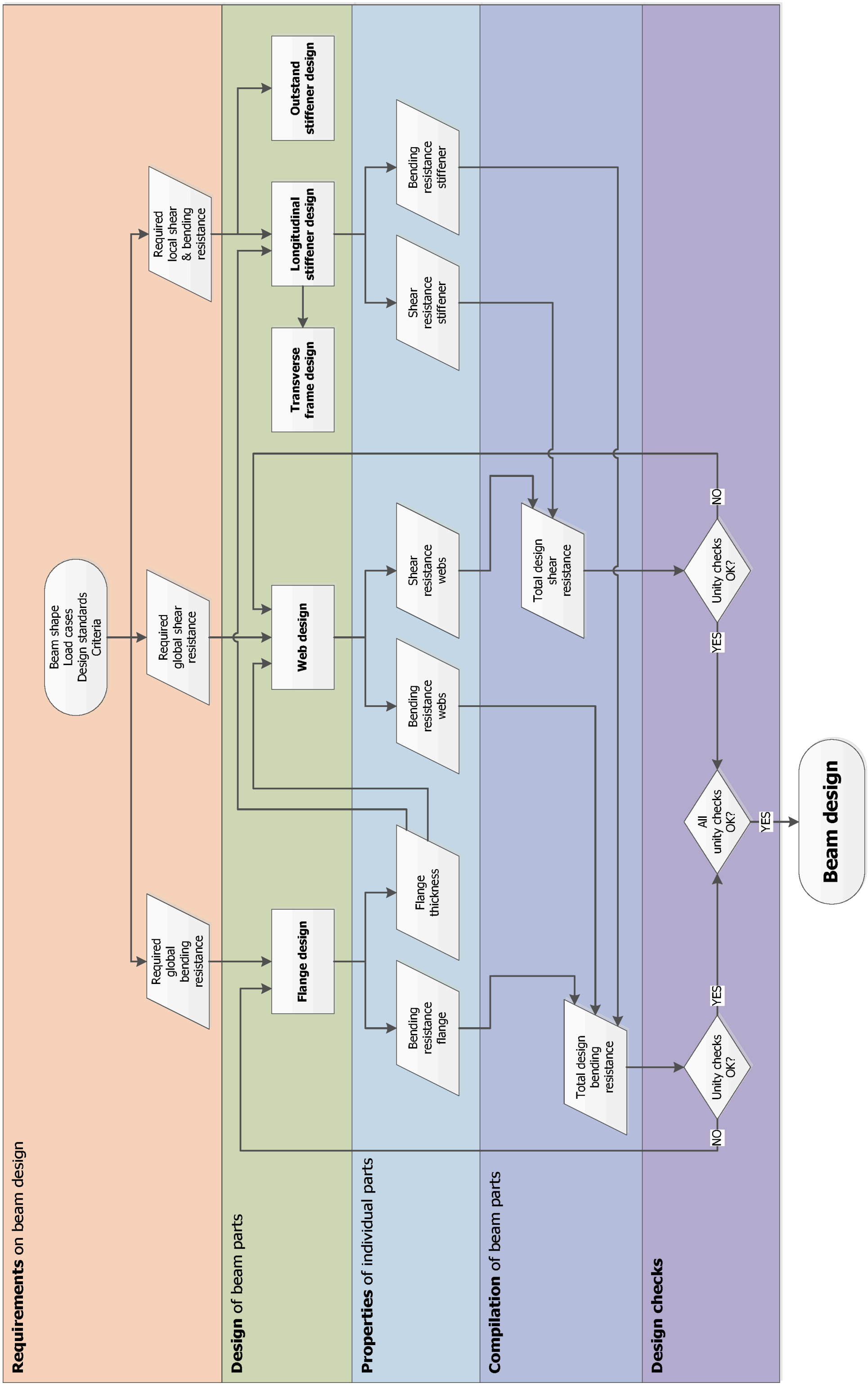


Figure 6.1: Simplified representation showing the influence of the design of parts on bending and shear resistance of the beam and the design of the other parts

6.2. Design of the components

6.2.1. Flanges

The flanges account for the global bending resistance together with the webs and longitudinal stiffeners. For local load introduction, the flanges also account for local bending resistance with an effective width around the stiffeners. Thus the design of the flanges is based on global bending requirements, as well as local bending requirements. The resulting flange dimensions are presented in Figure 6.2. The cross section properties are summarized in Table 6.1.

Table 6.1: Section properties of the flanges

Property	Value			Unit
Plate thickness	t_{flange}	=	35	mm
Plate width	w_{flange}	=	880	mm
Cross section area	$A_{flanges}$	=	61600	mm^2
Second moment of inertia	$I_{flanges}$	=	$2.52{\times}10^7$	mm^4
Location of neutral axis	$\overline{y}_{flanges}$	=	17.5	mm
Bending resistance	M_{Rd}	=	511	kNm

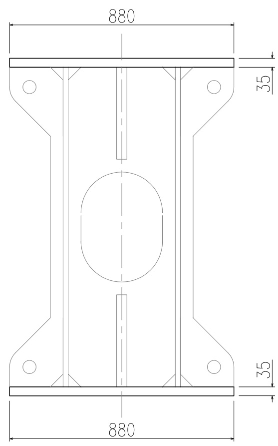


Figure 6.2: Dimensions of the flanges

6.2.2. Webs

The webs account for the global shear resistance together with the longitudinal stiffeners. The webs also add bending resistance together with the flanges and the longitudinal stiffeners. Thus the design of the webs is related to the global shear and bending requirements. The resulting web dimensions are presented in Figure 6.3. The cross section properties are summarized in Table 6.2.

Table 6.2: Section properties of the webs

Property	Value			Unit
Plate thickness	t_{web}	=	20	mm
Plate height	h_{web}	=	1250	mm
Area	A_{webs}	=	50000	mm^2
Shear area	$A_{v,webs}$	=	50000	mm^2
Second moment of inertia	I_{webs}	=	$6.51{\times}10^9$	mm^4
Location of neutral axis	$\overline{y}_{webs}$	=	660	mm
Bending resistance	M_{Rd}	=	3502	kNm
Shear resistance	V_{Rd}	=	10250	kN

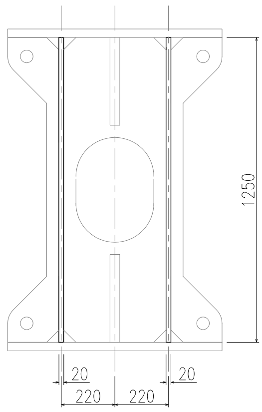


Figure 6.3: Dimensions of the webs

6.2.3. Longitudinal stiffeners

The longitudinal stiffeners account for the global shear resistance together with the webs. The longitudinal stiffeners also account for global bending resistance together with the webs and flanges. Moreover, these stiffeners also resist local bending and shear due to local load introduction into the flanges, where the effective width of the flanges adds local bending resistance to the longitudinal stiffener. The design of these stiffener thus depends on both global and local bending and shear requirements as well as dimensions of the flanges. The resulting dimensions are presented in Figure 6.4. The cross section properties are summarized in Table 6.3.

Table 6.3: Section properties of the longitudinal stiffeners

Property	**Value**			**Unit**
Plate thickness	t_{LS}	=	40	mm
Plate height	h_{LS}	=	360	mm
Area	A_{LS}	=	28800	mm^2
Shear area	$A_{v,LS}$	=	28800	mm^2
Second moment of inertia	I'	=	3.93×10^8	mm^4
	I_{LS}	=	3.11×10^8	mm^4
Location of neutral axis	$\overline{y}_{LS}$	=	215	mm
Bending resistance	M_{Rd}	=	514	kNm
Shear resistance	V_{Rd}	=	5904	kN

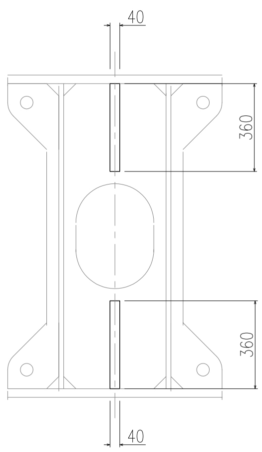


Figure 6.4: Dimensions of the longitudinal stiffeners

6.2.4. Transverse stiffeners

The transverse stiffeners transfer the loads introduced into the longitudinal stiffener to the webs. They also add to the resistance of support forces, which determines the locations of the transverse stiffeners over the length of the beam. Most locations are determined by intersections with bulkheads of the MC, see Figure 6.5. Two additional stiffeners are placed at the locations near the center of the beam to support a strand jack in the middle of a gantry girder. An overview of all transverse stiffeners and the spacing between them is given in Figure 6.6. These stiffeners do not add to the global resistances. The

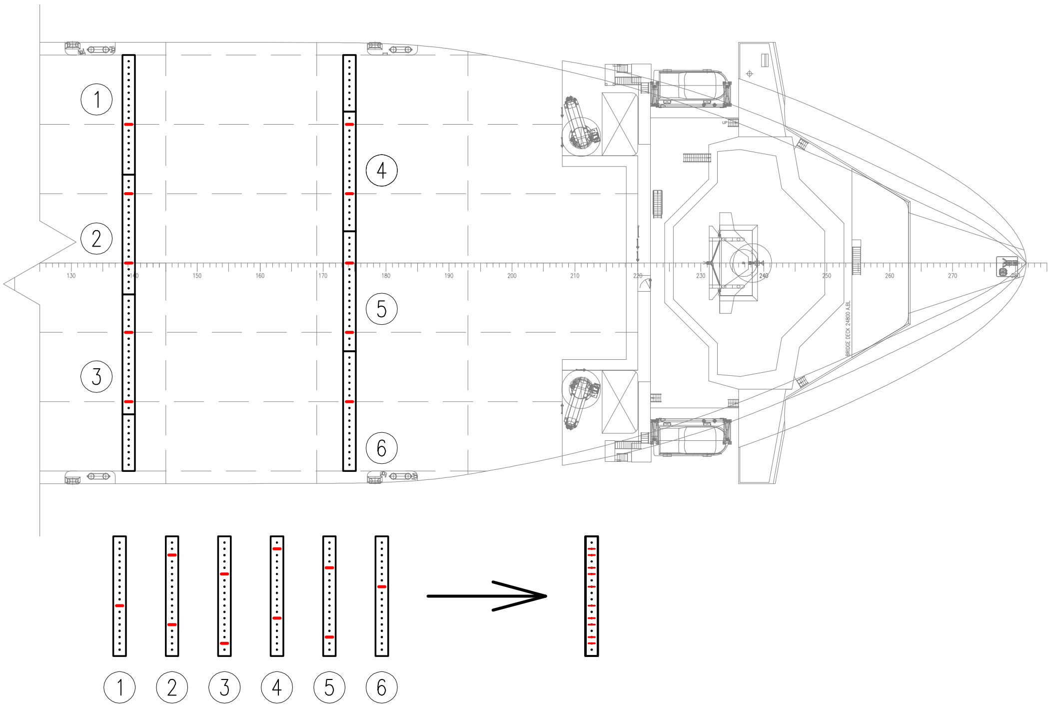


Figure 6.5: Determination of the locations of transverse stiffeners, based on intersections with bulkheads of the MC

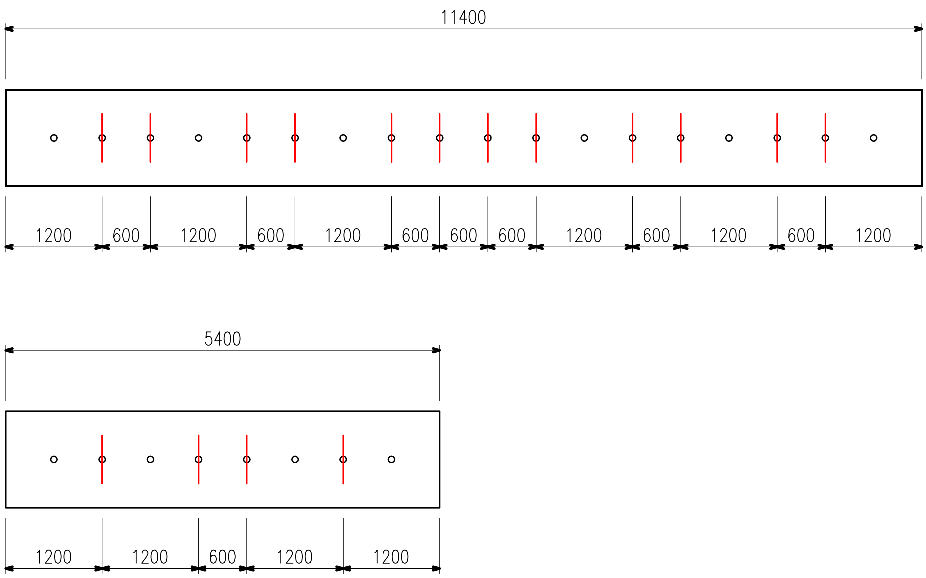


Figure 6.6: Details of the spacing between transverse stiffeners, the spacing in the 5400mm beam is based on the spacing in the 11400mm beam

dimensions of the transverse stiffeners depend on the loads introduced by the longitudinal stiffeners and the effective width and thickness of the flanges. The resulting dimensions are presented in Figure 6.7. The properties are summarized in Table 6.4.

Table 6.4: Section properties of the transverse stiffeners

Property	**Value**			**Unit**
Plate thickness	t_{TS}	=	30	mm
Plate height	h_{TS}	=	410	mm
Area	A_{TS}	=	12300	mm
Shear area (local)	$A_{v,TS}$	=	12300	mm^2
Second moment of inertia (local)	I_{TS}	=	3.18×10^8	mm^4
Location of neutral axis (local)	$\overline{y}_{LS}$	=	258	mm
Bending resistance (local)	M_{Rd}	=	438	kNm
Shear resistance (local)	V_{Rd}	=	1681	kN

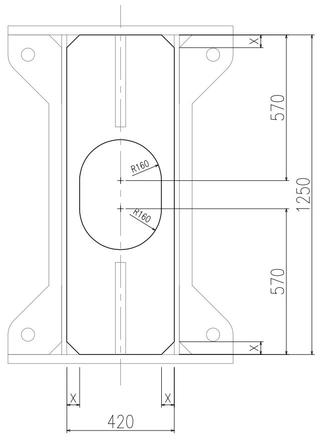


Figure 6.7: Dimensions of the transverse stiffeners

6.2.5. Outstand stiffeners

The outstand stiffeners are designed to support the outstand of the flanges and to create connection points on the beam. Locations of the outstand stiffeners over the length of the beam sections are the same as the transverse stiffeners, see Figure 6.6. The outstand stiffeners do not add to the global resistance. They only transfer local loads introduced on the outstand of the flanges to the webs. The dimensions of the outstand stiffeners are influenced by the effective width and thickness of the flanges. The resulting dimensions are presented in Figure 6.8. The properties are summarized in Table 6.5. A hole is made in each outstand stiffener for connecting lashing and lifting devices to the beam. Use is made of 25t shackles, which have a pin diameter of 50mm [19]. In this way, a assembly of two beam sections and two T-connectors can be lifted with two lifting points, which is a requirement of multi sling lifting operations [20]. The resulting hole geometry is shown in Figure 6.8. This leaves 100mm free length between the edge of the outstand stiffener and the inside of the shackle bow.

Table 6.5: Section properties of the outstand stiffeners

Property	Value			Unit
Plate thickness	t_{OS}	=	40	mm
Plate height	h_{OS}	=	270	mm
Area	A_{OS}	=	10800	mm
Shear area (local)	$A_{v,OS}$	=	10800	mm^2
Second moment of inertia (local)	I_{OS}	=	1.06×10^8	mm^4
Location of neutral axis (local)	$\overline{y}_{OS}$	=	141	mm
Bending resistance (local)	M_{Rd}	=	267	kNm
Shear resistance (local)	V_{Rd}	=	1476	kN

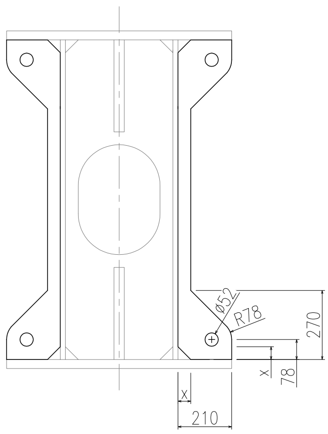


Figure 6.8: Dimensions of the outstand stiffeners

6.3. Total beam geometry

6.3.1. Total resistance

The combined properties of the components are presented in Table 6.6. The overall dimensions are shown in Figure 6.9.

Table 6.6: Properties of the assembled beam

Property	Value			Unit
Shear area	A_v	=	78800	mm^2
Second moment of inertia	I	=	3.80×10^{10}	mm^4
Distance to extreme fibre	y	=	660	mm
Bending resistance	M_{Rd}	=	20418	kNm
Shear resistance	V_{Rd}	=	10767	kN

6.3.2. Unity checks

The unity checks for the total geometry on global bending and shear are:

$$\frac{M_{Ed}}{M_{Rd}} = 0.71 \tag{6.1}$$

$$\frac{V_{Ed}}{V_{Rd}} = 0.82 \tag{6.2}$$

Both unity checks return a value smaller than 1, so the design resistance is high enough according to EC3.

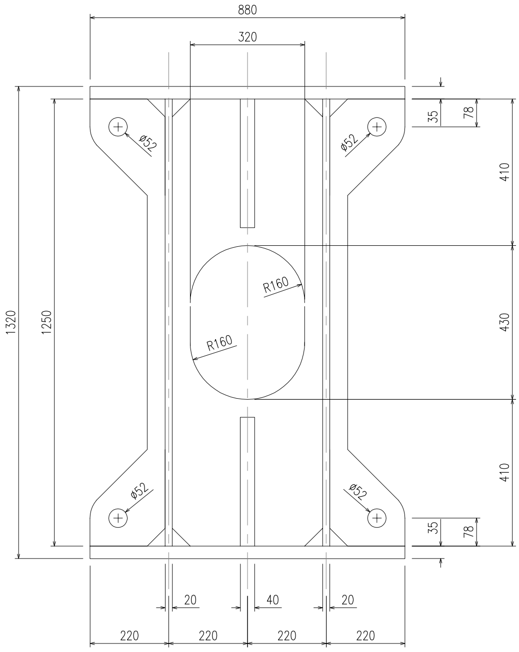


Figure 6.9: Total geometry of the beam

6.4. Connection between beam sections

Three types of forces act in the connection, which are transferred to the next section by different parts:

- Compression due to the global bending moment; this is transferred through the flanges
- Tension due to the bending moment; this is transferred through the pin-hole connection
- Shear; this is also transferred through the pin-hole connection

The values of these forces are taken as M_{Rd} and V_{Rd} of Section 6.3, to create a modular beam in which the connections between sections are equally strong as the design resistance of the beam sections. This results in the connection geometry shown in Figure 6.10. The connection is constructed of High Strength Steel (HSS), S690, because a connection from S355 steel takes too much space. Disadvantage of using HSS is that it is more expensive, but the advantages are that less steel is required and therefore less welding.

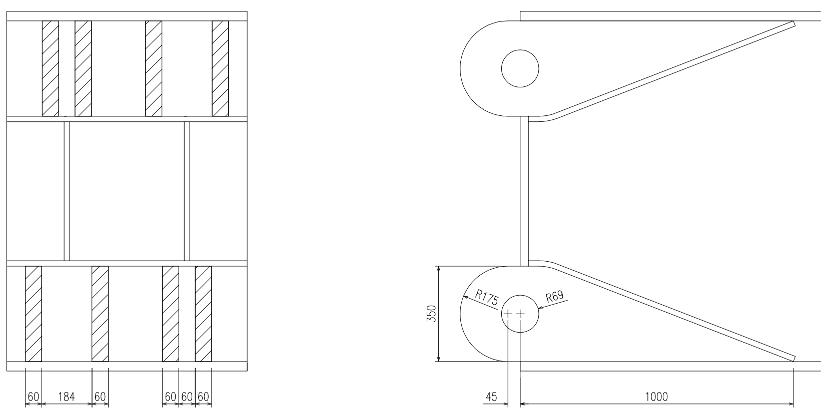


Figure 6.10: Dimensions of the pin-hole connection between beam sections

PART III

Concept evaluation

7
Structural analysis

This chapter evaluates the structural design of the modular beam concept. A Finite Element Method (FEM) model will be developed in Section 7.1 to look at the basic design, as presented in Chapter 5, more in detail. This model will be used in Section 7.2 to perform a Finite Element Analysis (FEA). Changes in the basic design due to the structural analysis are presented in the final concept design in Section 7.4.

7.1. Finite element model

The FEM model used for evaluating the structural properties of the basic beam design is shown in Figure 7.1. The model consists of plates constructed from S355 steel, which has a yield strength of 355N/mm^2. Depending on the load case to be analyzed, hinged and sliding line supports are placed under the appropriate transverse and outstand stiffeners over the entire with of the beam. Loads due to the functions and environmental factors are introduced to the top flange of the beam as either concentrated or distributed loads. The target mesh size is chosen to be 50mm×50mm squares.

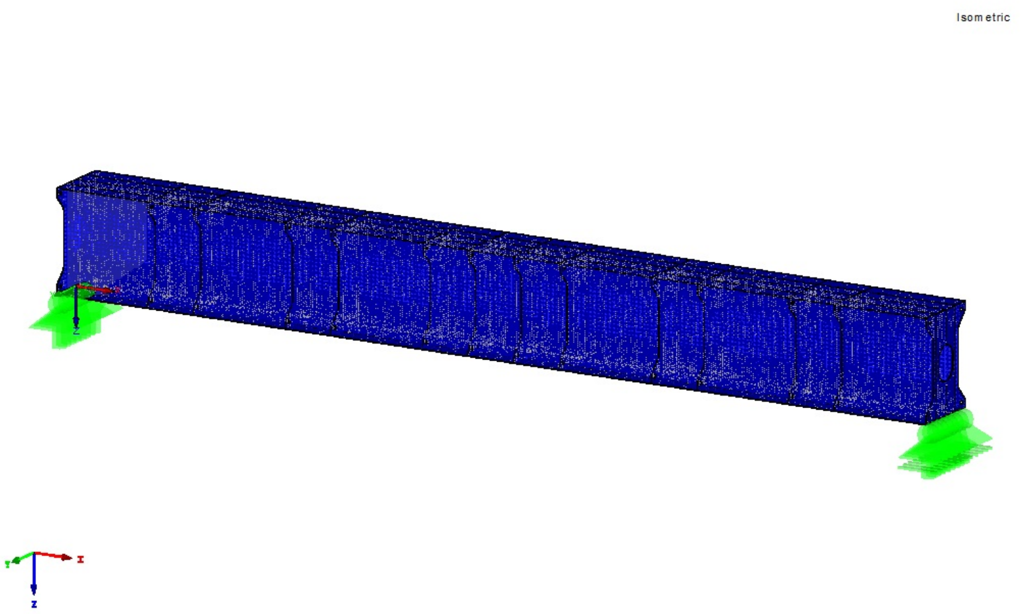


Figure 7.1: Basic FEM model, as developed in RFEM

Due to the use of a plate model, singularities arise at the locations of load introduction and supports. These singularities result in peak stresses in the model, see Figure 7.2. Although these stress peaks are higher than the yield stress of the material, this does not mean that the beam is not strong enough. Detailed engineering and modelling solves this problem. However, this model can be used for the global results and therefore the stress peaks at load introduction and support locations will be ignored in this research.

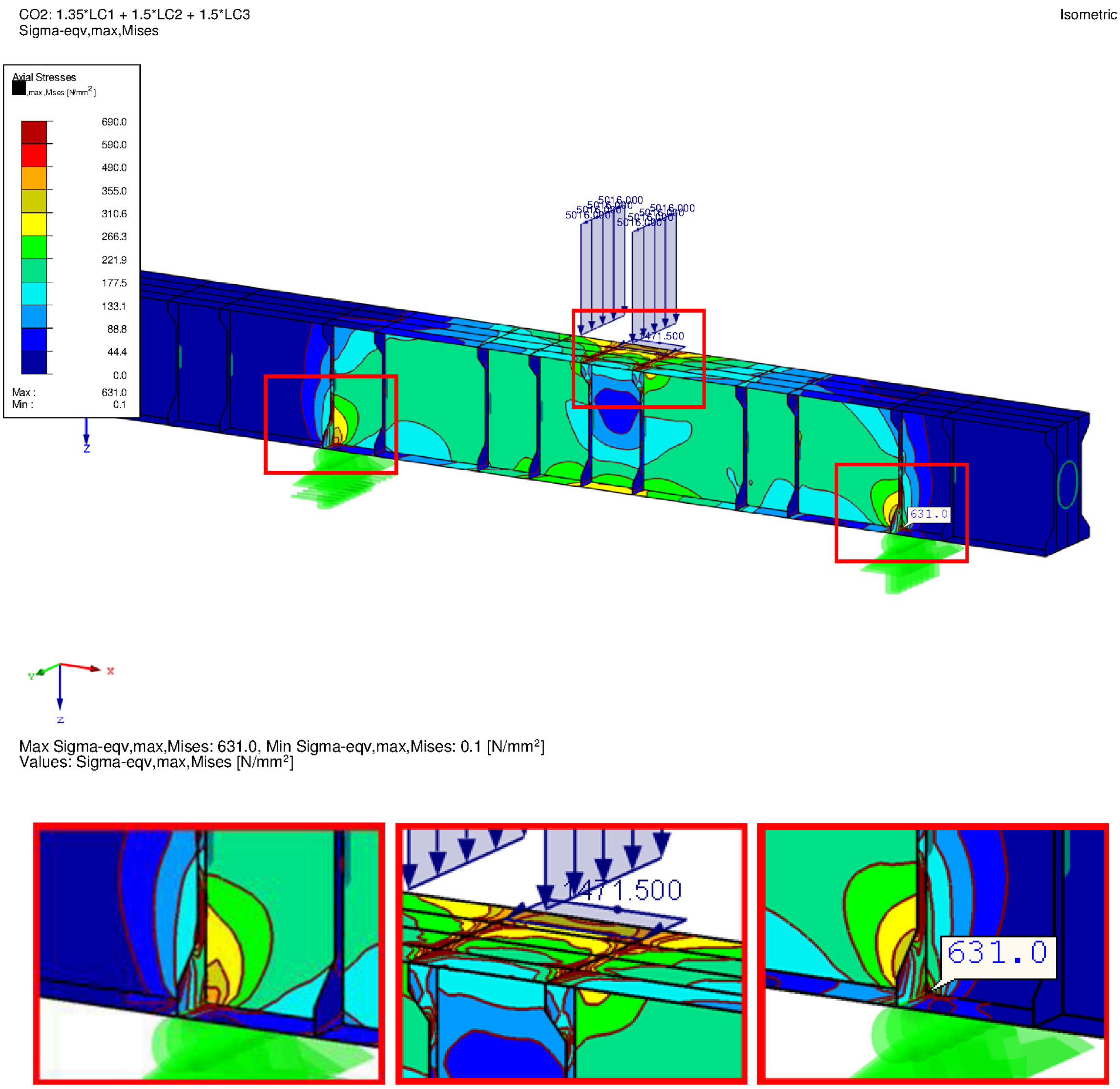


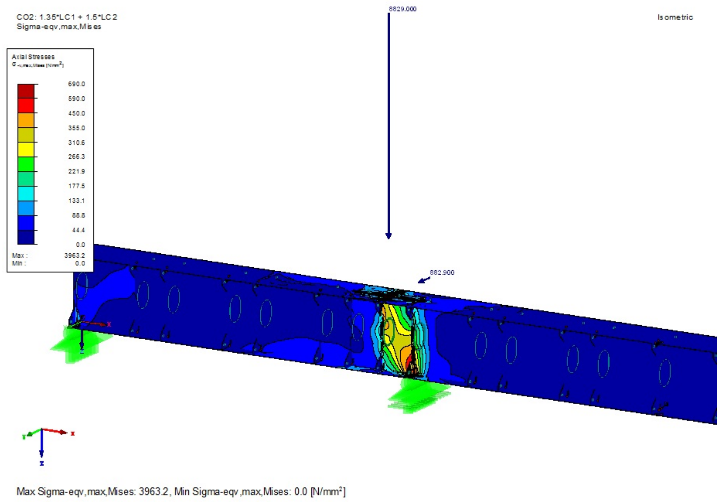
Figure 7.2: Example of stress peaks in the RFEM model due to load introduction and support modelling

7.2. Finite element analysis of the beam

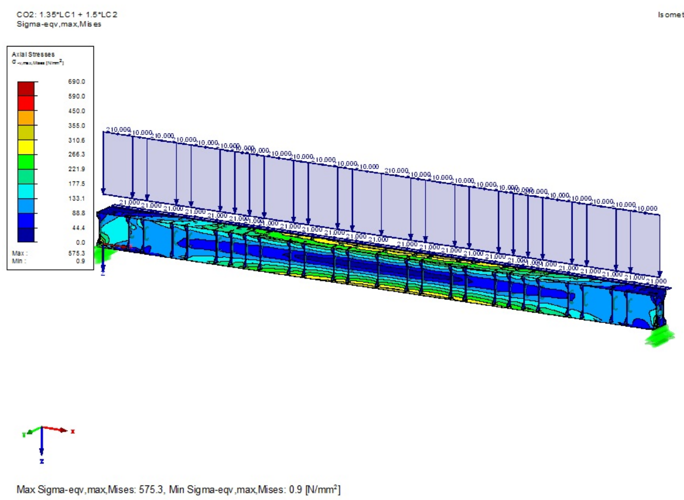
In this section, the worst load cases are shown regarding stresses and deflections. Buckling is not evaluated according to EC3. For an overview of the results of all load cases, see Appendix G.

7.2.1. Stress

The worst load cases regarding stresses, are load cases 6 *'Load on grillage (2)'* and 12 *'Temporary bridge'*. The maximum Von Mises stress of these load cases is shown in Figure 7.3. In the figure can be seen that local stress peaks with values above the yield stress occur at the location of the load introduction. However, this peak stress is caused by the modelling of the load. Another stress peak is caused by the supports in the bottom flange. These stress peaks can be dealt with through detailed design and modeling. By ignoring the local results for now, one can see that the highest global stress is dark yellow, which corresponds to a maximum stress of 355N/mm^2. Thus it can be concluded that the global strength of the beam is sufficient to fulfill the functions.



(a) Stress in the modular beam due to load case 6: Load on grillage (2)



(b) Stress in the modular beam due to load case 12: Temporary bridge

Figure 7.3: Von Mises stress results for the two worst case load cases

7.2.2. Deflection

The worst load case regarding deflections is load case 12 *'Temporary bridge'*, as shown in Figure 7.4. The maximum deflection is 125mm with a span of 22800mm. The maximum deflection according to EC3 is $1/150 \times 22800mm = 152mm$ at this span, so also the deflection is within the limits of the design criteria.

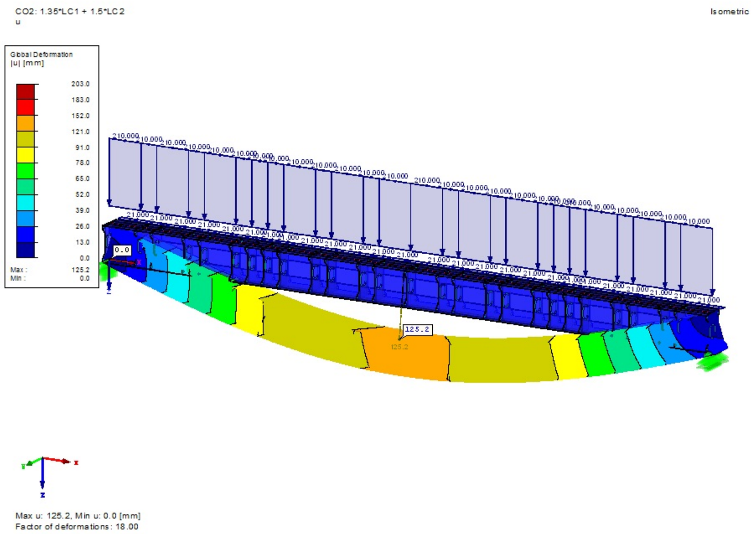


Figure 7.4: Deflection of the modular beam due to load case 12: Temporary bridge

7.2.3. Bending moment

The maximum bending moment of all load cases occurs in load case 12 *'Temporary bridge'*, as shown in Figure 7.5. This bending moment is 14776.90kNm, which is including a safety factor of 1.5 following from EC3. The bending resistance of the beam is 20418kNm including a safety factor of 1.5, which implies that the occurring bending moment is within design limits. The occurring bending moment without safety factor is approximately 9851kNm, which is close to the calculated value of 9106kNm. This difference is caused by the self weight and wind.

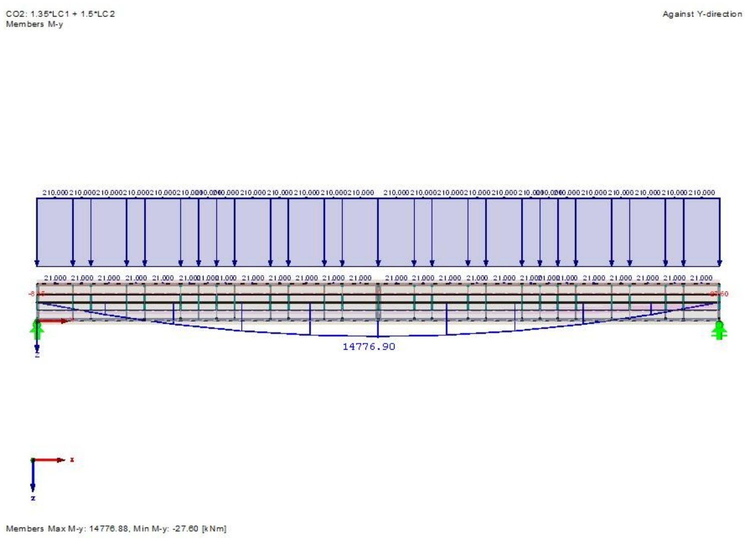


Figure 7.5: Bending moment in the modular beam due to load case 12: Temporary bridge

7.2.4. Shear force

The maximum shear force of all load cases occurs in load case 1 *'Skidding with Enerpac skids'*, as is shown in Figure 7.6. The maximum shear force occurring is 5441.19kN including a safety factor of 1.5 following from EC3. The shear resistance of the beam is 10767kN including safety factor of 1.5, so the shear force is within design limits. The occurring shear force without safety factor is approximately 3660kN, which is close to the calculated value of 3896kN. The difference is caused by wind and the exact location of the load introduction.

