Towards sustainable shipping: Recommendations for the telescopic mast design of a sailing cargo vessel

LUKAS BLAU
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Abstract

A comparative study is carried out to investigate the most promising route towards lightweight construction of a retractable mast for a sailing cargo vessel. Four design families are developed and compared. The primary criteria for judgement are the structural mass, strength and stiffness in relation to a provided benchmark design. Additional evaluation criteria are the capital costs for raw materials and manufacturing.

The design space includes isotropic materials as well as fiber reinforced polymer (FRP) solutions and is navigated by employing analytical evaluation methods supported by finite element analysis (FEA). Restrictions to the design space are given by a general arrangement of the benchmark design. This includes the limitation to the ULS loads and the overall mast geometry.

A review of relevant Det Norske Veritas (DNV) rules for classification is performed and the guidelines for wind turbine blades and wind powered units (WPU) are judged most suitable to the design challenge. Relevant design principles are implemented in the structural analysis.

It is concluded that pure metal constructions imply an unreasonably large weight penalty. Local buckling is found to disqualify FRP single skin solutions as successful candidates. Secondary to that, strength concerns are the major drivers for the structural mass.

The report presents two designs that are judged fit for the purpose, one is a hybrid truss structure from high strength low alloy steel (HSLA steel) and carbon fiber reinforced polymer (CFRP). The second design is a sandwich construction with CFRP face sheets, a PVC foam core and additional stiffening members in steel.
Acronyms

AiP  Approval in Principle.
AR   aspect ratio.
BC   boundary condition.
CFRP carbon fiber reinforced polymer.
CLT  Classical lamination theory.
DNV  Det Norske Veritas.
EBT  Euler beam theory.
FEA  finite element analysis.
FRP  fiber reinforced polymer.
GFRP glass fiber reinforced polymer.
GHG  greenhouse gas.
HSLA steel  high strength low alloy steel.
IMO  International Maritime Organization.
MOS  margin of safety.
PCTC pure car truck carrier.
ULS  ultimate limit state.
WPU  wind powered units.
XLS  extreme loads.
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Chapter 1

Introduction

In an effort to meet the UN goal for sustainable development No. 13 Climate Action, the International Maritime Organization (IMO) has defined a framework to reduce greenhouse gas (GHG) emissions from international shipping. In comparison to 2008 the CO2 emissions are to be lowered by 70% and the total GHG emission from seaborne trade by 50% by 2050 [1, p.5]. These targets are supported by the third GHG emissions report published by the IMO in 2014 [2]. The authors evaluated various scenarios for the energy markets and concluded, that despite a slight decrease in the percentual contribution in shipping related C02 emissions to the global level from 2007-2012 the respective figures could grow by 247% until 2050 [2, tab.5]. Similar trends are predicted for other climate active substances as a result of the ever-increasing demand for sea borne transportation.

The maritime sector needs to drastically reduce its GHG emissions and the urgency is high as commercial vessels typically have a life cycle of approximately 25 years. Newly built vessels from 2025 should be equipped with suitable technical solutions to meet the IMO targets for 2050 as they will actively contribute to the future emission levels.

A comparative review by Bouman et al. [3] of existing research into the potential to reduce GHG emissions from international shipping concluded, that none of the currently available technologies or operational measures alone can enable the maritime sector to reach the targets stipulated by the IMO [3, p.418]. The combined application of the most effective measures could yield a total reduction in C02 emissions of 75%, however, often at the price of emitting other climate active substances such as methane for LNG powered vessels or in case
of bio fuels the excessive need for farmland to ensure the applicability on a sufficiently large scale. One of the most promising technological approaches seems to be a modified hull design towards larger vessels to reduce the hull’s wave making resistance and thereby the GHG emissions per transported unit of cargo. The potential reduction in C02 emissions is reported up to 83% [3, tab.2] if implemented on a large scale. However, port restrictions such as maximum draft, the length of berths, cargo crane dimensions and recent events such as the grounding of the 199629 DWT container vessel Ever Given in the Suez Channel on March 23, 2021 indicate the limitations of this measure.

With the announcement of the Oceanbird project in September 2020 the Swedish ship design company Wallenius Marine AB proposed a century old technology in a new framework to meet the IMO targets for 2050 [4].

![Oceanbird Project](image)

**Figure 1.1:** *The Oceanbird: The hull design incorporates aspects of a classical pure car truck carrier (PCTC) combined with a more slender hull shape for enhanced sailing performance. The five wing sails are approximately 80m in height and contrast conventional sails with a rigid wing design in a symmetric profile.*

The Oceanbird project envisions a 200m long car carrier that can transport up to 7000 units of cargo using wind power as the major source of energy. As depicted in figure 1.1 five almost 80m tall wing sails will deliver most of the required energy to drive the vessel towards its destination. This will cut GHG emissions by approximately 90% compared to diesel for an average service speed of 10 knots [4]. To be able to de-power the vessel in critical weather conditions, during port stays and to be able to cross obstacles such as bridges the wing sails must be retractable and rely on a telescopic mechanism.

As shown in figure 1.1 the wing-sails do not have conventional stays to hold them upright. The masts inside the wings-sails are therefore to be seen as slen-
der telescopic cantilever beams. The masts are divided into sections and the thesis is focused around the second lowest one as it presents the highest loaded scenario in an intended composite solution and includes all major challenges that come with the concept.

### 1.1 Classification societies

In the marine industry it is common practice that ship designs undergo a classification process in which the designs are reviewed with regards to safety concerns retrieved from previous experience and failure analysis. Further requirements relate to the efficiency of the concept and environmental aspects. The organizations that guide and assess these technical reviews and eventually insure the vessels are typically referred to as classification societies.

For the Oceanbird project Det Norske Veritas (DNV) is appointed for the process of Approval in Principle (AiP).

An AiP is an early-stage assessment of the conceptual work to confirm the principle feasibility with regards to the rules for classification. It serves the purpose of identifying potential non-conformities at an early design stage and confirm the feasibility towards the stakeholders.

### 1.2 Scope and limitations

This thesis project presents conceptual designs for the telescopic mast of the Oceanbird. To streamline the AiP process all structural evaluations are carried out following selected guidelines by DNV.

The construction is heavily loaded and critical to the safe operation of the vessel, hence structural integrity is the governing objective. Rigid body accelerations and aerodynamic forces induced by the almost 80m tall masts require the ship to possess an exceptional transverse stability to ensure safe operation. To limit the heeling of the vessel an overall low vertical center of gravity (VCG) must be achieved. Consequently, a lightweight design is a crucial objective for the development of the mast sections. The research is focused on the following question:

*What is the most suitable design to achieve a strong, lightweight, and technically feasible mast section that complies with DNV rules for classification?*

To answer this question four conceptual design families are developed and
compared by their respective masses and costs. The range of concepts comprises of designs in FRP single skin as well as locally stiffened versions of it. Furthermore, sandwich constructions and the application of trusses are evaluated.

The two most promising concepts are designed in greater detail and presented as a recommendation for further development.

The study is limited with regards to the evaluation of natural frequencies, thermal loads and general fatigue analysis. Furthermore, the overall geometrical dimensions of the mast section provided by Wallenius Marine AB are kept constant and not used as design variables in this study, this holds true for the overlap between the mast sections too. The loads provided as input are grouped into ultimate limit state (ULS) and extreme loads (XLS), ULS representing the regular service in fully hoisted conditions. The XLS are a worst-case superposition of loads and the wing sails are to be reefed. The conceptual designs are evaluated against the ULS conditions only.
Chapter 2

Background

This section of the report discusses background information to facilitate the understanding of the report and the framework of the project.

2.1 Coordinate system and location IDs

The framework provided by Wallenius Marine AB defines, that the interfaces between the mast sections are to be designed in a sliding arrangement. On each wall of the mast sections "sliding pads" are mounted that enable reefing of the mast sections and transfer the loads between them. The pads have an important role in the transfer of internal loads. An in-depth understanding of them is required to follow the consequent report, hence, their locations and notation shall be introduced at this point, alongside with a global reference system used throughout the report. Figure 2.1 clarifies these definitions.
CHAPTER 2. BACKGROUND

Figure 2.1: Coordinate system and locations ID: The root of the cartesian coordinate system used throughout the project is located in the root of the mast, its x-axis oriented towards the leading edge and the z-axis towards the mast tip. The tags of the levels M12-M21 describe the z-location of the sliding pads that carry the internal forces from one mast section to the neighbouring ones. The total height of the mast from root to tip is denoted $h_m$. In the overlapping areas of two mast sections, each wall holds four sliding pads, which can be identified by layer number $i$ where $i$ is counted counterclockwise looking from above in negative $z$-direction. For example, the two sliding pads on level M22 that face towards the leading edge are denoted M22_5 and M22_6.

The respective z-coordinates of each layer are stated in table 2.1, the labels M12-M21 correspond to the notation in figure 2.1.

<table>
<thead>
<tr>
<th>Level</th>
<th>z-coordinate</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>M21</td>
<td>44264</td>
<td>mm</td>
</tr>
<tr>
<td>M22</td>
<td>39920</td>
<td>mm</td>
</tr>
<tr>
<td>M11</td>
<td>25336</td>
<td>mm</td>
</tr>
<tr>
<td>M12</td>
<td>21136</td>
<td>mm</td>
</tr>
</tbody>
</table>

2.2 Design principles Det Norske Veritas

In the shipping industry new built ships are to be approved by a classification society. Although the IMO cannot overrule national law, most member
countries adopt their standards and as international shipping requires at least two ports the guidelines are to be seen as quasi-mandatory. For the AiP of the Oceanbird’s design the classification society DNV is selected. An exploration is performed into their various rule sets to align the design principles in this work with the most likely rulebook to be applied in the AiP process. The principal design approach for composite structures displays minor differences over the various DNV classification rules and guidelines. Based on the similarities between the wing-sail structure and a wind turbine blade the decision is made, to incorporate the approach specified in the DNV rulebook for wind turbines into this project. The wing-sails of the Oceanbird project are operated in the same environmental conditions as offshore wind turbine blades, especially with regards to potential corrosion due to a saline environment, temperature ranges and the deteriorating influence of UV light. Furthermore, both structures are made from FRP, generate power from wind, are slender and bolted into the support at the root. This implies them being analysed as cantilever beams and inertial forces become of great importance.

In the rule book for wind turbine blades [5] the approach is based on a load factor to ensure a slightly over-estimated load case and a reduction factor to be applied to the material properties. The approach is expressed in the following form [5, p.22]

\[ S_d(\gamma_f \cdot F_k) \leq \frac{R_d}{\gamma_m} \tag{2.1} \]

where \( S_d \) represents the structural response (stress or strain) depending on the characteristic load \( F_k \) amplified by the load factor \( \gamma_f \). This factor shall account for unfavourable variation of loads from their design values [6, p.34, p.49] and is further specified in [7, tab. 4.2]. Assuming a design load case of normal operation and the occurrence of failure the partial load safety factor is to be taken as \( \gamma_f = 1.2 \) [7, tab. 4.3]. The loads provided by Wallenius Marine AB represent a worst-case scenario based on probabilistic considerations, thus it is assumed that the loads are to be interpreted as including the load factor \( \gamma_f \) already. On the right-hand side of the inequality equation stands the material property \( R_d \) reduced by the reduction factor \( \gamma_m \). The latter is described in more detail in the following equation:

\[ \gamma_m = \gamma_{m0} \cdot \gamma_{mc} \cdot \gamma_{m1} \cdot \gamma_{m2} \cdot \gamma_{m3} \cdot \gamma_{m4} \cdot \gamma_{m5} \tag{2.2} \]

where the subscripts indicate the following: \( m0 \): Base factor, \( mc \): Critically of failure, \( m1 \): Long term degradation of the resin, \( m2 \): Temperature effects, \( m3 \): Manufacturing, \( m4 \): Accuracy of the analysis, \( m5 \): Accuracy of the assumed
loads. Depending on the analytical setup and knowledge about the applied materials these factors can take slightly different values. The reduction factor can be mapped in the magnitude of \( \gamma_m \in [1.4, 2.4] \). Given the current state of the design the material factor as \( \gamma_m = 2.4 \) is used throughout the project.

### 2.3 Wear of lubricated sliding contacts

To avoid wear between two sliding metal surfaces two types of countermeasures can be employed [8, p.244]. They are characterized as measures that reduce the wear itself (surface treatment) and such that reduce the friction in the contacts (lubrication).

Lubrication is often provided, when large contact pressures are present (\( \geq 1.5 \text{GPa} \)). To achieve a maximum reduction in wear both methods should be applied simultaneously.

For low alloy steels the electro-chemical process of vanadizing and the surface infusion with boron particles have similar effects [8, p.253] and provide means to reduce wear for the given design case.
Chapter 3

Methods

In this chapter methods are introduced, that are utilized throughout the project. The background of these tools is described only to the degree, that is useful with regards to the application of the method. Sources are stated and give guidelines, where in-depth information, e.g. regarding the derivations of the formulae, can be found.

3.1 Material selection

The exploration of suitable materials is performed in GRANTA EduPack [9] on level 3 - Aerospace materials. In the software the concept of specific material properties is implemented and utilized within the project. When exploring material candidates for a lightweight structure one is not primarily interested in absolute material properties like strength or stiffness but instead these parameters should be judged in relation to the penalty they imply, for instance the increase in mass. An intuitive approach would be to divide the property of interest by the density of the material and compare these measures. However, this approach can be refined based on the type of structural member and what kind of parameter is to be optimized. For members in the construction the structural mass and stiffness can be expressed in two distinct equations. By substituting one into the other performance indices can be obtained that represent an optimized parameter for the selection of potential material candidates. Table 3.1 provides the performance indices [9] applicable when optimizing the material exploration for strength and stiffness at a low structural mass.
Table 3.1: Performance indices

<table>
<thead>
<tr>
<th>Structural member / optimize for:</th>
<th>Strength</th>
<th>Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam in bending</td>
<td>$\sigma^{2/3}/\rho$</td>
<td>$E^{1/2}/\rho$</td>
</tr>
<tr>
<td>Rod in tension/compression</td>
<td>$\sigma/\rho$</td>
<td>$E/\rho$</td>
</tr>
<tr>
<td>Panel in bending</td>
<td>$\sigma^{1/2}/\rho$</td>
<td>$E^{1/3}/\rho$</td>
</tr>
<tr>
<td>Panel in compression</td>
<td>$\sigma/\rho$</td>
<td>$E/\rho$</td>
</tr>
</tbody>
</table>

where $\rho$ denotes the density, $E$ the Young’s modulus and $\sigma$ the strength of the material. In GRANTA EduPack [9] further performance measures are available for various structural members and load cases. Also, other parameters combining three aspects (e.g. strength, density and costs) are presented.

### 3.2 General analytical relations

This section provides an overview of non-composite-specific analytical approaches utilized in the design of structural members.

#### 3.2.1 Euler type buckling of columns

The critical buckling loads for slender columns as described in equation 3.1 denote the critical bifurcation load at which the structure becomes unstable, meaning it can take one of two equilibrium states [10, p.3]. The critical load described in eq. 3.1 is sensitive to material imperfections or lateral disturbances, hence, structural members should have sufficient margins to this load to ensure a stable design.

$$P_{cr} = \frac{\pi^2 \cdot EI}{L_{eff}^2}$$  \hspace{1cm} (3.1)

$E$ and $I$ describe well-known stiffness parameters and $L_{eff}$ the effective length depending on the length of the member $L$ and the assumed BC.
Figure 3.1: Euler buckling: Effective length. The effective length in equation 3.1 is modified as \( L_{\text{eff}} = k \cdot L \). For case (a) \( k \) is to be taken as 0.5, for case (b) as 0.7, case (c) as 1.0 and case (d) and (e) as 2. The graphic is inspired by [10, tab.4.1]

### 3.2.2 Von Mises failure criterion

When subjecting a ductile isotropic material to a uniaxial stress it can be compared in a straightforward manner to the yield strength of the material to predict the onset of plastic deformation. In more complex loading scenarios this approach is too simplistic as different stress components add up. This gives rise to the question, how to assess the superposed stress states with respect the strength of the material. In 1913 R.v.Mises introduced an equivalent stress that can be computed based on equation 3.2 [11].

\[
\sigma_{\text{vM}} = \left[ \frac{1}{2} \cdot \sigma_1^2 + \frac{1}{2} \cdot \sigma_2^2 - \sigma_1 \sigma_2 + 3 \cdot \tau_{12}^2 \right]^{1/2}
\]  

(3.2)

The terms \( \sigma_i \) denote stresses in two directions, \( \tau_{12} \) the respective shear stress. The equivalent von Mises stress (\( \sigma_{\text{vM}} \)) provides a powerful tool to evaluate multi-axial stress states in metals and is, despite being a hypothesis, a widely accepted criterion for the onset of plastic deformation.

### 3.2.3 Stiffness of beam columns under axial and lateral load

The analytical analysis of beams is mostly done using Euler beam theory (EBT) which assumes small deflections. When beams are loaded in axial compression in combination with a lateral load this theory is not applicable since the two loads interact and amplify the lateral deflection as shown in figure 3.2.
EBT relates the deflection of a beam and the bending moment distribution in the form:

\[ EI \cdot \frac{d^2v(x)}{dx^2} = -M(x) \]  

(3.3)

where \( v \) represents the deflection in the y-direction, \( EI \) the bending stiffness and \( M(x) \) the bending moment distribution over the beam. As described above the inclusion of large deformations yields an additional bending moment component in the form \( M(x)_{\text{axial}} = v(x) \cdot P_{\text{axial}} \). It becomes clear, that the axial force will amplify the lateral deflection beyond the effect of the lateral load.

The presented method to compute the amplified deflection follows the approach presented in [10, p.173ff] in a modified version for a beam with both sides clamped.

The largest deflection can be attained at the mid of the beam and computed as

\[ v(L/2) = \frac{P_{\text{lateral}}L^3}{192EI} \cdot \eta(\psi) \]  

(3.4a)

\[ \eta(\psi) = \frac{3 \tan(\psi) - \psi}{\psi^3} \]  

(3.4b)

\[ \psi = \frac{\pi}{2} \cdot \sqrt{\frac{P_{\text{axial}}}{P_e}} \]  

(3.4c)

where \( P_{\text{lateral}} \) represents the lateral force and \( P_e \) the critical load for Euler Buckling according to equation 3.1.

From equation 3.4a it can be observed that the deflection \( v(L/2) \) consists of the deflection from the lateral force according to EBT for a beam with both sides clamped, amplified by the factor \( \eta(\psi) \), which represents the effect of the axial load.

For very small ratios \( P_{\text{axial}}/P_e \) equation 3.4c yields \( \psi \approx 0 \) and \( \eta(\psi) \approx 1 \). In this case the expression 3.4a converges to the solution according to EBT for a lateral force only.
When the axial force approaches the critical Euler buckling load $\psi \approx \pi/2$ for which $\eta(\psi)$ peaks to infinity. This is of course not a realistic solution, however, it shows, that for ratios of $P_{axial}/P_c \approx 1$ even the smallest lateral disturbance will cause great deflections.

### 3.3 Analytical equations for FRP structures

In this section the analytical approaches applied throughout the designs are presented.

#### 3.3.1 Failure criteria for FRP

The predication of failure in composite materials differs from the approach on metals as described in 3.2. Firstly, in contrast to metals, yielding in polymers and glass or carbon fibers is not governed by a shear driven mechanism [12, p.191]. Secondly, the potential anisotropy and general laminate architecture imply a direction-oriented failure behaviour. Assuming a unidirectional laminae of an arbitrary fiber and matrix two aspects shall be discussed to underline the difference to metal failure modes:

1. **Influence of the loading direction:** One can imagine, that pulling the ply in the fiber direction will stress mainly the fibers. Once their baring capacity is reached, they would fail and the matrix would likely not be able to sustain the tensile load in itself. Alternative, if one would pull transverse to the fibers the only continuous material that could support would be the matrix. Hence, in fiber direction, the ultimate strength would be dominated by the fibers, in transverse direction by the matrix.

2. **Influence of the sign of the loading:** Given that the load would be applied in the fiber-direction there is a difference in failure modes depending on if this load is compressing or tensioning the fibers. In tension the failure of the laminate is governed mainly by the strength of the fibers. In compression other physical processes such as fiber splitting, crushing or micro buckling by kink-band formations compete [13, ch.6].

Methods as the von Mises equivalent stress hypothesis (eq.3.2) cannot be used in the analysis of FRP materials as they are based on assumptions that hold for metals. Instead, a large variety of FRP failure criteria has been developed, the well-known **Maximum Stress, Maximum Strain** and **Tsai-Hill** criteria are discussed in detail below.
The Maximum Stress and Maximum Strain criteria are based on the idea, that laminates can be seen as being built up from many laminae (single plys of uni-directional fibers embedded in a matrix). Depending on the material choice the laminae have allowable strain and stress values depending on the loading direction and the signs indicating tension or compression. The fiber direction is denoted as 1, transverse to the fibers as 2, shear denoted as 12. The subscripts c and t represent the sign of the stresses, meaning tension or compression. The occurring stresses are divided by the respective allowables and for indices exceeding 1 failure is predicted.

\[
f_{\text{MaxStress}} = \max \left[ \frac{\sigma_1}{\hat{\sigma}_1}, \frac{\sigma_2}{\hat{\sigma}_2}, \frac{\tau_{12}}{\hat{\tau}_{12}} \right] \leq 1 \tag{3.5}
\]

\[
f_{\text{MaxStrain}} = \max \left[ \frac{\epsilon_1}{\hat{\epsilon}_1}, \frac{\epsilon_2}{\hat{\epsilon}_2}, \frac{\gamma_{12}}{\hat{\gamma}_{12}} \right] \leq 1 \tag{3.6}
\]

where the hat symbolizes the material strength and \( \sigma_1, \sigma_2 \) and \( \tau_{12} \) the occurring stresses in the individual laminae. As previously discussed, the direction of the occurring stress (1,2) has to be taken into account as well as its sign (t/c). Figure 3.3 depicts these relations and shows, that the allowables form a safe envelope in 2D, if one imagines the shear strength in out-of-plane direction one would obtain a 3D block.

Concerns have been raised regarding the corners of the envelope in figure 3.3. A laminae that is loaded in tension in 1-direction only until it almost
reaches the allowable would show roughly the same failure index, if it was loaded in tension in 1 and 2-direction up to the allowable. Alternatively formulated, the stresses in 1 and 2-direction do not compromise one another.