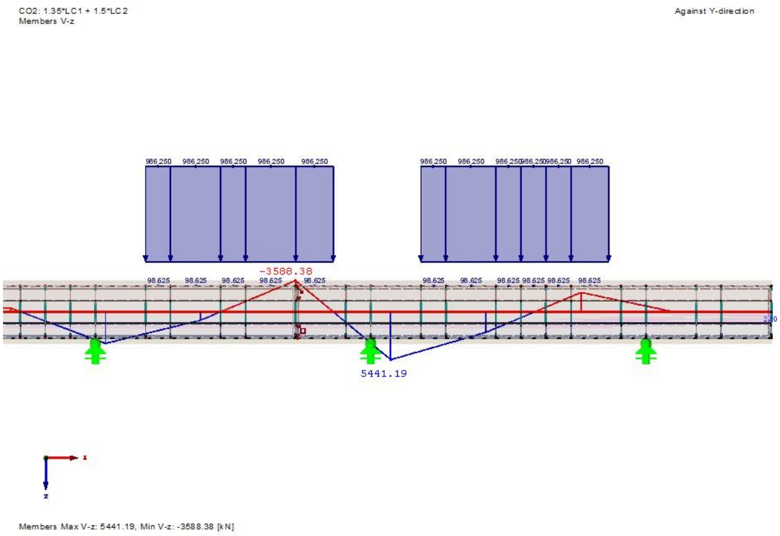


Figure 7.6: Shear force in the modular beam due to load case 1: Skidding with Enerpac (1)

7.3. Finite element analysis of the connection

In this section, a FEA is performed for the connection between beam elements. In Chapter 6 it is determined that four pad eyes of 60mm thickness on both the top and bottom of the beam are required to transfer the forces between beam elements. In this FEA only one pad eye is modelled and loaded with a quarter of the total load transferred through the connection. The forces are applied to the center of the pin, which is modelled by a *'No tension'* element, so that only compressive forces are transferred to the pad eye.

7.3.1. Model and mesh

In this section, an overview of the model is given in Figure 7.7. In this model, the mesh is shown as well. The mesh size chosen is squares of 50mm×50mm. Supports are applied to the top edge and the inclined line.

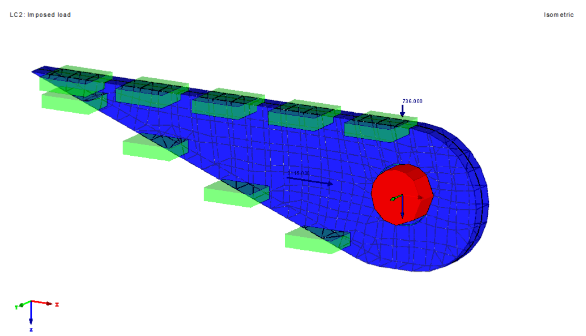


Figure 7.7: Overview of the model of one pad eye and the mesh size applied

7.3.2. Stress

The resulting stresses in the pad eye are presented in Figure 7.8. The maximum value is 521.3N/mm^2, which is below the allowable value of 690N/mm^2 for the pad eye material (S690).

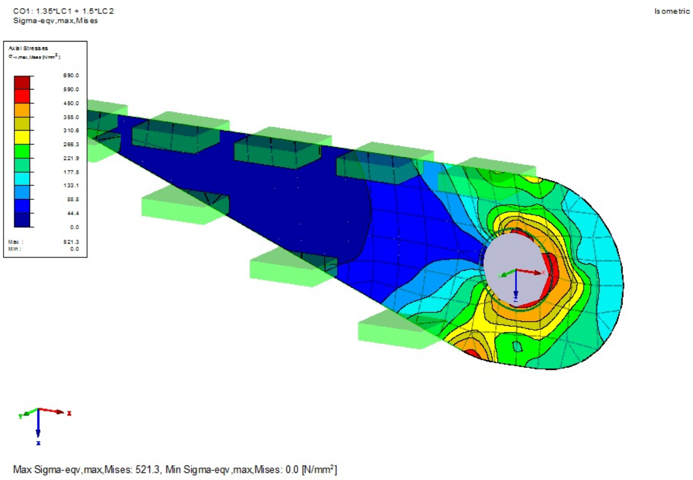


Figure 7.8: Von Mises stress in the pad eye due to application of maximum axial and shear force

7.3.3. Deflection

The maximum deflection of the pad eye occurs in the hole of the pad eye, as can be seen in Figure 7.9. The deflection is 0.5mm, which is within the limits.

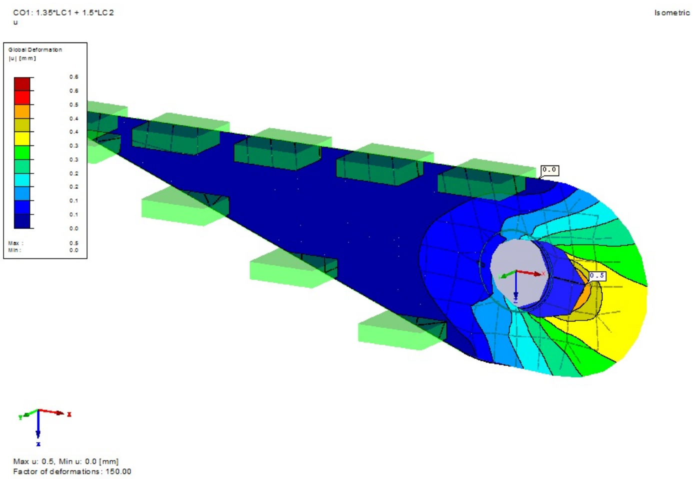


Figure 7.9: Deflections in the pad eye after application of maximum axial and shear force

7.4. Final beam concept

The beam design proposed after hand calculations in Chapter 6 is tested in this chapter using a FEA. It has shown that the beam design is adequate for the functions that the beam has to fulfill. An overview of the cross section properties is shown in Figure 7.10.

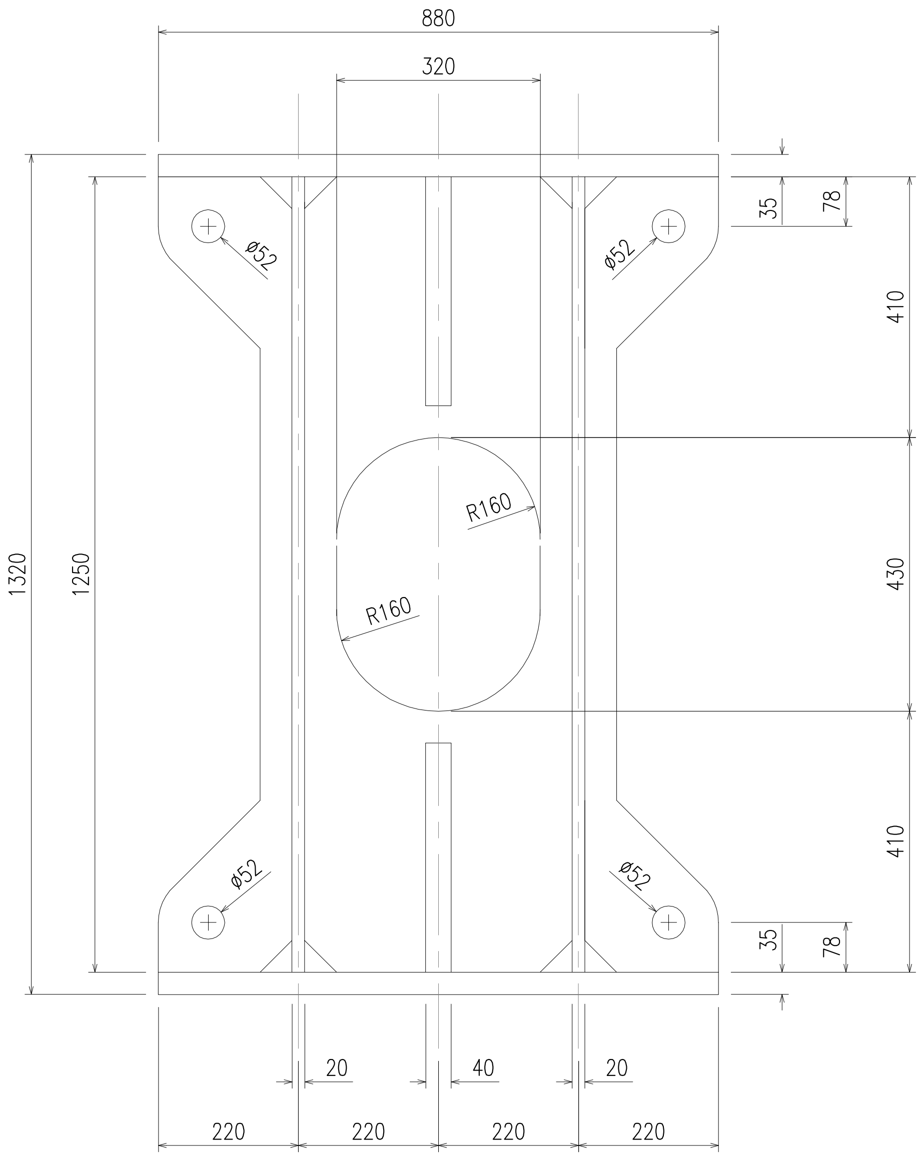


Figure 7.10: Main properties of the cross section of the beam after FEA

8
Economic analysis

8.1. Implementation costs

The implementation costs for the beam set shown in Figure 8.1 are summarized in Table 8.1. With this set, all functions mentioned in Chapter 2 can be fulfilled. This is also visualized in Figure 8.2. An evaluation per item of this table is given in the next sections.

Table 8.1: Summary of implementation costs

Item	Costs	
Engineering costs	€	127,500
Manufacturing costs	€	1,455,000
Purchase of containers	€	27,100
Total implementation costs	**€**	**1,609,600**

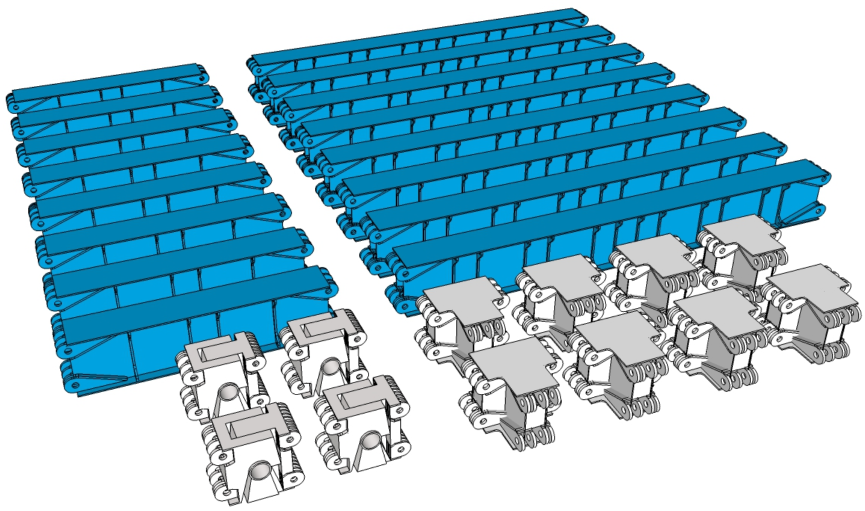
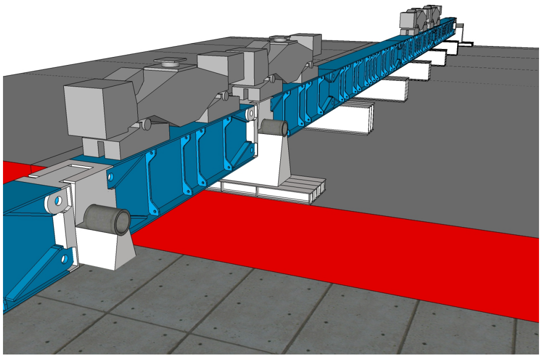
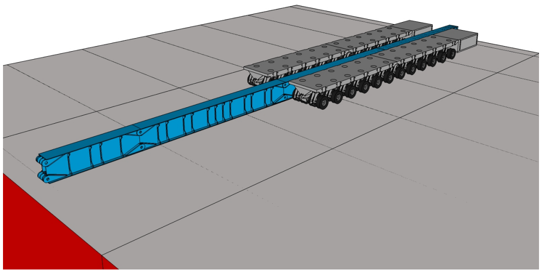


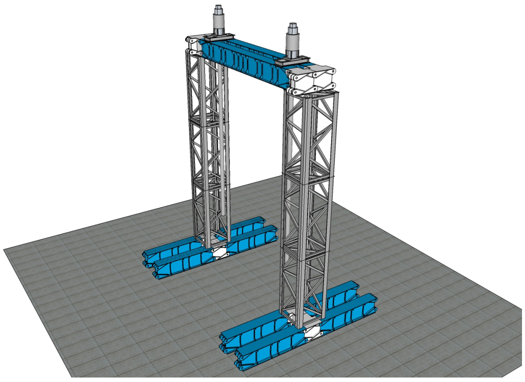
Figure 8.1: Set of modular beam components to be able to fulfill all beam functions mentioned in this report



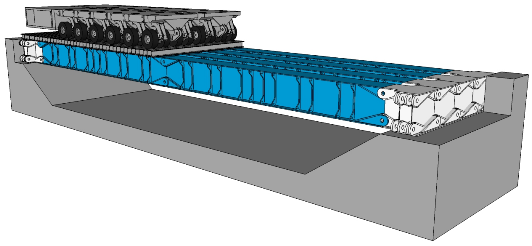
(a) Skidding a load onto the MC, deployed components are beam 11400mm, beam 5400mm, link beam attachments, supports



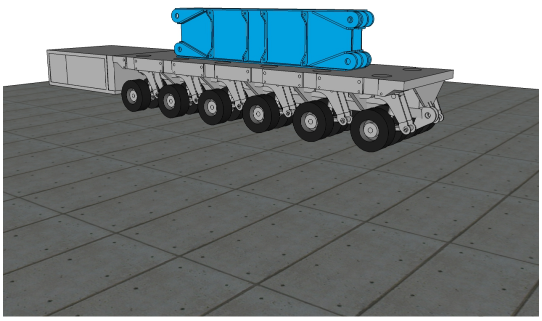
(b) Grillage on the deck of the MC, deployed components are beam 11400mm, beam 5400mm



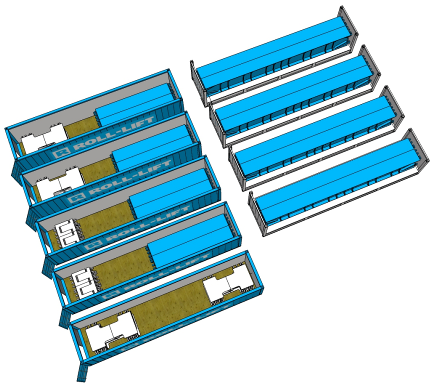
(c) Gantry with foundation, deployed components are beam 11400mm, beam 5400mm, T-connector



(d) Temporary bridge with a span of approximately 25m, deployed components are beam 11400mm, T-connector



(e) Spacer on a trailer, deployed component is beam 5400mm



(f) Division of the beam components over 40ft containers, considering weight and size limitations

Figure 8.2: Visualization of the beam functions and beam components used for these functions

8.1.1. Engineering costs

The engineering costs include everything up to the fabrication drawings: the concept design, detailed design, drawing, classification and approval of the design. It is expected that 1500 man hours are needed for this phase of the implementation of the modular beam. At a rate of 85 €/hour, the total engineering costs become as calculated in Formula (8.1).

$$man\ hours \times costs\ per\ hour = 1500 \times 85 = €127,500 \tag{8.1}$$

8.1.2. Manufacturing costs

The manufacturing costs cover all the costs involved for the fabrication of the beam: materials, machining, manufacturing, man hours, conservation and testing. All these costs are translated into a price per kilogram, which is based on the price of Roll-Lift lifting beams purchased in 2013. This price is 6 €/kg. The mass of the modular beam system is calculated with Formula (8.2). The individual values per component are evaluated in Table 8.3.

$$Total\ mass = \sum component\ mass \times number\ of\ components = 242.5t \tag{8.2}$$

This brings the manufacturing costs calculated in Formula (8.3).

$$Manufacturing\ cost = total\ mass \times price\ per\ kilo = 242,500 \times 6 = €1,455,000 \tag{8.3}$$

8.1.3. Purchase of containers

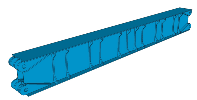
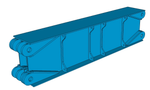
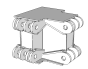
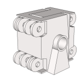
The modular beam components will be transported and stored in containers, purchased by Roll-Lift. In order to calculate the costs of containers required, the number of container needs to be determined. The distribution of the components, based on weight and size criteria, is shown in Table 8.2. The division of the beam components over the containers is also visualized in Figure 8.2(f). Inquiry at Hacon Container BV gives a cost price of €3,500 for a 40ft open top container and €2,400 for a flat rack, the total purchasing costs of containers are calculated with Formula (8.4).

$$Container\ cost = \sum number\ of\ containers \times price = 4 \times 2,400 + 5 \times 3,500 = €27,100 \tag{8.4}$$

Table 8.2: Distribution of the beam components over containers

#	Container type	Contents
1	Flat rack	2× Beam 11400mm
2	Flat rack	2× Beam 11400mm
3	Flat rack	2× Beam 11400mm
4	Flat rack	2× Beam 11400mm
5	Open top	2× Beam 5400mm 2× Link beam attachment
6	Open top	2× Beam 5400mm 2× Link beam attachment
7	Open top	2× Beam 5400mm 2× T-connector
8	Open top	2× Beam 5400mm 2× T-connector
9	Open top	4× T-connector

Table 8.3: Calculation of the masses of the individual components and the total mass of the modular beam

Section	Section properties		Beam 11400mm		Beam 5400mm		T-connector		Link beam	
										
			amount	subtotal [t]	amount	subtotal [t]	amount	subtotal [t]	amount	subtotal [t]
Beam	0.88	t/m	11.4m	10.0	5.4m	4.8	2.06m	1.8	0m	0.0
Connection	1.11	t/end	2	2.2	2	2.2	3	3.3	2	2.2
End plate	0.06	t/end	2	0.1	2	0.1	3	0.2	2	0.1
Transverse frame	0.08	t/location	12	1.0	4	0.3	0	0.0	0	0.0
Outstand stiffener	0.07	t/location	12	0.8	4	0.3	0	0.0	0	0.0
Other mass				0.0		0.0		0.3		3.3
Component mass				14.2		7.7		5.6		5.6
Number of components				8		8		8		4
Total component mass				113.0		61.5		45.0		23.0

8.2. Annual costs

The annual costs of the beam that are not project specific consist of several parts: depreciation, financing cost, maintenance cost, insurance, storage, corporate overhead, taxes and profit and risk cost. The sum of these parts is the minimum required annual revenue for the modular beam to be economically feasible, this subject will be handled in Section 8.3. An elaboration per part of the annual cost is given in the next sections.

8.2.1. Depreciation

The beam system will be amortized to scrap value over 15 years, which is the same period as Roll-Lift uses for crawler cranes. Because a crawler crane consists of moving parts and the modular beam only of steel, this is a safe estimation. With a scrap value of €0.19 ([21], consulted March 10 2014), the scrap value is calculated with Formula (8.5). The depreciation cost is then calculated with Formula (8.6).

$$Scrap\ value = totalmass \times scrapprice = 242500 \times 0.19 \approx €46,100 \tag{8.5}$$

$$Depreciation\ cost = \frac{implementation\ costs - scrap\ value}{depreciation\ period} \approx €104,200 \tag{8.6}$$

8.2.2. Financing

The implementation costs are financed by a bank. The interest rate is assumed 5% of the average debt with the bank. The annual financing costs are calculated with Formula (8.7).

$$Financing\ cost = interestrate \times \frac{implementation\ costs - scrap\ value}{2} \approx €39,000 \tag{8.7}$$

8.2.3. Maintenance

There are no moving parts in the modular beam system, so maintenance of the beam is limited to touching up the paint and occasionally refurbishing the pin-hole connections. In consultation with Roll-Lift, the maintenance cost are estimated to 0.5% of the implementation cost. This gives the annual maintenance cost as calculated in Formula (8.8).

$$Maintenance\ cost = 0.5\% \times implementation\ costs \approx €8,000 \tag{8.8}$$

8.2.4. Insurance

Insurance for the beam system is estimated to be 0.5%, in consultation with Roll-Lift. This value is low, because the beam is a dead object: it cannot move on its own (like a crane) and it has no operator of its own. The annual insurance costs are calculated with Formula (8.9).

$$Insurance\ cost = 0.5\% \times implementation\ costs \approx €8,000 \tag{8.9}$$

8.2.5. Storage costs

When not in use, the beam must be stored. For the basic beam set mentioned in Section 8.1, at least nine containers are required to transport and store all beam elements. Roll-Lift stores equipment in a yard in Rotterdam for 4.50 €/m^2/month. In consultation with Roll-Lift, it is estimated that the beam is used 4 months per year, in transit 4 months per year and in storage 4 months per year. This gives annual storage costs calculated with Formula (8.10).

$$Annual\ storage\ cost = 4.50 \times 9 \times area\ of\ 40ft\ container \times number\ of\ months\ in\ storage = €4,800 \tag{8.10}$$

8.2.6. Corporate overhead

Most costs made during the execution of projects are directly charged to the particular projects. However, indirect cost such as rent of offices or payments to general personnel must also be accounted for. Therefore, a fraction of the implementation cost is reserved to pay these indirect cost. The percentage used is 2%, which is based on the value Roll-Lift uses for cranes. The corporate overhead is calculated using Formula (8.11).

$$Corporate\ overhead = 2\% \times implementation\ costs = €32,000 \tag{8.11}$$

8.2.7. Taxes

When the beam is used abroad and makes money, Roll-Lift has to pay taxes over the earned money. The amount of taxes to be paid is based on Roll-Lift data for cranes, which yields 0.5% of the implementation cost of the beam. This gives the annual tax cost as calculated in Formula (8.12).

$$Tax\ cost = 0.5\% \times implementation\ costs \approx €8,000 \tag{8.12}$$

8.2.8. Profit and risk

A fraction of the annual cost consists of a compensation to the investors for the risk of their investment or a share of the profit that their investment makes. The amount of profit and risk accounted is based on values used for cranes within Roll-Lift, which is 2.1% of the implementation costs. This gives the profit and risk costs as calculated in Formula (8.13).

$$Profit\ and\ risk\ cost = 2.1\% \times implementation\ costs = €33,800 \tag{8.13}$$

8.3. Breakdown cost price

In this section the minimum rent price of the beam system is calculated, based on the annual costs of the beam system. The breakdown cost price is shown in Table 8.4. This yields a minimum rate of €61.00 to break even. This will be the price Roll-Lift uses for internal invoices. In comparison to hiring heavy lift beams, which cost between €80 and €100 per ton per week (from previous Roll-Lift projects), it can be concluded that the modular beam is economically feasible. It can also be concluded that a profit margin exists between the minimum required revenue and the market price of comparable heavy lift beams so that the modular beam is profitable when rented to third parties. An indication of the minimum rent price per beam component is shown in Table 8.5.

Table 8.4: Breakdown cost price of the basic modular beam set

Breakdown cost price			
Depreciation	€	104,200	44%
Interest	€	39,000	17%
Maintenance	€	8,000	3%
Insurance	€	8,000	3%
Storage	€	4,800	2%
Corporate overhead	€	32,000	14%
Taxes	€	8,000	3%
Profit and risk	€	33,600	14%
Required annual revenue	**€**	**237,600**	**100%**
Yearly deployment		16	weeks
Avg. deployed beam mass		242.5	t
Minimum rate/ton/week	**€**	**61.00**	

Table 8.5: Indication of minimum rent prices of beam component

Component	**Mass [t]**	**Minimum rent [€/week]**
Beam section 11400mm	14.2	870
Beam section 5400mm	7.7	470
T-connector	5.6	350
Hinge	5.6	350

PART IV

Conclusions and Recommendations

9
Conclusions and recommendations

The goal of this research was to develop a concept design of a modular, containerized, multi functional beam for heavy lift purposes, with the main purpose to be used for loading the BigRoll MC using platform trailers or skidding. In this chapter, the conclusions and recommendations of this research will be presented.

9.1. Conclusions

9.1.1. Multi functional design

The design of the beam is based on a set of functions, namely skid support beam, grillage on deck of the MC, gantry girders and foundations, vertical spacer and temporary bridge. Equipment used for the functions is chosen to be in the range around 600t, which yields comparable loads on the beam and therefore comparable requirements. It has shown that none of the functions have extreme load requirements compared to the other functions. The functions skid support beam and grillage posed requirements on the width and height respectively. These requirements have shown not to interfere with the requirements and performance of other functions. The conclusion about multi functional design is that all functions mentioned can be united in one multi functional heavy lift beam.

9.1.2. Modular design

The beam concept proposed in this research is modular due to the application of connections on the ends of beam sections. These connections are standardized for the entire beam set with all of its components. Through the symmetric design of the connections, beam elements always fit, even upside down and backwards. Furthermore, the connections are made equally strong as the rest of the beam, so that no weak spots occur in an assembly. The beam concept consists of several components, such as long beam sections, short beam sections, T-connectors and link beam attachments. These components allow for a multitude of arrangements, from a long beam assembly to a frame structure.

9.1.3. Containerized design

The dimensions of the beam are based on the functions. These dimensions are limited to a maximum length, width and height, so that the beam elements fit in standard shipping containers. One requirement for the design was that the beam must be constructed from S355 steel. The disadvantage of this steel grade compared to HSS is that relatively a lot of material is required to obtain sufficient strength. Despite of this requirement, the basic design proves that the mass of the beam elements is well within the limits for containerized transport, and that multiple elements can be transported in one container.

9.1.4. Module carrier

The concept beam fits the deck structure of the MC, with regard to the length and stiffeners. The length of beam sections is determined using the transverse direction on deck of the MC to ensure a proper fit on deck. To cope with high bearing loads at intersections with the vessels bulkheads, stiffeners are placed at these locations in the beam.

9.1.5. Materials

The material used for the overall design of the beam is S355 steel. It is proven through hand calculations and FEA that sufficient strength is achieved with the proposed beam design constructed from S355 steel. The connection between beam elements however, cannot be constructed from S355 steel because this would require a connection that is wider than the beam. The material chosen for the connection is HSS S690, which is roughly twice as strong as S355 steel.

9.1.6. Cost

The cost of the beam concept are estimated for a set of beam components that is sufficient to perform all functions mentioned. This set consists of 8 long beam sections, 8 short beam sections, 8 T-connectors and 4 link beam attachments. A minimum revenue per week per ton is determined to cover the cost. It is concluded that the minimum revenue is lower than the rent price of comparable beams of other companies. This gives two scenarios. In the first scenario for own use, the internal beam rent is lower than hiring beams from competitors. In the second scenario, when the beam is rented to other companies, profit can be made.

9.2. Recommendations

9.2.1. Detailed design

The design of the beam is still basic at the end of this research. Before it can be implemented, further engineering is required to look into the weld details, construction sequence, fit of the connections and all other details.

9.2.2. Risk of damaging the beam

The requirement of S355 steel for construction of the beam follows from the idea that a damaged beam must be able to be repaired on site. Higher grades of steel need special provisions when they are welded, so repairs to these steel types cannot be done on site. An analysis of the risk of the beam being damaged may provide more insight, and may prove that the risk is low enough to drop this criterion and justify higher steel grades.

9.2.3. Use of high strength steel

For the concept design of the modular beam in this research, plate thicknesses are required up to 40mm. With the use of HSS, smaller thicknesses are required, which yields a less heavy beam structure. Although the material cost of HSS are higher and welding is more expensive, the reduction of materials and welds may lead to less production cost compared to steel grade S355. However, due to the application of thinner plates in HSS, stability of the beam becomes an issue. To prevent buckling, additional stiffeners may be required which implies again an increase in material and welds and thus an increase in production cost. More investigation into the application of HSS may yield a more cost effective concept.

9.2.4. Design of auxiliary components

In this research, auxiliary components for the modular beam are proposed, such as a T-connector and link beam attachments. These auxiliary components are rough ideas to aid in the multi functional aspect of the modular beam. To check whether these ideas are actually implementable, more engineering is required on these components.

9.2.5. Storage location

For the economical analysis in this research, it is assumed that the modular beam set is stored near the Roll-Lift headquarters, in Rotterdam, The Netherlands. For cost reasons, it may be useful to investigate in other storage locations around the globe to reduce on (de)mobilization cost and time.

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PART V

Appendices

A

Scientific research paper

The scientific research paper is presented on page 71.

Concept design of a modular heavy lift beam

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ABSTRACT

A piece of equipment often used in the heavy lift industry is the heavy lift beam. This type of beam is used in a wide variety of purposes, however each beam is often designed to fulfill only one of these purposes. In some cases, the heavy lift beam is even designed and built for one individual project and scrapped afterwards. RollDock and Roll-Lift are two sister companies providing heavy lift services, over sea and on land respectively. They requested a heavy lift beam design that can be used for multiple purposes, that is modular and can be transported in standard shipping containers. An added challenge of the design is that the beam must fit on the deck of RollDock's new wide deck heavy lift vessel, the Module Carrier. In this paper, a concept design of a heavy lift beam is proposed that can fulfill the functions skid support beam, grillage, gantry girder, gantry foundation, vertical spacer and temporary bridge. The concept is developed in four parts: determination of the cross section of the beam, determination of the section lengths, determination of the connection between beam sections and the proposal of auxiliary beam components. The basic structural design of the beam is carried out with hand calculations, followed by a Finite Element Analysis in RFEM. The design standard used as a basis for the design, loads, load combinations and safety factors is NEN-EN 1993 (Eurocode 3). It is shown that within the dimensional limitations imposed by the multiple functions and containerized transport, a heavy lift design can be made that meets all requirements to fulfill the functions. Finally, a economic analysis is performed resulting in a breakdown cost price, to prove the economic feasibility of the concept design.

I. Introduction

THE heavy lift market is characterized by ultra heavy and odd sized loads. Often heavy transports are unique projects, which are engineered for that one time job. Standard heavy lift and transport equipment is used to the extent possible for each project, but often specialized structures are required. A structure that is often used for this purpose is a heavy lift beam. Heavy lift beams are used for load spreading, as well as applications in horizontal and vertical transport.

RollDock is a shipping company that provides worldwide heavy lift services over sea. RollDock is currently in the process of designing a new wide deck heavy lift vessel for loads up to 15000 tons, the Module Carrier (MC) operated by BigRoll. This type of vessel will be put into service in 2015. When loading ultra heavy loads, a support structure is required to prevent damage to the deck due to concentrated loads. Current practice is that this support structure is engineered and manufactured individually for each project and discarded afterwards. To reduce on engineering and material cost, RollDock has requested to design a standardized steel beam that can be re-used for several projects. For increased productivity of this standardized beam, it must be multi functional to be used for other heavy lift purposes than solely as supports on the MC. These purposes are also found in RollDocks sister company Roll-Lift, which facilitates heavy lift projects on land.

The functions of the beam are determined using past RollDock and Roll-Lift projects, as well as functions that may be useful in future projects. The functions, as further elaborated in Section II, used for the modular beam design are:

- *Skid support beam*, on board the BigRoll MC for loads up to 600 tons per skid shoe
- *Grillage*, on board the Bigroll MC for loads transported by platform trailers
- *Gantry girder*, for a gantry with lifting capacity up to 1200 tons
- *Gantry foundation*, for a gantry with lifting capacity up to 1200 tons
- *Vertical spacer*, to create extra height on platform trailers
- *Temporary bridge*, to support fully loaded platform trailers over a span of 25 meters

The design of this heavy lift beam (Section III and IV) is different compared to other heavy lift beams in the following aspects:

- *Multi functional design*, other heavy lift beams are dedicated to one function or even to one project
- *Modular design*, through the use of standardized connections between beam elements, the beam system can be expanded/adapted depending on the functions
- *Containerized design*, smart dimensioning and the choice of materials, and thus weight of the elements, must enable relatively cheap and convenient transport in standard shipping containers
- *Fit for the MC*, strong points in the beam must line up with strong points in the vessels deck, to enable proper load transfer to the MC

Furthermore both a structural and economic analysis are performed on the concept design to test its feasibility (Section V).

The goal of this research is to develop a concept design of a modular, containerized, multi functional beam for heavy lift purposes, with the main purpose to be used for loading the BigRoll MC using platform trailers or skidding.

II. Background

The heavy lift beam proposed in this paper will be used for loading the MC, as skid support beam or as grillage for loads on platform trailers. Furthermore it will be used for different purposes in a heavy lift gantry, as a spacer and a temporary bridge for heavy transport.

Skidding ([1], [2], [3], [4]) is the process of sliding a load on skid shoes over a skid beam in order to transport it. The skid beam must prevent the skid shoe from damaging the deck of the MC. Friction is reduced through the use of grease or teflon pads. Skidding is often used for heavy loads with a small footprint, too small for other transport methods. Skidding is only used for loads that need to be transported over short distances. Different skid systems with a capacity of approximately 600 tons are analyzed for the concept design.

Grillage is the description of the support structure for loads on the deck of a vessel. The grillage is installed on the deck before the load is brought on board with platform trailers ([1], [5], [6], [7], [8]). Once in the right place, the load is lowered onto the grillage using the suspension stroke of the trailers. The grillage prevents the load from damaging the deck of the MC. The height of the grillage must be between the minimum and maximum height of the trailer in order to drive the load over the grillage, lower it onto the grillage and drive the trailers away. Three types of platform trailers are analyzed: conventional trailers, self propelled trailers (SPT) and self propelled modular trailers (SPMT), with capacities up to 40 tons per axle line.

A gantry is a portal type of crane that is capable of lifting very heavy loads. In fact, a gantry is a bridge structure with a lifting device on top. For heavy gantries, the lifting devices of choice are strand jacks ([2], [4], [9], [10]). A gantry for heavy lift purposes consists from top to bottom of strand jacks, gantry girders, towers, a foundation and for high gantries guy wires. The strand jacks do the lifting, the rest of the components is the steel structure supporting the strand jacks. For the concept design, strand jacks with a capacity of approximately 600 tons are analyzed. The beam in this paper will be used as gantry girder and as gantry foundation.

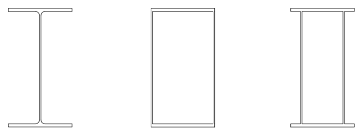
A vertical spacer for heavy lift purposes is in general any wooden or steel structure that is strong enough to carry the load placed on top. Vertical spacers are often used in combination with platform trailers, when the height of the trailer itself is insufficient. The trailers on which the spacer will be used are the platform trailers as described with the function grillage.

A temporary bridge is a temporary structure, often made of steel. For a small span, steel mats provide enough strength to support the trailer. Larger spans are crossed with beams and H-profiles or steel mats on top, to create a stronger bridge. The temporary bridge in this paper is used for fully loaded platform trailers over a maximum span of 25 meters.

III. Design approach

The concept design is divided into four subjects:

- The *shape of the cross section* is determined using dimensional requirements from the functions as well as structural and stability requirements as prescribed by design standards. The options considered are an H-profile, a box beam and a box beam with offset webs, see Figure 1.
- The *length of beam sections* is determined using the dimensions of the deck of the MC, the layout of the deck support structure and the limitations imposed by the requirement of transport in standard shipping containers (maximum length <40ft or <12m). Four options are considered, namely only sections of 9900mm, only 10200mm, a combination of 11400mm and 5700mm and a combination of 11400mm and 5400mm (see Figure 2)
- The *connections between sections* are determined using functional and operational criteria, such as a flat top and bottom and a low number of parts. The options considered are bolts, a pin-hole connection, clamps, container twist locks and a shape fit connection, see Figure 3.
- *Auxiliary beam components* are proposed to aid the straight beam components in fulfilling all the functions. Through the application of the same connections as the beam sections, the auxiliary components can be connected to the beam sections. Two auxiliary components are proposed, namely a T-connector and a hinge.