To circumvent this problem the Tsai-Hill criterion was developed [14]:

\[ f_{TH} = \frac{\sigma_1^2}{\sigma_1^2} - \frac{\sigma_1 \sigma_2}{\sigma_1^2} + \frac{\sigma_2^2}{\sigma_2^2} + \frac{\tau_{12}^2}{\tau_{12}^2} \leq 1 \]  

(3.7)

It takes this concern into account by adding combined terms into the equation. Figure 3.4 depicts the safe envelope predicted by the Tsai-Hill criterion:

![Safe envelope for the Tsai-Hill criterion. Compared to the approach of the Maximum Stress/Strain criteria the maximum allowable values can only be reached, without predicting failure, when the loading is applied strictly unidirectional. As the allowables remain the same in all three approaches the oval shape of the Tsai-Hill criterion could be fully mapped inside the envelope for the Maximum Stress criterion. This indicates that the Tsai-Hill criterion is more conservative than the respective counterparts. The figure is inspired by [13].](image)

The equations 3.5, 3.6 and 3.7 can only be evaluated, if the stresses and strains in the individual layers of the laminate are known. These can be obtained, by applying classical lamination theory (CLT), which shall not be described in detail here. The principal relation is given by the following equation:

\[
\begin{bmatrix}
N_x \\
N_y \\
N_{xy} \\
M_x \\
M_y \\
M_{xy}
\end{bmatrix} = \begin{bmatrix}
A_{11} & A_{12} & A_{16} & B_{11} & B_{12} & B_{16} \\
A_{12} & A_{22} & A_{26} & B_{12} & B_{22} & B_{26} \\
A_{16} & A_{26} & A_{66} & B_{16} & B_{26} & B_{66} \\
B_{11} & B_{12} & B_{16} & D_{11} & D_{12} & D_{16} \\
B_{12} & B_{22} & B_{26} & D_{12} & D_{22} & D_{26} \\
B_{16} & B_{26} & B_{66} & D_{16} & D_{26} & D_{66}
\end{bmatrix} \begin{bmatrix}
\varepsilon_x \\
\varepsilon_y \\
\gamma_{xy} \\
\kappa_x \\
\kappa_y \\
\kappa_{xy}
\end{bmatrix}
\]

(3.8)

The equation relates the external loads \( (N_i) \) and moments \( (M_i) \) on the left-hand side by the central \( ABD-Matrix \) to the structural response in terms of
strains ($\varepsilon_i$) and curvatures ($\kappa_i$). The central ABD-matrix is the description of the laminate itself. On closer inspection it becomes apparent, that for ABD-matrices with non-zero $B_{ij}$ terms the in-plane loads are related to curvatures. This is the case for non-symmetric layups and should in most cases be avoided. However, in some it can be a valuable property to utilize in the design process.

3.3.2 Buckling of orthotropic composite plates

The buckling load of a simply supported orthotropic single skin composite plate is given by the following equation [13, eq.5.56].

$$P_{cr} = \pi^2 \left[ D_{11} \left( \frac{m}{a} \right)^2 + 2 \left( D_{12} + 2D_{66} \right) \left( \frac{1}{b} \right)^2 + D_{22} \left( \frac{a}{m} \right)^2 \left( \frac{1}{b} \right)^4 \right]$$

(3.9)

Where the terms $D_{ij}$ represent the entries of the D-matrix that can be obtained for a FRP by the application of CLT. The number of half-wave lengths over the short span of the panel is set to $n = 1$, $a$ and $b$ describe the panel dimensions.

3.4 Analytical equations for sandwich structures

In this section the methods used in the design of sandwich member are discussed. In the analysis of sandwich structures a distinction is made between beams and panels. In the light of the geometrical parameters of the mast wall sections they can be treated as beams [15, example 9.5] and the methods provided in the following sections describe the beam solutions to failure modes. An exception is made for general buckling of sandwich structures (ref. 3.4.2).

3.4.1 Basic parameters for sandwich structures

Two vital parameters to describe the properties of sandwich panels are the bending stiffness $D$ and the shear stiffness $S$. These properties are described in the following equations [15, ch.3].

$$D = \frac{E_f \cdot b^3}{6} + \frac{E_f \cdot t_f \cdot d^2}{2} + \frac{E_c \cdot t_c^3}{12}$$

(3.10a)

$$S = \frac{G_c \cdot d^2}{t_c} \approx G_c \cdot d$$

(3.10b)
In these equations \( t_f \) represents the face sheet thickness, \( t_c \) the thickness of the core and \( E_f \) and \( E_c \) the respective Young’s moduli. \( d \) is the distance between the centroids of the face sheets and computes as \( d = t_c + t_f \).

### 3.4.2 Buckling of sandwich panels with isotropic face sheets

In the analysis of sandwich panels in general but particularly for stability the shear deformability of the core has to be taken into account. This is important when evaluating the boundary condition (BC) of a sandwich panel and it implies the need to further refine the classical BC. The terms \textit{hard} and \textit{soft} BCs are clarified in figure 3.5.

![Figure 3.5: Simply supported boundary conditions for sandwich panels. Both figures represent a simple support on the left-hand side of the panel, however in contrast to other structural members for sandwich panels a refinement of the definition of the BC is required. Sketch (a) represents a hard and sketch (b) a soft BC. For the hard BC the moment along the out-of-plane axis is locked whereas for the other one this DoF is free.](image.png)

The critical buckling loads for a sandwich panels with all edges hard simply supported and isotropic face sheets can be evaluated according to equation 3.11 [15, ch.9.6].

\[
P_{cr} = \frac{\pi^2 \cdot D}{b^2 \cdot (1 - \nu^2)} \cdot \frac{\left[ \frac{m}{mb} + \frac{a}{mb} \right]^2}{1 + \frac{\pi^2 \cdot D}{S \cdot (1 - \nu^2) \cdot b^2} \cdot \left[ \frac{mb}{a} \right]^2 + 1}
\]

(3.11)

where \( m \) and \( n \) represent the numbers of half waves over the panel, \( D \) the flexural rigidity according to equation 3.10a, \( S \) the shear stiffness according to equation 3.10b, \( b \) the shorter span of the panel and \( a \) the longer side.

In most cases it is sufficient to evaluate the critical buckling load for \( n = 1 \), however, for which number of half wavelengths \( m \) the lowest buckling load would occur is not obvious and depends on the aspect ratio (AR) \( AR = a/b \). Figure 3.6 visualizes this behaviour.
Figure 3.6: Critical buckling loads for a sandwich panel with isotropic FRP face sheets. The lowest critical buckling load would only be given by mode number 5 for aspect ratios below roughly 5.5, for more slender panels other mode numbers become dominant in the design process.

3.4.3 Core shear failure

For a sandwich plate subjected to bending moment and under the assumptions of thin face sheets and a weak \((E_f >> E_c)\) core, the approximation holds, that the face sheets carry the bending stresses whereas the core takes on the shear stresses. Under shear loading the core can fail, a phenomenon that is described as core-shear failure. Equation 3.12 provides a tool to assess this risk [15, p.7.3].

\[
\tau_{c,sf} = \frac{T(z)}{d} \leq \hat{\tau}_c \tag{3.12}
\]

\(T(z)\) denotes the shear force, \(d\) the distance between the centroids of the face sheets and \(\hat{\tau}_c\) the shear strength of the core.

3.4.4 Face wrinkling

Under in-plane compressive stresses the face sheets can either locally indentate the core or separate from it. Both phenomena occur in form of sinusoidal shape deformations of the face sheet characterized by a short wavelength. Equation 3.13 provides a method to assess this concern [15, p.7.3].

\[
\sigma_{f,fw} = 0.5\sqrt{E_F \cdot E_c \cdot G_c} \tag{3.13}
\]
It shall be noted, that although the equation only holds material properties it is to be interpreted as an upper allowable stress limit and therefore relates to the geometry of the construction too.

### 3.4.5 Shear crimping

Shear crimping or shear buckling is a stability concern, that is related to the low shear modulus $G_c$ of the conventional core materials compared to the one of the face sheets. It realizes in short half waves along the panel and its critical load threshold can be evaluated according to equation 3.14. [15, p.7.4]

$$\sigma_{f,sc} = \frac{S}{2 \cdot t_f}$$

(3.14)

It relates the shear stiffness $S$ (see eq. 3.10b) to the combined thickness of the two face sheets $2 \cdot t_f$ and is to be interpreted as a maximum allowable compressive stress.

### 3.4.6 Local indentation

When subjecting a sandwich element to a point load normal to the in-plane directions the core might crush. As stress relates to force divided by area this concern can usually be avoided by increasing the contact surface or by the selection of a core with a higher compressive strength $\sigma_{cc}$ [15, p.7.5]. Equation 3.15 provides a simple way to assess this concern.

$$A_{f,li} \geq \frac{P}{\sigma_{cc}}$$

(3.15)

### 3.5 Composites evaluation tools

In this work two tools for the evaluation of laminates are used. One is an implementation of CLT in Python, the other is FEA of FRP constructions in ANSYS R02.

The CLT code allows the user to evaluate a sheet of FRP for uniform load cases. The solution can be obtained in minutes and the results are intuitive and provide sufficient information to improve the design. The downside is, that no complex geometries or loading scenarios can be assessed.

The FEA analysis in ANSYS R02 enables the structural analysis of the fully modelled geometry in FRP in almost any load case and loading condition. It comes at the disadvantage, that the computational effort in terms of time and memory requirements are substantial.
3.5.1 CLT code

The evaluation of stresses and strains in composite materials and ultimately the judgement, if failure in the individual laminae of a composite is to be expected is not as straightforward as for isotropic materials as the failure mechanism is generally different and more complex in a sense, that two materials (fibers and resin) are acting together.

To enable a quick judgement for simple load cases CLT and the failure criteria described in section 3.3.1 are implemented in Python. Figure A.6 in the appendix provides an overview of the code architecture.

3.5.2 ANSYS ACP

With its static structural cell ANSYS R02 provides a powerful tool to carry out finite element analysis (FEA) to evaluate structures with a complex geometry and/or loading scenario.

Alongside the static structural cell an additional analysis system is available, that enables the user to carry out the analysis of composite or sandwich structures, ANSYS ACP. It contains a pre (ACP PRE) and a post processor (ACP POST). The first one allows the user to define the laminate in terms of material, orientation angles and thicknesses. The FEA is consequently carried out in the static structural cell and the results further processed in ACP POST. A total of 13 failure criteria are available to judge the structures fitness for the load case.

3.5.2.1 Convergence

In FEA a trade-off must be made between a refinement of the analysis and computational effort. In non-composite FEA the static structural cell allows the user to enable adaptive meshing. During the solution the solver varies the mesh density to find the best trade of where the least computational effort leads to stable results. When using the ACP modules this option is not available as the mesh is generated in the pre-processor and cannot be influenced by the static structural cell during the solution. To allow the user to judge a suitable mesh density a routine is implemented in Python. The ACP post processor facilitates the extraction of the results of the failure criteria at each node of the mesh as .csv files. The Python routine allows the import of these files and the graphical display of the results as shown in figure 3.7.
Figure 3.7: Boxplots over the highest 1% of the inverse reserve factor for the failure indices exported from ANSYS ACP POST for FEA solutions to different mesh configurations. The conceptual idea is to not only look into the highest result of the failure indices and thereby judge convergence but to assess the highest x%-.

The boxes indicating the range between the 25% - 75% quartiles, alongside the medians plotted in red give an indication of movement in the center of the top x%-data sets. The whiskers and outliers (these should not be judged as such in this application) enable simple access to the information, if the highest failure indices are converging.

Whether or not the most extreme data points are to be seen as a convergence issue or a singularity is to be judged by the user. The range between the 25% and the 75% quartile alongside the median over the highest x% of the entire data give a good indication, if the overall mesh density is yielding convergence. In the example provided in figure 3.7 convergence seems to have been reached in the configuration Mesh110.
Chapter 4

Design loads

The loading of the mast structure is rather complex and related to various decisions in the design cycle of the vessel. Major components are listed below alongside with the influencing variables in brackets.

1. Aerodynamic forces on the wings in terms of lift and drag (angle of attack, wind speed, moisture content in the air, boundary layer of the wind, wing-wing, and wing-hull interaction).

2. Rigid body accelerations (masses, global accelerations in six degrees of freedom of the hull).

3. Aeroelasticity (structural mass, stiffness, damping).

Further aspects that influence the loading of the structure are the frequency of the changes in amplitude in cyclic loading and vibrations.