(a) H-beam (b) Box beam (c) Box beam with offset of webs

Fig. 1. Cross sections of several options for the shape of the modular beam

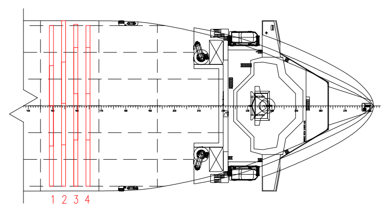
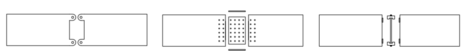
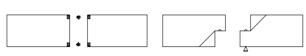


Fig. 2. Fit of the section length options on the deck of the MC: Sections of 9900mm (No.1), 10200mm (No.2), 11400mm+5700mm (No.3) and 11400mm+5400mm (No.4)



(a) Pin-hole (b) Bolts (c) Clamps



(d) Twist locks (e) Shape fit

Fig. 3. Options for connecting the beam sections

The height is chosen to be 1320mm, to stay within the height range of platform trailers. The width of the beam is chosen to accommodate the widest skid system: 880mm. The choice of the offset of the webs is made using the local bending moment in the flanges due to the functions. An offset of 220mm from the centerline is chosen to minimize the local bending moment due to the worst load cases.

The length of the beam sections is chosen using *Pugh's Method* [11], which compares the options relative to each other to a set of criteria. This gives as best option: the combination of sections of 11400mm and 5400mm.

The choice of connection between beam sections is also made using *Pugh's Method*. This gives the pin-hole connection as best option.

The auxiliary components are a T-connector and a hinge. These auxiliary components are not engineered into detail in this research.

The T-connector is dimensioned to provide the same spacing between beams if connected sideways as the wheel bogey spacing of platform trailers for the use as temporary bridge. If connected head to head, the ideal spacing is created for a gantry tower foundation.

The hinge is used with a beam section of 5400mm to create a link beam, a hinged part of the skid support beam to create a bridge between quay and vessel. Due to the hinges, this bridge allows for a small amount of movement between quay and vessel while still providing a continuous support beam along the entire length of skid track.

IV. Basic design

In this section, the basic design including plate thickness of the beam will be determined. A requirement from RollDock and Roll-Lift is that the beam must be constructed in S355 steel, which is considered normal construction steel. This material choice enables on site repairs, if necessary. If the beam would be constructed in High Strength Steel (HSS), the elements can become less heavy. For example, if the beam is constructed in S690 steel which has approximately twice the yield strength of S355, roughly half the weight can be saved. However, S690 may only be welded indoors, needs preheating and must be welded by certified welders for HSS. Therefore the beam will be constructed in S355 steel.

The load cases following from the functions are presented in Table I. The highest values for bending moment and shear are printed in bold face, these are the design values for the structural design.

The load combinations are defined according to Eurocode 3 (EC3) [12]. Load combinations are classified as characteristic, Ultimate Limit States (ULS) and Serviceability Limit States (SLS). The load combinations appropriate for the design are defined as follows [13]:

- Characteristic combination, Formula (1)
- ULS EQU: Loss of static equilibrium of the structure, Formula (2)
- ULS STR: Internal failure or excessive deformation of the structure, Formula (3)
- SLS: Characteristic combination, Formula (4)

TABLE I
BENDING MOMENTS AND SHEAR FORCE DUE TO FUNCTIONS

Load	case	M [kNm]	V [kN]
1	Enerpac skid (1)	5771	3896
2	Enerpac skid (2)	99	395
3	Mammoet skid (1)	5740	3871
4	Mammoet skid (2)	98	392
5	Grillage (1)	**9712**	2943
6	Grillage (2)	0	**5886**
7	Grillage (3)	2453	2943
8	Gantry girder (1)	8388	1472
9	Gantry girder (2)	5886	2943
10	Gantry girder (3)	5886	2943
11	Gantry foundation	0	1472
12	Temporary bridge	9106	1598

$$CO_{k,1} = G_z\text{"}+\text{"}Q_{load,k}\text{"}+\text{"}Q_{wind,k} \quad (1)$$
$$CO_{k,2} = \gamma_G G_z\text{"}+\text{"}\gamma_Q Q_{load,k}\text{"}+\text{"}\gamma_Q Q_{wind,k} \quad (2)$$
$$CO_{k,3} = \xi\gamma_G G_z\text{"}+\text{"}\gamma_Q Q_{load,k}\text{"}+\text{"}\gamma_Q \psi_0 Q_{wind,k} \quad (3)$$
$$CO_{k,4} = G_z\text{"}+\text{"}Q_{load,k}\text{"}+\text{"}\psi_0 Q_{wind,k} \quad (4)$$
$$Q_{wind} = 0.10 \times q_{load} \quad (5)$$

Where	$CO_{k,i}$	=	Load combination i of load case k
	γ_G	=	Partial safety factor for permanent actions (=1.35)
	G_z	=	Self weight
	γ_Q	=	Partial safety factor for variable actions (=1.5)
	Q_{load}	=	Value of the leading variable action
	Q_{wind}	=	Value of the variable action due to wind
	ξ	=	Reduction factor (=0.85)
	ψ_0	=	Factor for combination value of a variable action (=1.0)
	"+"		means *combined with*

The plate thickness of the cross section of the beam are determined using hand calculations. For the hand calculations, only the leading variable actions multiplied by their partial safety factor are considered. The design loads of the beam are (from Table I):

- Design bending moment:
  $M_{Ed} = 9712 \times 1.5 = 14568$kNm
- Design shear force:
  $V_{Ed} = 5886 \times 1.5 = 8829$kN

Using these design loads, the cross section of the beam is determined. To resist local load introduction due to one type of skid system, a steel strip is added at the center of the beam under the flange. This steel strip also aids in the global resistance of the beam to bending and shear. The cross section properties are summarized in Figure 4.

Although stability of this cross section is not an issue according to EC3, transverse stiffeners are applied to the beam. This has four purposes. The first is to transfer the loads on the steel strip at the centerline to the webs. The second is to retain the shape of the beam and support the outstand of the flanges. The third purpose is to create an interface (holes)

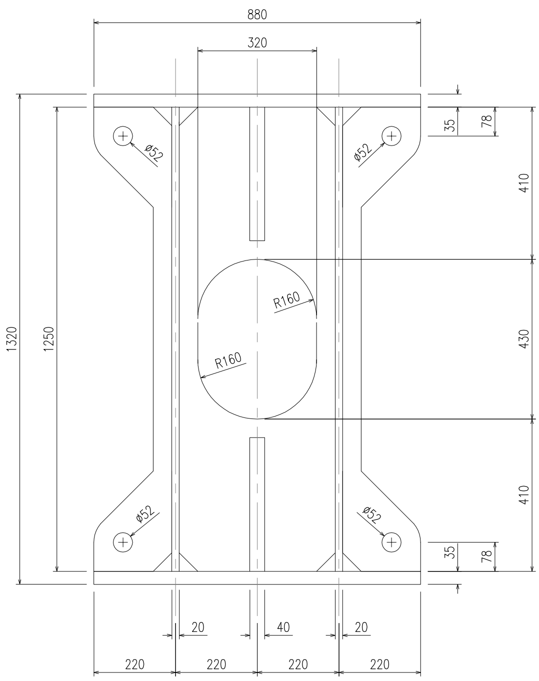


Fig. 4. Main properties of the cross section of the beam

for lifting and lashing the beam. The last purpose is to cope with high bearing stresses where the beam crosses the deck support structure of the MC. This is also how the spacing of the transverse stiffeners is determined. The beam is placed on deck from port to starboard and vice versa. Intersections of beam sections with the deck structure are marked and all sections are combined to get the total picture, see Figure 5.

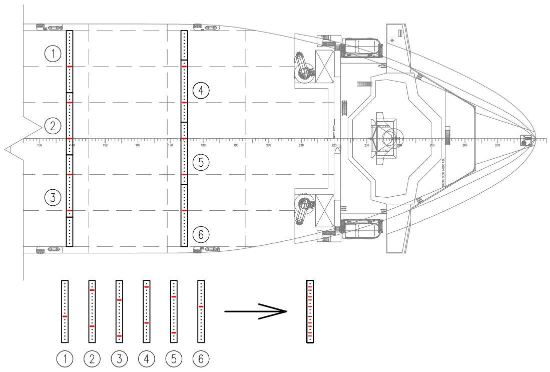


Fig. 5. Determination of the locations of transverse stiffeners, based on intersections with bulkheads of the MC

The pin-hole connection is executed in S690 due to the limited space available for the connection. S355 steel would take too much space and too large plate thicknesses. The connection in S690 consists of pairs of pad eyes, each 60mm thick. The connection will be locked with a pin of 135mm diameter. See Figure 6 for the details of the connection.

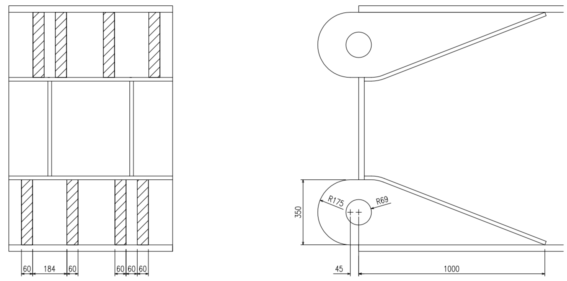


Fig. 6. Dimensions of the pin-hole connection between beam sections

V. Design evaluation

The basic beam design, as proposed in the previous section, is evaluated using a Finite Element Analysis (FEA) in RFEM and tested on it's economic feasibility.

A. Structural analysis

A model of the beam design and connection pad eye are created in RFEM and all load cases and combinations are calculated. The worst case results concerning Von Mises stress and deflections are shown in this paper, for the beam in Figure 7 and 8 and for the pad eye in Figure 9 and 10. It can be seen that for both models, the global results are within the material limits as stated in EC3.

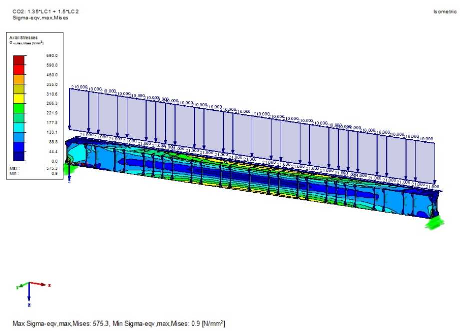


Fig. 7. Von Mises stress in the modular beam due to load case 12: Temporary bridge

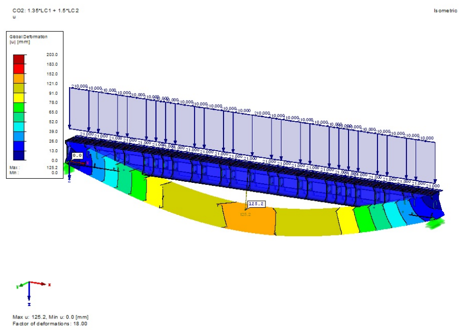


Fig. 8. Deflection of the modular beam due to load case 12: Temporary bridge

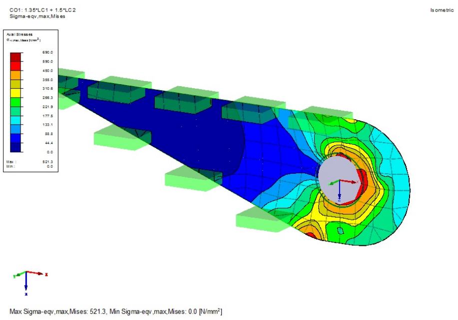


Fig. 9. Von Mises stress in the pad eye due to application of maximum axial and shear force

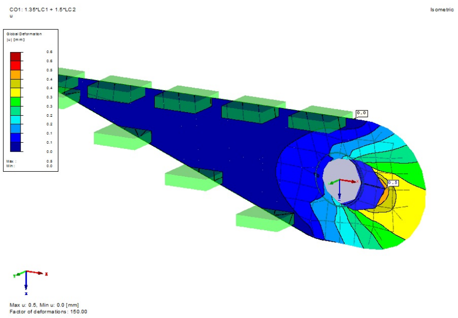


Fig. 10. Deflections in the pad eye after application of maximum axial and shear force

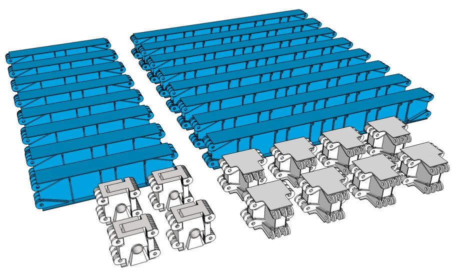


Fig. 11. Basic set of modular beam components to be able to fulfill all beam functions mentioned in this paper

B. Economic analysis

For the economic analysis, a basic set of beam components is composed with all components necessary to fulfill the functions of the beam. The basic beam set consists of 8 beam sections of 5400mm, 8 sections of 11400mm, 8 T-connectors and 4 hinges. This set is shown in Figure 11. The implementation cost, consisting of engineering cost, manufacturing cost and the purchase of containers, are summarized in Table II.

With the implementation cost, an estimate is made of the required annual revenue, which must be at least equal to the annual cost. An overview of the break down cost price, which

TABLE II
SUMMARY OF IMPLEMENTATION COSTS

Item	Costs	
Engineering costs	€	127,500
Manufacturing costs	€	1,455,000
Purchase of containers	€	27,100
Total implementation costs	**€**	**1,609,600**

TABLE III
BREAKDOWN COST PRICE OF THE BASIC MODULAR BEAM SET

Breakdown cost price			
Depreciation	€	104,200	44%
Interest	€	39,000	17%
Maintenance	€	8,000	3%
Insurance	€	8,000	3%
Storage	€	4,800	2%
Corporate overhead	€	32,000	14%
Taxes	€	8,000	3%
Profit and risk	€	33,600	14%
Required annual revenue	**€**	**237,600**	**100%**
Yearly deployment		16	weeks
Avg. deployed beam mass		242.5	t
Minimum rate/ton/week	**€**	**61.00**	

leads to the required annual revenue, is given in Table III. The calculation of the depreciation is based on a depreciation period of 15 years to scrap value. The interest is calculated as 5% of the average debt. Maintenance and insurance are both assumed 0.5% of the implementation cost. For the storage of the entire beam system, 9 containers are required. The storage cost are determined on storage rates in Rotterdam, The Netherlands of €4.50 per square meter. Total storage cost are the area of 9 containers times the storage rate. Corporate overhead consists of costs that can not directly be allocated to projects and amounts 2% of the implementation cost. Taxes are estimated as 0.5% of the implementation cost. Finally profit and risk are estimated as 2.1% of the implementation cost. All percentages mentioned are based on values used for existing Roll-Lift equipment.

The breakdown cost price results in a rate of €61.00 per ton per week. This will be the price Roll-Lift uses for internal invoices. In comparison to hiring heavy lift beams, which cost between €80 and €100 per ton per week (from previous Roll-Lift projects), it can be concluded that the modular beam is economically feasible. It can also be concluded that a profit margin exists between the minimum required revenue and the market price of comparable heavy lift beams so that the modular beam is profitable when rented to third parties.

VI. RESULTS

The results are split in structural results and economical results.

The structural strength of the beam design is shown sufficient for the functions of the beam. The cross section of the beam is shown in Figure 4. A summary of used materials and plate thicknesses is given in Table IV.

TABLE IV
MATERIALS AND THICKNESS OF PARTS OF THE CROSS SECTION

Component	Material	Thickness [mm]
Flanges	S355	35
Webs	S355	20
Longitudinal stiffeners	S355	40
Transverse stiffeners	S355	30
Outstand stiffeners	S690	40

The economic feasibility of the concept is also shown in this paper. The minimum revenue is lower than the rent price of other companies. This implies that for own use the heavy lift beam is less expensive and that a profit margin exists if the beam is rented to other companies. The minimum rent price per beam component is shown in Table V.

TABLE V
INDICATION OF MINIMUM RENT PRICES OF BEAM COMPONENT

Component	Mass [t]	Minimum rent [€/week]
Beam section 11400mm	14.2	870
Beam section 5400mm	7.7	470
T-connector	5.6	350
Hinge	5.6	350

VII. CONCLUSION

In this section, conclusions are drawn about the proposed beam design, regarding the modular, containerized, multi functional design, the fit on deck of the MC, the materials used in the design and the cost of the concept.

The design of the beam is based on a set of functions, namely skid support beam, grillage on deck of the MC, gantry girders and foundations, vertical spacer and temporary bridge. Equipment used for the functions is chosen to be in the range around 600t, which yields comparable loads on the beam and therefore comparable requirements. It has shown that none of the functions have extreme load requirements compared to the other functions. The functions skid support beam and grillage posed requirements on the width and height respectively. These requirements have shown not to interfere with the requirements and performance of other functions. The conclusion about multi functional design is that all functions mentioned can be united in one multi functional heavy lift beam.

The beam concept proposed in this research is modular due to the application of connections on the ends of beam sections. These connections are standardized for the entire beam set with all of its components. Through the symmetric design of the connections, beam elements always fit, even upside down and backwards. Furthermore, the connections are made equally strong as the rest of the beam, so that no weak spots occur in an assembly. The beam concept consists of several components, such as long beam sections, short beam sections, T-connectors and link beam attachments. These components allow for a multitude of arrangements, from a long beam assembly to a frame structure.

The dimensions of the beam are based on the functions. These dimensions are limited to a maximum length, width and height, so that the beam elements fit in standard shipping containers. One requirement for the design was that the beam must be constructed from S355 steel. The disadvantage of this steel grade compared to HSS is that relatively a lot of material is required to obtain sufficient strength. Despite of this requirement, the basic design proves that the mass of the beam elements is well within the limits for containerized transport, and that multiple elements can be transported in one container.

The concept beam fits the deck structure of the MC, with regard to the length and stiffeners. The length of beam sections is determined using the transverse direction on deck of the MC to ensure a proper fit on deck. To cope with high bearing loads at intersections with the vessels bulkheads, stiffeners are placed at these locations in the beam.

The material used for the overall design of the beam is S355 steel. It is proven through hand calculations and FEA that sufficient strength is achieved with the proposed beam design constructed from S355 steel. The connection between beam elements however, cannot be constructed from S355 steel. The material chosen for the connection is HSS S690, which is roughly twice as strong as S355 steel.

The cost of the beam concept are estimated for a set of beam components that is sufficient to perform all functions mentioned. This set consists of 8 long beam sections, 8 short beam sections, 8 T-connectors and 4 link beam attachments. A minimum revenue per week per ton is determined to cover the cost. It is concluded that the minimum revenue is lower than the rent price of comparable beams of other companies. This gives two scenarios. In the first scenario for own use, the internal beam rent is lower than hiring beams from competitors. In the second scenario, when the beam is rented to other companies, profit can be made.

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B

Equipment specifications

B.1. Skidding equipment

B.1.1. Friction reduction

A skid system can be as simple as hard wood timber on steel lubricated with grease, or more sophisticated as steel on steel with graphite or stainless steel on Teflon. The similarity of these skid systems is the combination of materials to reduce the friction, this is presented in Table B.1. The skid systems presented in this research typically use stainless steel on Teflon pads as friction reducing material.

B.1.2. Skid system specifications

The relevant dimensions of the different skid systems for this research are indicated in Figure B.1 to B.4. If a dimension range is shown, it indicates the minimum and maximum dimension in retracted and extended state respectively. The specifications are also summarized in Table B.2. The average line load of the skid systems is calculated by dividing the total capacity of skid shoes plus push-pull unit by the total length of that combination (L_1 in Figure B.5). The maximum line load is calculated by dividing the capacity by the supported length (L_2 in Figure B.5). According to heavy lift specialists, the Dorman Long systems are rarely used in practice because of the high line loads and corresponding ground pressures. Therefore the Dorman Long skid systems will not further be used in the design of the skid support beam. In the heavy lift industry, a shift is seen from the 'old' Mammoet skid system which is in use for many years to the 'new' Enerpac skid system. The latter has the advantage not only to spread the load over the length of the skid shoe, but also to use more width to spread the load. However, both Mammoet and Enerpac systems are often seen in heavy lift projects, so both systems will be used for the design of the skid support beam.

B.2. Platform trailers

The relevant dimensions of the two types of platform trailers for this research are indicated in Figure B.6 and B.7. For the height of the trailers, the minimum height is indicated together with the maximum suspension stroke.

B.3. Strand jacks

The relevant dimensions of the strand jacks presented in this research are shown in Figure B.8 to B.4.

B.4. Lifting and lashing: shackles

For lifting and lashing, shackles are used. These shackles must be used the right way, as side loads on the shackles reduce the capacity of the shackles, see Figure B.12 and B.13. The specifications of Green Pin shackles up to a capacity of 55t are shown in Figure B.14.

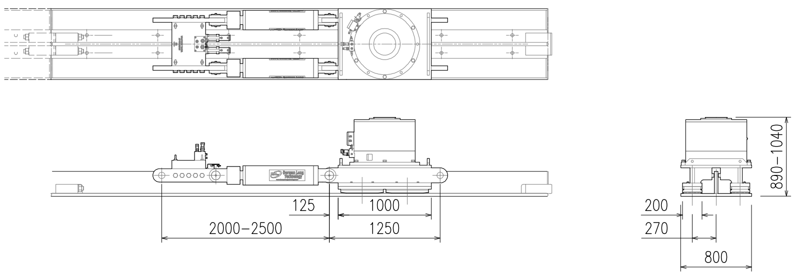


Figure B.1: Dimensions of the Dorman Long SU666 skid system [9]

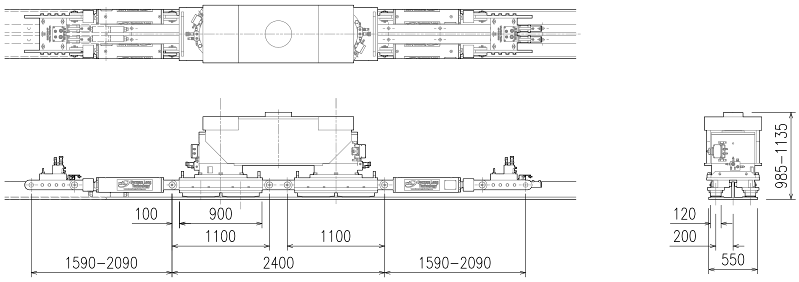


Figure B.2: Dimensions of the double Dorman Long SU333 skid system with bridge [9]

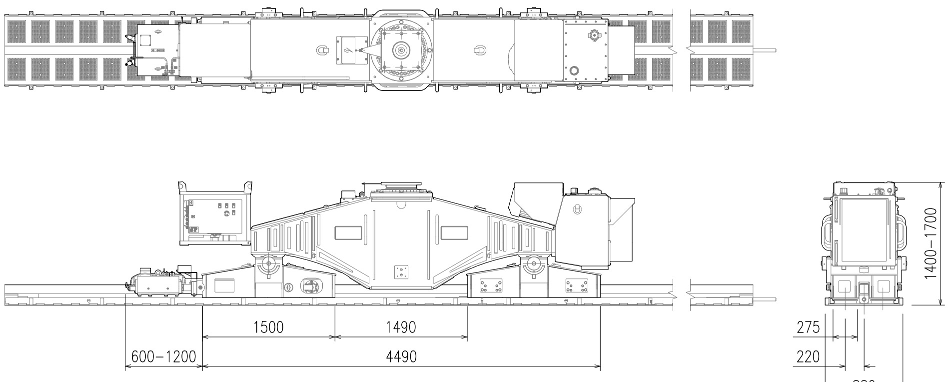


Figure B.3: Dimensions of the Enerpac HSK6000 skid system [4]

Table B.1: Friction of different skid materials [1]

Material	Lubrication	Friction [% of load]
Hard wood on steel	Grease	4-10
Steel on steel	Graphite or grease	6-20
Stainless steel on Teflon	–	2-6

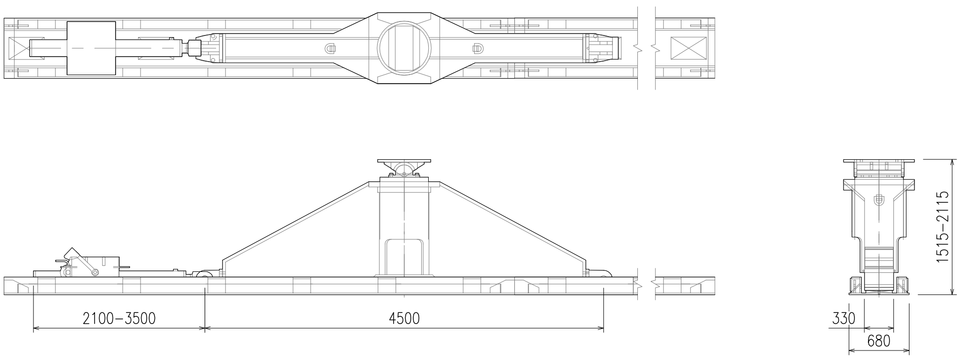


Figure B.4: Dimensions of the Mammoet heavy skid system [2]

Table B.2: Specifications of four 600t skid systems [2, 4, 9]

	Unit	Dorman Long	Dorman Long	Enerpac	Mammoet
Type		SU666	2x SU333	HSK6000	Heavy system
Capacity	t	666	666	600	600
Skid shoe length	mm	1250	2400	4490	4500
Supported length	mm	1000	1800	3000	4000
Offset of supports	mm	270	200	220	0
Skid shoe height	mm	890-1040	985-1135	1400-1700	1515-2115
Skid track width	mm	800	550	880	680
Push-pull unit length	mm	2500	4180	1200	3500
Skid shoes/push-pull unit		1	1	1	2
Average line load	t/m	176	95	105	96
Maximum line load	t/m	666	370	200	150

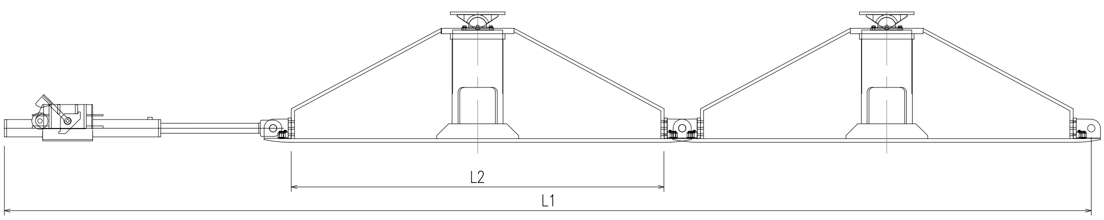


Figure B.5: Lengths used for calculating line loads, L_1 for average line load and L_2 for maximum line load

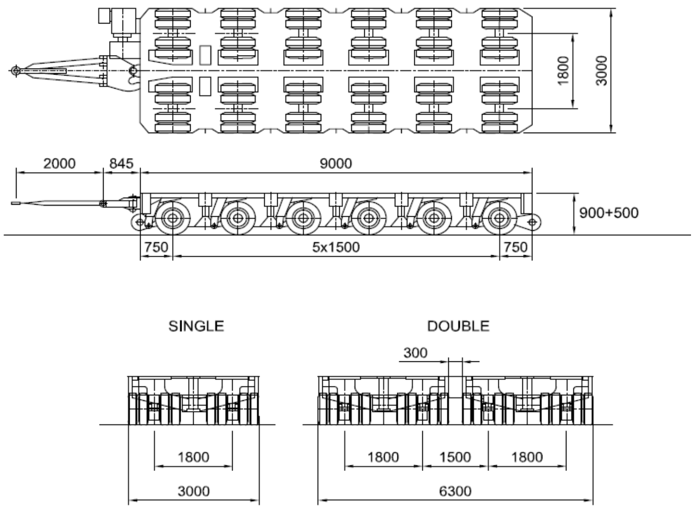


Figure B.6: Dimensions of conventional trailers and SPTs [2]

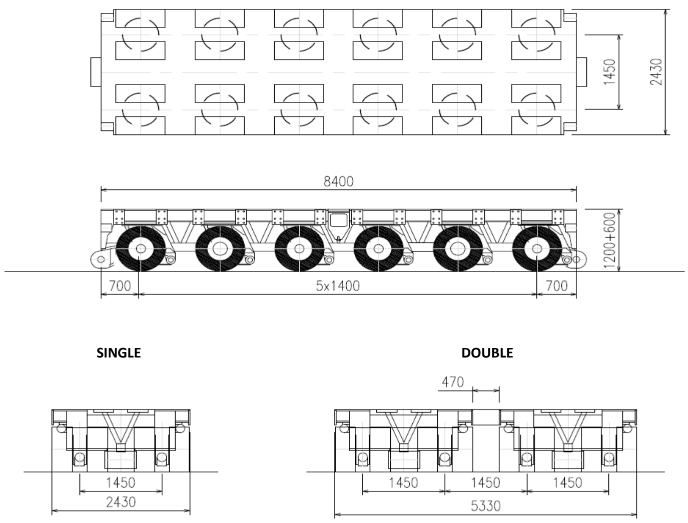


Figure B.7: Dimensions of SPMTs [8]

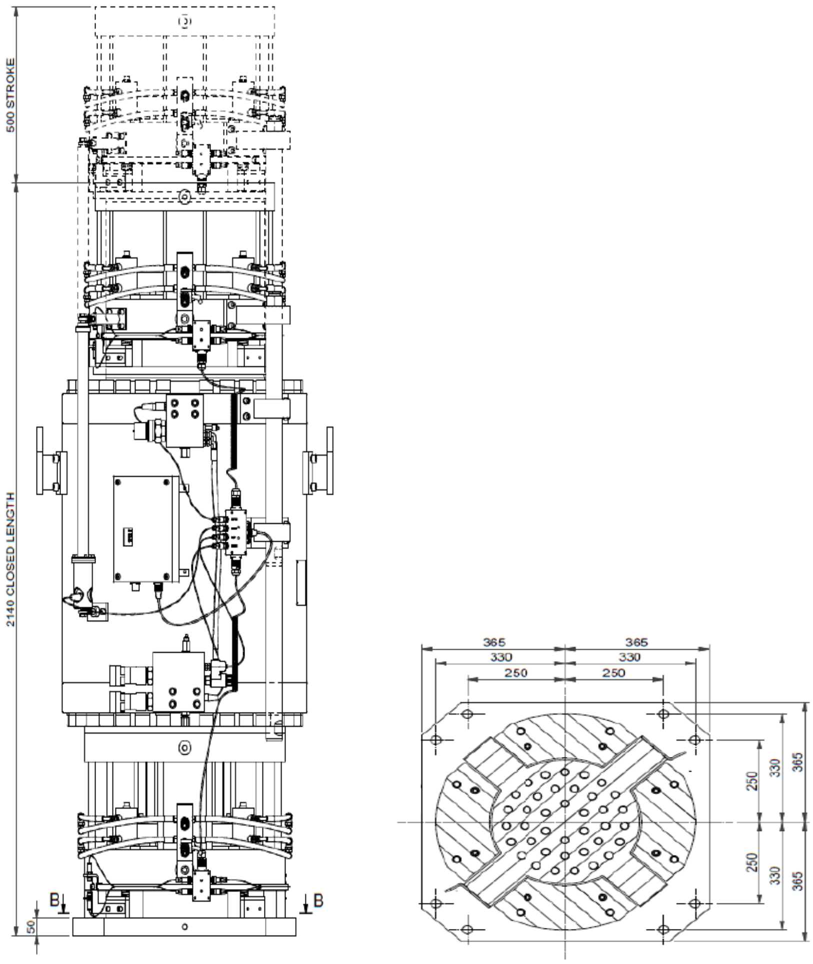


Figure B.8: Dimensions of the Dorman Long DL-S588 strand jack [9]

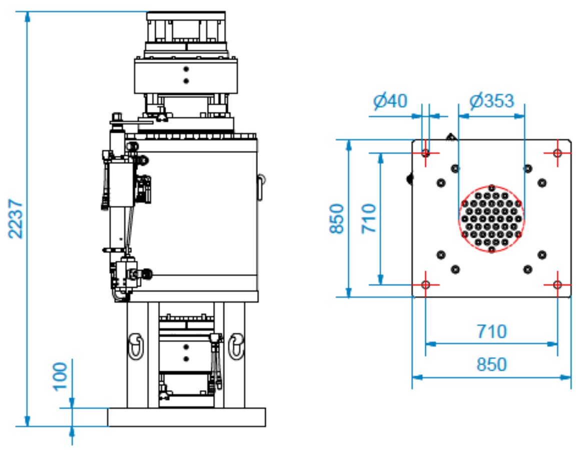


Figure B.9: Dimensions of the Enerpac HSL6500 strand jack [4]

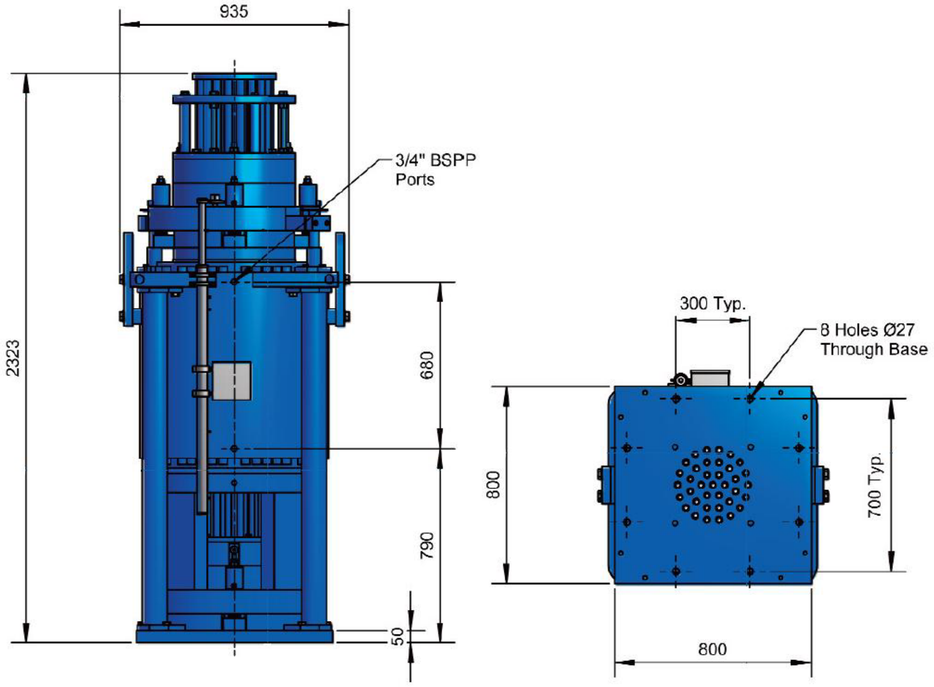
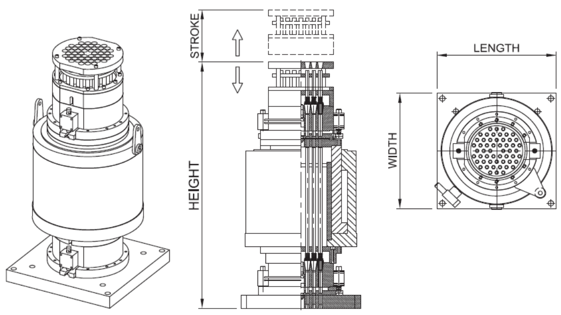


Figure B.10: Dimensions of the Fagioli L600 strand jack [10]



Strand jack					
Specifications	**SSL830 (Double Anchored)**	**SSL830**	**SSL550**	**SSL300**	**SSL100**
Capacity:	900	900	600	300	100
Strands (Qty):	54	54	36	18	7
Strand diameter:	18	18	18	18	18
Stroke:	400	400	400	400	480
Weight	4350	3850	2390	2000	940
Length:	880	880	790	550	800
Width:	880	880	790	550	500
Height:	2590	1880	1845	1560	1765

Figure B.11: Dimensions of the Mammoet SSL550 strand jack [2]

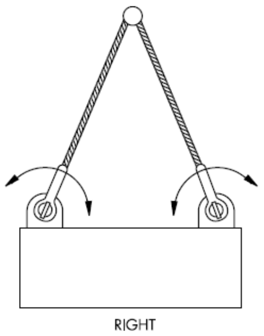


Figure B.12: Directions for assembly of shackle and load [19]

Load angle	Reduction for side loading New Working Load Limit
0 °	100% of original Working Load Limit
45 °	70% of original Working Load Limit
90 °	50% of original Working Load Limit

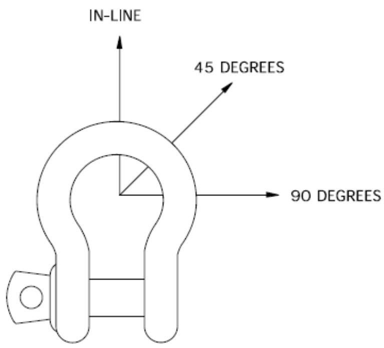
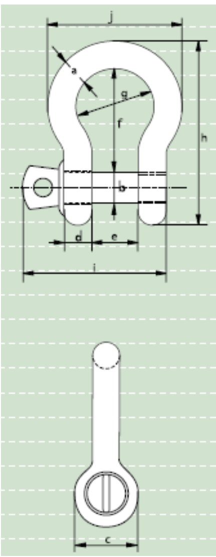


Figure B.13: Capacity reduction of shackle due to side loads [19]



working load limit	diameter bow	diameter pin	diameter eye	width eye	width inside	length inside	width bow	length	length bolt	width	weight each
	a	b	c	d	e	f	g	h	i	j	
t	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	kg
0.33	5	6	12	5	9.5	22	16	36	29.5	26	0.02
0.5	7	8	16.5	7	12	29	20	48.5	38	34	0.05
0.75	9	10	20	9	13.5	32	22	56	46.5	40	0.1
1	10	11	22.5	10	17	36.5	26	63.5	54	46	0.14
1.5	11	13	26.5	11	19	43	29	74	59.5	51	0.19
2	13.5	16	34	13	22	51	32	89	73	58	0.36
3.25	16	19	40	16	27	64	43	110	89	75	0.63
4.75	19	22	46	19	31	76	51	129	103	89	1.01
6.5	22	25	52	22	36	83	58	144	119	102	1.5
8.5	25	28	59	25	43	95	68	164	137	118	2.21
9.5	28	32	66	28	47	108	75	185	153	131	3.16
12	32	35	72	32	51	115	83	201	170	147	4.31
13.5	35	38	80	35	57	133	92	227	186	162	5.55
17	38	42	88	38	60	146	99	249	203	175	7.43
25	45	50	103	45	74	178	126	300	243	216	12.84
35	50	57	111	50	83	197	138	331	272	238	18.15
42.5	57	65	130	57	95	222	160	377	310	274	26.29
55	65	70	145	65	105	260	180	433	344	310	37.6

Figure B.14: Specifications of Green Pin shackles up to 55t [19]

C

Calculation of the deck capacity

At the time of the start of this research, the design of the MC-class is still ongoing. It is however already in an advanced stage. The assumptions and calculations made in this report are based on the design state of June 28th, 2013.