Wallenius Marine AB selected a probabilistic approach on the definition of the design loads. For the intended route wind and wave statistics are evaluated and further processed into design accelerations following the approaches described in the set of DNV guidelines for ships and wind turbines. By means of superposition according to the DNV document [16] two load cases are defined:

- **ULS - Ultimate limit state**: They describe the loads when the masts are fully hoisted and the wings-sails are in operation and also referred to as regular service loads.

- **XLS - Extreme loads**: These cover more extreme superpositions of loads, in this configuration the wing-sails are to be reefed.
Both cases comprise of a set of design loads in terms of a maximum bending and torsional moment at the mast root, that stem from transverse aerodynamic and inertial forces. Furthermore, an upper limit for the vertical acceleration is defined. Effectively the approach imposes operational restrictions for the vessel to maintain these design parameters below the stated thresholds. The structural evaluations presented in this thesis project are based on the ULS definition of loads, respective parameters are described in table A.1.

4.1 Global loads

As only extreme values at specific locations for the ULS are given, the distribution of bending moment in x and y direction, as well as the shear forces over the wings must be approximated.

The span of the wing sections decreases stepwise from bottom to top of the mast, it is therefore assumed, that the aerodynamic pressure and distribution of mass is decreasing linearly with height as well.

By means of integration of the general distributed load with an unknown initial value \( b \) and a slope \( a \), \( p(z) = b - a \cdot z \) the shear force and bending moment distribution over the mast can be obtained in a general form. This approach yields unknown integration constants that are obtained by substitution of the BCs: \( T(h_m) = M(h_m) = 0 \), \( h_m \) representing the total height of the mast.

\[
T(z) = b \left[ z - \frac{1}{2h_m}z^2 - \frac{1}{2}h_m \right] \quad (4.1a)
\]

\[
M(z) = b \left[ \frac{1}{2}z^2 - \frac{1}{6h_m}z^3 - \frac{1}{2}h_m z + \frac{1}{6}h_m^2 \right] \quad (4.1b)
\]

By inserting the ULS into eq. 4.1b the amplitude \( b \) can be found as \( b = 6M_{b,max}/L^2 \) where \( M_{b,max} \) is the maximum bending moment at the root of the mast according tab. A.1.

Assuming the bending moment towards the leading edge to be two thirds of the one in y-direction, expressions for the bending moment distribution in x and y-direction are now available for the design process (eq. 4.2a, 4.2b).

It is furthermore assumed, that the distribution of the torsional moment \( M(z) \)
is linearly decreasing towards the tip of the mast.

\[ M_x(z) = \frac{2M_{b,\text{max}}}{h_m^3} \left[ z^2 - \frac{1}{3h_m}z^3 + h_mz + \frac{1}{3}h_m^2 \right] \quad (4.2a) \]

\[ M_y(z) = \frac{3M_{b,\text{max}}}{h_m^3} \left[ z^2 - \frac{1}{3h_m}z^3 - h_mz + \frac{1}{3}h_m^2 \right] \quad (4.2b) \]

\[ M_z(z) = M_{z,\text{max}} [h_m - z] \quad (4.2c) \]

where \( M_{z,\text{max}} \) describes the maximum torsional moment at the root of the mast according tab. A.1.

This set of equations combined with the information presented in table 2.1 can be used to evaluate the external loads at each location of interest. The distributions above represent the extreme conditions in regular service and are assumed not to occur in conjunction. If a bending moment of \( M_y(0) = M_{b,\text{max}} \) is present at the mast foot, the respective component in x-direction is to be taken as \( M_x(0) = 0 \text{Nm} \).

### 4.2 Local loads

In between the mast section sliding pads are mounted to enable low-friction hoisting and reefing operation of the wing-sails. Hence, the load paths are defined by the placement of these pads. A total of 16 load pads are placed in each overlap between two mast sections (ref. fig.2.1). This makes the construction highly over-determined and gives rise to the concern, that loads will not be evenly distributed. It is thereby decided, to assume the transfer of torsional loads by all pads, however, the much more dominant force components induced by the global bending moment are assumed to be taken on by two load pads per mast section overlap only. This represents a (perhaps unreasonably) conservative approach but within the given information in the framework of this project this measure is judged necessary to safeguard the crew and ship.

The local loads are derived from the global loads as described in section 4.1 for torsion and bending of the masts individually and consequently superposed. The respective \( z \)-coordinates for each point of interest in the structure are listed in table 2.1. The transfer of the global torsional moment is assumed to take place as shown in figure 4.1.
According to the approach described in figure 4.1 the torsional components of the local loads $F_t$ are computed according eq. 4.3. For small values of $\alpha$ the lever arm can be approximated as $r_t \approx d_{xi}/2$.

$$F_{t,i} = \frac{M_t(z_i)}{4r_t}$$

(4.3)

The transfer of the global bending moment according to equation 4.2a and 4.2b is predicted to occur according to figure 4.2.
Table 4.1 summarizes the local loads as a superposition of the torsional and bending components. The applied code corresponds to the notation discussed in section 2.1, e.g., $F_{M12,6}$ refers to the load on sliding pad 6 on level M12, oriented towards the leading edge. All local loads are acting normal to the load pads, on the levels M21 and M22 towards the center of the mast (mast section 3 is on the outside of section 2) and outwards at the levels M11 and M12.

For smoother referencing it is advised to revisit figure 2.1 before inspecting table 4.1. It describes the magnitudes of the local loads acting on the sliding pads.

Table 4.1: Local design loads.

<table>
<thead>
<tr>
<th>Local load / Level</th>
<th>M12</th>
<th>M11</th>
<th>M22</th>
<th>M21</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{Mi,1}$</td>
<td>1.365</td>
<td>1.697</td>
<td>0.411</td>
<td>0.554</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,2}$</td>
<td>1.365</td>
<td>1.697</td>
<td>0.411</td>
<td>0.554</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,3}$</td>
<td>1.962</td>
<td>2.467</td>
<td>0.560</td>
<td>0.781</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,4}$</td>
<td>1.962</td>
<td>2.467</td>
<td>0.560</td>
<td>0.781</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,5}$</td>
<td>1.365</td>
<td>1.697</td>
<td>0.411</td>
<td>0.554</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,6}$</td>
<td>1.365</td>
<td>1.697</td>
<td>0.411</td>
<td>0.554</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,7}$</td>
<td>1.962</td>
<td>2.467</td>
<td>0.560</td>
<td>0.781</td>
<td>MN</td>
</tr>
<tr>
<td>$F_{Mi,8}$</td>
<td>1.962</td>
<td>2.467</td>
<td>0.560</td>
<td>0.781</td>
<td>MN</td>
</tr>
</tbody>
</table>

On each layer (M11-M22) eight pads are mounted, to obtain for instance the load on pad 8 on level M12 the reader should refer to the entry in row eight and column one.
Chapter 5

Conceptual designs

The initial geometry provided by Wallenius Marine is characterised by a hollow, thin-walled, rectangular structure with large cross-sectional dimensions as presented in appendix A.1.

In an initial approach the mast section is designed as a single-skin construction from FRP based on the unmodified geometrical layout. The assessment of the structural stability revealed, that, regardless of the material choice, the wall sections are unfit to carry the large compressive forces induced predominately by the global bending moment and would collapse under global loads far below the ULS.

To cope with these challenges several conceptual designs are developed and benchmarked against the initial design (in the following referred to as benchmark design BD, see section 5.1). The assessment is performed based on the total structural mass, the costs in the light of prices for the raw material and the roughly estimated costs for manufacturing. Figure 5.1 depicts the evaluated conceptual design families. To limit the further design work the two most promising concept families, namely the design of a truss structure and the construction of the mast section in sandwich panels are selected and developed in a greater resolution.
Figure 5.1: Conceptual designs families: To cope with inability of the benchmark design to safely sustain the compressive loads in the wall sections four concept families are developed. In the first group the walls are strengthened solely by increasing the wall thickness in GFRP or CFRP. The second family is composed around the idea, to add material in strategic locations rather than simply increasing the wall thickness everywhere to increase the local buckling strength. The truss structure presents a more consequent approach on the idea for the locally stiffened walls. Instead of adding stiffening members the problematic continuous walls are removed and the loads are carried by the stiffening member only. The last conceptual group includes variations of a sandwich concept. The governing idea is to utilize the outstanding stiffness properties of sandwich panels to prevent the wall sections to buckle under the compressive loads.

The base for the decision process is presented in appendix A.6. It shall be made clear that some of the entries of the table are outdated, representing the state of knowledge at the time of the decision, and are presented for the sake of transparency only.

5.1 The benchmark design

As part of the framework for the thesis an initial design has been provided by Wallenius Marine AB. For mast section 2 the walls are intended to be manufactured in FRP.
Regardless if the walls are manufactured in GFRP or CFRP, the critical thresholds for local buckling according to equation 3.9 would be far below the compressive stresses induced by the ULS loads. This finding injects the need to the project to evaluate the following conceptual families. For this structure in GFRP the mass is estimated to be 11.8ton. This value serves as a rough estimate to benchmark the other concepts.

### 5.2 Truss structure

A truss structure is developed and presented in figure 5.3. The overarching idea is to combine the benefits of two material classes to obtain a simple, strong, stiff, and lightweight design.
5.2.1 Conceptual Idea

The four side pillars as indicated in figure 5.3 are designed in high strength low alloy steel (HSLA steel). The high Young’s modulus ensures global bending stiffness and resilience against Euler type buckling of the entire structure. The slenderness of the mast in conjunction with the high loading in bending and the applied material factor as described in section 2.2 yield tough strength requirements on the side pillars, for a design in carbon steel this would have implied considerably larger scantlings and resulted in an unacceptably high structural mass.

The outer surfaces of the side pillars will provide the contact with the adjacent mast sections. Steel is more robust than composite materials and require less efforts to cope with localized loads. The contact surfaces between the mast sections are designed as sliding supports and offer the opportunity to act as sliding surfaces, given that they are prepared according section 2.3.
The horizontal and cross-braced members are hollow tubes designed from a 0-dominated CFRP, meaning most fibers are oriented in axial direction. The requirements on the braces are such, that they need to be stiff enough to be safe from Euler type buckling, simultaneously they need to be light to keep the total structural mass within reasonable limits. Despite its higher price carbon is selected as the material with the highest specific stiffness.

The connections between the structural members are achieved by means of connecting elements in HSLA steel that are adhesively bonded into the CFRP tubes and bolted to steel brackets that are welded to the side pillars. This reveals an additional benefit of designing the side pillars in steel. If they were made from FRP, the joining of side pillars and braces would require a more complex manufacturing technique.

5.2.2 Analysis of the side pillars

In the design of the side pillars five concerns are identified. They are depicted and described in figure 5.4 and the respective analytical approach described in the following sections.

![Figure 5.4: Design challenges in the side pillars of the truss structure. (a) Euler type buckling in sections of the side pillars between two horizontal braces. (b) Euler type buckling of the entire structure under self-weight in design acceleration. (c) Yielding of the material in compression or tension. (d) Yielding as a result of large deflections caused by the interaction between axial and lateral forces. When the mast sections slide into each other the lower edge of the upper mast section presses inwards at locations, where no horizontal brace can counteract the deflection. In superposition with the compressive forces described in case (a) this makes an additional load case of concern. (e) Non-conformity with global stiffness requirements.](image-url)
(a) Euler type buckling of sections of the side pillars: The loads in the side pillars are evaluated according the equations below:

\[
\sigma_t(z_{12}) = -\frac{M_i(z_{12})}{I_{ii}} \cdot \frac{d_{yi}}{2}
\]

\[
\sigma_c(z_{12}) = \frac{M_i(z_{12})}{I_{ii}} \cdot \frac{d_{yi}}{2} + \frac{3}{2} \cdot \frac{m_{sec234} \cdot a_z}{4A_{SP}}
\]

\[
F_c = \sigma_c \cdot A_{SP}
\]

where \( A_{SP} \) denotes the cross-sectional area of a single hollow side tube and \( d_{yi} \) the distance between the pillars according to figure 5.3. \( M_i \) denotes the global bending moment the mast is subjected to and \( I_{ii} \) its moment of inertia. The first term in equation 5.1b represents the stresses caused by the global bending moment, the second term the contribution of the weight from the sections 2,3 and 4 (\( m_{sec234} \)) in design acceleration \( a_z \). Here the assumption is made, that caused by global deflections of the mast the distribution of loads is uneven over the 4 side pillars. This concern is addressed by introducing a generic amplification factor \( 3/2 \) into the expression 5.1b.

The critical compressive load in the vertical members is evaluated according to equation 3.1, where the BCs are assumed to be pinned on both sides and consequently \( L_{eff} = k \cdot L = 1 \cdot 2.29m = 2.29m \).

(b) Euler type buckling of the entire mast structure: The slenderness of the mast give rise to the concern, that the structure could collapse under its own structural mass in design acceleration. To evaluate this risk the critical buckling load is derived using equation 3.1. The evaluated load case is to assume the entire mass of a wing sail (125ton) concentrated on top of a 77m high cantilever beam. Consequently \( L_{eff} = k \cdot L = 2 \cdot 77m = 154m \).

This load case is judged highly conservative, as the structural mass is distributed over the height of the masts.

(c) Yielding under design loads: The compressive and tensile stresses are computed according to equation 5.1a and 5.1b. These stress levels are compared to the materials yield strength reduced by the material factor \( \gamma_m \).

\[
|\sigma_{c/t}| \leq \frac{\hat{\sigma}}{\gamma_m}
\]

where the subscript \( t \) and \( c \) indicate the tensile and compressive load case and \( \hat{\sigma} \) the yield strength of the material.
(d) Yielding under combined axial and lateral loads: The load case described in figure 5.4 case (d) can only occur on the upper side of mast section 2, not on the lower end. This is due to the fact, that the mast sections are increasing in diameter towards the top of the masts. While on the upper side this load case yields a superposition of the lateral with compressive loads, on the lower side it always yields a combination of lateral and tensile forces. Hence the critical load case caused by large deformation described in section 3.2.3 must be evaluated at the overlap between mast section 2 and 3. At the overlap between mast section 1 and 2 the tensile stresses from global bending and local, lateral deflection are summed and compared to the yield point of the material.

The lateral load is derived from the situation depicted in figure 4.2. It can be observed that the magnitude of this force largely depends, on how far the supports are located from each other. Figure 5.5 displays, how this effect is used to obtain a lightweight design.

Figure 5.5: Lateral forces at the connections of two mast sections due to the global bending moment $M_x$. As the mast sections slide into each other two observations can be made: Firstly, the upper contact in mast section two always remains at a location, where a horizontal brace prevents lateral deflection. Secondly, as mast section 3 slides down, the distance between the supports increases. Consequently, the magnitude of the lateral loads decreases. The situation on the left-hand side depicts the mast at full extension. In the operational condition on the right-hand side the lateral force from the lower supports acts at half length between two supporting braces and the consequent deflection works as a lever arm for the axial forces in the vertical members.
The lateral deflection under combined axial and lateral loads as depicted in the situation on the right-hand side of figure 5.5 and described in section 3.2.3 is evaluated according to equation 3.4a. The obtained mid-point deflection is then expressed in terms of an equivalent maximum bending moment and by the use of standard beam-theory solutions and a variation of equations 5.1a, 5.1b and 5.2 connected to an allowable upper stress level. The results are expressed graphically below:

Figure 5.6: Deflection under axial and lateral load. On the x-axis the ratio of the actual compressive force $P$ over the critical force for Euler type buckling $P_e$ are marked. The y-axis shows the lateral deflection in meters. The dotted horizontal line corresponds to the deflection as if no axial force would be present and the dashed horizontal line displays the upper limit for the deflection before yielding of the material is to be expected. For a known axial and lateral load, the intersection between the blue and horizontal, dashed line defines a minimum critical Euler load the pillar must be designed for to stay within safe limits against failure in yielding. The vertical line marks the current design stage.

In the current design, the sections between two braces can sustain such high compressive loads before Euler type buckling is expected, that the ratio of the actual and the sustainable forces is far from the threshold, were the combined effect of lateral and axial loading would cause deflections and stresses approaching the critical stress limit.

The lower side of the mast section holds the critical load case. Here a superposition of tensile loads from global bending and the ones induced by the lateral deflection defines the load case. The analytical approach is given by

$$\sigma_{t\text{glob}} + \sigma_{t\text{lat}} \leq \frac{\sigma}{\gamma_m}$$

(5.3)

where $\sigma_{t\text{glob}}$ corresponds to the tensile axial loads from global bending and $\sigma_{t\text{lat}}$ to the ones caused by the lateral deflection due to the lateral point load.
To ensure that the superposition of tensile loads does not suffer from effects that are difficult to capture in an analytical way, the situation is recreated in ANSYS and the tensile stresses are evaluated using the von Mises stress hypothesis (eq. 3.2). Since the stresses are mostly oriented in z-direction, the comparison between analytical stresses compared to the von Mises hypothesis holds and the results indicate a difference in the solutions of approximately 5.5% referencing the analytical expression.

(e) Non-conformity with global stiffness requirements: The deflection of the mast tip depends on the BCs, the distributed bending moment, and the moment of inertia. In the computations of the moments of inertia, it is for reasons of simplicity assumed that only the side pillars contribute to it. The mast tip deflection is computed by application of EBT for a cantilever beam in the form:

\[ v_{\text{max}} = \frac{Wa^2}{6EI}(3z - a) \]  

(5.4)

where \( W \) denotes a point load and \( a \) the distance of the load from the fixed support. In this application \( a = CEA \) according table A.1 and the point load is obtained by dividing the maximum bending moment at the mast root by the center of effort \( W = M_{\text{bmax}}/CEA \).

Table 5.1 summarizes the final scantlings for the side pillars (SP) of the truss structure. The table forms the basis for the results presented in table 5.2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>HSLA steel</td>
<td>-</td>
<td>SP</td>
</tr>
<tr>
<td>Outer diameter</td>
<td>210.00</td>
<td>mm</td>
<td>SP</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>16.00</td>
<td>mm</td>
<td>SP</td>
</tr>
<tr>
<td>Total length</td>
<td>23130</td>
<td>mm</td>
<td>SP</td>
</tr>
<tr>
<td>Max. distance between braces</td>
<td>2290</td>
<td>mm</td>
<td>SP</td>
</tr>
<tr>
<td>Min. distance between braces</td>
<td>1150</td>
<td>mm</td>
<td>SP</td>
</tr>
</tbody>
</table>

The results of the evaluation of failure modes are presented in form of an actual value, describing the structural response in terms of force, stress or deflection side by side with the critical threshold for the respective parameter (e.g., critical Euler buckling load, max. deflections). The latter values contain the material factor \( \gamma_m \) as discussed in section 2.2 except of case (e), as the
material factor applies to strength and stability but not to stiffness evaluations. The column margin of safety (MOS) describes, how critical the current solution is with regards to the limiting thresholds. MOS > 1 describes a scenario, that is to be seen as safe and in accordance with the selected DNV principles for classification.

Table 5.2: Truss structure: Summary side pillar design

<table>
<thead>
<tr>
<th>Concern</th>
<th>Actual value</th>
<th>Crit. value</th>
<th>Unit</th>
<th>MOS</th>
<th>Keyword/Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>1.42</td>
<td>7.42</td>
<td>MN</td>
<td>5.23</td>
<td>Critical Euler load</td>
</tr>
<tr>
<td>(b)</td>
<td>1.26</td>
<td>1.52</td>
<td>MN</td>
<td>1.21</td>
<td>Critical Euler load</td>
</tr>
<tr>
<td>(c)</td>
<td>145.66</td>
<td>625.00</td>
<td>MPa</td>
<td>4.29</td>
<td>Compressive stress</td>
</tr>
<tr>
<td>(c)</td>
<td>108.22</td>
<td>625.00</td>
<td>MPa</td>
<td>5.78</td>
<td>Tensile stress</td>
</tr>
<tr>
<td>(d)</td>
<td>1.92</td>
<td>6.37</td>
<td>mm</td>
<td>3.31</td>
<td>Upper side</td>
</tr>
<tr>
<td>(d)</td>
<td>601.55</td>
<td>625.00</td>
<td>MPa</td>
<td>1.04</td>
<td>Lower side</td>
</tr>
<tr>
<td>(e)</td>
<td>3.01</td>
<td>5350</td>
<td>mm</td>
<td>1.78</td>
<td>Mast tip deflection</td>
</tr>
</tbody>
</table>

The driving case for the design of the side pillars is case (d) on the lower end of the mast section. The decisive facts are the low location on the mast (high bending moment), superposition of the tensile loads from global bending and the lateral force that is not directly supported by a horizontal brace.

### 5.2.3 Analysis of the braces

In truss structures the common assumption is, that the braces are pin-supported, meaning that both sides of the members are free of bending moment. As no bending moment can be introduced from either side, the entire brace is assumed to be loaded in compression and tension only. This simplifies the structural analysis to the following two design concerns (note, the itemisation from the side pillar structure is continued for simplified referencing):

- f. Euler type buckling of the brace under compressive loads.
- g. Fiber failure under design loads.

**Axial forces:** Usually the axial forces in a truss construction are evaluated according the cutting method proposed by August Ritter [17], however, this method assumes, that all elements are held in place by a pinned support. This simplifies the analysis as it provides a way to work with a statically determined structure. The perspective holds, if the connection of the members is
realized in a joint, in which all attached members connect to one point of rotation. Hence no bending stresses occur in the analysis. This assumption does not hold for the proposed structure. The side pillars are continuous, with a rather massive cross section, which implies, that the axial forces in the braces are somewhat averaged. Furthermore, the hybrid material selection implies a variation in stiffness between the braces and the side pillars. Hook’s law relates stresses and strains by the Young’s modulus. In a hybrid structure, where two materials share the same strain, they experience different stress levels due to the variation in stiffness. This is an additional indication, that the derivation of axial forces in the braces according [17] would be an overestimation, as the cutting method by Ritter does not provide an opportunity, to implement, this phenomenon.

To circumvent the challenge a simulation is set up in ANSYS R02 to obtain the axial forces, detailed insights into the FEM analysis can be found in appendix A.3.

According to figures A.1 and A.2 the axial forces in the braces vary significantly depending on the location, orientation in x- or y-direction and weather they are horizontal members or angled ones. To enable easier production the following approach on the scantlings for the truss structure is selected: A distinction is made between horizontal and angled members and the respective orientation along the x- or y-axis is considered. The location of the members is not further considered and the highest loaded member over the height of the mast dictates the design load. The approach leads to the design loads described in table 5.3. $HB_x$ and $HB_y$ represent the horizontal members in x- and y-direction, $CB_x$ and $CB_y$ the respective forces in the crossed-members. The entries in the table are taken from figures A.1 and A.2 and 5.3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$HB_x$</th>
<th>$HB_y$</th>
<th>$CB_x$</th>
<th>$CB_y$</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial design load</td>
<td>±0.29</td>
<td>±0.36</td>
<td>±0.95</td>
<td>±1.24</td>
<td>MN</td>
</tr>
<tr>
<td>Length in ST* spacing</td>
<td>2.09</td>
<td>2.29</td>
<td>3.10</td>
<td>3.24</td>
<td>m</td>
</tr>
<tr>
<td>Length in HD* spacing</td>
<td>2.09</td>
<td>2.29</td>
<td>2.38</td>
<td>2.56</td>
<td>m</td>
</tr>
</tbody>
</table>

* ST and HD representing areas of standard and high density spacing of the braces (see fig. 5.3).