C.1. Overview of the deck structure

The deck of the MC is completely flat for maximum flexibility in loading arrangements. Below the deck surface, a grid of support structures is designed. The strong support structures are the bulkheads (Transverse Bulkhead (TB) and Longitudinal Bulkhead (LB)) and Web Frames (WF), as can be seen in Figure C.1. Transverse structures are counted as frame numbers from the stern of the vessel, longitudinal structures are counted as distance from the centerline of the vessel. An overview of the support structures and their capacities is given in Table C.1 and Figure C.1. The reinforced bulkheads are designed to have a higher capacity if the load is concentrated on 10m length of the bulkhead.

Table C.1: Properties of deck support structure

Structure	Locations	Spacing [mm]	Line load [t/m]	
			Overall	Max 10m
Transverse bulkhead	From frame 25 every 24th frame	14400	200	200
Reinforced TB	Frame 73, 97, 121, 145 and 169	14400	200	375
Longitudinal bulkhead	Centerline and every 6600mm	6600	200	200
Reinforced LB	Centerline, at 6600mm and 13200mm	6600	200	250
Webframe	From frame 1 every 4th frame	2400	50	50

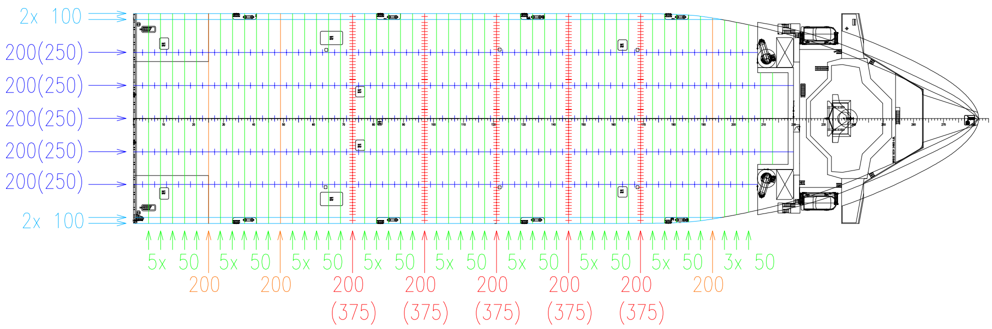


Figure C.1: Overview of the the MCs deck support structure, values are line loads in [t/m], values between parentheses indicate maximum line loads over max 10m in [t/m]

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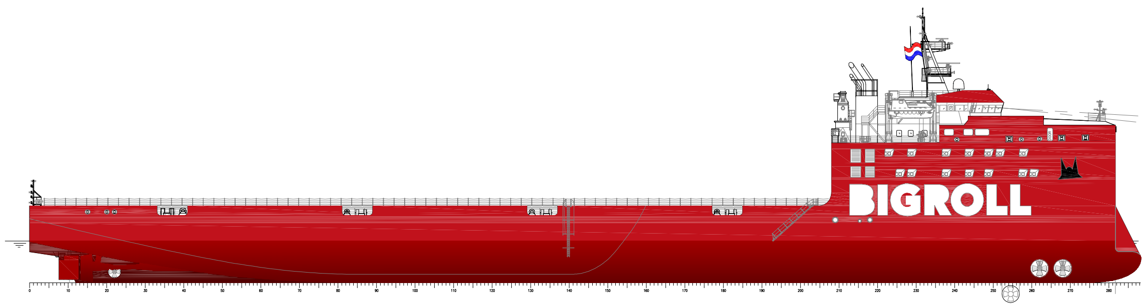
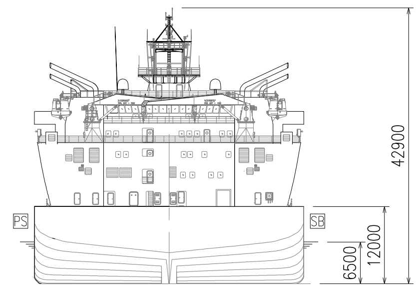
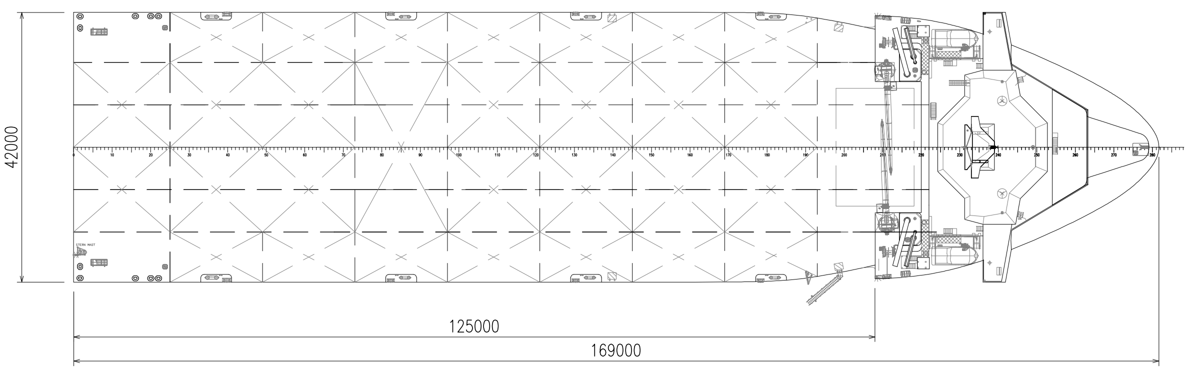
SIDE VIEW



AFT VIEW



TOP VIEW



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C.2. Loading positions

The locations of support points under the load are often determined by the manufacturer of the heavy load. The support structure on deck of the MC has to line up with these support points. Therefore the position of the beams on board needs to be flexible. Figure C.3 gives an overview of loading positions for the MC. For longitudinal loading, the complete width of the vessel can be used. Transverse loading of ultra heavy units however is due to bending of the vessel restricted to the mid section of the vessel, from frame 61 to 181. The bulkheads in these areas, both LB and TB, are reinforced with brackets (see Figure C.2).

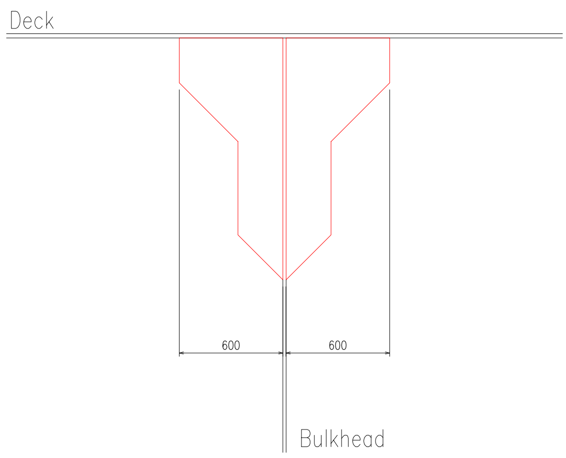
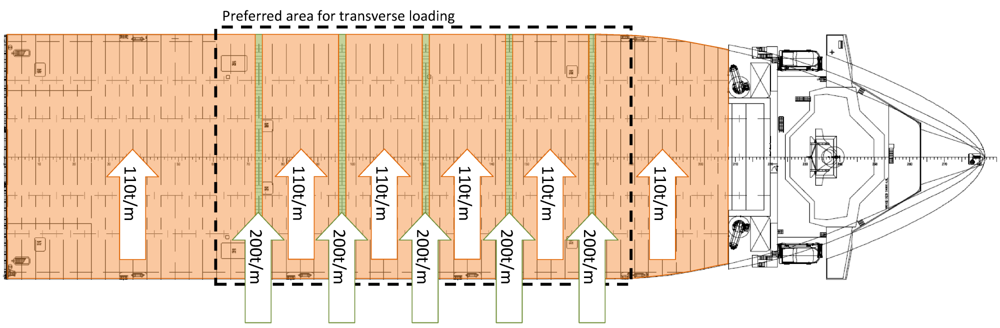
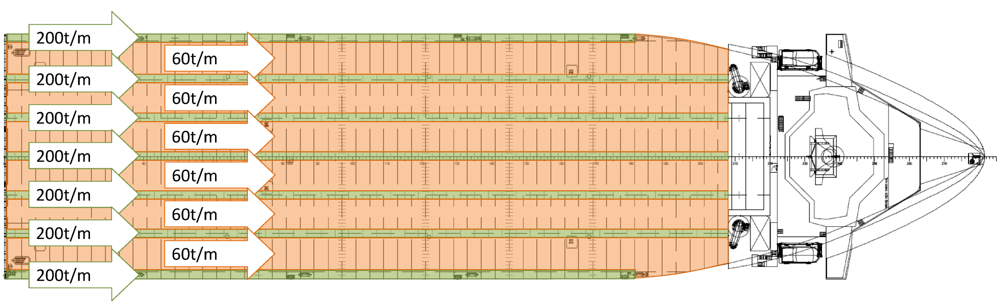


Figure C.2: Detail of brackets on a reinforced bulkhead



(a) Transverse loading positions and capacities



(b) Longitudinal loading positions and capacities

Figure C.3: Loading positions and capacities of the MC

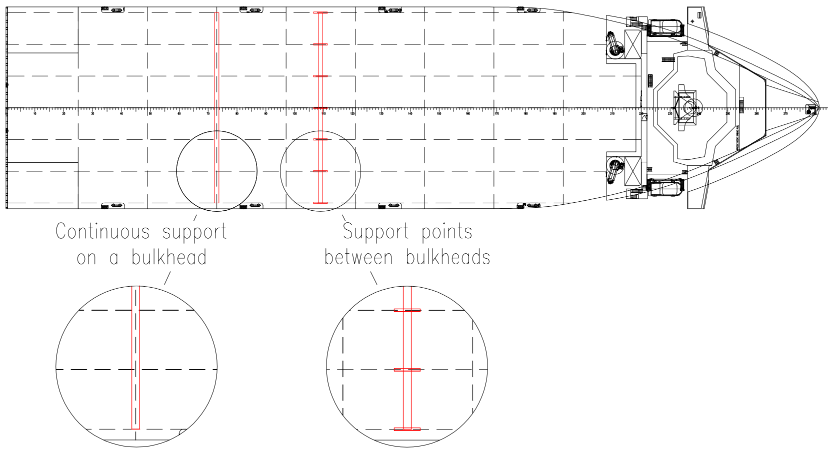


Figure C.4: Beam support arrangements for transverse loading between bulkheads and on top of a bulkhead

C.3. Maximum loads

Figure C.3 also shows the capacity of deck at the loading positions. If the beam is located directly above a bulkhead, a semi-continuous support structure can be placed under the beam, thereby using the line load of that bulkhead as capacity. If the beam however is placed between bulkheads, the beam is only supported on points where a strong deck structure crosses the path of the beam. In that case, the width of the support determines the capacity of the support point. This is clarified for transverse loading in Figure C.4, the same applies for longitudinal loading. The width of the support structure is obviously determinative for the capacity of any loading position that is not on top of a bulkhead. In Chapter 5 is is shown that sufficient load spreading can be achieved by placing HEB1000 profiles underneath the beam. For the beam design calculations, it will be assumed that the maximum load spread required can be achieved with the HEB1000 profiles and thus no special supports are needed. The load spread widths are based on an assumed load spread angle of $45°$ [15]. The maximum activated support width becomes

$$w_{sup} = 2(h_{sup} + W_{beam}) \tag{C.1}$$

The values of h_{sup} and W_{beam}, based on expected loads and loading equipment, are 1000mm and 8800mm respectively. This results in a support width $w_{sup} = 2880mm$. The capacity of the support points is now simply calculated by multiplying the width of the support and the allowable line load of the deck supporting structure as presented in Table C.1. By dividing this capacity by the distance between the support points, the allowable average line load on the beam is calculated. In formula:

$$q_{allow,avg} = \frac{q_{sup} \times w_{sup}}{l_{rep}} \tag{C.2}$$

Table C.2 gives an overview of the capacity of the support points and the resulting allowable line loads. These line loads are also visualized in Figure C.3.

Table C.2: Allowable line loads on the beam in different positions

Direction	**Position**	**q$_{sup}$ [t/m]**	**w$_{sup}$ [m]**	**d$_{sup}$ [m]**	**q$_{allow,avg}$ [t/m]**	**F$_{sup,max}$ [t]**
Transverse	On BH	200	continuous	0.6	200	-
	Between BHs	250	2.88	6.6	110	720
Longitudinal	On BH	200	continuous	1.2	200	-
	Between BHs	50	2.88	2.4	60	144

D

Applicable sections of the Eurocodes

This appendix gives an overview of applicable sections from EC0, EC1 and EC3 in a reader-friendly format, as far as that is possible. EC3 is used as a basis, relevant parts of EC0 and EC1 are copied in at logical places [16, 22, 23].

D.1. Materials

The basic design rules of EC3 are applicable for steel structures with material thicknesses $t \geq 3$mm.

D.1.1. Material properties

The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel are obtained by using the simplification given in Table 3.1 of NEN-EN 1993-1-1, which is presented in Figure D.1.

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550

Figure D.1: Relevant section of NEN-EN 1993-1-1 Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

D.1.2. Ductility requirements

The requirements on ductility follow from the national annex to NEN-EN 1993-1-1. The following values are used:

- $\frac{f_u}{f_y} \geq 1.20$
- Elongation at fracture may not be less than 15%
- $\epsilon_u > 15\epsilon_y$, where $\epsilon_y = \frac{f_y}{E}$

D.1.3. Fracture toughness

No further checks against brittle fracture are needed, because the conditions given in NEN-EN 1993-1-10 Table 2.1 are satisfied, see Figure D.2. From the national annex:

- $T_{Ed} = -20^o C$, because the construction will be used in outside environments
- For compression parts, $\sigma_{Ed} = 0.25 f_y(t)$ must be used
- $f_y(t) = f_{ynom} - 0.25 \frac{t}{t_0}$ and $t_0 = 1mm$

Steel grade	Sub-grade	Charpy energy CVN at T [°C]	Charpy energy CVN J_{min}	Reference temperature T_{Ed} [°C] $\sigma_{Ed} = 0{,}75\, f_y(t)$: 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed} = 0{,}50\, f_y(t)$: 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed} = 0{,}25\, f_y(t)$: 10	0	-10	-20	-30	-40	-50
S235	JR	20	27	60	50	40	35	30	25	20	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	90	75	60	50	40	35	30	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	125	105	90	75	60	50	40	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S275	JR	20	27	55	45	35	30	25	20	15	80	70	55	50	40	35	30	125	110	95	80	70	60	55
	J0	0	27	75	65	55	45	35	30	25	115	95	80	70	55	50	40	165	145	125	110	95	80	70
	J2	-20	27	110	95	75	65	55	45	35	155	130	115	95	80	70	55	200	190	165	145	125	110	95
	M,N	-20	40	135	110	95	75	65	55	45	180	155	130	115	95	80	70	200	200	190	165	145	125	110
	ML,NL	-50	27	185	160	135	110	95	75	65	200	200	180	155	130	115	95	230	200	200	200	190	165	145
S355	JR	20	27	40	35	25	20	15	15	10	65	55	45	40	30	25	25	110	95	80	70	60	55	45
	J0	0	27	60	50	40	35	25	20	15	95	80	65	55	45	40	30	150	130	110	95	80	70	60
	J2	-20	27	90	75	60	50	40	35	25	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	110	90	75	60	50	40	35	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	155	130	110	90	75	60	50	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	95	80	65	55	45	35	30	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	135	115	95	80	65	55	45	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	70	60	50	40	30	25	20	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	90	70	60	50	40	30	25	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	105	90	70	60	50	40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	125	105	90	70	60	50	40	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	150	125	105	90	70	60	50	200	180	155	130	110	95	75	215	200	200	200	175	155	130
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

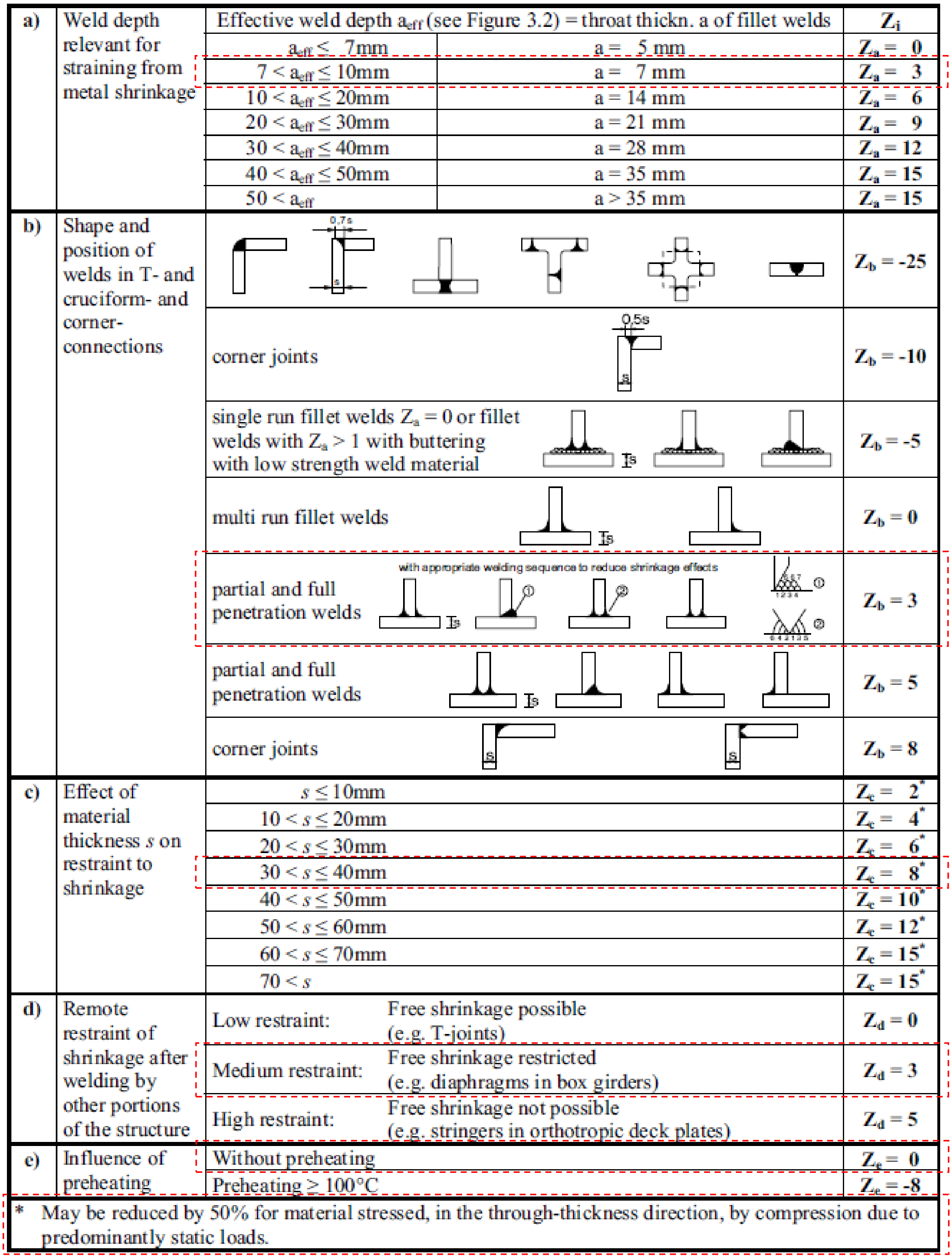
Figure D.2: NEN-EN 1993-1-10 Table 2.1: Maximum permissible values of element thickness t in mm

D.1.4. Through thickness properties

The quality class of the material used should at least have a design Z-value of Z15, see Figure D.3. The value for Z_{Ed}=15, which is the sum of Z_a to Z_e according to NEN-EN 1993-1-10 Formula (3.2) and NEN-EN 1993-1-10 Table 3.2, which is presented in Figure D.4.

Target value of Z_{Ed} according to EN 1993-1-10	Required value of Z_{Rd} expressed in terms of design Z-values according to EN 10164
$Z_{Ed} \leq 10$	—
$10 < Z_{Ed} \leq 20$	Z 15
$20 < Z_{Ed} \leq 30$	Z 25
$Z_{Ed} > 30$	Z 35

Figure D.3: NEN-EN 1993-1-1 Table 3.2: Choice of quality class according to EN 10164



a)	Weld depth relevant for straining from metal shrinkage	Effective weld depth a_{eff} (see Figure 3.2)	= throat thickn. a of fillet welds	Z_i
		$a_{eff} \leq 7mm$	a = 5 mm	$Z_a = 0$
		$7 < a_{eff} \leq 10mm$	a = 7 mm	$Z_a = 3$
		$10 < a_{eff} \leq 20mm$	a = 14 mm	$Z_a = 6$
		$20 < a_{eff} \leq 30mm$	a = 21 mm	$Z_a = 9$
		$30 < a_{eff} \leq 40mm$	a = 28 mm	$Z_a = 12$
		$40 < a_{eff} \leq 50mm$	a = 35 mm	$Z_a = 15$
		$50 < a_{eff}$	a > 35 mm	$Z_a = 15$
b)	Shape and position of welds in T- and cruciform- and corner-connections			$Z_b = -25$
		corner joints		$Z_b = -10$
		single run fillet welds $Z_a = 0$ or fillet welds with $Z_a > 1$ with buttering with low strength weld material		$Z_b = -5$
		multi run fillet welds		$Z_b = 0$
		partial and full penetration welds	with appropriate welding sequence to reduce shrinkage effects	$Z_b = 3$
		partial and full penetration welds		$Z_b = 5$
		corner joints		$Z_b = 8$
c)	Effect of material thickness s on restraint to shrinkage	$s \leq 10mm$		$Z_c = 2^*$
		$10 < s \leq 20mm$		$Z_c = 4^*$
		$20 < s \leq 30mm$		$Z_c = 6^*$
		$30 < s \leq 40mm$		$Z_c = 8^*$
		$40 < s \leq 50mm$		$Z_c = 10^*$
		$50 < s \leq 60mm$		$Z_c = 12^*$
		$60 < s \leq 70mm$		$Z_c = 15^*$
		$70 < s$		$Z_c = 15^*$
d)	Remote restraint of shrinkage after welding by other portions of the structure	Low restraint:	Free shrinkage possible (e.g. T-joints)	$Z_d = 0$
		Medium restraint:	Free shrinkage restricted (e.g. diaphragms in box girders)	$Z_d = 3$
		High restraint:	Free shrinkage not possible (e.g. stringers in orthotropic deck plates)	$Z_d = 5$
e)	Influence of preheating	Without preheating		$Z_e = 0$
		Preheating $\geq 100°C$		$Z_e = -8$

* May be reduced by 50% for material stressed, in the through-thickness direction, by compression due to predominantly static loads.

Figure D.4: NEN-EN 1993-1-1 Table 3.2: Criteria affecting the target value of Z_{Ed}

D.1.5. Material coefficients

The material coefficients to be adopted in calculations for the structural steels covered by EC3 are presented in Table D.1.

Table D.1: Material coefficients according to NEN-EN 1993-1-1

Property	**Value**
Young's modulus	E=210000N/mm^2
Shear modulus	$G=\frac{E}{2(1+\nu)}$ ≈81000N/mm^2
Poisson's ratio in elastic stage	ν=0.3
Coefficient of linear thermal expansion	α=12×10^{-6} K^{-1} (for $T \leq 100^oC$)

D.2. Connections made with pins

The design of pin-hole connections is performed according to NEN-EN 1993-1-8.

D.2.1. General

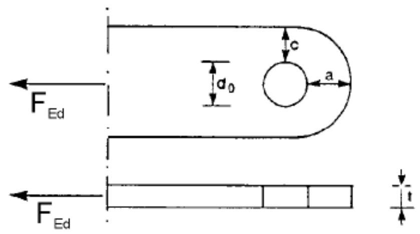
The pin-hole connection must satisfy:

- The pin must be secured if there is a risk of pins becoming loose
- The geometry of the unstiffened element that contains a hole for the pin should satisfy the requirements given in Figure D.5
- Pin connected members should be arranged such to avoid excentricity

D.2.2. Design of pin ended members

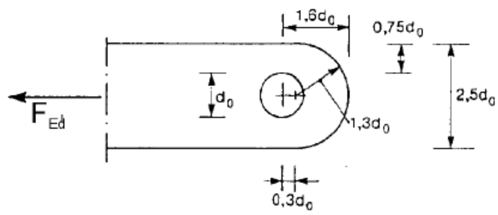
The geometrical requirements for pin ended members follow from NEN-EN 1993-1-8 Table 3.9, which is presented in Figure D.5

Type A: Given thickness t



$$a \geq \frac{F_{Ed}\gamma_{M0}}{2\,t\,f_y} + \frac{2\,d_0}{3} \quad : \quad c \geq \frac{F_{Ed}\gamma_{M0}}{2\,t\,f_y} + \frac{d_0}{3}$$

Type B: Given geometry



$$t \geq 0{,}7\sqrt{\frac{F_{Ed}\gamma_{M0}}{f_y}} \quad : \quad d_0 \leq 2{,}5\,t$$

Figure D.5: NEN-EN 1993-1-8 Table 3.9: Geometrical requirements for pin ended members

D.2.3. Design of pins

The design criteria for solid circular pins follow from NEN-EN 1993-1-8 Table 3.10, which is presented in Figure D.6

Failure mode	Design requirements
Shear resistance of the pin	$F_{v,Rd} = 0{,}6\, A\, f_{up} / \gamma_{M2} \geq F_{v,Ed}$
Bearing resistance of the plate and the pin	$F_{b,Rd} = 1{,}5\, t\, d\, f_y / \gamma_{M0} \geq F_{b,Ed}$
If the pin is intended to be replaceable this requirement should also be satisfied.	$F_{b,Rd,ser} = 0{,}6\, t\, d\, f_y / \gamma_{M6,ser} \geq F_{b,Ed,ser}$
Bending resistance of the pin	$M_{Rd} = 1{,}5\, W_{el}\, f_{yp} / \gamma_{M0} \geq M_{Ed}$
If the pin is intended to be replaceable this requirement should also be satisfied.	$M_{Rd,ser} = 0{,}8\, W_{el}\, f_{yp} / \gamma_{M6,ser} \geq M_{Ed,ser}$
Combined shear and bending resistance of the pin	$\left[\frac{M_{Ed}}{M_{Rd}}\right]^2 + \left[\frac{F_{v,Ed}}{F_{v,Rd}}\right]^2 \leq 1$
d is the diameter of the pin;	
f_y is the lower of the design strengths of the pin and the connected part;	
f_{up} is the ultimate tensile strength of the pin;	
f_{yp} is the yield strength of the pin;	
t is the thickness of the connected part;	
A is the cross-sectional area of a pin.	

Figure D.6: NEN-EN 1993-1-8 Table 3.10: Design criteria for pin connections

D.3. Durability

The durability of a steel structure is defined in NEN-EN 1990, and is described as follows: The structure shall be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance. The indicative design working life is indicated in Figure D.7. In order to achieve an adequately durable structure, the following should be taken into account according to NEN-EN 1990:

- The intended or foreseeable use of the structure
- The required design criteria
- The expected environmental conditions
- The composition, properties and performance of the materials and products
- The properties of the soil
- The choice of the structural system
- The shape of members and the structural detailing
- The quality of workmanship, and the level of control
- The particular protective measures
- The intended maintenance during the design working life

D.4. Structural analysis

D.4.1. Structural modelling for analysis

Structural modelling and basic assumptions

Analysis is based upon calculation models of the structure that are appropriate for the limit state under consideration.

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures [(1)]
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.		

Figure D.7: NEN-EN 1990 Table 2.1: Indicative design working life

Joint modelling
The effect of the joint may be assumed to have no effect on the structural analysis, because the joint is classified as continuous according to NEN-EN 1993-1-8 Table 5.1 (see Figure D.8.

Method of global analysis	Classification of joint		
Elastic	~~Nominally pinned~~	Rigid	Semi-rigid
Rigid-Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-Plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and partial-strength Semi-rigid and full-strength Rigid and partial-strength
Type of joint model	Simple	Continuous	Semi-continuous

Figure D.8: NEN-EN 1993-1-8 Table 5.1: Type of joint model

Ground-structure interaction
Account should be taken of the deformation characteristics of the supports where significant.

D.4.2. Global analysis

The internal forces and moments may be determined using the first order analysis, using the initial geometry of the structure, if the following criterion is satisfied (NEN-EN 1993-1-1, Formula (5.1)): $\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$

D.4.3. Methods of analysis considering material non-linearities

Elastic global analysis is used in all cases, according to NEN-EN 1993-1-1. The analysis should be based on the assumption that the stress-strain behaviour of the material is linear, whatever the stress level is.

D.4.4. Finite element methods of analysis

The criteria for FEM analysis follow from NEN-EN 1993-1-5 Annex C.

Modeling

- The choice of finite element models and the size of the mesh determine the accuracy of results. For validation sensitivity checks with successive refinement may be carried out

- The finite element modelling may be carried out for the structure as a whole or a substructure as a part of the whole structure
- The boundary conditions for supports, interfaces and applied loads should be chosen such that results are conservative
- Geometric properties should be taken as nominal

Documentation
The mesh size, loading, boundary conditions and other input data as well as the output should be documented in a way that they can be reproduced by third parties.

Loads
The loads applied to the structures should include relevant load factors and load combination factors. For simplicity a single load multiplier α may be used.

Limit state criteria
The ultimate limit state criteria should be used as follows:

- for structures susceptible to buckling: attainment of the maximum load
- for regions subjected to tensile stresses: attainment of a limiting value of 5% of the principal membrane strain

D.4.5. Classification of cross sections

The role of cross section classification is to identify the extent to which the resistance and rotation capacity of cross sections is limited by its local buckling resistance. Four classes of cross-sections are defined:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

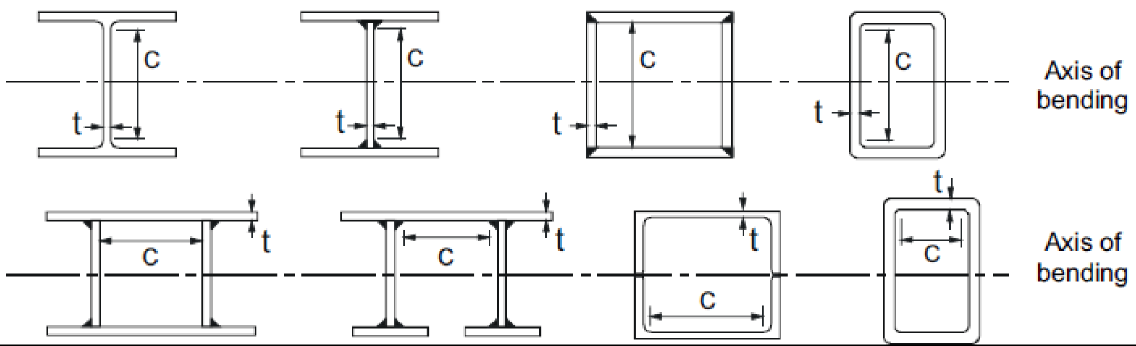
A cross section is classified according to the highest (least favourable) class of its parts. The class of each part of the cross section is determined using NEN-EN 1993-1-1 Table 5.2, which is presented in Figure D.9 and D.10.

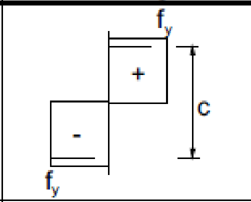
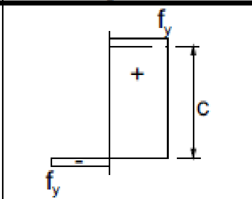
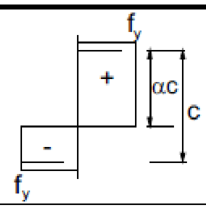
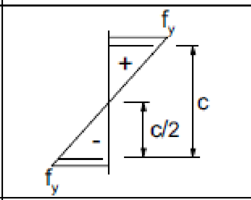
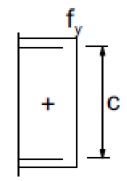
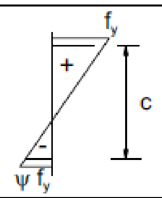
D.5. Ultimate limit states

D.5.1. General

Ultimate limit states are the limit states that concern the safety of people and/or the structure, according to NEN-EN 1990. The partial factors γ_M that should be applied to the various characteristic values of resistance for the ultimate limit states are defined in the National Annex to NEN-EN 1993-6 Table 6.1, as presented in Figure D.11.