(f) Euler type buckling: The braces are designed against Euler type buckling using equation 3.1. The BCs are assumed pinned on both sides [18, p.50].
Therefore, \( L_{\text{eff}} = k \cdot L = 1 \cdot L \). The stiffness is assumed 100GPa for a 0-dominated CFRP.

**g) Fiber failure under axial loads:** The concern regarding the tensile loads in the braces is laminate failure. The assessment of this concern is done using the Maximum Stress, the Maximum Strain and the Tsai-Hill criterion for fiber failure in composites described in equations 3.5 - 3.7 by applying the loads in the CLT code described in appendix A.7.

For simplification the four-ply laminate \([0 \, 45 \, -45 \, 0]\) is used and the thickness of each layer adjusted, so that the required wall-thickness is reached without having to model large amounts of layers. The input for the CLT is commonly done in form of line loads and moments. In case of cylindrical tube loaded in axial tension/compression only the line load can be expressed in N/mm as:

\[
N_x = \pm \frac{F \cdot \gamma_m}{\pi \cdot d} \quad (5.5)
\]

where \( F \) represents the axial design load and \( \pi \cdot d \) the circumference of the tube’s cross section. In this case the DNV material factor \( \gamma_m \) is implemented on the load-side of the equation instead of the material side as otherwise done throughout this report. The reason is the simplified implementation of the factor into the Tsai-Hill criterion.

Table 5.4 summarizes the scantlings for the bracing members.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>( HB_x )</th>
<th>( HB_y )</th>
<th>( CB_x )</th>
<th>( CB_y )</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer diameter</td>
<td>105</td>
<td>115</td>
<td>170</td>
<td>190</td>
<td>mm</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>10</td>
<td>10</td>
<td>15</td>
<td>15</td>
<td>mm</td>
</tr>
</tbody>
</table>

Based on the scantling presented in table 5.4 the concerns (f) and (g) are evaluated and the results presented in the table below:
Table 5.5: Truss structure: Results structural evaluations

<table>
<thead>
<tr>
<th>Case</th>
<th>Parameter</th>
<th>$HB_x$</th>
<th>$HB_y$</th>
<th>$CB_x$</th>
<th>$CB_y$</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(f)</td>
<td>Crit. Euler load</td>
<td>0.32</td>
<td>0.36</td>
<td>0.95</td>
<td>1.25</td>
<td>MN</td>
</tr>
<tr>
<td>(f)</td>
<td>Euler buckling MOS</td>
<td>1.1</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>(g)</td>
<td>Max. Stress (t/c)</td>
<td>0.3/0.3</td>
<td>0.3/0.4</td>
<td>0.4/0.4</td>
<td>0.5/0.5</td>
<td>-</td>
</tr>
<tr>
<td>(g)</td>
<td>Max. Strain (t/c)</td>
<td>0.3/0.4</td>
<td>0.3/0.5</td>
<td>0.4/0.6</td>
<td>0.5/0.6</td>
<td>-</td>
</tr>
<tr>
<td>(g)</td>
<td>Tsai-Hill (t/c)</td>
<td>0.1/0.2</td>
<td>0.2/0.2</td>
<td>0.3/0.3</td>
<td>0.4/0.4</td>
<td>-</td>
</tr>
</tbody>
</table>

In the entries for the composite failure criteria t/c corresponds to the results under tension and compression. Furthermore the braces are designed to be safe against instabilities, while being as lightweight as possible. Simultaneously they are supposed to be modular, hence the slightly increased MOS value for the horizontal braces along the x-axis.

5.2.4 Connecting elements

The connection between the composite tubes and the side pillars is done by means of metal inserts that are adhesively bonded into the braces and bolted to brackets that are welded to the side pillars. In this way it can be ensured, that the bracing members are free of any bending moment and the structure is simple to produce and to assemble.

Adhesively bonded joints  Figure 5.7 shows the schematics of the intended insert.

Figure 5.7: The inserts are to be manufactured from HSLA steel and bonded into the tube. Two brackets are mounted for the bolted connection to avoid bending of the fastener.
The selected adhesive is marine grade epoxy. To have the resin of the CFRP to be the same material as the adhesive has the advantage, that no difference in stiffness between the two materials occur. The concerns regarding these connections are:

h. Pull-out of these under tension.

i. Uneven stress distribution. If the length of the part of the inserts that overlaps with the carbon tube would become too long, there might be the risk, that the deeper part of the bond could not contribute to the strength of the connection, as the stresses would be accumulated at the beginning of the connection.

To cope with both concerns the design approach captured in the following equation is used.

\[
\frac{F \cdot \gamma_m}{\pi \cdot d_i \cdot \hat{\sigma}_{\text{epoxy}}} \leq L_{\text{bond}} \leq d_i
\]  

(5.6)

\(d_i\) represents the inner tube diameter, \(F\) the axial force, \(\hat{\sigma}_{\text{epoxy}}\) the ultimate strength of the adhesive and \(L_{\text{bond}}\) the length of the overlap between the insert and carbon tube.

The left-hand side of the equation represents the idea, that the axial force over the bonded area should never exceed the strength of the adhesive reduced by the DNV material factor \(\gamma_m\). The right-hand side of the equation is based on the assumption, that as long as the length of the bond is smaller than the tube diameter the stress distribution should be sufficiently evenly distributed, so that concern (i) would not occur.

**Welded connections** Initially the side pillars were designed from common carbon steel but the introduction of a material factor \(\gamma_m\) (see ch.2.2) cause the design to be highly strength driven. This requirement would drive the dimensions of the side tube to an unreasonable total mass, therefore the design is presented in high strength low alloy (HSLA) steel. Typically, the loss of strength in weld lines is of concern, when using these types of alloys. Hochhauser et Al. [19, sec. 4.2] have shown, that with correctly set welding parameters, the loss in tensile strength can be restrained to below 8% of the original property. The mass of the required brackets is assumed to be 8kg/piece. This assumption is supported by a preliminary analysis into the fatigue design of the weld according the S-N curve based Nominal Stress approach described in [20].
5.2.5 Operational machinery

A conceptual design of a mechanism to lock the mast sections together is developed and presented in figure 5.8.

![Figure 5.8: Truss structure](image)

The mechanism depicted in figure (ii) is to be placed inside the side pillars as shown in (i). The mechanism relies on a bracket, that is rotating around a pin as long as it is in unloaded condition. Once the mass of the upper sections is placed on the bracket the right-hand side spring in (iii) compresses and the load transfer takes places through a mechanical interlock. The spring on the left-hand side of (iii) presses the bracket out of the tube, the small channel in the back enables the retraction of the brackets by means of a pulley.

The device might need to be reverted, so that in the fail-safe state the bracket is retracted and the masts can be reefed in all circumstances.

5.3 Sandwich construction

In the following sections the conceptual design in sandwich panels is discussed.

5.3.1 Conceptual idea

The benchmark design as described in section 5.1 suffered from the problem of local buckling induced by compressive bending stresses in the thin walls of the mast cross section. Sandwich constructions have an outstanding bending stiffness, as the face sheets are separated by a light core and therefore bend
over a common neutral axis rather than their own one. This effect shall be utilized to circumvent the local stability problems in the benchmark design. A downside of sandwich constructions is, that the typically rather weak cores set drastic limitations on the applicability of point loads. To enable the sandwich concept to carry the local loads listed in Table 4.1 the corners of the mast section are designed in a single skin, carbon fiber reinforced polymer (CFRP). Their strength and stiffness are intended to prevent local deformations induced by the local loads and disperse them over a larger cross-sectional area, before they are induced into the face sheets of the sandwich walls. Figure 5.9 provides a coarse overview over the sandwich concept.

Figure 5.9: Conceptual idea of the sandwich construction: The single skin CFRP edges are intended to take up the local loads and disperse them, before they enter the face sheets of the sandwich panels. Furthermore, their stiffness prevents indentations by the local forces that would hinder the operation of the mast sections. The wall sections are built from CFRP face sheets and to obtain high resilience against local buckling at a low weight penalty.

5.3.2 Moments of inertia

The occurring stresses from global bending in the structure largely depend on the total moment of inertia of the sandwich structure and are evaluated in a similar fashion as done for the side pillars of the truss construction (compare eq. 5.1a, 5.1b). To enable this approach the moments of inertia for the sandwich concept must be found. Here it is assumed, that due to its comparably low stiffness the core can be neglected and will not contribute to the moments of inertia.
5.3.3 Analysis of the wall panels

In the design of the wall panels for the sandwich construction the following concerns are identified:

(a) **Face fracture**: Assuming the mast cross section in a monolithic material concept it becomes apparent, that the wall sections must carry compressive and tensile forces induced by bending and local loads. This is no different when the walls are built in a sandwich construction. As the cores are less stiff than the face sheets they will not contribute significantly, and the bending stresses have to be carried by the face sheets. In this light the concern of total face fracture or inter-laminate damages such a individual fiber fracture is raised.

(b) **Wrinkling**: The compressive stresses in the wall sections might induce wrinkling in the face sheets. In this failure mode small ripples form over the surface of the face sheets, either indenting the core or separating from it.

(c) **Stability**: The concern of instability of the wall sections is addressed in two regards: Firstly, instabilities under bending stresses of the mast might lead to general buckling of the sandwich panels. Secondly, the same load case can cause shear induced instability. Here a core failure would be initiating the collapse.

(d) **Local indentations**: The construction must be capable to withstand the local loads described in table 4.1. When point loads are induced in out-of-plane direction into the sandwich the compressive strength of the core could be insufficient, leading to local indentation caused by the collapse of the cores cell structure.

Typically, one evaluates failure of the core under shear loading too, however, the presence of the face sheets in the adjacent walls mitigates this concern. The evaluated cores are by margins less stiff than the face sheets, hence, it is assumed that contradicting the classical view on sandwich panels, the core does not have to carry the transverse forces.

The failure mode of face dimpling would only be applicable, if honeycomb cores were used. Here, the face sheets form small curvatures in the unsupported area in the cells of the honeycomb. As this core type is not introduced in the construction this mode of failure is neglected.
(a) **Face fracture:** This concern is utilizing the CLT code described in section 3.5.1 and the application of the laminate failure criteria according to equations 3.3.1. The compressive/tensile forces and the shear are expressed in terms of line loads $N_x$ and $N_{xy}$ as inputs for the CLT code. Furthermore, the structure is simulated in ANSYS ACP for verification purposes.

(b) **Wrinkling:** The failure mode of wrinkling is evaluated according to equation 3.13.

(c) **Stability:** As described in section 3.4.2 the evaluation of sandwich panels depends on which type of BCs are applied. In case of the mast design the lower and upper edges are (more or less) free, the sides of the panels are hard simply supported. These conditions present a challenge to an analytical approach as equation 3.11 is valid for all sides simply supported under hard BCs. In the light of these circumstances a sensitivity study is performed, comparing first the analytical solution for hard BCs to the results obtained from an ANSYS ACP solution. In a second step the BCs are varied from hard simply supported to hybrid as described above in ANSYS to evaluate, if the analytical equations yield sufficiently accurate results to support the assumption, that the varying BCs in the mast design can be neglected. The respective analysis can be found in the appendix A.4 and indicates, that for a conceptual approach the evaluation of the analytical equation 3.11 is sufficient. In addition, the risk of shear-induced buckling is implemented in the considerations according to equation 3.14.

(d) **Local indentation:** The evaluation of local indentations is initially performed using equation 3.15 and the results indicate, that unacceptable large contact surfaces or high performance core materials would be required. Instead, this issue is addressed by the introduction of single-skin CFRP edges in the mast design and respective structural concerns are discussed in the consequent section.

Table 5.6 summarizes the scantlings and material selections for the sandwich panels. The notation x- and y-walls represent the axis, the wall is aligned with.
Table 5.6: Sandwich panels: Scantlings

<table>
<thead>
<tr>
<th>Symbol</th>
<th>X-walls</th>
<th>Y-walls</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>- CFRP, DC160</td>
<td>CFRP, DC160</td>
<td>-</td>
</tr>
<tr>
<td>Face sheet thickness</td>
<td>$t_f$</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Core thickness</td>
<td>$t_c$</td>
<td>30</td>
<td>40</td>
</tr>
</tbody>
</table>

Based on the selected scantlings table 5.7 presents loads and structural parameters of the sandwich walls, table 5.8 the results of the evaluations of the listed concern.

Table 5.7: Sandwich panels: Loads and structural parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>X-walls</th>
<th>Y-walls</th>
<th>Unit</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive stress</td>
<td>$\sigma_{z,c}$</td>
<td>65.5</td>
<td>36.0</td>
<td>MPa</td>
<td>-</td>
</tr>
<tr>
<td>Tensile stress</td>
<td>$\sigma_{z,t}$</td>
<td>61.7</td>
<td>32.2</td>
<td>MPa</td>
<td>-</td>
</tr>
<tr>
<td>Compressive line load</td>
<td>$N_{x,c}$</td>
<td>1769.3</td>
<td>996.4</td>
<td>N/mm</td>
<td>CLT input</td>
</tr>
<tr>
<td>Tensile line load</td>
<td>$N_{x,t}$</td>
<td>1877.7</td>
<td>1113.4</td>
<td>N/mm</td>
<td>CLT input</td>
</tr>
<tr>
<td>Shear line load</td>
<td>$N_{xy}$</td>
<td>±171.8</td>
<td>±188.2</td>
<td>N/mm</td>
<td>CLT input</td>
</tr>
<tr>
<td>Bending stiffness</td>
<td>$D$</td>
<td>0.6</td>
<td>0.4</td>
<td>MNm</td>
<td>-</td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>$S$</td>
<td>3.2</td>
<td>3.7</td>
<td>MN/m</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.8: Sandwich panels: Failure modes

<table>
<thead>
<tr>
<th>Concern/ Wall orientation</th>
<th>Actual value</th>
<th>Crit. value</th>
<th>Unit</th>
<th>MOS</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Face fracture*</td>
<td>0.4</td>
<td>0.3</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>(b) Face wrinkling</td>
<td>65.5</td>
<td>36.0</td>
<td>344.2</td>
<td>344.2</td>
</tr>
<tr>
<td>(c) Glob. buckling</td>
<td>0.56</td>
<td>0.34</td>
<td>1.5</td>
<td>1.2</td>
</tr>
<tr>
<td>(c) Shear buckling</td>
<td>65.5</td>
<td>36.0</td>
<td>78.2</td>
<td>66.7</td>
</tr>
<tr>
<td>(d) Core indent.**</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*The driving load case for the face sheet thickness is the capability of the face sheets to sustain localized loads. This phenomenon is discussed in the following section.

**Local indentations were judged too critical, thus single skin edges are implemented that make this failure mode obsolete.
5.3.4 Edge design

The implementation of single-skin CFRP edges is motivated by the weakness of the cores in sandwich structures, to carry the large out of plane point loads (local loads). Furthermore, the face sheets of the adjacent wall panels must be able to sustain the respective in plane loads. Figure 5.10 depicts the intended function of the monolithic mast edge.

![Figure 5.10: Indented edge functionality](image)

Figure 5.10: Indented edge functionality: Figure (i) shows the challenge to the sandwich core induced by the localized loads. This problem roots in a rather low compressive strength of standard cores and can only be mitigated resorting to high performance materials such as honeycombs or by the implementation of monolithic edges in the sandwich mast design. Figure (ii) depicts the implemented monolithic edge. Besides circumventing the situation in figure (i) the beam type element has to ensure a small deflection normal to the z-axis. This is to safeguard the core in the adjacent wall and ensure, that the local loads will enter the respective face sheets only after having been reduced and dispersed over a larger cross-sectional area.

The evaluation of the above-described situation gives rise to the concern that either the laminate in the single skin edge or in the face sheets of the attached sandwich panels could fail. As the loading situation (especially for the sandwich face sheets) is not evenly distributed but depends on the stiffness and deflection of the edge the analysis is carried out in an FRP FEA. This is to determine the required scantlings of the edge and test for potential implication regarding the thickness of the sandwich face sheets. The analysis is reported in appendix A.5. Figure 5.11 depicts the design of the mast corner:
Figure 5.11: Mast corner design: The edge of the mast is designed in CFRP single skin and shielded by HSLA steel to obtain a sliding surface, that can be further processed to be robust against wear and tear (ref 2.3). The length of the contact $L_C$ is kept short to avoid the introduction of a bending moment into the edge. The length $L_E$ is selected considerably longer to provide moment of inertia and to avoid the challenges described in figure 5.10. To prevent the metal contact from taking bending stresses and to avoid, thermal strains in the bond line the metal patches must be discretized over the length of the mast. The parameter $t_E$ denotes the thickness of the edge.

The results of the FEA indicate, that the implementation of the single skin edges into the sandwich design are a suitable solution to sustain the local loads on the levels M21 and M22 and the reefing operation of section 3. However, to provide sufficient support to sustain these loads in the levels M11 and M12 a too high mass penalty would need to be accepted. It is therefore decided, to introduce a localized solution for these levels, assuming that reefing would begin in the upper sections of the mast. To provide additional transverse stiffness to the lower levels two frames from HSLA steel are embedded in the xy-plane into the structure as indicated in figure 5.12.
Figure 5.12: H-profiles embedded into the concept on the levels M11 and M12. The FEA results depict the inverse reserve factor for the FRP failure criteria described in section 3.3.1. If the index remains below $1/\gamma_m = 0.42$ no failure is predicted, and the results are in line with DNV design principles. The results indicate that the single skin composite edge provides sufficient support for the local loads in the upper sections, however, not in the lower ones. To rectify this shortcoming an H-profile from HSLA steel is embedded into the structure to support in fully hoisted condition.

The H-profiles are evaluated against yielding (e), global buckling in x- and y-direction (f) and local buckling of flanges (g) and web (h) as if no sandwich panels would support the member. The bond is evaluated by superposing the bending stresses in z-direction with the ones, induced by the local loads in x-direction or y respectively. The dominating failure modes are plastic deformation and global instabilities. Table 5.9 reports the selected scantlings of the corner and steel frames.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>x-wall</th>
<th>y-wall</th>
<th>Unit</th>
<th>Member</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web thickness</td>
<td>$t_{HB_{,w}}$</td>
<td>22</td>
<td>22</td>
<td>mm</td>
<td>H beam</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>$t_{HB_{,f}}$</td>
<td>22</td>
<td>22</td>
<td>mm</td>
<td>H beam</td>
</tr>
<tr>
<td>Flange height</td>
<td>$t_{HB_{,h}}$</td>
<td>185</td>
<td>185</td>
<td>mm</td>
<td>H beam</td>
</tr>
<tr>
<td>Corner thickness</td>
<td>$t_e$</td>
<td>70</td>
<td>70</td>
<td>mm</td>
<td>CFRP corner</td>
</tr>
<tr>
<td>Corner length</td>
<td>$L_e$</td>
<td>300</td>
<td>300</td>
<td>mm</td>
<td>CFRP corner</td>
</tr>
<tr>
<td>Contact length</td>
<td>$L_c$</td>
<td>100</td>
<td>100</td>
<td>mm</td>
<td>CFRP corner</td>
</tr>
</tbody>
</table>
It shall be noted that the manufacturing of a 70mm thick CFRP edge might be a challenge in itself. Furthermore, the height of the web is not indicated, as it is defined by the sandwich panel thickness. Table 5.10 presents the results of the structural evaluations around the H-beams.

Table 5.10: H-beams: Failure modes

<table>
<thead>
<tr>
<th>Concern/ Wall orientation</th>
<th>Actual value X</th>
<th>Actual value Y</th>
<th>Crit. value X</th>
<th>Crit. value Y</th>
<th>Unit</th>
<th>MOS X</th>
<th>MOS Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>(e) Yielding</td>
<td>624.6</td>
<td>440.8</td>
<td>625.0</td>
<td>625.0</td>
<td>MPa</td>
<td>1.0</td>
<td>1.4</td>
</tr>
<tr>
<td>(f) Glob. buckling</td>
<td>2.5</td>
<td>1.7</td>
<td>2.8</td>
<td>1.8</td>
<td>MN</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>(g) Flange buckling</td>
<td>0.6</td>
<td>0.4</td>
<td>4.5</td>
<td>4.5</td>
<td>GPa</td>
<td>7.1</td>
<td>10.1</td>
</tr>
<tr>
<td>(h) Web buckling</td>
<td>0.6</td>
<td>0.4</td>
<td>10.6</td>
<td>15.3*</td>
<td>GPa</td>
<td>16.9</td>
<td>34.7</td>
</tr>
</tbody>
</table>

* The panel thickness dictates the height of the web. Hence the increased MOS for concern (h) of the member oriented along the y-axis.
Chapter 6

Cost assessment

Besides the structural mass the capital costs for the developed concepts are of interest to decide for the most suitable conceptual approach on the mast design. To evaluate monetary aspects of the designs a simple cost model is applied. First, the estimation of the raw material costs will be discussed, followed by an approximation of the manufacturing investments.

6.1 Investments for raw materials

6.1: The costs for raw materials are modelled by the approach described in equation 6.1:

\[ P_{RM} = \sum m_i \cdot C_{m_i} \]  

\( P_{RM} \) indicates the prices for raw materials, \( m_i \) the individual mass components derived from the scantling described in the tables 5.1, 5.4, 5.6 and 5.9. The factor \( C_{m_i} \) describes the material factors that are stated below [9].

<table>
<thead>
<tr>
<th>Material</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy resin</td>
<td>( C_{m,R} )</td>
<td>90</td>
<td>SEK/kg</td>
</tr>
<tr>
<td>Carbon fibers</td>
<td>( C_{m,CF} )</td>
<td>270</td>
<td>SEK/kg</td>
</tr>
<tr>
<td>Core (DC160)</td>
<td>( C_{m,C} )</td>
<td>100</td>
<td>SEK/kg</td>
</tr>
<tr>
<td>HSLA steel</td>
<td>( C_{m,HSLA} )</td>
<td>12</td>
<td>SEK/kg</td>
</tr>
</tbody>
</table>

A slight modification is applied to the modelling of the costs for the CFRP braces in the sandwich structure. It is assumed, that these members will be
purchased pre-manufactured, thus an addition of 20% is added to the raw material costs.

6.2 Investments for manufacturing

The investments to produce the concepts are modelled using the approach described in equation 6.2. It is decided to exclude capital cost to purchase manufacturing equipment. These components depend on the scope of the production process with regards to time and the size of the series. Further assumptions or scenarios would need to be applied to relate these costs to a robust judgement on the feasibility of the conceptual designs. In absence of this data the results would become blurred and less straight forward.

\[ P_M = \sum m_i \cdot C_{L,i} \cdot C_{BR} \]  

(6.2)

\( P_M \) denotes the manufacturing costs, \( m_i \) the individual mass components, \( C_{L,i} \) a labor time factor and \( C_{BR} \) a constant factor for the costs of labour time for man hours. In this model two modifications are made. First, the manufacturing costs for the CFRP laminate of truss braces is set to zero as discussed previously. However, the bonding procedure at both side of the tube shall be considered: \( P_{Braces} = 2 \cdot n_{braces} \cdot 15\, \text{min} \cdot C_{BR} \). Table 6.2 gives inside into the basic cost factors applied to the model [21].