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1^{*)}$: $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Figure D.9: NEN-EN 1993 Table 5.2: Maximum width-to-thickness ratios for compression parts (sheet 1)

D.5.2. Resistance of cross sections

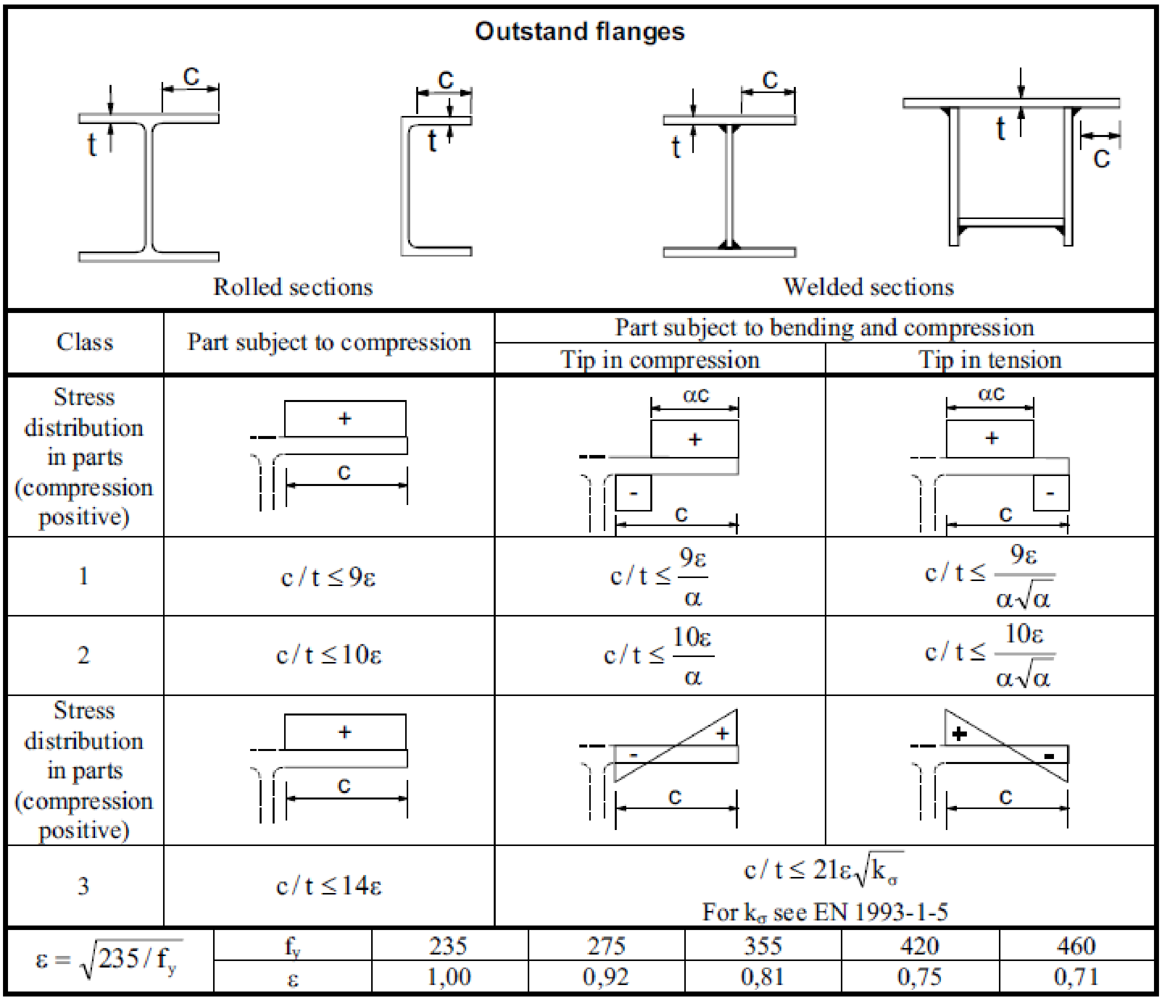
Tension

The design value of the tension force N_{Ed} at each cross section should satisfy Formula (D.1).

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \tag{D.1}$$

Where N_{Ed} = Design normal force

$N_{t,Rd}$ = Design tension resistance



Outstand flanges

Rolled sections | Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Figure D.10: NEN-EN 1993 Table 5.2: Maximum width-to-thickness ratios for compression parts (sheet 2)

The design tension resistance should be calculated with Formula (D.2).

$$N_{t,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{D.2}$$

Where $N_{t,Rd}$ = Design tension resistance
A = Area
f_y = Yield strength
γ_M = Partial factor for resistance of cross sections

Compression

The design value of the compression force N_{Ed} at each cross section should satisfy Formula (D.3).

$$\frac{N_{Ed}}{N_{c,Rd}} \le 1.0 \tag{D.3}$$

Where N_{Ed} = Design normal force
$N_{c,Rd}$ = Design compression resistance

The design compression resistance should be calculated with Formula (D.4).

Resistance of	**Value of relevant partial factor** γ
a) members and cross-section	
Resistance of cross-sections to excessive yielding including local buckling	$\gamma_{M0} = 1,00$
Resistance of members to instability assessed by member checks	$\gamma_{M1} = 1,00$
Resistance of cross-sections in tension to fracture	$\gamma_{M2} = 1,25$
b) joints	
Resistance of bolts Resistance of rivets Resistance of pins at ultimate limit states Resistance of welds Resistance of plates in bearing	$\gamma_{M2} = 1,25$
Slip resistance: — at ultimate limit state (category C) — at serviceability limit state (category B)	 $\gamma_{M3} = 1,25$ $\gamma_{M3,ser} = 1,10$
Bearing resistance of an injection bolt	$\gamma_{M4} = 1,00$
Resistance of joints in hollow section lattice girders	$\gamma_{M5} = 1,00$
Resistance of pins at serviceability limit states	$\gamma_{M6,ser} = 1,00$
Preload of high strength bolts	$\gamma_{M7} = 1,10$

Figure D.11: National Annex to NEN-EN 1993-6 Table 6.1: Partial factors for resistance

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{D.4}$$

Where $N_{c,Rd}$ = Design compression resistance
A = Area
f_y = Yield strength
γ_M = Partial factor for resistance of cross sections

Bending moment

The design value of the bending moment M_{Ed} at each cross section should satisfy Formula (D.5).

$$\frac{M_{Ed}}{M_{Rd}} \leq 1.0 \tag{D.5}$$

Where M_{Ed} = Design bending moment
M_{Rd} = Design bending resistance

The design bending resistance should be calculated with Formula (D.6).

$$M_{Rd} = \frac{W_{el} f_y}{\gamma_{M0}} \tag{D.6}$$

Where M_{Rd} = Design bending resistance
W_{el} = Design elastic section modulus
f_y = Yield strength
γ_M = Partial factor for resistance of cross sections

Shear

The design value of the shear force V_{Ed} at each cross section should satisfy Formula (D.7).

$$\frac{V_{Ed}}{V_{Rd}} \leq 1.0 \tag{D.7}$$

Where V_{Ed} = Design shear force
V_{Rd} = Design shear resistance

The shear area A_v for box sections should be calculated with Formula (D.8).

$$A_v = \eta \sum h_{web} t_{web} \tag{D.8}$$

Where A_v = Shear area
η = 1.20 (according to NEN-EN 1993-1-5)
h_{web} = Heigth of the web
t_{web} = Thickness of the webs

For verifying the design elastic shear resistance V_{Rd}, Formula (D.9) may be used for a critical point of the cross section.

$$\frac{\tau_{Ed}}{f_y / \left(\sqrt{3}\gamma_{M0}\right)} \leq 1.0 \tag{D.9}$$

$$\tau_{Ed} = \frac{V_{Ed} S}{It} \tag{D.10}$$

Where τ_{Ed} = Design shear stress
f_y = Yield strength
γ_M = Partial factor for resistance of cross sections
V_{Ed} = Design shear force
S = First moment of inertia
I = Second moment of inertia
t = Plate thickness

Torsion

The design value of the torsional moment T_{ed} at each cross section should satisfy Formula (D.11).

$$\frac{T_{ed}}{T_{Rd}} \leq 1.0 \tag{D.11}$$

Where T_{ed} = Design torsional moment
T_{Rd} = Design torsional resistance

The total torsional moment T_{ed} should be considers as the sum of two internal effects according to Formula (D.12).

$$T_{ed} = T_{t,Ed} + T_{w,Ed} \tag{D.12}$$

Where T_{ed} = Design torsional moment
$T_{t,Ed}$ = Internal St. Vernant torsion
$T_{w,Ed}$ = Internal warping torsion

D.6. Serviceability limit states

D.6.1. General

The verification of serviceability limit states is described in NEN-EN 1990 and should be based on criteria concerning the following aspects:

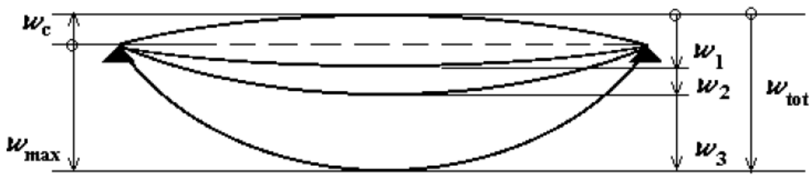
- deformations that affect the appearance or the functioning of the structure
- vibrations that limit the functional effectiveness of the structure
- damage that is likely to adversely affect the appearance or the functioning of the structure

D.6.2. Vertical deflections

Definitions of deflections are presented in the National Annex to NEN-EN 1990, these are shown in Figure D.12. The maximum value of w_{tot} is defined by Formula (D.13).

$$w_{tot,max} = \frac{l_{rep}}{150} \tag{D.13}$$

Where w_{tot} = Total vertical deflection
l_{rep} = Unsupported length



Key :

w_c Precamber in the unloaded structural member
w_1 Initial part of the deflection under permanent loads of the relevant combination of actions according to expressions (6.14a) to (6.16b)
w_2 Long-term part of the deflection under permanent loads
w_3 Additional part of the deflection due to the variable actions of the relevant combination of actions according to expressions (6.14a) to (6.16b)
w_{tot} Total deflection as sum of w_1 , w_2 , w_3
w_{max} Remaining total deflection taking into account the precamber

Figure D.12: National Annex to NEN-EN 1990 Figure A1.1: Definitions of vertical deflections

D.7. Actions on structures

The actions on structures are extracted from NEN-EN 1990 [22].

D.7.1. Design situations
The relevant design situations shall be selected taking into account the circumstances under which the structure is required to fulfil its function. Design situations shall be classified as follows:

- persistent design situations, which refer to the conditions of normal use
- transient design situations, which refer to temporary conditions applicable to the structure, e.g. during execution or repair
- accidental design situations, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact or the consequences of localised failure
- seismic design situations, which refer to conditions applicable to the structure when subjected to seismic events

D.7.2. Basic variables
Classification of actions
Actions shall be classified by their variation in time as follows :

- permanent actions (G), e.g. self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements
- variable actions (Q), e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads
- accidental actions (A), e.g. explosions, or impact from vehicles

Actions shall also be classified

- by their origin, as direct or indirect
- by their spatial variation, as fixed or free
- by their nature and/or the structural response, as static or dynamic

Characteristic values of actions
The characteristic value F_k of an action is its main representative value and shall be specified as a mean value, an upper or lower value, or a nominal value. The self-weight of the structure may be represented by a single characteristic value and be calculated on the basis of the nominal dimensions and mean unit masses. For variable actions, the characteristic value (Q_k) shall correspond to either an upper value with an intended probability of not being exceeded or a nominal value, which may be specified in cases where a statistical distribution is not known.

D.8. Partial factor method for verification
D.8.1. General
For the design in this research, the following rules of NEN-EN 1990 apply for the partial factor method:

- When using the partial factor method, it shall be verified that, in all relevant design situations, no relevant limit state is exceeded when design values for actions or effects of actions and resistances are used in the design models
- For the selected design situations and the relevant limit states the individual actions for the critical load cases should be combined as detailed in this section. However actions that cannot occur simultaneously, for example due to physical reasons, should not be considered together in combination
- Design values should be obtained by using the characteristic or other representative values, in combination with partial and other factors as defined in this section and EN 1991 to EN 1999

Table D.2: ψ values used for this research (Conservative approach)

Category E: Storage areas	
ψ_0	1.0
ψ_1	0.9
ψ_2	0.8

D.8.2. Design values of actions

The design value F_d of an action F can be expressed in general terms according to (6.1a) and (6.1b) of NEN-EN 1990 as Formula (D.14). The values for ψ_i in EC0 are only available for buildings, for the application in this research Category E actions (with least reduction of actions) will be used for a conservative estimation. The resulting ψ values are presented in Table D.2.

$$F_d = \gamma_f F_{rep} \tag{D.14}$$
$$F_{rep} = \psi F_k \tag{D.15}$$

Where F_d = Design value of an action
F_k = Characteristic value of an action
F_{rep} = Representative value of an action
γ_f = Partial factor for an action
ψ = either 1.00 or ψ_0, ψ_1 or ψ_2

D.8.3. Ultimate limit states

The following ultimate limit states are verified as relevant for this research:

- EQU: Loss of static equilibrium of the structure or any part of it considered as a rigid body, where minor variations in the value or the spatial distribution of actions from a single source are significant and the strengths of construction materials or ground are generally not governing
- STR: Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs

The following ultimate limit states are verified as not relevant for this research:

- GEO : Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance
- FAT : Fatigue failure of the structure or structural members

Verifications

When considering a limit state of static equilibrium of the structure (EQU), it shall be verified that Formula (D.16) is valid.

$$E_{d,dst} \leq E_{d,stb} \tag{D.16}$$

Where $E_{d,dst}$ = Design value of the effect of destabilising actions
$E_{d,stb}$ = Design value of the effect of stabilising actions

When considering a limit state of rupture or excessive deformation of a section, member or connection (STR), it shall be verified that Formula (D.17) is valid.

$$E_d \leq R_d \tag{D.17}$$

Where E_d = Design value of the effect of actions
R_d = Design value of the resistance to the corresponding action

Combination of actions
For each critical load case, the design values of the effects of actions (E_d) shall be determined by combining the values of actions that are considered to occur simultaneously. Each combination of actions should include a leading variable action. For this research, the design value of the effect of actions is written in Formula (D.18). The combination of actions in brackets {..} may be expressed as Formula (D.19) for EQU, or alternatively for STR as the less favourable of Formula (D.20) and (D.21).

$$E_d = E\{\gamma_{G,j}G_{k,j}; \gamma Q, 1Q_{k,1}; \gamma_{Q,i}\psi_{0,i}Q_{k,i}\} \quad j \geq 1; i > 1 \tag{D.18}$$

$$\sum_{j\geq 1} \gamma_{G,j}G_{k,j}" + "\gamma_{Q,1}Q_{k,1}" + "\sum_{i>1} \gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{D.19}$$

$$\sum_{j\geq 1} \gamma_{G,j}G_{k,j}" + "\gamma_{Q,1}\psi_{0,1}Q_{k,1}" + "\sum_{i>1} \gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{D.20}$$

$$\sum_{j\geq 1} \xi_j\gamma_{G,j}G_{k,j}" + "\gamma_{Q,1}Q_{k,1}" + "\sum_{i>1} \gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{D.21}$$

Design values of actions
The design values of actions for serviceability limit states are given in Figure D.13 and D.14.

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}\,Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{Gj,sup} = 1{,}10$

$\gamma_{Gj,inf} = 0{,}90$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with the following set of recommended values. The recommended values may be altered by the National annex.

$\gamma_{Gj,sup} = 1{,}35$

$\gamma_{Gj,inf} = 1{,}15$

$\gamma_{Q,1} = 1{,}50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1{,}50$ where unfavourable (0 where favourable)

provided that applying $\gamma_{Gj,inf} = 1{,}00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Figure D.13: NEN-EN 1990 Table A1.2(A): Design values of actions (EQU)

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions (*)	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable	Action	Main	Others
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National annex. In case of 6.10a and 6.10b, the National annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b.
$\gamma_{Gj,sup} = 1,35$
$\gamma_{Gj,inf} = 1,00$
$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)
$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)
$\xi = 0,85$ (so that $\xi\gamma_{Gj,sup} = 0,85 \times 1,35 \cong 1,15$).
See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.

NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self weight of the structure may be considered as coming from one source ; this also applies if different materials are involved.

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,05 to 1,15 can be used in most common cases and can be modified in the National annex.

Figure D.14: NEN-EN 1990 Table A1.2(B): Design values of actions (STR)

D.8.4. Serviceability limit states

Verifications

For the serviceability limit states, it shall be verified according to NEN-EN 1990 that Formula (D.22) is valid.

$$E_d \leq C_d \tag{D.22}$$

Where E_d = Design value of the effect of actions
C_d = Design value of the effect of actions specified in the serviceability criterion

Combination of actions

The combinations of actions to be taken into account in the relevant design situations should be appropriate for the serviceability requirements and performance criteria being verified. The characteristic combination of actions for serviceability limit states is defined symbolically by Formula (D.23) in which the combination of actions in brackets {..} is expressed as Formula (D.24). The frequent combination is defined by Formula (D.25) and {(D.26)} and the quasi-permanent combination by Formula (D.27) and {(D.28)}. For serviceability limit states the partial factors γ_M for the properties of materials are taken as 1.0.

$$E_d = E\{G_{k,j}; Q_{k,1}; \psi_{0,i}Q_{k,i}\} \quad j \geq 1; i > 1 \tag{D.23}$$

$$\sum_{j\geq 1} G_{k,j}\text{"} + \text{"}Q_{k,1}\text{"} + \text{"}\sum_{i>1} \psi_{0,i}Q_{k,i} \tag{D.24}$$

$$E_d = E\{G_{k,j}; \psi_{1,1}Q_{k,1}; \psi_{2,i}Q_{k,i}\} \quad j \geq 1; i > 1 \tag{D.25}$$

$$\sum_{j\geq 1} G_{k,j}\text{"} + \text{"}\psi_{1,1}Q_{k,1}\text{"} + \text{"}\sum_{i>1} \psi_{2,i}Q_{k,i} \tag{D.26}$$

$$E_d = E\{G_{k,j}; \psi_{2,i}Q_{k,i}\} \quad j \geq 1; i > 1 \tag{D.27}$$

$$\sum_{j\geq 1} G_{k,j}\text{"} + \text{"}\sum_{i\geq 1} \psi_{2,i}Q_{k,i} \tag{D.28}$$

Design values of actions

The design values of actions for serviceability limit states are given in Figure D.15.

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

Figure D.15: NEN-EN 1990 Table A1.4: Design values of actions for use in the combination of actions

E

Concept development

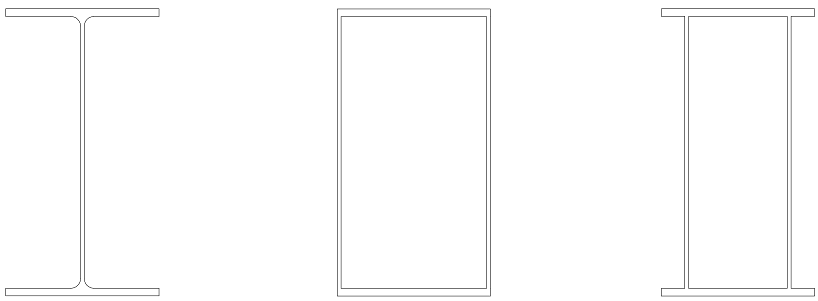
In this appendix, an elaboration will be presented on the design choices made per sub-concept that lead to the best option per sub-concept. This elaboration is divided into sections per sub-concept.

E.1. Beam shape

The options considered are a standard H-profile and two box girder shapes constructed from steel plates, as is presented in Figure E.1. These beam shapes are examined on criteria for stability and bending of the upper flange. In order to do so, an estimate of the basic geometry is required. The geometry is determined with the criteria due to the different functions, the maximum loads following from the load cases and the material properties in Chapter 4.

E.1.1. Bending moment in the flanges

The offset of the webs from the centerline of the beam, and thereby the shape of the beam, is determined using the maximum bending moment in the flanges due to the functions of the beam. Especially skid systems produce high loads at different offsets from the centerline of the system. For the other functions, steel structures or contact plates are used to spread the load over the width of the beam. Figure E.2 shows the different geometries and loads that cause bending of the top flange of the beam. The magnitude of the load (F or q) combined with the offset (x) from the webs creates a local bending moment in the top flange of the beam, according to Formula (E.1) to (E.4). Which formula is valid for each situation is shown in Figure E.3.



(a) H-beam (b) Box beam (c) Box beam with offset of webs

Figure E.1: Cross sections of several options for the shape of the modular beam

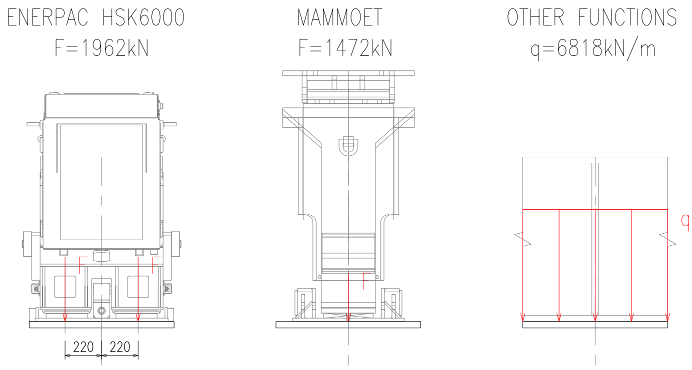


Figure E.2: Loads introduced on the top flange of the beam

$$M = Fx \tag{E.1}$$

$$M = \frac{Fx}{2} \tag{E.2}$$

$$M_1 = -\frac{qx^2}{2} \tag{E.3}$$

$$M_2 = R\left(\frac{R}{2q} - l_{out}\right) \tag{E.4}$$

$$R = \frac{qw_{flange}(w_{flange} - 2l_{out})}{2l_{rep}} \tag{E.5}$$

Where	M_i	=	Bending moment, in [Nmm]
	F	=	Concentrated load, in [N]
	q	=	Distributed load, in [N/mm]
	x	=	Distance, in [mm]
	R	=	Reaction force, in [N]
	l_{out}	=	Length of outstand, in [mm]
	w_{flange}	=	Width of the flange, in [mm]
	l_{rep}	=	Unsupported length, in [mm]

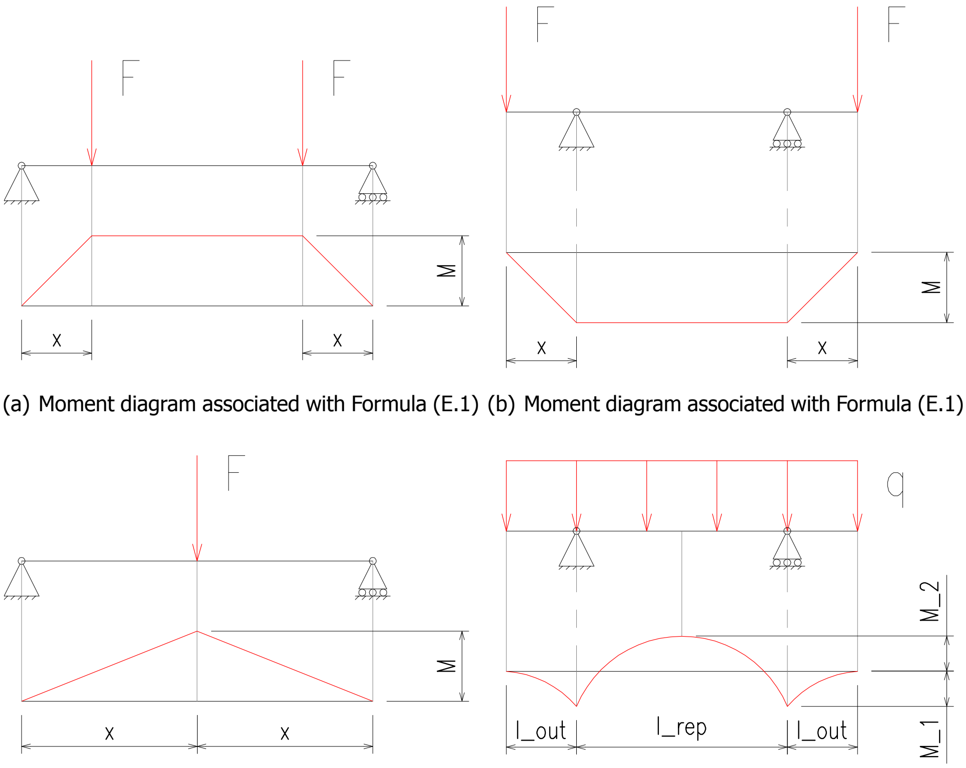
The values of the loads are given, but the position of the webs supporting the top flange determines the moment that the top flange encounters. The relation between the offset of the webs and the resulting bending moments for the two skid systems and the other functions is shown in Figure E.4.

E.1.2. Determination of the basic geometry

The outer dimensions of the beam follow from the functions skid support beam and grillage:

- Width of the beam: W_{beam} = 880mm
- Height of the beam: H_{beam} = 1320mm

A width-to-height ratio of 1:3 for the area enclosed by the webs and flanges is indicated to provide a structure with a good strength-to-mass ratio [17]. For a beam height between 1200mm and 1400mm, the offset of the webs should therefore be 200mm to 233mm from the centerline of the beam. The design choice is made to take a web offset of 220mm from the centerline of the beam. By doing so, the bending moment in the top flange due to the preferred skid system is reduced to zero, while the bending moment due to other functions than skidding is near its minimum value. Only skidding with the Mammoet skid system, which is used less and less, creates a high bending moment in the top flange. For that system, special provisions need to be taken, as will be shown in Appendix F. By implementing the width-to-height ratio to the web offset of 220mm, the beam height becomes H_{beam}=3×(2×



(a) Moment diagram associated with Formula (E.1) (b) Moment diagram associated with Formula (E.1)

(c) Moment diagram associated with Formula (E.2) (d) Moment diagram associated with Formula (E.3) and (E.4)

Figure E.3: Moment diagrams associated with Formula (E.1) to (E.4)

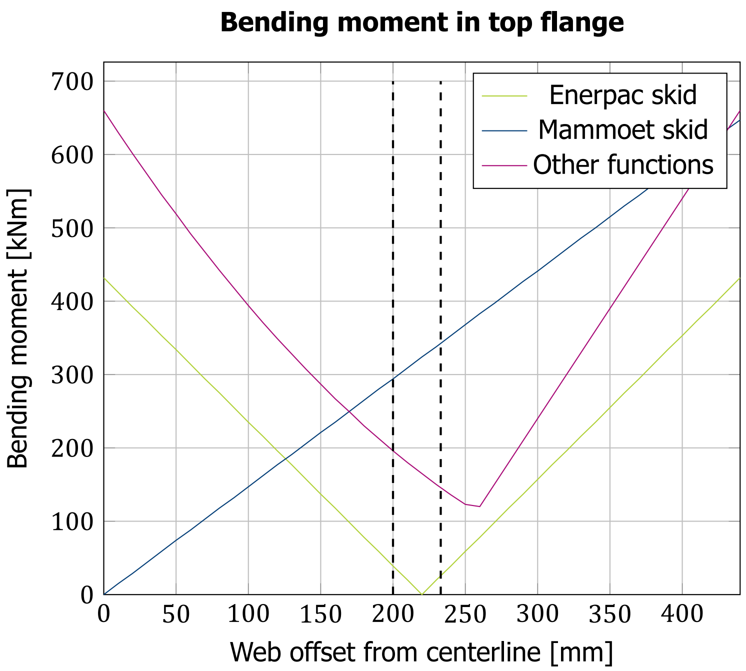


Figure E.4: Bending moments in the top flange due to the different skid systems and other functions

220)=1320mm. The plate thickness of the webs is determined using the criteria and material properties on shear. The maximum shear force occurring due to the functions is in load case 3 (Table 4.3), with a value of V=5886kN. Using Formula (E.6) for maximum shear stress in the cross section of the beam, the minimum required shear area A_v can be calculated by taking τ_{max}=τ_{allow}=205N/mm^2 from the material properties and γ_{SF}=1.5 from EC3. Then A_v=64602mm^2 minimum to stay within the limits for shear stress.

$$\tau_{max} = 1.5 \times \gamma_{SF} \frac{V}{A_v} \tag{E.6}$$

Where τ_{max} = Maximum shear stress, in [N/mm^2]
γ_{SF} = Safety factor, [-]
V = Shear force, in [N]
A_v = Shear area, in [mm^2]

The plate thickness of the flanges follows from the criteria and material properties on bending. The maximum bending moment occurring due to the functions is in load case 4 (Table 4.3), with a value of M=9712kNm. Using Formula E.7 for normal stress in the cross section of the beam, the minimum required second moment of inertia can be calculated if σ=σ_y=355N/mm^2. y=$\frac{1}{2}H_{beam}$ for a symmetric beam, which gives y=660mm.

$$\sigma = \gamma_{SF} \frac{My}{I} \tag{E.7}$$

Where σ = Normal stress, in [N/mm^2]
γ_{SF} = Safety factor, [-]
M = Bending moment, in [Nmm]
y = Distance from neutral line to extreme fibre, in [mm]
I = Second moment of inertia, in [mm^4]

Now A_v, I and h_{web} are defined as

$$A_v = h_{web} t_{web} \tag{E.8}$$
$$I = \frac{1}{12} W_{beam} H_{beam}{}^3 - \frac{1}{12}(W_{beam} - 2t_{web}) h_{web}{}^3 \tag{E.9}$$
$$h_{web} = H_{beam} - 2t_{flange} \tag{E.10}$$

Where A_v = Shear area, in [mm^2]
h_{web} = Heigth of the web, in [mm]
t_{web} = Thickness of the webs, in [mm]
I = Second moment of inertia, in [mm^4]
W_{beam} = Width of the beam, in [mm]
H_{beam} = Heigth of the beam, in [mm]
t_{flange} = Thickness of the flanges, in [mm]

After a couple of iterations, an estimate of the web and flange thickness can be found:

- t_{flange} = 30mm
- t_{web} = 30mm (in case of two webs, t_{web} = 60mm in case of one web)
- h_{web} = 1260mm

A summary of this estimate of basic dimensions is given in Figure E.5.

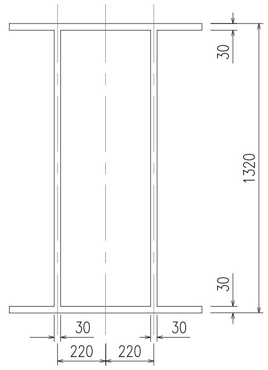
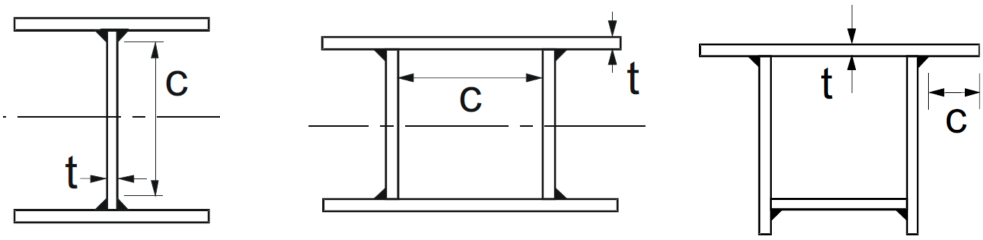


Figure E.5: Estimation of the basic geometry of cross section of the modular beam



(a) Stability parameters for the webs (b) Stability parameters for the midsection of the flanges (c) Stability parameters for the outstand of the flanges

Figure E.6: Stability parameters for different beam sections [16]

E.1.3. Stability of the beam geometry

The stability of the basic beam geometry is assessed according to EC3. For calculation of stability, the ratio between the plate thickness t and plate width c are used. The relevant cases for the determination of t and c are shown in Figure E.6. This gives the following classification of the cross section of the beam:

- Webs: $\frac{c}{t} = 42$, which implies for a part subject to bending: class 1
- Midsection of flanges: $\frac{c}{t} = 14$, which implies for a part subject to compression: class 1
- Outstand of flanges: $\frac{c}{t} = 7$, which implies for a part subject to compression: class 1

The overall beam classification is thus class 1. This means that the cross section of the beam can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of resistance. In other words, the global stability of the beam with these parameters is ensured, even without stiffeners [16].

E.2. Section length

Four combinations of beam lengths are considered (see also Figure E.7):

- Only sections of 9900mm long
- Only sections of 10200mm long
- A combination of sections 11400mm long and half sections of 5700mm
- A combination of sections 11400mm long and 'half' sections of 5400mm

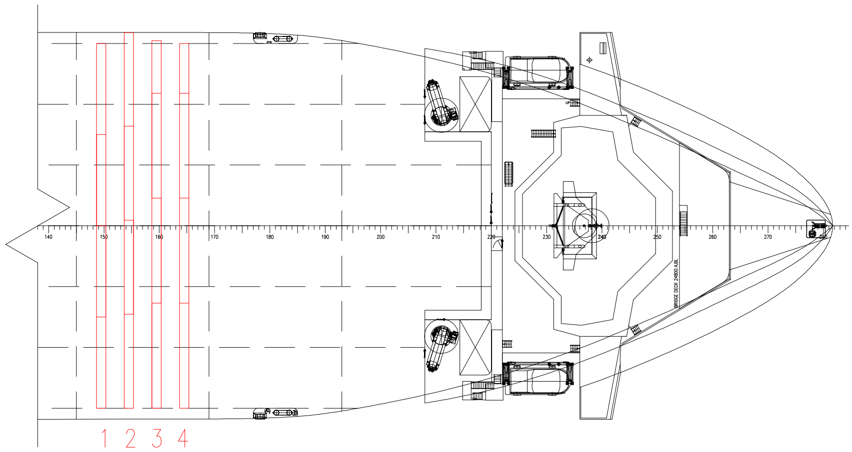


Figure E.7: Fit of the section length options on the deck of the MC: Sections of 9900mm (No.1), 10200mm (No.2), 11400mm+5700mm (No.3) and 11400mm+5400mm (No.4)

E.2.1. Selection method

For the selection of the section length, *Pugh's method* [18] will be used. This is an effective method for comparing alternatives relative to each other in their ability to meet the applicable criteria. The decision criteria are carefully selected, to represent all design requirements without covering requirements twice. At the start a reference concept is chosen to be used as datum, the other concepts are scored compared to this datum. After a scoring round, the best concept is chosen as datum in the next round until all other alternatives return a negative total score. The scoring options are presented in Table E.1.

Table E.1: Scoring options for Pugh's method

Score	Meaning
-1	Worse than datum
0	Same as datum
1	Better than datum

E.2.2. Scoring criteria

Transverse fit on deck

All section lengths considered fit the deck of the MC in transverse direction. Support locations, such as longitudinal bulkheads or longitudinal girders, are located at centerline and at a distance of 6600mm, 13200mm, 19800mm and 21000mm from centerline of the vessel, see Figure E.8. A combination of section lengths that adds up to $2 \times 19800mm = 39600mm$ is considered optimal, this leaves 1200mm on both sides of the deck for connecting a link beam.

Multiples of 600mm

Support locations underneath the beam are determined by the deck support structure of the MC. In transverse direction, the interval is 6600mm. In longitudinal direction, the interval between strong points is 1200mm. The largest common denominator between those intervals is 600mm. Section lengths that are multiples of 600mm are considered optimal.

Maximized within container

The beam sections are transported in standard shipping containers of 20ft and 40ft, approximately 6000mm and 12000mm respectively. To make optimal use of the space within the container, and moreover to create the largest span possible with one beam section within the container limits, a section length that is close to the limits is considered optimal.

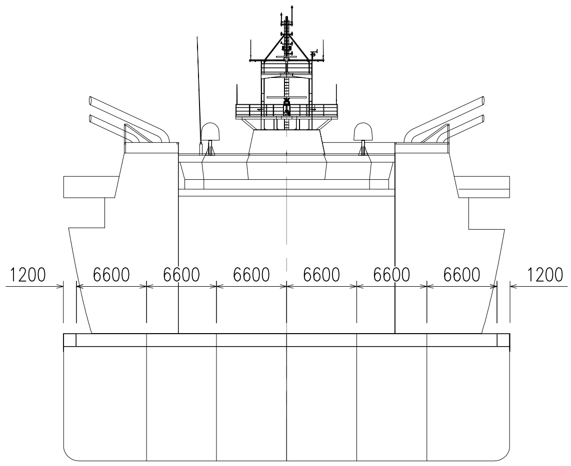


Figure E.8: Distances between longitudinal deck support structures

Flexibility in beam length
The beam will be multi functional, which implies that different lengths are required for different applications. A larger diversity in possible beam lengths by combining different sections, is considered better for the design of the modular beam.

E.2.3. Selection of section length

Pugh's method: first iteration
The decision matrix for the first iteration of the selection of the section length can be found in Table E.2. Length option 11400mm+5700mm is chosen as datum.

Table E.2: Pugh's method, iteration 1: datum is '11400+5700'

Criterion	9900	10200	11400+5700	11400+5400
Transverse fit on deck	1	0	0	1
Multiples of 600mm	0	1	0	1
Maximized within container	-1	-1	0	0
Flexibility in beam length	-1	-1	0	0
TOTAL	-1	-1	0	2

Pugh's method: second iteration
The decision matrix for the second iteration of the selection of the section length can be found in Table E.3. Length option 11400mm+5400mm had the best results in the first iteration, it is chosen as datum in this iteration.

Table E.3: Pugh's method, iteration 2: datum is '11400+5400'

Criterion	9900	10200	11400+5700	11400+5400
Transverse fit on deck	0	-1	-1	0
Multiples of 600mm	-1	0	-1	0
Maximized within container	-1	-1	0	0
Flexibility in beam length	-1	-1	0	0
TOTAL	-3	-3	-2	0

E.3. Connections between sections

Options for the connections between beam sections are several implementations of a pin-hole connection, bolted connections, connections using clamps, container twist locks and shape fit connections as is shown in Figure E.9. Because of the large number of options, the connections will be grouped into the basic working principle of the connection for the analysis in this section.

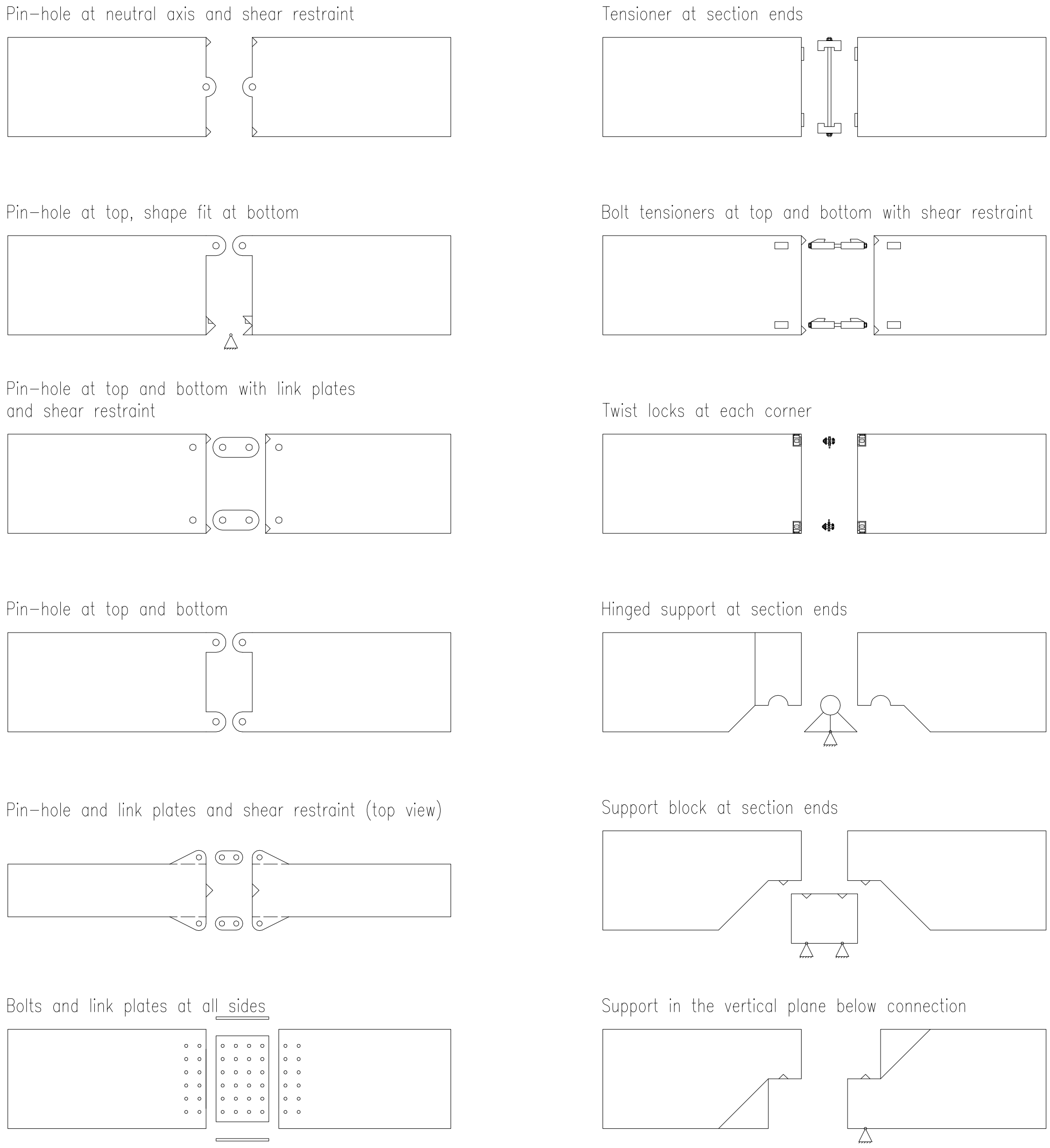


Figure E.9: Options for connecting the beam sections

E.3.1. Selection method

For the selection of the best connection option, the same method is used as for the selection of the section length: *Pugh's method*. For the selection process, not all connections shown in Figure E.9 will be analyzed individually. Instead they will be grouped according to their connection type: pin-hole connections, bolted connections, connections using clamps, twist lock connections and shape fit connections. The final design of the chosen connection type will be determined in a later stadium of this research.

E.3.2. Scoring criteria

Low number of parts

Most connection options require additional parts to establish the connection between two sections, such as the pin in a pin-hole connection. The criterion of a low number of parts yields two advantages. The first advantage is that less parts require less handling, which means less installation time. The second advantage is that the risk of loosing parts is low for a system with little loose elements. A connection which requires no additional parts is considered optimal.

Easy installation

The amount of effort needed to establish the connection between beam sections is judged by this criterion. A beam system that is easy to install and remove and is therefore convenient for the people who have to work with the system. Note that this criterion is based on the amount of actions required to establish the connection and the effort to perform these actions, not on the amount of parts. No effort, apart from lifting the beam section, is considered optimal.

Flat top and bottom

The modular beam needs a flat top to accommodate the skid systems, the bottom must be flat to be able to lay on deck or on the ground surface without supports. A connection which has no protrusions though the top and bottom flange is considered optimal.

Independent of supports

The locations of the supports of the beam are not fixed, because of the deck support structure of the MC. Therefore the connection should not need a support to establish or close the connection. A connection that is independent of the location of the supports is considered best.

Resistance to bending

Resistance to bending is defined as the capability of the connection to transfer the tension and compression forces in the connection due to bending of the beam. Due to the irregular support locations underneath beam sections, the connection can be located on the point with the highest bending moment. Therefore the resistance to bending loads of the connection must be at least equal to the bending resistance of the beam. A connection option with the capability to resist these bending loads without creating a gap between the beam sections is considered best.

Resistance to shear

For some functions, the beam is loaded with moving loads. This, in combination with irregular support locations can create high shear loads in the connection plane. These shear forces must be resisted by the connection. As the previous criterion, the shear resistance of the connection must be at least equal to the shear resistance of the beam. A connection type with the capability to resist these shear forces without creating height difference between the beam sections is considered best.

E.3.3. Selection of the connection

Pugh's method: first iteration

The decision matrix for the first iteration of the selection of the connection type can be found in Table E.4. The shape fit connection is chosen as datum for this iteration.

Pugh's method: second iteration

The decision matrix for the second iteration of the selection of the connection type can be found in Table E.5. Both the pin-hole and clamped connections had equal results as the datum in the previous iteration. In this iteration, the clamped connection is chosen as datum.

Pugh's method: third iteration

The decision matrix for the third iteration of the selection of the connection type can be found in Table E.6. The pin-hole connection had the best result in the previous iteration, it is chosen as datum for this iteration.

Table E.4: Pugh's method, iteration 1: datum is 'Shape fit'

Criterion	Pin-hole	Bolts	Clamps	Twist locks	Shape fit
Low number of parts	-1	-1	-1	-1	0
Easy installation	-1	-1	-1	-1	0
Flat top and bottom	0	-1	0	0	0
Independent of supports	1	1	1	1	0
Resistance to bending	1	1	1	0	0
Resistance to shear	0	0	0	0	0
TOTAL	0	-1	0	-1	0

Table E.5: Pugh's method, iteration 2: datum is 'Clamps'

Criterion	Pin-hole	Bolts	Clamps	Twist locks	Shape fit
Low number of parts	0	-1	0	0	1
Easy installation	1	0	0	1	1
Flat top/bottom	0	-1	0	0	0
Independent of supports	0	0	0	0	-1
Resistance to bending	0	0	0	-1	-1
Resistance to shear	0	0	0	0	0
TOTAL	1	-2	0	0	0

Table E.6: Pugh's method, iteration 3: datum is 'Pin-hole'

Criterion	Pin-hole	Bolts	Clamps	Twist locks	Shape fit
Low number of parts	0	-1	0	0	1
Easy installation	0	-1	-1	0	0
Flat top/bottom	0	-1	0	0	0
Independent of supports	0	0	0	0	-1
Resistance to bending	0	0	0	-1	-1
Resistance to shear	0	0	0	0	0
TOTAL	0	-3	-1	-1	-1

F

Structural design

In this appendix, the detailed geometry of the beam and corresponding dimensions will be determined. This appendix is divided into sections, in which every section describes a part of the cross section. These parts are the webs, flanges, longitudinal stiffeners, transverse stiffeners for the closed part of the beam cross section (frames) and transverse stiffeners for the outstand of the flanges. Although stiffeners are not required for global stability of the beam, it will be shown that they are needed for resistance of local load introduction.

F.1. General design information

In practice, the structural design of the beam parts is complicated, because the design of one part influences not only the beam design as a whole, but also the design of other individual parts. This is illustrated in Figure 6.1. The dimensions presented in this appendix are the results of several iterations in the design scheme shown in the figure.

F.1.1. Global requirements

The required global design values for shear area (A_v) and second moment of inertia (I) follow from rewriting Formula (E.6) and (E.7) into Formula (F.1) and (F.3) respectively. This results in:

- Required shear area: A_v= 64615mm^2
- Required second moment of inertia: I= 2.71$\times 10^{10}$mm^4

$$A_v = 1.5 \times \frac{V_{Ed}}{\tau_{max}} \tag{F.1}$$

$$V_{Ed} = \gamma_{SF} V \tag{F.2}$$

Where A_v = Shear area, in [mm^2]
γ_{SF} = Safety factor, [-]
V = Shear force, in [N]
τ_{max} = Maximum shear stress, in [N/mm^2]

$$I = \frac{M_{Ed} y}{\sigma_y} \tag{F.3}$$

$$M_{Ed} = \gamma_{SF} M \tag{F.4}$$

Where I = Second moment of inertia, in [mm^4]
γ_{SF} = Safety factor, [-]
M = Bending moment, in [Nmm]
y = Distance from neutral line to extreme fibre, in [mm]
σ_y = Yield stress, in [N/mm^2]

F.1.2. Combination rules

The combination rules for shear area and second moment of inertia of the different beam parts are elaborated in Formula (F.5) and (F.6) respectively [24].

$$A_v = \sum_{i=2}^{3} A_{v,i} \tag{F.5}$$

Where A_v = Shear area, in [mm^2]
2, 3 = Webs, longitudinal stiffeners

$$I = \sum_{i=1}^{3} I_i A_i (\overline{y} - \overline{y}_i)^2 \tag{F.6}$$

$$\overline{y} = \frac{\sum_{i=1}^{3} \overline{y}_i A_i}{\sum_{i=1}^{3} A_i} \tag{F.7}$$

Where I = Second moment of inertia, in [mm^4]
A = Area, in [mm^2]
$\overline{y}$ = Location of the neutral axis in z-direction, in [mm]
1, 2, 3 = Flanges, webs, longitudinal stiffeners

F.1.3. Unity checks

The unity checks for the different beam parts are calculated with Formula (F.30) and (F.31). A unity check of 1 means that the resistance of the structure is exactly equal to the required strength to deal with the design loads.

$$\frac{M_{Ed}}{M_{Rd}} \leq 1 \tag{F.8}$$

$$\frac{V_{Ed}}{V_{Rd}} \leq 1 \tag{F.9}$$

$$M_{Rd} = \frac{I\sigma_y}{\overline{y}} \times 10^{-6} \tag{F.10}$$

$$V_{Rd} = A_v \tau_{allow} \times 10^{-3} \tag{F.11}$$

Where	M_{Ed}	=	Design bending moment, in [kNm]
	M_{Rd}	=	Design bending resistance, in [kNm]
	V_{Ed}	=	Design shear force, in [kN]
	V_{Rd}	=	Design shear resistance, in [kN]
	I	=	Second moment of inertia, in [mm^4]
	σ_y	=	Yield stress, in [N/mm^2]
	$\overline{y}$	=	Location of the neutral axis in z-direction, in [mm]
	A_v	=	Shear area, in [mm^2]
	τ_{allow}	=	Allowable shear stress, in [N/mm^2]

F.2. Web design

The design of the webs is based on the global requirements on shear (see Formula (F.14)). The thickness of the webs depends on the beam height and flange thickness according to Formula (F.12). The calculation of t_{flange} will be performed in Section F.3. $A_{v,LS}$ will be determined in Section F.4. H_{beam} is in Chapter 5 determined to be 1320mm. The resulting section properties of the webs are summarized in Table F.1.

F.2.1. Design formulas

$$t_{web} = \frac{A_{v,webs}}{h_{web}} \tag{F.12}$$

$$h_{web} = H_{beam} - 2t_{flange} \tag{F.13}$$

$$A_{v,webs} = A_v - A_{v,LS} \tag{F.14}$$

$$I_{webs} = 2 \times \frac{1}{12} t_{web} {h_{web}}^3 \tag{F.15}$$

$$\overline{y}_{webs} = t_{flange} + \frac{h_{web}}{2} \tag{F.16}$$

Where	$A_{v,webs}$	=	Shear area of the webs, in [mm^2]
	A_v	=	Shear area, in [mm^2]
	$A_{v,LS}$	=	Shear area of the longitudinal stiffener, in [mm^2]
	t_{web}	=	Thickness of the webs, in [mm]
	h_{web}	=	Heigth of the web, in [mm]
	H_{beam}	=	Heigth of the beam, in [mm]
	t_{flange}	=	Thickness of the flanges, in [mm]

F.2.2. Section properties

F.2.3. Unity checks

The webs are only part of the global resistance to bending and shear. Therefore, the unity checks for global shear and bending are calculated when the entire geometry is known.

Table F.1: Section properties of the webs

Property	Value			Unit
Plate thickness	t_{web}	=	20	mm
Plate height	h_{web}	=	1250	mm
Area	A_{webs}	=	50000	mm^2
Shear area	$A_{v,webs}$	=	50000	mm^2
Second moment of inertia	I_{webs}	=	6.51×10^9	mm^4
Location of neutral axis	$\overline{y}_{webs}$	=	660	mm
Bending resistance	M_{Rd}	=	3502	kNm
Shear resistance	V_{Rd}	=	10250	kN

F.3. Flange design

The design of the flanges is determined using the global requirements on bending. The flange is assumed not to contribute to the shear area of the beam. The thickness of the flanges is calculated using the second moment of inertia. As stated at the begin of this appendix, the total second moment of inertia is built up from the combination of individual second moment of inertias of each individual part. The thickness of the flanges is determined by solving the the flange part of Formula (F.17), which is in fact Formula (F.3) in rewritten form. The required second moment of inertia (I) and the part about the webs are already determined in previous sections of this appendix, the part of the longitudinal stiffener will be determined in Section F.4.

F.3.1. Design formulas

$$I_{flanges} + A_{flanges}(\overline{y} - \overline{y}_{flanges})^2 = I - \sum_{i=2}^{3}(I_i + A_i(\overline{y} - \overline{y}_i)^2 \quad \text{(F.17)}$$

$$I_{flanges} = 2 \times \frac{1}{12} w_{flange} t_{flange}{}^3 \quad \text{(F.18)}$$

$$A_{flanges} = w_{flange} t_{flange} \quad \text{(F.19)}$$

$$\overline{y}_{flanges} = \frac{t_{flange}}{2} \quad \text{(F.20)}$$

Where			
	I_i	=	Second moment of inertia of the respective part, in [mm^4]
	A_i	=	Cross sectoin area of the flanges of the respective part, in [mm^2]
	$\overline{y}_i$	=	Location of the neutral axis in z-direction of the respective part, in [mm]
	w_{flange}	=	Width of the flange, in [mm]
	t_{flange}	=	Thickness of the flanges, in [mm]
	2, 3	=	Webs, longitudinal stiffeners

F.3.2. Section properties

Table F.2: Section properties of the flanges

Property	Value			Unit
Plate thickness	t_{flange}	=	30	mm
Plate width	w_{flange}	=	880	mm
Cross section area	$A_{flanges}$	=	61600	mm^2
Second moment of inertia	$I_{flanges}$	=	2.52×10^7	mm^4
Location of neutral axis	$\overline{y}_{flanges}$	=	17.5	mm
Bending resistance	M_{Rd}	=	511	kNm

F.3.3. Unity checks
The flanges are only a part of the global resistance to bending. Therefore, the unity checks for global bending and shear are calculated when the entire geometry is determined.

F.4. Design of longitudinal stiffeners
Based on the EC3 criteria on the design of steel structures, no stiffeners are required to ensure stability of the beam. However, due to local load introduction of the Mammoet skid system and grillage directly on top of a bulkhead at the centerline of the beam, reinforcements are needed to prevent local failure of the flanges. Because of the fact that both local load introductions are line loads, the design choice is made to construct longitudinal stiffeners at the centerline at both flanges over the entire length of the beam, to create more local bending and shear resistance. The loads are transferred from the longitudinal stiffener to the webs by transverse frames, which will be determined in Section F.5.

F.4.1. Loads
The design load on the longitudinal stiffener is taken as the maximum value of the two situations multiplied with a γ_{SF}. This yields the shear and bending requirements as stated in Table F.3.

Table F.3: Loads on the longitudinal stiffeners

Load introduction	Line load [kN/m]	l_{rep} [m]	V_{Ed}' [kN]	M_{Ed}' [kNm]
Mammoet skid system	1308	1.2	1177	353
Grillage on bulkhead	1817	1.2	1635	491

F.4.2. Requirements
This results in the following requirements for the cross section of the longitudinal stiffener:

- Required local shear area: A_v'= 11966mm^2
- Required local second moment of inertia: I'= 3.52$\times 10^8$mm^4

F.4.3. Design formulas
Now the stiffener dimensions are determined using these local requirements on shear and bending and Formula (F.21) to (F.26).

$$I' = I_{LS} + A_{LS}(\overline{y}' - \overline{y}_{LS}{}')^2 + I_{Eff,flange} + A_{Eff,flange}(\overline{y}' - \overline{y}_{flanges}{}')^2 \quad \text{(F.21)}$$

$$I_{LS} = \frac{1}{12} t_{LS} h_{LS}{}^3 \quad \text{(F.22)}$$

$$A_v{}' = A_{LS} = t_{LS} h_{LS} \quad \text{(F.23)}$$

$$\overline{y}_{LS}{}' = \frac{1}{2} h_{LS} \quad \text{(F.24)}$$

$$\overline{y}' = \frac{A_{LS}\overline{y}_{LS}{}' + A_{Eff,flange}\overline{y}_{flanges}{}'}{A_{LS} + A_{Eff,flange}} \quad \text{(F.25)}$$

$$I_{Eff,flange} = \frac{1}{12} w_{eff} t_{flange}{}^3 \quad \text{(F.26)}$$

$$A_{Eff,flange} = w_{eff} t_{flange} \quad \text{(F.27)}$$

$$w_{eff} = \frac{1}{8} l_{rep} \quad \text{(F.28)}$$

$$\overline{y}_{flanges}{}' = h_{LS} + \frac{1}{2} t_{flange} \quad \text{(F.29)}$$

Where
I' = Second moment of inertia for local calculations of the stiffener, in [mm^4]
I_i = Second moment of inertia of the respective part, in [mm^4]
A_i = Area of the respective part, in [mm^2]
A_v = Shear area for local stiffener calculations, in [mm^2]
$\overline{y}_i{}'$ = Location of the neutral axis in z-direction of the respective part, in [mm]
t_{LS} = Thickness of the longitudinal stiffeners, in [mm]
h_{LS} = Height of the longitudinal stiffeners, in [mm]
t_{flange} = Thickness of the flanges, in [mm]
w_{eff} = Effective width of the flange from centerline of the stiffener, in [mm]
l_{rep} = Unsupported length, in [mm]

F.4.4. Section properties

The results of the calculations after several iterations are shown in Table F.4.

Table F.4: Section properties of the longitudinal stiffeners

Property	Value			Unit
Plate thickness	t_{LS}	=	40	mm
Plate height	h_{LS}	=	360	mm
Area	A_{LS}	=	28800	mm^2
Shear area	$A_{v,LS}$	=	28800	mm^2
Second moment of inertia	I'	=	3.93×10^8	
	I_{LS}	=	3.11×10^8	mm^4
Location of neutral axis	$\overline{y}_{LS}$	=	215	mm
Bending resistance	M_{Rd}	=	514	kNm
Shear resistance	V_{Rd}	=	5904	kN

F.4.5. Unity checks

The unity checks for local bending and shear in this section are

$$\frac{M_{Ed}}{M_{Rd}} = 0.98 \quad \text{(F.30)}$$

$$\frac{V_{Ed}}{V_{Rd}} = 0.83 \quad \text{(F.31)}$$

Both unity checks return a value smaller than 1, so the design resistance is high enough according to EC3.

F.5. Design of transverse frames

In order to transfer the loads from the longitudinal stiffener to the webs, transverse frames are implemented. The frames, also known as transverse stiffeners, are installed between the webs against the flanges. If the upper and lower transverse stiffener are connected and form one frame, it also helps to maintain the shape of the beam.

F.5.1. Loads and requirements

The design load on the transverse stiffener is a concentrated load imposed by the longitudinal stiffener or a external line load if the beam is used as grillage. The concentrated load due to the longitudinal stiffener is equal to the reaction force of the longitudinal stiffener when loaded with its maximum load. This yields the shear and bending requirements as stated in Table F.5.

Table F.5: Loads on the transverse stiffeners

Load introduction	**Load**	l_{rep} **[m]**	V' **[kN]**	M' **[kNm]**
Longitudinal stiffener	2180kN	0.44	1635	360
Grillage	3344kN/m	0.44	1104	121

This results in the following requirements for the cross section of the longitudinal stiffener:

- Required local shear area: A_v'= 7976mm^2
- Required local second moment of inertia: I'= 2.62$\times 10^8$mm^4

F.5.2. Design formulas

The stiffener dimensions are now determined using these local requirements on shear and bending and Formula (F.32) to (F.37).

$$I' = I_{TS} + A_{TS}(\bar{y}' - \bar{y}_{TS}')^2 + I_{Eff,flange} + A_{Eff,flange}(\bar{y}' - \bar{y}_{flanges}')^2 \tag{F.32}$$

$$I_{TS} = \frac{1}{12} t_{TS} h_{TS}^{3} \tag{F.33}$$

$$A_v' = A_{TS} = t_{TS} h_{TS} \tag{F.34}$$

$$\bar{y}_{TS}' = \frac{1}{2} h_{TS} \tag{F.35}$$

$$\bar{y}' = \frac{A_{TS}\bar{y}_{TS}' + A_{Eff,flange}\bar{y}_{flanges}'}{A_{TS} + A_{Eff,flange}} \tag{F.36}$$

$$I_{Eff,flange} = \frac{1}{12} w_{eff} t_{flange}^{3} \tag{F.37}$$

$$A_{Eff,flange} = w_{eff} t_{flange} \tag{F.38}$$

$$w_{eff} = \frac{1}{8} l_{rep} \tag{F.39}$$

$$\bar{y}_{flanges}' = h_{TS} + \frac{1}{2} t_{flange} \tag{F.40}$$

Where I' = Second moment of inertia for local calculations of the stiffener, in [mm^4]
I_i = Second moment of inertia of the respective part, in [mm^4]
A_i = Area of the respective part, in [mm^2]
A_v = Shear area for local stiffener calculations, in [mm^2]
$\bar{y}_i'$ = Location of the neutral axis in z-direction of the respective part, in [mm]
t_{LS} = Thickness of the longitudinal stiffeners, in [mm]
h_{LS} = Height of the longitudinal stiffeners, in [mm]
t_{flange} = Thickness of the flanges, in [mm]
w_{eff} = Effective width of the flange from centerline of the stiffener, in [mm]
l_{rep} = Unsupported length, in [mm]

F.5.3. Section properties

The results after several iterations of the design loop are shown in Table F.6. Bending and shear resistance of the transverse stiffener is only expressed as local values, because they do not account for the global resistance.

Table F.6: Section properties of the transverse stiffeners

Property	Value			Unit
Plate thickness	t_{TS}	=	30	mm
Plate height	h_{TS}	=	410	mm
Area	A_{TS}	=	12300	mm
Shear area (local)	$A_{v,TS}$	=	12300	mm^2
Second moment of inertia (local)	I_{TS}	=	3.18×10^8	mm^4
Location of neutral axis (local)	$\bar{y}_{LS}$	=	258	mm
Bending resistance (local)	M_{Rd}	=	438	kNm
Shear resistance (local)	V_{Rd}	=	1681	kN

F.5.4. Unity checks

The unity checks for local bending and shear in this section are

$$\frac{M_{Ed}'}{M_{Rd}'} = 0.82 \tag{F.41}$$

$$\frac{V_{Ed}'}{V_{Rd}'} = 0.97 \tag{F.42}$$

Both unity checks return a value smaller than 1, so the design resistance is high enough according to EC3.

F.6. Design of outstand stiffeners

The outstand stiffeners are installed to support the outstand of the flange in case the beam is loaded as grillage. The outstand stiffeners will also be used to connect lifting devices and fasten equipment. For that reason, holes will be made in the outstand stiffeners.

F.6.1. Loads and requirements

The design load on the outstand stiffener is line load if the beam is used as grillage. This yields the shear and bending requirements as stated in Table F.7. This results in the following requirements for

Table F.7: Loads on the outstand stiffeners

Load introduction	**Load**	l_{rep} **[m]**	V' **[kN]**	M' **[kNm]**
Grillage	3344kN/m	0.22	1104	121

the cross section of the longitudinal stiffener:

- Required local shear area: A_v'= 8077mm^2
- Required local second moment of inertia: I'= 4.73$\times 10^7$mm^4

F.6.2. Design formulas

The stiffener dimensions are now determined using these local requirements on shear and bending and Formula (F.43) to (F.48).

$$I' = I_{OS} + A_{OS}(\overline{y}' - \overline{y}_{OS}')^2 + I_{Eff,flange} + A_{Eff,flange}(\overline{y}' - \overline{y}_{flanges}')^2 \tag{F.43}$$

$$I_{OS} = \frac{1}{12} t_{OS} {h_{OS}}^3 \tag{F.44}$$

$$A_v' = A_{OS} = t_{OS} h_{OS} \tag{F.45}$$

$$\overline{y}_{OS}' = \frac{1}{2} h_{OS} \tag{F.46}$$

$$\overline{y}' = \frac{A_{OS}\overline{y}_{OS}' + A_{Eff,flange}\overline{y}_{flanges}'}{A_{OS} + A_{Eff,flange}} \tag{F.47}$$

$$I_{Eff,flange} = \frac{1}{12} w_{eff} {t_{flange}}^3 \tag{F.48}$$

$$A_{Eff,flange} = w_{eff} t_{flange} \tag{F.49}$$

$$w_{eff} = \frac{1}{8} l_{rep} \tag{F.50}$$

$$\overline{y}_{flanges}' = h_{OS} + \frac{1}{2} t_{flange} \tag{F.51}$$

Where
I' = Second moment of inertia for local calculations of the stiffener, in [mm^4]
I_i = Second moment of inertia of the respective part, in [mm^4]
A_i = Area of the respective part, in [mm^2]
A_v = Shear area for local stiffener calculations, in [mm^2]
$\overline{y}_i'$ = Location of the neutral axis in z-direction of the respective part, in [mm]
t_{OS} = Thickness of the outstand stiffeners, in [mm]
h_{OS} = Height of the outstand stiffeners, in [mm]
t_{flange} = Thickness of the flanges, in [mm]
w_{eff} = Effective width of the flange from centerline of the stiffener, in [mm]
l_{rep} = Unsupported length, in [mm]

F.6.3. Section properties

The results after several iterations of the design loop are shown in Table F.8. Bending and shear resistance of the transverse stiffener is only expressed as local values, because they do not account for the global resistance.

Table F.8: Section properties of the outstand stiffeners

Property	Value			Unit
Plate thickness	t_{OS}	=	40	mm
Plate height	h_{OS}	=	270	mm
Area	A_{OS}	=	10800	mm
Shear area (local)	$A_{v,OS}$	=	10800	mm^2
Second moment of inertia (local)	I_{OS}	=	1.06×10^8	mm^4
Location of neutral axis (local)	$\bar{y}_{OS}$	=	141	mm
Bending resistance (local)	M_{Rd}	=	267	kNm
Shear resistance (local)	V_{Rd}	=	1476	kN

F.6.4. Unity checks

The unity checks for local bending and shear in this section are

$$\frac{M_{Ed}'}{M_{Rd}'} = 0.45 \tag{F.52}$$

$$\frac{V_{Ed}'}{V_{Rd}'} = 0.75 \tag{F.53}$$

Both unity checks return a value smaller than 1, so the design resistance is high enough according to EC3.

F.6.5. Hole dimensions

A hole is machined in each outstand stiffener for connecting lashing and lifting devices to the beam. Use is made of 25t shackles, which have a pin diameter of 50mm [19]. The radius and location of the hole are calculated using Roll-Lift document *'Pad eye calculation according to NEN-EN 1993-1-8: 2005'*, see page 127. This results in a hole with a diameter of 52mm and its center 78mm from the edge of the outstand stiffener. This leaves 100mm free length between the edge of the outstand stiffener and the inside of the shackle bow.

ROLL-LIFT

Pad eye calculation according to NEN-EN 1993-1-8: 2005

INPUT

Input parameters

Geometry

Diameter pin	d_p	=	50	[mm]
Diameter pin hole	d_o	=	52	[mm]
Main plate radius	r	=	78	[mm]
Eccentricity	e	=	0	[mm]
Dist. edge hole to edge plate in line of force	a_p	=	52	[mm]
Dist. edge hole to edge plate perpendicular	c_p	=	52	[mm]

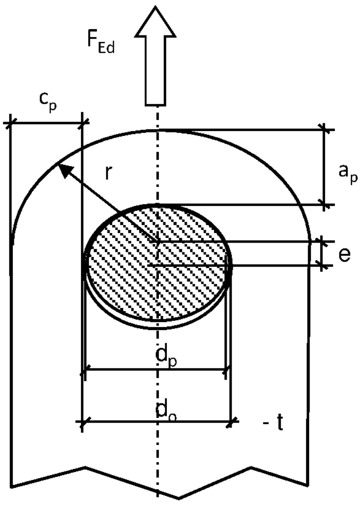
Calculation of the plate geometry with the formulas given in article 3.13 of prEN 1993-1-8:2005.
The optimum geometry is reached at the following requirements:
$e \approx 0.33 \times d_o$
$r \approx 1.60 \times d_o$
$t \approx 0.45 - 0.65 \times d_o$

Material

Main plate thickness	t	=	40	[mm]
Yield stress plate material	$f_{y;d}$	=	355	[N/mm²]

Load

Actual load (without load factors)	F_{rep}	=	245	[kN]
Load factor	■	=	2,00	[-]
Dynamic amplification factor (ina dyn = 1,00)	■yn	=	1,00	[-]
Material factor; M0 = 1.00 in prEN 1993-1-8	■o	=	1,00	[-]
Overal safety factor = g x dyn x mp	■tal	=	2,00	[-]
Design load = Frep x total	F_{Ed}	=	490,5	[kN]



OUTPUT

Decisive check: **1,00** ≤ **1,00** OK
Plate geometry in line of force

Check strength in line of force

a_p = Fa;d / (2 x t x $f_{y;d}$) + 2/3 x d_o — Formula table 3.9

$F_{a;d}$ = (a_p -2/3 x d_o) x 2 x t x $f_{y;d}$ — $F_{a;d}$ = plate strength based on measure a_p

$F_{a;d}$ = (52-2/3 x 52) x 2 x 40 x 355

$F_{a;d}$ = 492 kN

$\frac{F_{Ed}}{F_{a;d}} = \frac{491}{492} =$ **1,00** ≤ **1,00** OK

Check strength perpendicular to line of force

c_p = $F_{c;d}$ / (2 x t x $f_{y;d}$) + 1/3 x d_o — Formula table 3.9

$F_{c;d}$ = (c_p -1/3 x d_o) x 2 x t x $f_{y;d}$ — $F_{c;d}$ = plate strength based on measure c_p

$F_{c;d}$ = (52-1/3 x 52) x 2 x 40 x 355

$F_{c;d}$ = 985 kN

$\frac{F_{Ed}}{F_{c;d}} = \frac{491}{985} =$ **0,50** ≤ **1,00** OK

Check ULS bearing strength

$F_{b;Rd}$ = 1.5 x d_p x t x $f_{y;d}$ — Formula table 3.10

$F_{b;Rd}$ = 1.5 x 50 x 40 x 355

$F_{b;Rd}$ = 1065 kN

$\frac{F_{Ed}}{F_{b;Rd}} = \frac{491}{1065} =$ **0,46** ≤ **1,00** OK

Check SLS bearing strength

$F_{b;Rd;ser}$ = 0.6 x dp x t x fy;d — Formula table 3.10

$F_{b;Rd;ser}$ = 0.6 x 50 x 40 x 355

$F_{b;Rd;ser}$ = 426 kN

$\frac{F_{rep}}{F_{b;Rd;ser}} = \frac{245}{426} =$ **0,58** ≤ **1,00** OK

Check contact bearing stress

Contact bearing stress following Hertz — Formulas 3.15 and 3.16

$\sigma_{h;Ed}$ = 0.591 x sqrt((E x F_{rep} x (d_0 - d_p))/(d_p^2 x t))

$\sigma_{h;Ed}$ = 600 N/mm²

$\frac{s_{h;Ed}}{2.5 \times f_{y;d}} = \frac{600}{888} =$ **0,68** ≤ **1,00** OK

F.7. Total geometry

F.7.1. Geometry

The total geometry of the assembly of all beam components is shown in Figure F.1.

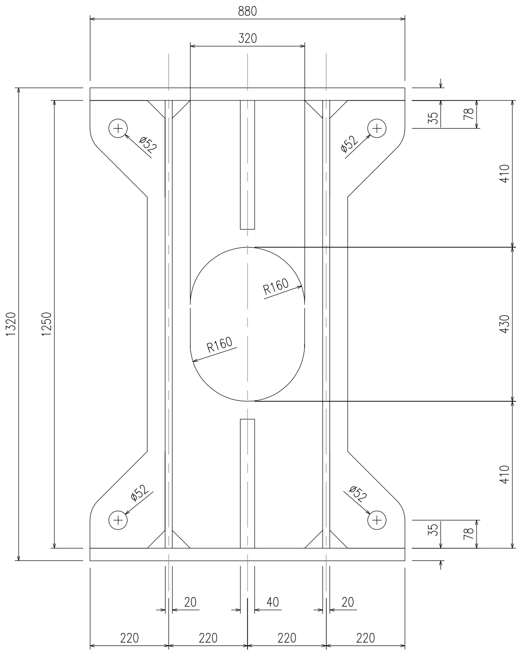


Figure F.1: Total geometry of the beam

F.7.2. Total resistance

Now that the geometry of all beam parts is known, their properties are combined to calculate the global bending and shear resistance of the beam. These properties are combined using Formula (F.5) and (F.6).

$$\begin{aligned} I = {} & 2 \times \left(I_{webs} A_{webs} \left(\overline{y} - \overline{y}_{webs} \right)^2 \right) \\ & + 2 \times \left(I_{flanges} A_{flanges} \left(\overline{y} - \overline{y}_{flanges} \right)^2 \right) \\ & + 2 \times \left(I_{LS} A_{LS} \left(\overline{y} - \overline{y}_{LS} \right)^2 \right) \end{aligned} \tag{F.54}$$

$$\overline{y} = \frac{\overline{y}_{webs} A_{webs} + \overline{y}_{flanges} A_{flanges} + \overline{y}_{LS} A_{LS}}{A_{webs} + A_{flanges} + A_{LS}} \tag{F.55}$$

$$M_{Rd} = \frac{I \sigma_y}{y} \times 10^{-6} \tag{F.56}$$

Where
I_i = Second moment of inertia of the respective part, in [mm^4]
A_i = Area of the respective part, in [mm^2]
$\overline{y}_i$ = Location of the neutral axis in z-direction of the respective part, in [mm]
M_{Rd} = Design bending resistance, in [kN]
σ_y = Yield stress, in [N/mm^2]
y = Distance from neutral line to extreme fibre, in [mm]

$$A_v = A_{v,webs} + A_{v,LS} \tag{F.57}$$
$$V_{Rd} = A_v \tau_{allow} \times 10^{-3} \tag{F.58}$$

Where A_{vi} = Shear area of the respective part, in [mm^2]
V_{Rd} = Design shear resistance, in [kN]
A_v = Shear area, in [mm^2]
τ_{allow} = Allowable shear stress, in [N/mm^2]

Table F.9: Properties of the assembled beam

Property	**Value**			**Unit**
Shear area	A_v	=	78800	mm^2
Second moment of inertia	I	=	3.80×10^{10}	mm^4
Distance to extreme fibre	y	=	660	mm
Bending resistance	M_{Rd}	=	20418	kNm
Shear resistance	V_{Rd}	=	10767	kN

F.7.3. Unity checks

The unity checks for the total geometry on global bending and shear are:

$$\frac{M_{Ed}}{M_{Rd}} = 0.71 \tag{F.59}$$

$$\frac{V_{Ed}}{V_{Rd}} = 0.82 \tag{F.60}$$

Both unity checks return a value smaller than 1, so the design resistance is high enough according to EC3.

F.8. Connection between beam sections

F.8.1. Loads and requirements

The design load on the connection between beam sections is taken equal to the design resistances calculated in the previous section. This yields the shear and bending requirements as stated in Table F.10.

Table F.10: Loads (including safety factor of 1.5) on the connection between sections

Load type	**On component**	**Magnitude [kN]**
Compression	Flanges	18689
Tension	Pin-hole connection	18689
Shear	Pin-hole connection	8829

F.8.2. Calculations

Compression

Compressions is transferred through the flanges. These are already designed in Section F.3.

Tension

Tension is transferred through pad eyes. These are calculated according to EC3 with Roll-Lift calculation sheet *'Pad eye calculation according to NEN-EN 1993-1-8: 2005'*, which is presented on page 131.

Shear

Shear is also transferred through the pad eyes. It is assumed that the shear is distributed over all pad eyes, resulting in a vertical component of the force transferred through the pad eyes. This vertical component is not calculated with the calculation sheet on page 131, but added and analyzed in the FEA of the pad eye.

F.8.3. Connection geometry

The geometry of the connection between beam sections is shown in Figure F.2.

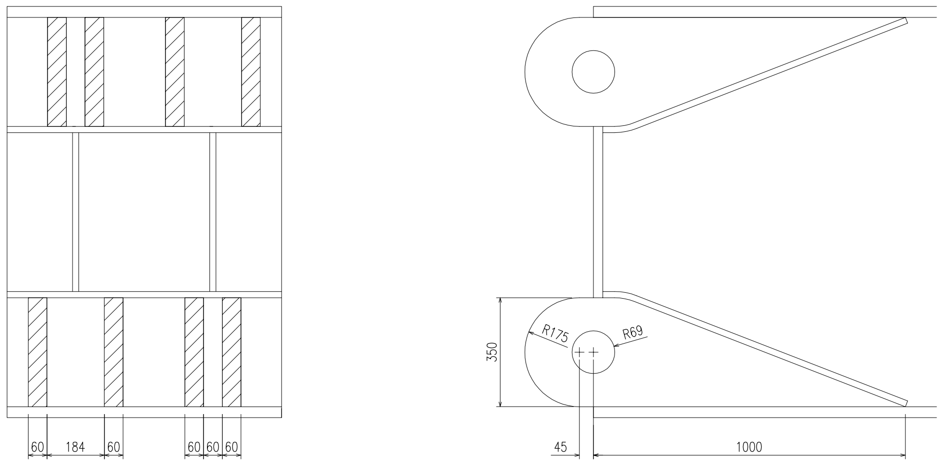


Figure F.2: Dimensions of the pin-hole connection between beam sections

ROLL-LIFT

Pad eye calculation according to NEN-EN 1993-1-8: 2005

INPUT

Input parameters

Geometry

Diameter pin	d_p	=	135	[mm]
Diameter pin hole	d_o	=	137	[mm]
Main plate radius	r	=	175	[mm]
Eccentricity	e	=	45	[mm]
Dist. edge hole to edge plate in line of force	a_p	=	151,5	[mm]
Dist. edge hole to edge plate perpendicular	c_p	=	106,5	[mm]

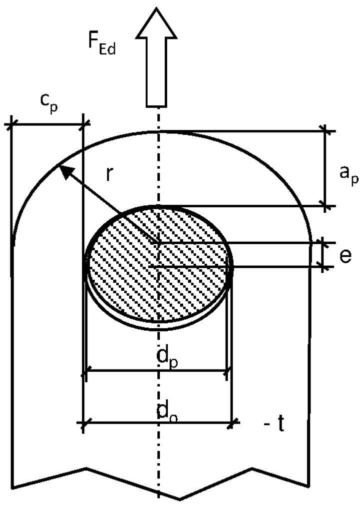
Calculation of the plate geometry with the formulas given in article 3.13 of prEN 1993-1-8:2005.
The optimum geometry is reached at the following requirements:
$e \approx 0.33 \times d_o$
$r \approx 1.60 \times d_o$
$t \approx 0.45 - 0.65 \times d_o$

Material

Main plate thickness	t	=	240	[mm]
Yield stress plate material	$f_{y;d}$	=	690	[N/mm²]

Load

Actual load (without load factors)	F_{rep}	=	12459	[kN]
Load factor	■	=	1,50	[-]
Dynamic amplification factor (ina dyn = 1,00)	■yn	=	1,00	[-]
Material factor; $_{M0}$ = 1.00 in prEN 1993-1-8	■10	=	1,00	[-]
Overal safety factor = g x dyn x mp	■tal	=	1,50	[-]
Design load = Frep x total	F_{Ed}	=	18689	[kN]



OUTPUT

Decisive check: **0,94** ≤ **1,00** OK
Plate geometry in line of force

Check strength in line of force

a_p = $F_{a;d} / (2 \times t \times f_{y;d}) + 2/3 \times d_o$ — Formula table 3.9

$F_{a;d}$ = $(a_p - 2/3 \times d_o) \times 2 \times t \times f_{y;d}$ — $F_{a;d}$ = plate strength based on measure a_p

$F_{a;d}$ = (151,5-2/3 x 137) x 2 x 240 x 690

$F_{a;d}$ = 19927 kN

F_{Ed}		18689					
$F_{a;d}$	=	19927	=	**0,94**	≤	**1,00**	OK

Check strength perpendicular to line of force

c_p = $F_{c;d} / (2 \times t \times f_{y;d}) + 1/3 \times d_o$ — Formula table 3.9

$F_{c;d}$ = $(c_p - 1/3 \times d_o) \times 2 \times t \times f_{y;d}$ — $F_{c;d}$ = plate strength based on measure c_p

$F_{c;d}$ = (106,5-1/3 x 137) x 2 x 240 x 690

$F_{c;d}$ = 20148 kN

F_{Ed}		18689					
$F_{c;d}$	=	20148	=	**0,93**	≤	**1,00**	OK

Check ULS bearing strength

$F_{b;Rd}$ = $1.5 \times d_p \times t \times f_{y;d}$ — Formula table 3.10

$F_{b;Rd}$ = 1.5 x 135 x 240 x 690

$F_{b;Rd}$ = 33534 kN

F_{Ed}		18689					
$F_{b;Rd}$	=	33534	=	**0,56**	≤	**1,00**	OK

Check SLS bearing strength

$F_{b;Rd;ser}$ = 0.6 x dp x t x fy;d — Formula table 3.10

$F_{b;Rd;ser}$ = 0.6 x 135 x 240 x 690

$F_{b;Rd;ser}$ = 13414 kN

F_{rep}		12459					
$F_{b;Rd;ser}$	=	13414	=	**0,93**	≤	**1,00**	OK

Check contact bearing stress

Contact bearing stress following Hertz — Formulas 3.15 and 3.16

$\sigma_{h;Ed}$ = $0.591 \times \mathrm{sqrt}((E \times F_{rep} \times (d_0 - d_p))/(d_p^2 \times t))$

$\sigma_{h;Ed}$ = 646 N/mm²

$S_{h;Ed}$		646					
$2.5 \times f_{y;d}$	=	1725	=	**0,37**	≤	**1,00**	OK

Pin calculation according to NEN-EN 193-1-8: 2005

INPUT

Input parameters

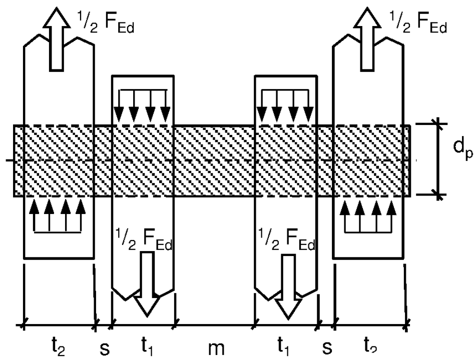
Geometry

Diameter pin	d_p	=	135	[mm]
Thickness inner plates	t_1	=	60	[mm]
Thickness outer plate	t_2	=	60	[mm]
Gap	s	=	2	[mm]
Gap between inner plates	m	=	60	[mm]

Calculation of a pin in a pin connection with the rules given in article 3.13 of EN 1993-1-8:2004. The optimum pin diameter and plate geometry is obtained at the following values:
$t_1 = t_2 \approx 0.20 - 0.35 \times d_p$

Material

Yield stress inner plate material (t_1)	$f_{1y;d}$	=	690	[N/mm2]
Yield stress outer plate material (t_2)	$f_{2y;d}$	=	690	[N/mm2]
Yield stress pin material	$f_{y;p}$	=	800	[N/mm2]
Tensile strength pin material	$f_{u;p}$	=	1000	[N/mm2]
Model factor; always 1.25	[redacted]ode	=	1,25	[-]



Load

Actual load (without load factors)	F_{rep}	=	6230	[kN]
Load factor	[redacted]	=	1,50	[-]
Dynamic amplification factor	[redacted]yn	=	1,00	[-]
Material factor	[redacted]0	=	1,00	[-]
Overall safety factor = [redacted] x [redacted]yn x [redacted]0	[redacted]tal	=	1,50	[-]
Design load = F_{rep} x [redacted]tal	F_{Ed}	=	9344	[kN]
Maximum bending moment	M_{Ed}	=	289,7	[kNm]
Actual bending moment	M_{rep}		99,7	[kNm]
Max bending moment at max shear force	$M_{v;m}$	=	149,5	[kNm]

Static values pin

A_p	=	14314	mm2
$W_{el;p}$	=	241547	mm3

OUTPUT

Decisive check	**1,00**	≤	1,00	OK
ULS bearing strength of the pin				

Check shear strength of the pin

$F_{v;Rd}$ = 0.6 x 2 x A_p x $f_{t;p}$ / [redacted]lodel — Formula table 3.10

$F_{v;Rd}$ = 0.6 x (2 x 14314) x 1000 / 1,25

$F_{v;Rd}$ = 13741 kN

$\frac{F_{Ed}}{F_{v;Rd}} = \frac{9344}{13741} =$ **0,68** ≤ 1,00 OK

Check ULS bending strength of the pin

M_{Rd} = 1,5 x $W_{el;p}$ x $f_{y;p}$ — Formula table 3.10

M_{Rd} = 1,5 x 241547 x 800

M_{Rd} = 289,9 kNm

$\frac{M_{Ed}}{M_{Rd}} = \frac{289,7}{289,9} =$ **1,00** ≤ 1,00 OK

Check SLS bending strength of the pin

$M_{Rd,ser}$ = 0,8 x $W_{el;p}$ x $f_{y;p}$ — Formula table 3.10

$M_{Rd,ser}$ = 0,8 x 241547 x 800

$M_{Rd,ser}$ = 154,6 kNm

$\frac{M_{rep}}{M_{Rd,ser}} = \frac{99,7}{154,6} =$ **0,64** ≤ 1,00 OK

Check combined shear and bending strength of the pin

$\frac{F_{Ed}}{F_{v;Rd}}$ = 0,68

$\frac{M_{v;max}}{M_{Rd}} = \frac{149,5}{289,9} =$ 0,52

0,68^2 + 0,52^2 = **0,73** ≤ 1,00 OK

Check bearing strength of the plate and pin

$F_{b;Rd}$ = 1.5 x 2 x t1 x d_p x f_1y;d — Formula table 3.10

$F_{b;Rd}$ = 1.5 x 2 x 60 x 135 x 690

$F_{b;Rd}$ = 16767 kN

$\frac{F_{Ed}}{F_{b;Rd}} = \frac{9344}{16767} =$ **0,56** ≤ 1,00 OK

G

Results of the FEM analysis

In this appendix, an overview is presented of the results of the FEA per load case. Section G.1 gives an overview of the finite element model, the input parameters and a short explanation of the results. The results per load case are presented in Section G.2 to G.13.

G.1. General properties of the model

For the analysis in RFEM, a plate model is used. All plates for the beam are constructed of S355 steel, which has a yield stress of $355N/mm^2$. The loads follow from the load cases as presented in Chapter 4. The load combinations consist of self weight of the beam multiplied with a safety factor of 1.35 and all loads acting on the beam multiplied with a safety factor of 1.5 as described by EC3. The target mesh size is chosen as $50mm \times 50mm$ squares. Due to the modelling, singularities exist at the locations of supports and load introduction, see Figure G.1. These singularities cause local stress peaks, which will be ignored in this research while this it only concerns global results. The legends to the stress and strain diagrams are shown in Figure G.2. For the stress diagrams, $355N/mm^2$ corresponds to the dark yellow color, every color above it corresponds to the subsequent HSS option. This means that stress diagrams showing orange or red in the global results have stresses above the yield stress of the material. For the strain diagrams, 0.2% corresponds to the dark yellow color. Strain diagrams showing orange or red will thus sustain excessive strain.

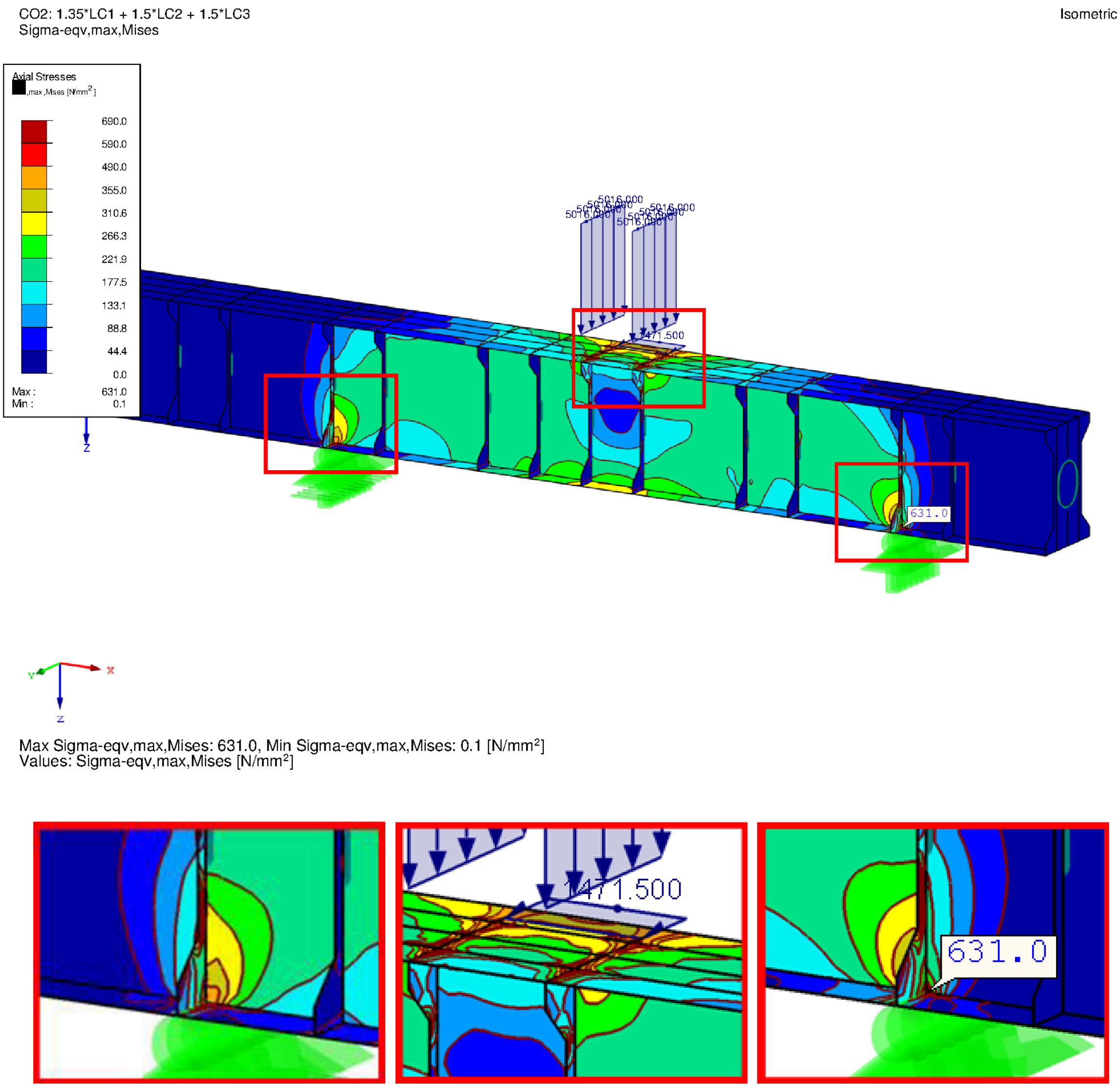
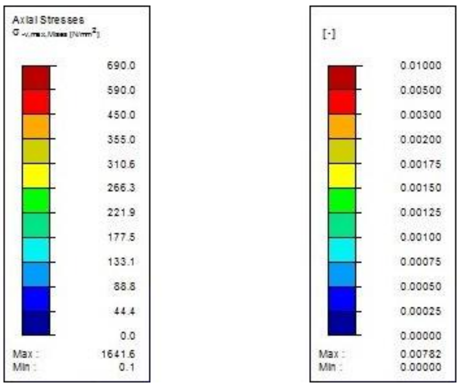


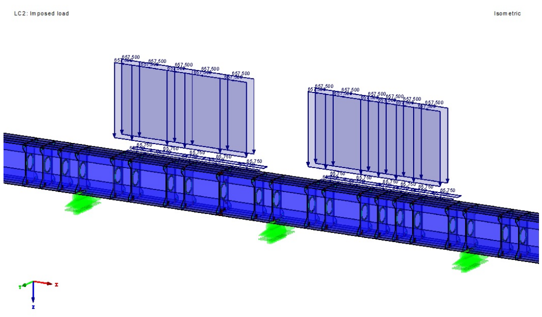
Figure G.1: Stress peaks in the RFEM model due to load introduction and support modelling



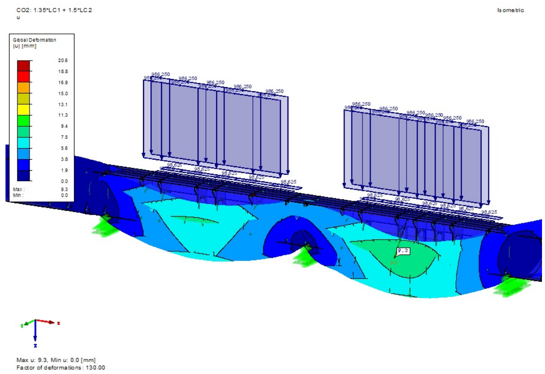
(a) Scale of the stress diagrams (b) Scale of the strain diagrams

Figure G.2: Scales used for the legends in this appendix

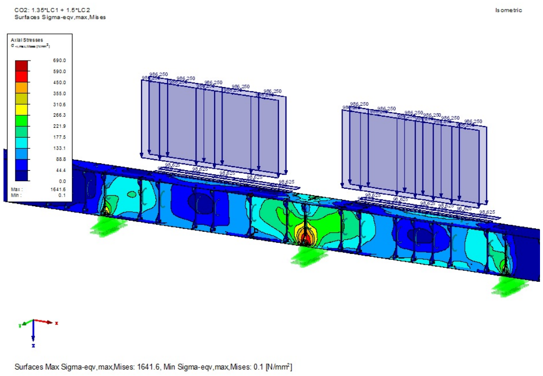
G.2. Load case 1: Skidding with Enerpac skids (1)



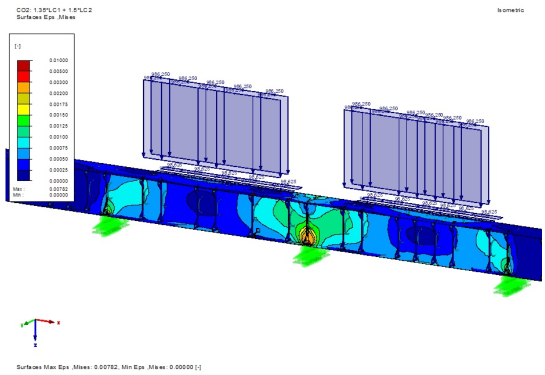
(a) Overview of loads and supports



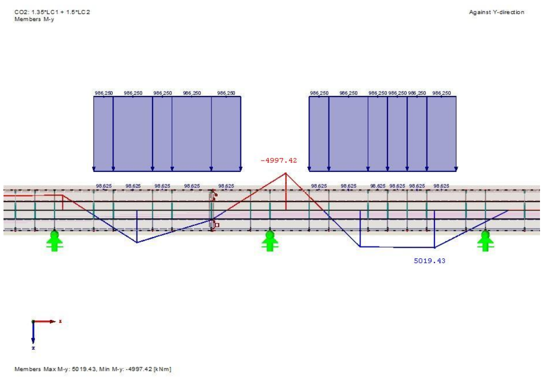
(b) Deflections of the beam



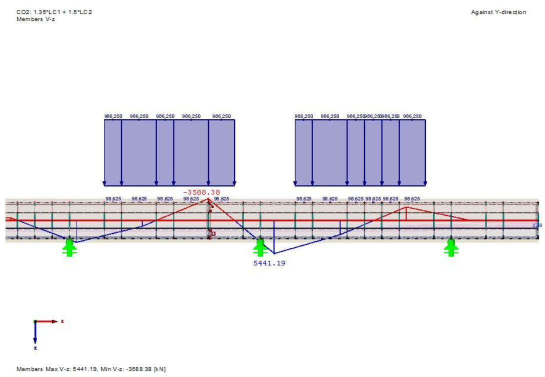
(c) Von Mises stress in the beam



(d) Strains in the beam



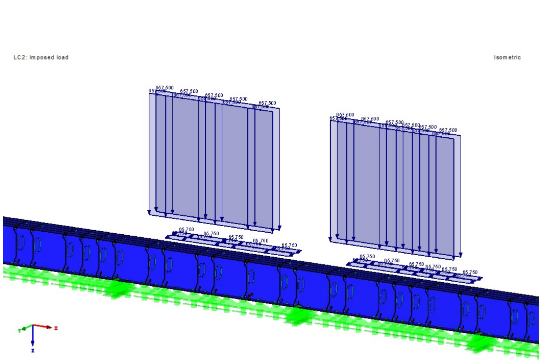
(e) Bending moment in the beam



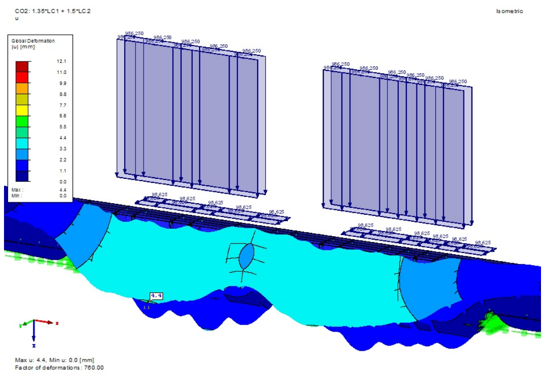
(f) Shear force in the beam

Figure G.3: Overview of input and output of load case 1

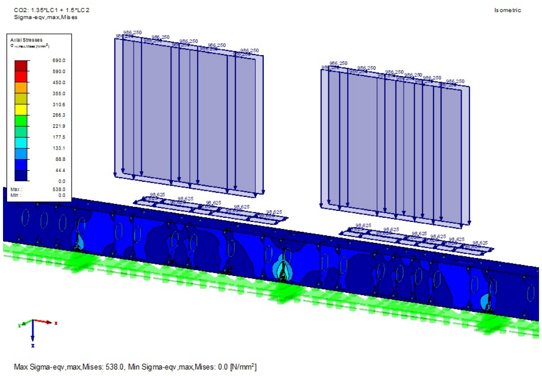
G.3. Load case 2: Skidding with Enerpac skids (2)



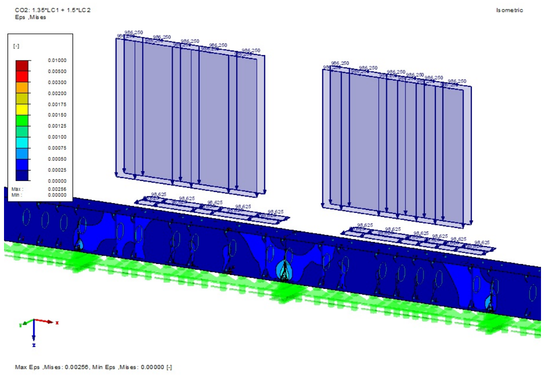
(a) Overview of loads and supports



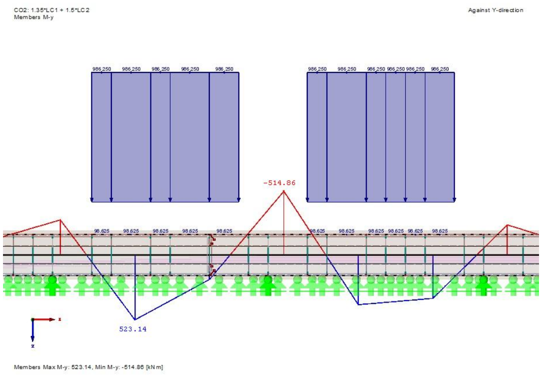
(b) Deflections of the beam



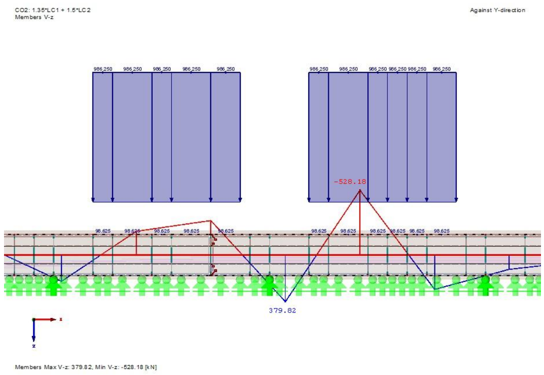
(c) Von Mises stress in the beam



(d) Strains in the beam



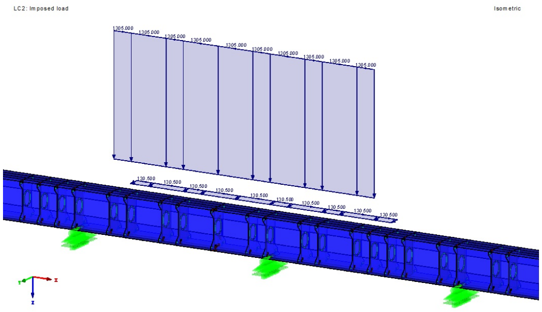
(e) Bending moment in the beam



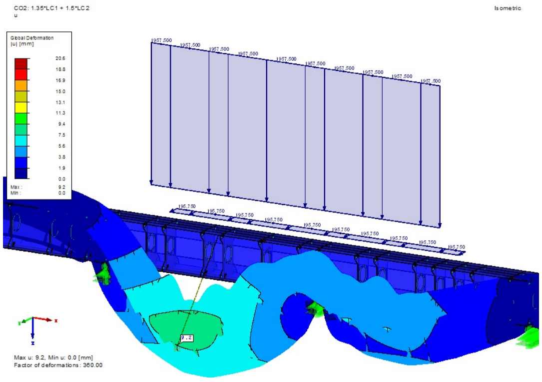
(f) Shear force in the beam

Figure G.4: Overview of input and output of load case 2

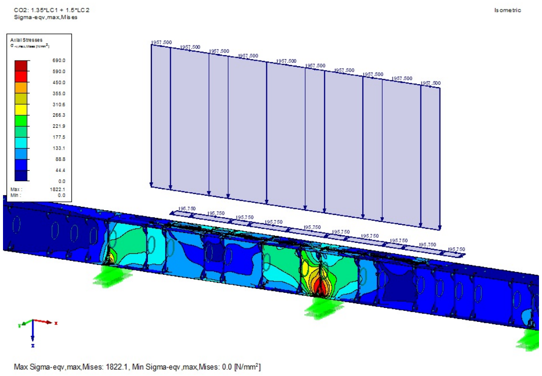
G.4. Load case 3: Skidding with Mammoet skids (1)



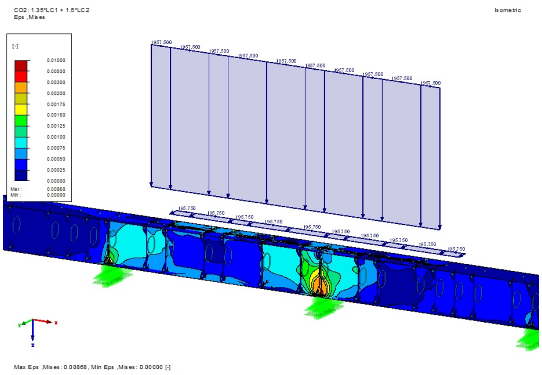
(a) Overview of loads and supports



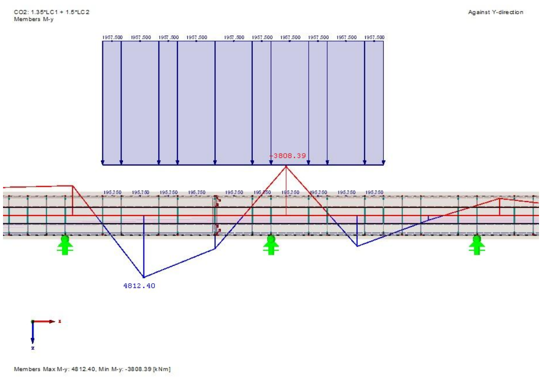
(b) Deflections of the beam



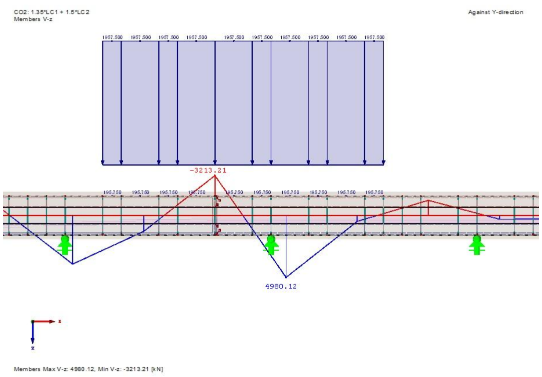
(c) Von Mises stress in the beam



(d) Strains in the beam



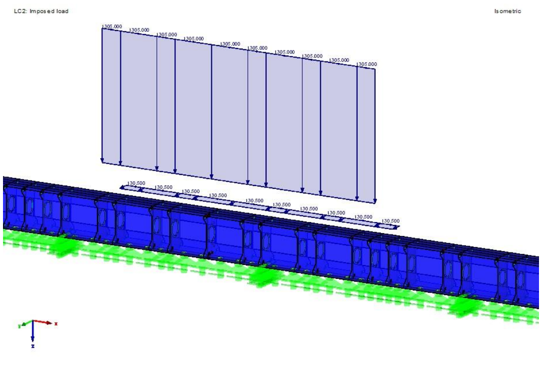
(e) Bending moment in the beam

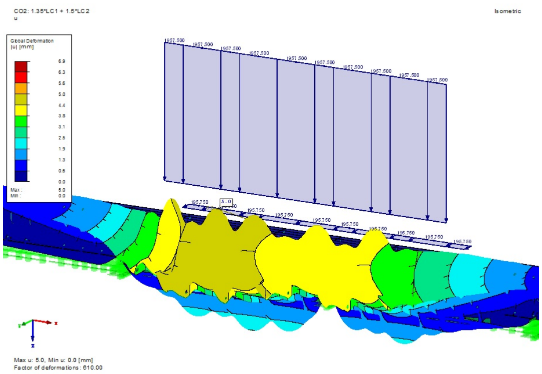


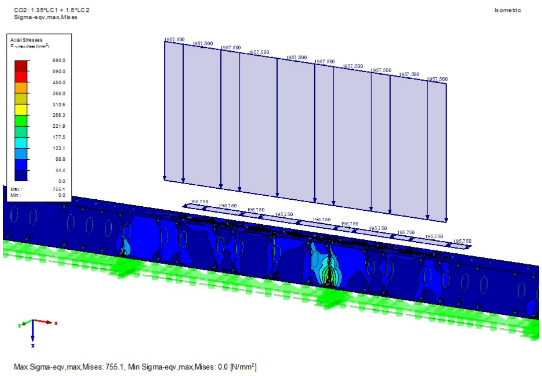
(f) Shear force in the beam

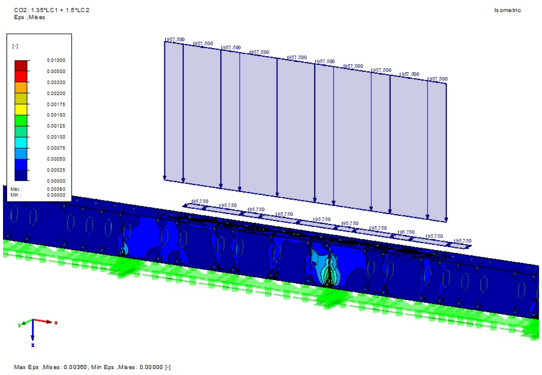
Figure G.5: Overview of input and output of load case 3

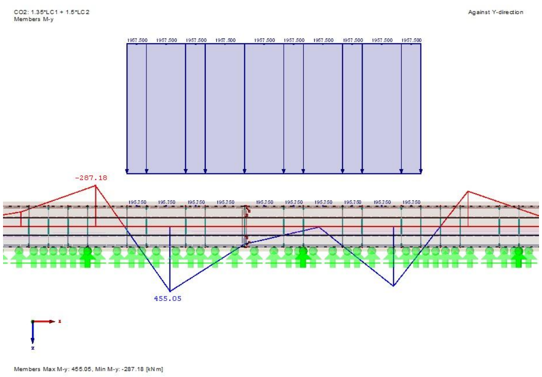
G.5. Load case 4: Skidding with Mammoet skids (2)


(a) Overview of loads and supports


(b) Deflections of the beam


(c) Von Mises stress in the beam


(d) Strains in the beam


(e) Bending moment in the beam

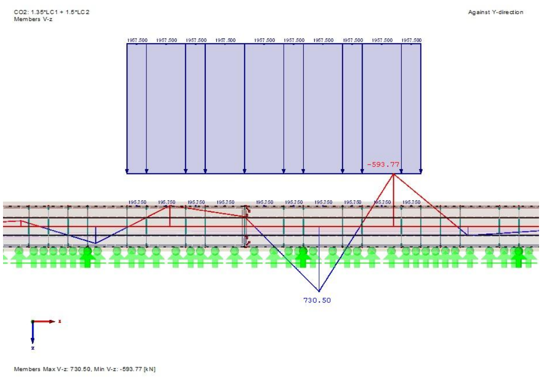
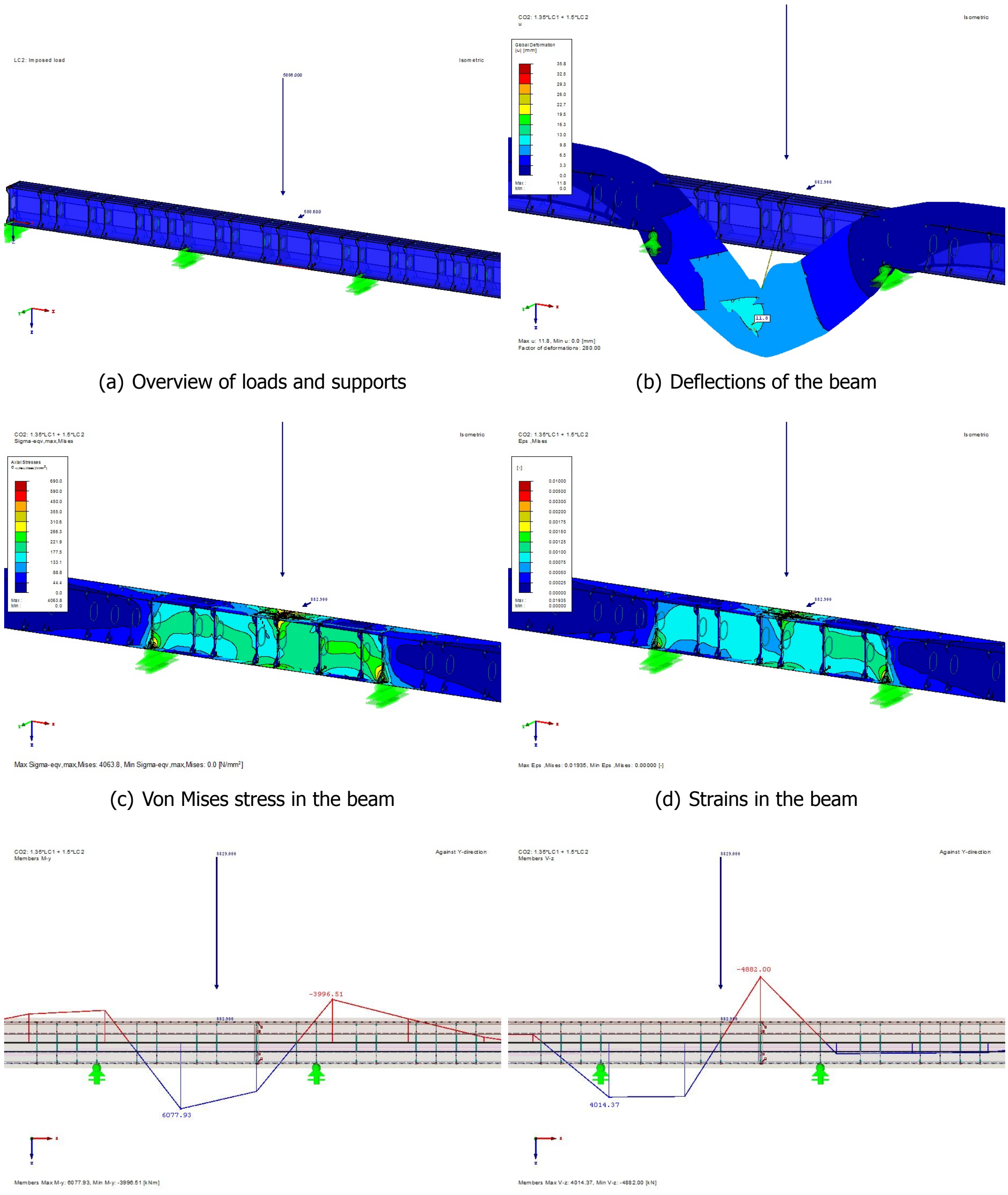

(f) Shear force in the beam

Figure G.6: Overview of input and output of load case 4

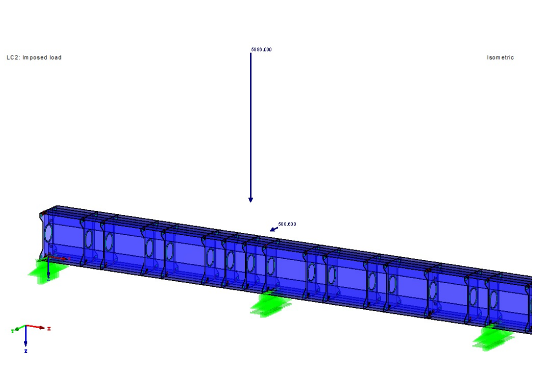
G.6. Load case 5: Load on grillage (1)



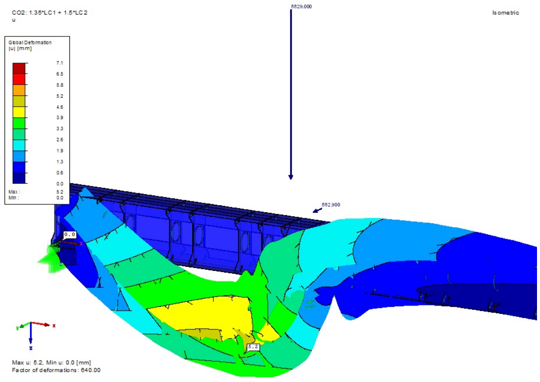
(a) Overview of loads and supports
(b) Deflections of the beam
(c) Von Mises stress in the beam
(d) Strains in the beam
(e) Bending moment in the beam
(f) Shear force in the beam

Figure G.7: Overview of input and output of load case 5

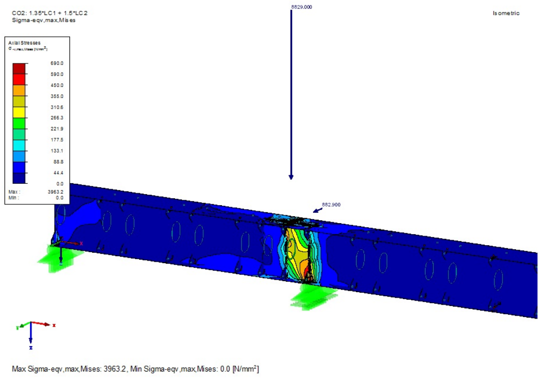
G.7. Load case 6: Load on grillage (2)



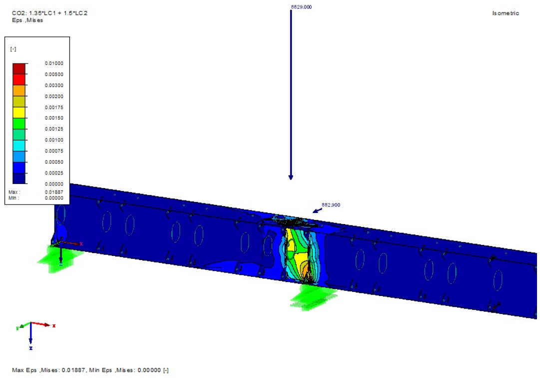
(a) Overview of loads and supports



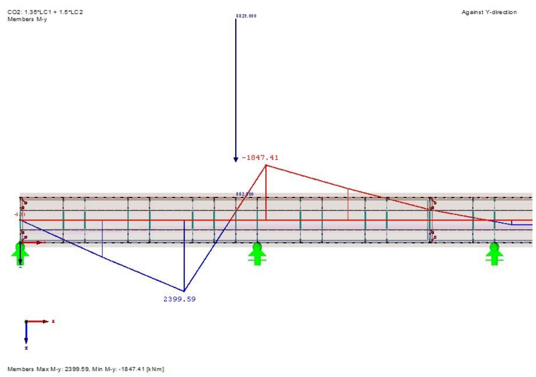
(b) Deflections of the beam



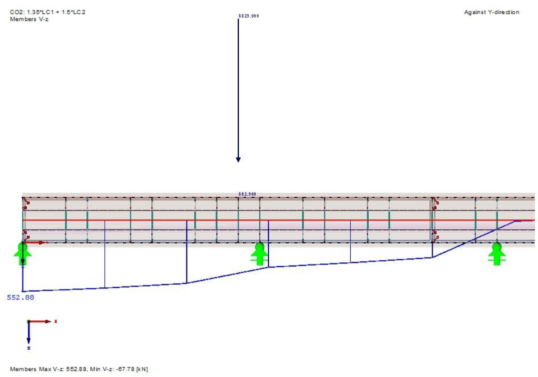
(c) Von Mises stress in the beam



(d) Strains in the beam



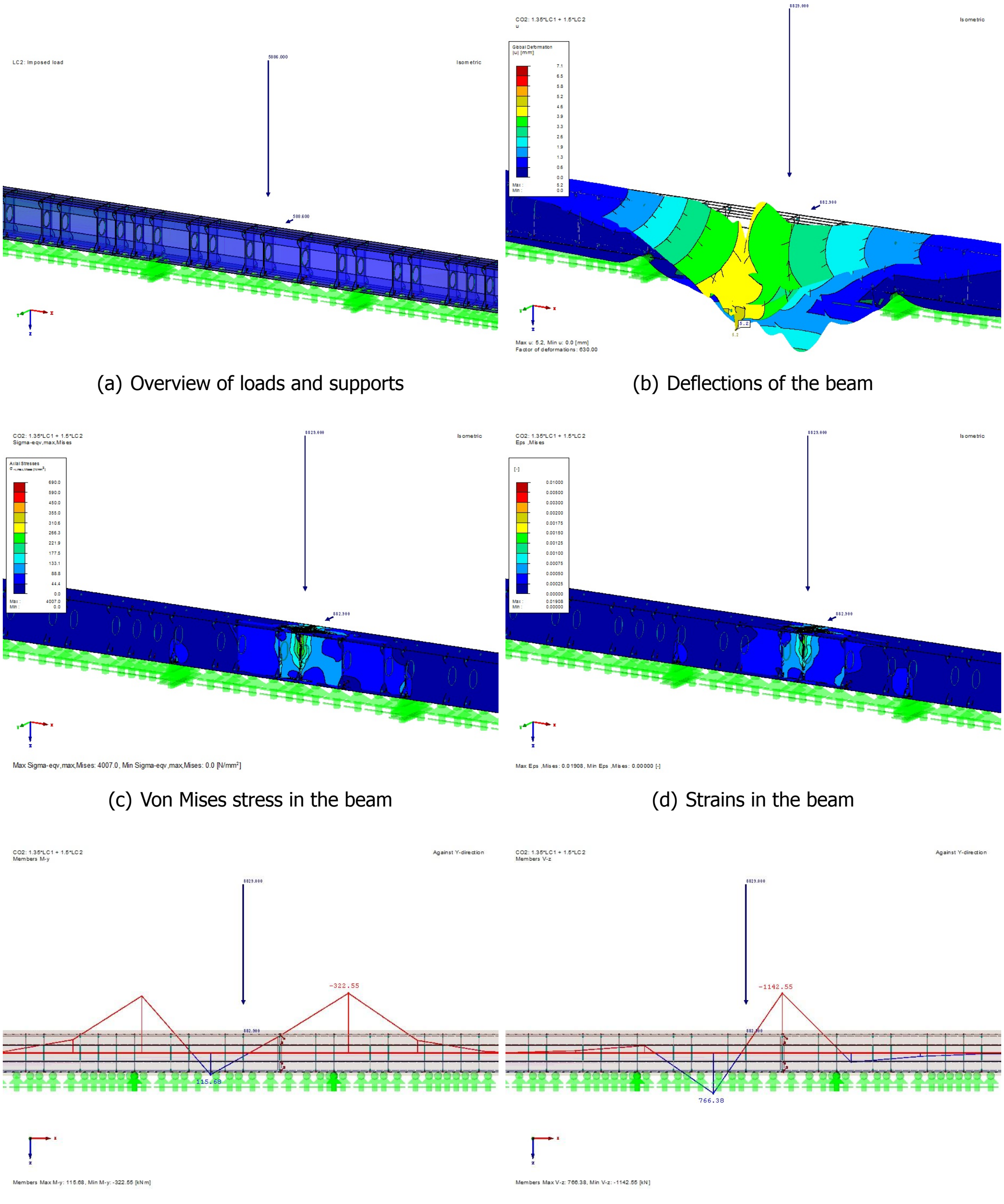
(e) Bending moment in the beam



(f) Shear force in the beam

Figure G.8: Overview of input and output of load case 6

G.8. Load case 7: Load on grillage (3) or vertical spacer



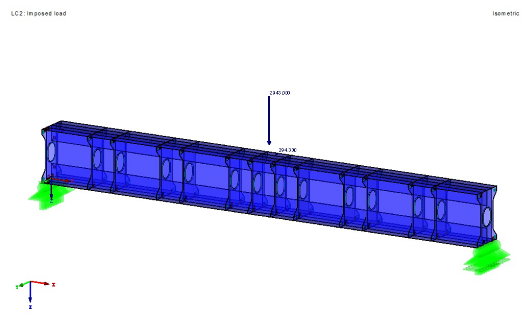
(a) Overview of loads and supports (b) Deflections of the beam

(c) Von Mises stress in the beam (d) Strains in the beam

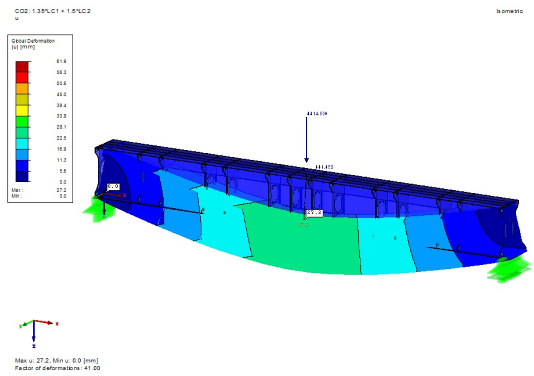
(e) Bending moment in the beam (f) Shear force in the beam

Figure G.9: Overview of input and output of load case 7

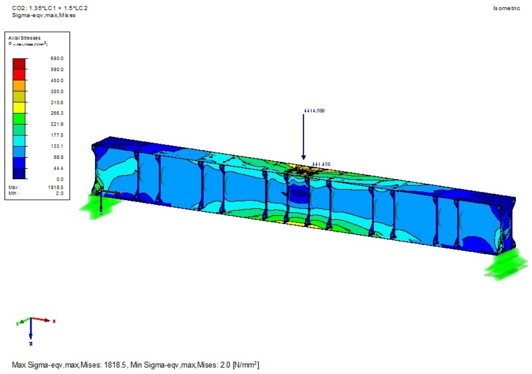
G.9. Load case 8: Gantry girder (1)



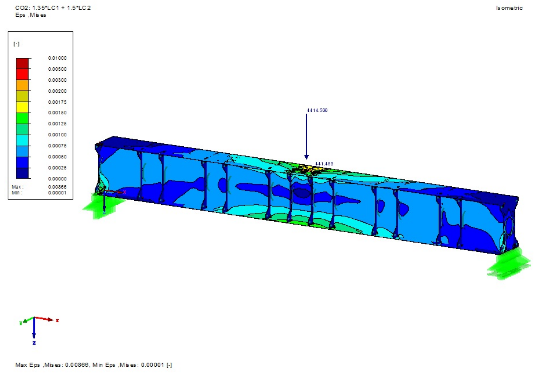
(a) Overview of loads and supports



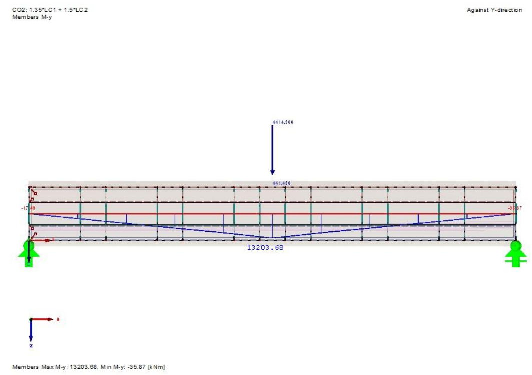
(b) Deflections of the beam



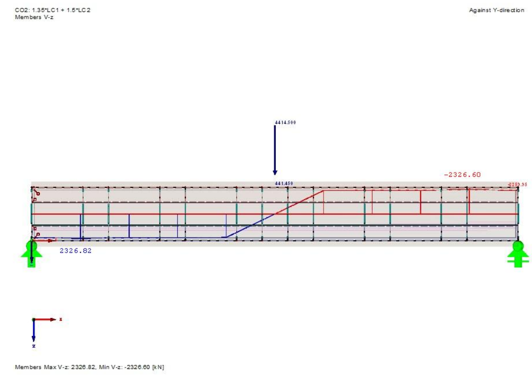
(c) Von Mises stress in the beam



(d) Strains in the beam



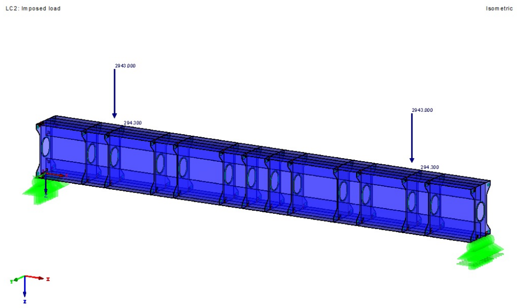
(e) Bending moment in the beam



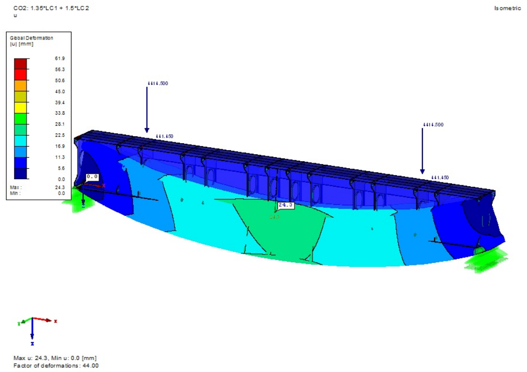
(f) Shear force in the beam

Figure G.10: Overview of input and output of load case 8

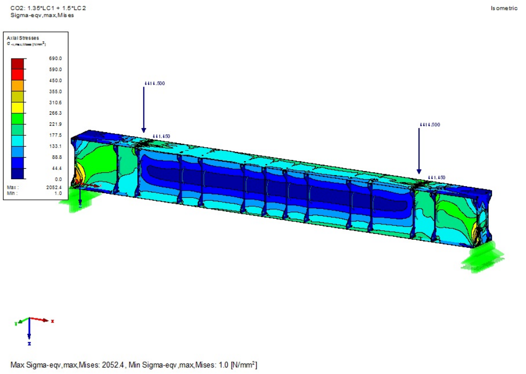
G.10. Load case 9: Gantry girder (2)



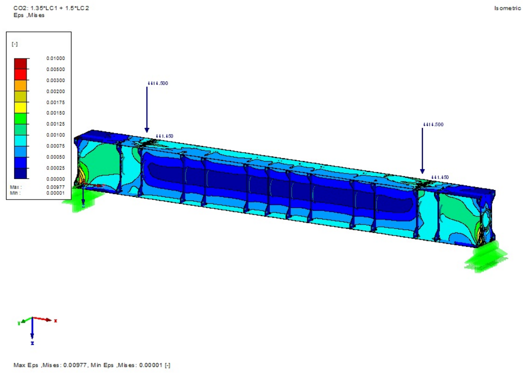
(a) Overview of loads and supports



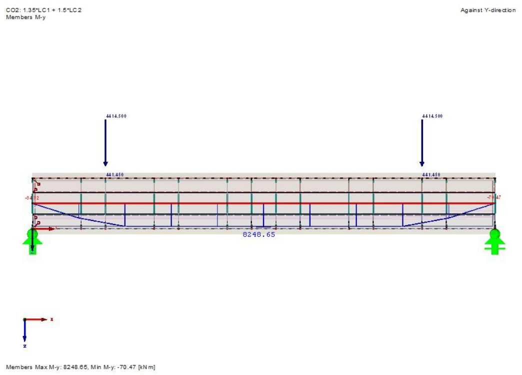
(b) Deflections of the beam



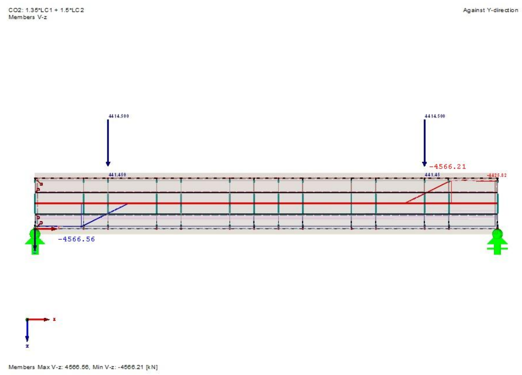
(c) Von Mises stress in the beam



(d) Strains in the beam



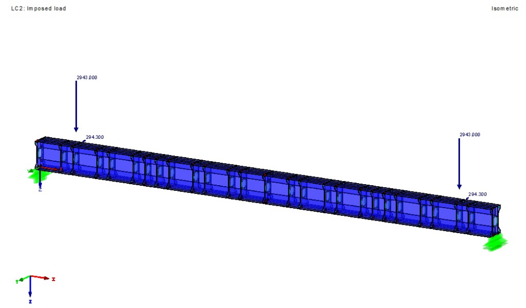
(e) Bending moment in the beam



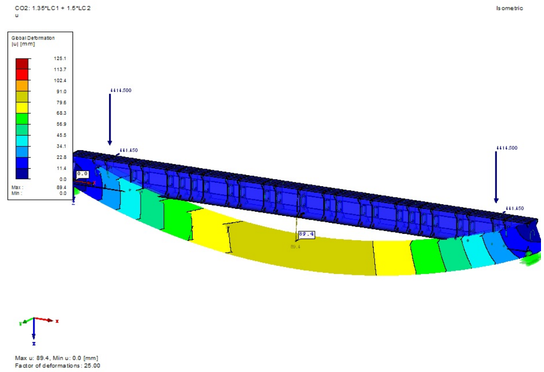
(f) Shear force in the beam

Figure G.11: Overview of input and output of load case 9

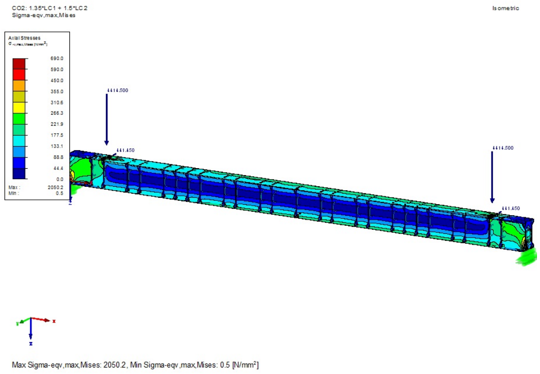
G.11. Load case 10: Gantry girder (3)



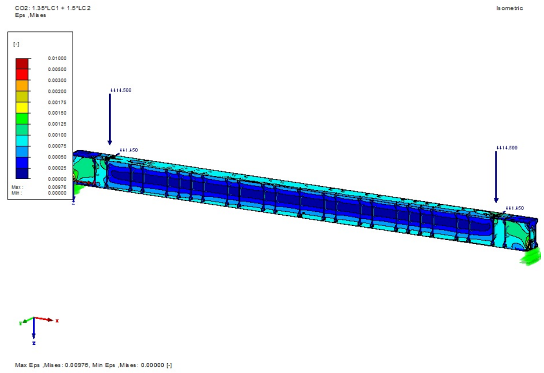
(a) Overview of loads and supports



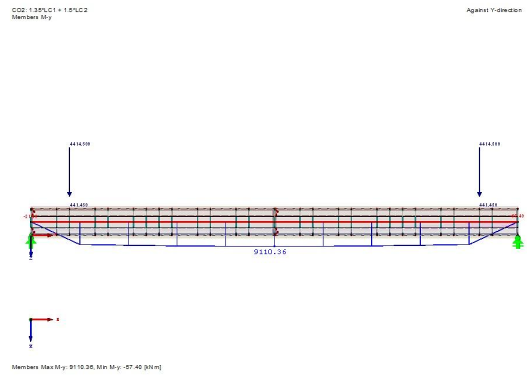
(b) Deflections of the beam



(c) Von Mises stress in the beam



(d) Strains in the beam

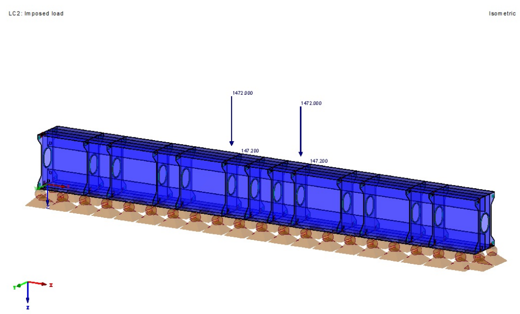


(e) Bending moment in the beam

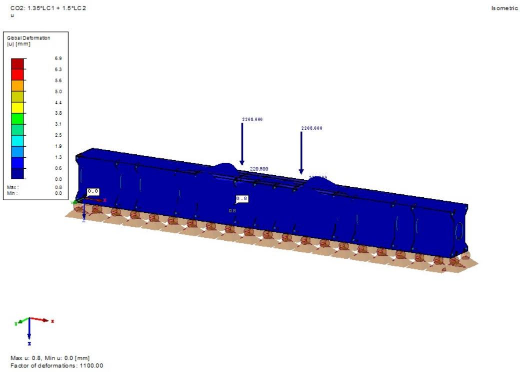
(f) Shear force in the beam

Figure G.12: Overview of input and output of load case 10

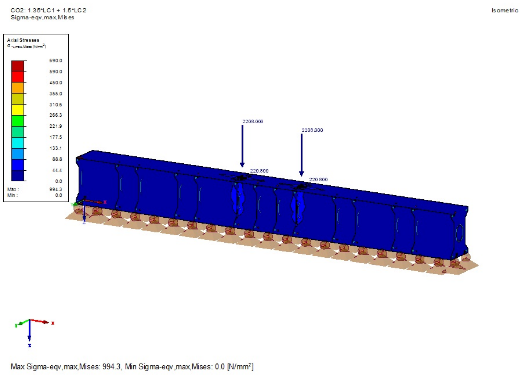
G.12. Load case 11: Gantry foundation



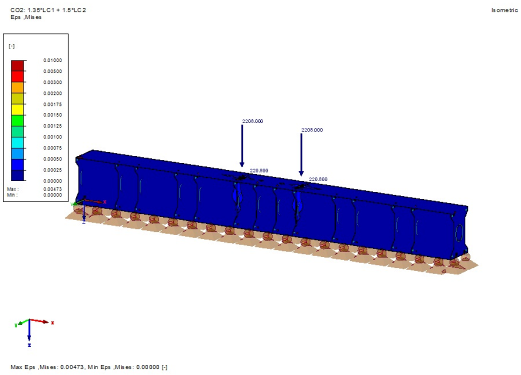
(a) Overview of loads and supports



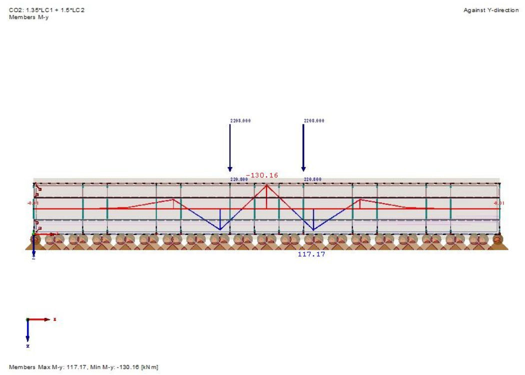
(b) Deflections of the beam



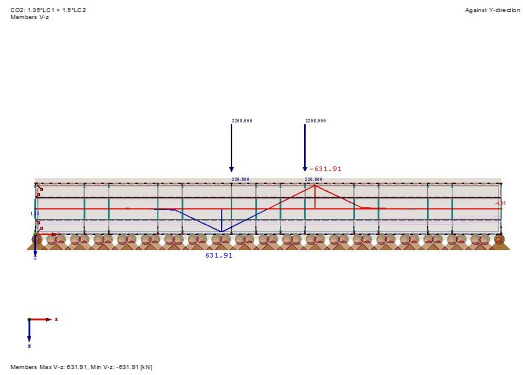
(c) Von Mises stress in the beam



(d) Strains in the beam



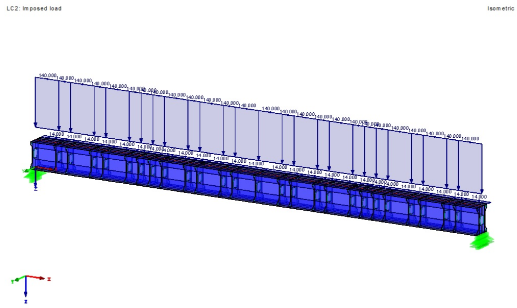
(e) Bending moment in the beam



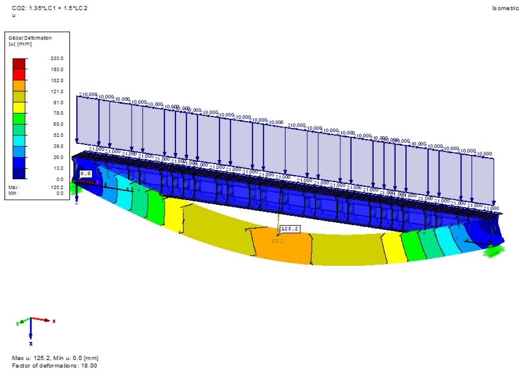
(f) Shear force in the beam

Figure G.13: Overview of input and output of load case 11

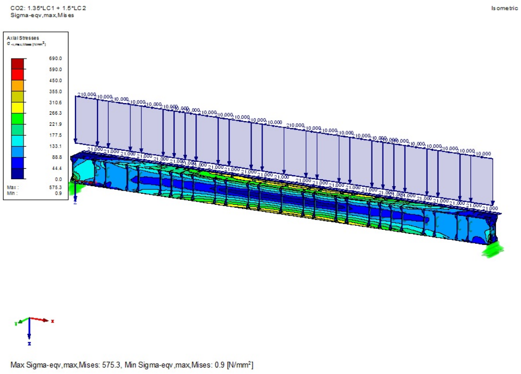
G.13. Load case 12: Temporary bridge



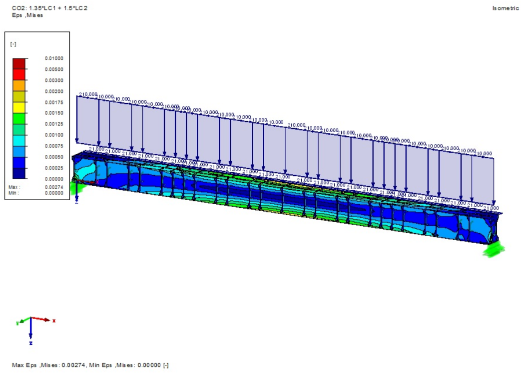
(a) Overview of loads and supports



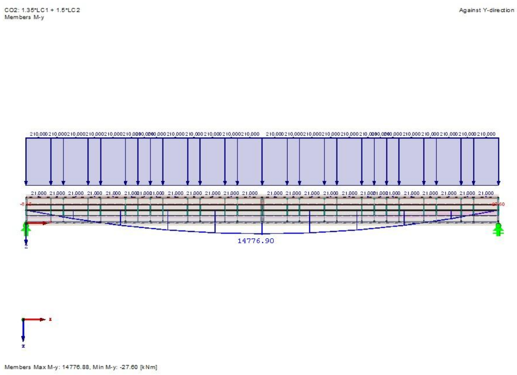
(b) Deflections of the beam



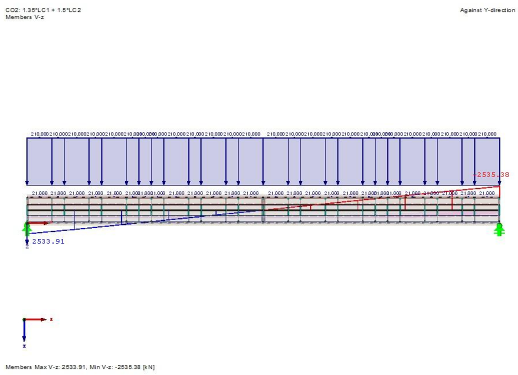
(c) Von Mises stress in the beam



(d) Strains in the beam



(e) Bending moment in the beam



(f) Shear force in the beam

Figure G.14: Overview of input and output of load case 12