<table>
<thead>
<tr>
<th>Cost factor</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base rate</td>
<td>( C_{BR} )</td>
<td>500</td>
<td>SEK/h</td>
</tr>
<tr>
<td>Labour time metals</td>
<td>( C_{L,Steel} )</td>
<td>0.5</td>
<td>h/kg</td>
</tr>
<tr>
<td>Labour time composite single skin</td>
<td>( C_{L,SingleSkin} )</td>
<td>1.5</td>
<td>h/kg</td>
</tr>
<tr>
<td>Labour time sandwich</td>
<td>( C_{L,Sandwich} )</td>
<td>1.0</td>
<td>h/kg</td>
</tr>
</tbody>
</table>
Chapter 7

Results

Local buckling of the mast wall sections under compressive loads is identified as the failure mode, that prevent single skin FRP solutions to become competitive candidates.

Four different concept families are developed around the challenge of localized instabilities and evaluated against applicable failure modes. Based on the expected structural masses and estimates on prices and the complexity in manufacturing two concepts are selected and designed in greater resolution. One is a hybrid truss structure in HSLA steel and CFRP, the other is a sandwich design in a foam core, CFRP face sheets, monolithic CFRP corners and additional stiffening members in HSLA steel. The two remaining concepts are investigated with respect to their structural mass and capital costs for raw materials and manufacturing.

Table 7.1 presents the structural masses for the conceptual designs of mast section 2. The entries are derived from the scantling presented in the tables 5.1, 5.4, 5.6 and 5.9.

<table>
<thead>
<tr>
<th>Mass</th>
<th>Sandwich</th>
<th>Truss</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core</td>
<td>0.99</td>
<td>0.00</td>
<td>ton</td>
</tr>
<tr>
<td>CFRP</td>
<td>10.05</td>
<td>2.30</td>
<td>ton</td>
</tr>
<tr>
<td>HSLA steel</td>
<td>1.47</td>
<td>9.67</td>
<td>ton</td>
</tr>
<tr>
<td>Sum</td>
<td>12.5</td>
<td>12.0</td>
<td>ton</td>
</tr>
</tbody>
</table>

Figure 7.1 expresses the results presented in table 7.1 graphically in form of pie charts.
Figure 7.1: Distribution of the structural mass for the truss and the sandwich concept.

An inside into the cost distribution according to section 6 of both concepts is given in table 7.2:

<table>
<thead>
<tr>
<th>Costs</th>
<th>Sandwich</th>
<th>Truss</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw material</td>
<td>2,106,092</td>
<td>661,546</td>
<td>SEK</td>
</tr>
<tr>
<td>Manufacturing</td>
<td>7,289,500</td>
<td>2,446,600</td>
<td>SEK</td>
</tr>
<tr>
<td>Sum</td>
<td>9,395,592</td>
<td>3,108,146</td>
<td>SEK</td>
</tr>
</tbody>
</table>

Figure 7.2 expresses the results presented in table 7.2 graphically in form of bar charts.

Figure 7.2: Distribution of costs for the truss and the sandwich concept.
Chapter 8

Discussion

In this chapter critical aspects of the report are discussed, followed by a direct comparison between the two remaining concepts in chapter 9.

The path from the research question to the conclusions of this thesis project is done by the derivation of distributed and local loads, followed by the development of conceptual designs and the refinement of the most promising ones against obvious failure. Finally, the conclusions are drawn by comparing the concepts in terms of structural mass and capital costs. These steps are taken from a conceptual perspective, often done by simplification and the application of basic analytical engineering concepts. This approach leaves gaps, and the presented designs are not to be seen as ready for production. Localized challenges, like stress concentrations around edges, might require additional design work. However, it has not been the aim of the project to provide manufacturing drawings. Instead, the value of this thesis lays in the fact, that by these simplifications the design space for the masts could be effectively restrained and the report indicates a well-founded recommendation, which concept provides the most promising path to a successful design. The method section provides a set of tools to ease the development of a final design.

8.1 Design loads

In the derivation of the global loads (ref. 4.1) the assumption is made, that the distributed load in its general form can be seen as linearly decreasing with the height of the mast. In absence of more accurate data this seems to be a reasonable approximation, as the span and thereby the structural mass and transverse forces from aerodynamic loads should display a similar behaviour. However,
should for instance investigations into the distribution of wind speeds over the length of the mast indicate an increase towards the mast tip, this assumption would no longer be valid. This could imply that, assuming the maximum bending moments at the root to remain constant, that the overlap between the mast sections 2 - 3 could experience loads of greater magnitude.

In the derivation of the local loads (ref. 4.2) acting on the sliding pads the assumptions are made, that four pads per layer carry the torsional moment and two pads in each overlap between two mast sections carry the bending loads. These assumptions are tremendously conservative but in absence of a verified design, that would lead to more evenly distributed loads, it is judged necessary to take this measure. These presumptions have great influence on the scantlings in the conceptual designs in terms of masses and costs.

8.2 Truss structure

When the concept of the truss structure was induced into the scope of the thesis project the idea was to simplify the design as much as possible. This implies that the presented version must not be the most optimized one and especially the application of steel in the side pillars can be questioned. Although it enables classical joining methods and simplifies the sliding contacts between two mast sections it comes at the disadvantage of driving the structural mass up. As depicted in figure 7.1 the steel accounts for more than 80% of the total mass. Given, that suitable joining techniques are found alongside with a robust concept to enable sliding contacts between two FRP surfaces this presents a great potential to further decrease the mass of the construction.

Two assumptions influence the static determination of the truss structure. Firstly, in the derivation of the local loads two pads are assumed to provide the load transfer. Secondly, in the analysis of the axial forces the joints of the bracing members are regarded as free of bending moment. A violation of any of the two assumptions would result in statically indeterminate structure and the axial forces in the braces would need to be evaluated again. The MOS values for stability in table 5.5 indicate the sensitivity for changes in the axial forces.
8.3 Sandwich structure

As a result of the design work carried out around the sandwich concept the author concluded that, expressed in a general way, structural sandwiches have a disadvantage compared to other material classes in their ability to carry point loads. As described above the local loads are derived in a highly conservative manner, which implies a slight imbalance in the thesis project as the disadvantages of the sandwich are amplified. This might distort the results. Further aspects on this matter are discussed in the section, that is concerned with the comparison of both concepts.

The single skin CFRP edge design in the sandwich concept itself only provides sufficient support in the upper levels (M21, M22) and when reefing the upper mast sections. In the lower levels, stiffening members from steel are embedded at the load points for the fully hoisted mast. This implies additional operation restrictions for reefing mast section 2, that are judged concerning for the operation of the vessel. Imposing operational restriction on when the mast section 2 can be reefed means that the system could never be operated above these restrictions, as weather and sea conditions could continuously worsen. However, the design includes the worst-case material factor $\gamma_m$ and it is judged that when progressing with the classification process, this factor will decrease. This would leave head space to deal with the required operational restrictions and therefore they are not judged critical at this stage. Nonetheless these arguments should be investigated thoroughly before progressing with a sandwich design.

8.4 Derivation of the capital costs

The costs are derived based on rough figures for prices for material and manufacturing. The report cannot claim these numbers to be fully accurate for this application, nonetheless their reliability is judged sufficient to describe relations and thereby serve the purpose of the project, to find the most suitable path towards a successful design.
Chapter 9

Conclusions

For the intended use the truss structure is judged most suited, despite the fact that the two surviving concepts imply an almost equal mass penalty. The conclusion is drawn based on three arguments:

1. **Problem approach:** The need to investigate conceptual designs for the mast originates in the failure mode of local buckling in the wall sections of the benchmark design. The sandwich solution presents a method to circumvent this problem, but the truss structure removes the wall and thereby the problem as a whole. In the light that there is no need to have continuous walls inside the wing-sails it seems to be more pragmatic to remove the problem rather than working around it.

2. **Fitness for purpose:** The sandwich design provides an elegant way to increase the critical general buckling loads in the wall sections, however, it induces new failure modes into the structure, increases the costs and complexity and implies a decreased ability to carry localized loads. The predominant advantage of sandwich panels is their specific stiffness when subjected to a bending moment. This load case is not present in the construction as the panels act as flanges of a beam with a hollow rectangular cross section and the members are mostly loaded in in-plane tension or compression. Therefore, the benefit of applying sandwich panels reduces to preventing instabilities in the wall sections. The advantage of a sandwich is thus utilized in a very localized way but the mentioned disadvantages apply to the entire structure. It is thereby judged that sandwich panels are not the ideal material concept for the given load case. This view is supported by the MOS values provided in table 5.8. The two dominating failure modes are (a) face fracture under the influence of...
localized loads and (c) shear buckling under compressive loads. Especially the latter case underlines this judgement. To circumvent general local buckling in the benchmark design sandwich panels are induced at the cost of inducing shear buckling as a new failure mode into the design. This mode is one of the dominant mass drivers, hence one stability concern is traded for another.

3. **Simplicity:** This term applies to various aspects of the truss structure. Although not evaluated in depth the manufacturing costs described in figure 7.2 provide an indication for the complexity of the concepts. The investments for the sandwich are \( \approx 200\% \) higher than the for the truss construction.

It is to be expected, that once approved by classification societies and accepted by the general public as a feasible concept, the demand for the system will be large, driven by the potential for fuel savings and sustainable aspects of the system. A decisive factor in the success of Wallenius Marine AB in capitalizing on it will be the ability to roll out the concept over a large fleet of vessels and to provide service to the systems in operation. Here the modularity of the truss concept will provide a great advantage. The side pillars as well as the braces can be produced independent from one another and transported and assembled at a later stage. Furthermore, if damages would occur in the masts over their lifetime, individual components can be replaced or repaired comparably easy. This holds too for crews maintaining the system as for instance braces could be kept spare on board and exchanged during port stays. Minor damages to the steel pillars could be rectified by the crew, provided sufficient qualification in advanced welding procedures would be ensured.

Although the direct comparison between the two surviving concepts presents the sandwich wall design as less suited, it shall be emphasised, that amongst all evaluated concepts this is a survivor and judged the second-best conceptual approach.
Chapter 10

Acknowledgements

First and foremost, I would like to express my gratitude for the continuous and professional guidance by Stefan Hallström and Emil Kotz. When navigating complex design spaces it is crucial to select an appropriate level of detail in the analysis and to adapt it at all stages to the needs of the project. With their technical expertise and structured mindset both have helped me to find this balance and corrected my path when I was too focused on details.

Secondly, I want to thank Mr. Zuheir Barsoum for taking time to share his valuable knowledge regarding high strength steel alloys, specifically in regard to the design of welded connections against failure and fatigue.

Lastly, I am thankful for having been given the opportunity by Wallenius Marine AB to work on such an innovative project as the Oceanbird. I have selected Naval Architecture as my profession because I see a great potential for sustainable innovations in the marine sector and I regard the topic and background of the thesis project as a formidable way to finalize my education and begin my professional career.


Appendix A

Appendices

A.1 Framework information for the project

The following table presents information provided by Wallenius Marine AB to be seen as the framework/general arrangement of the project.

Table A.1: Framework provided by Wallenius Marine

<table>
<thead>
<tr>
<th>ID</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Max. bending moment at mast root</td>
<td>$M_{b_{max}}$</td>
<td>Confidential</td>
<td>MNm</td>
</tr>
<tr>
<td>2</td>
<td>Max. torsional moment at mast root</td>
<td>$M_{t_{max}}$</td>
<td>Confidential</td>
<td>MNm</td>
</tr>
<tr>
<td>3</td>
<td>Max. vertical acceleration</td>
<td>$a_{z_{max}}$</td>
<td>Confidential</td>
<td>m/s²</td>
</tr>
<tr>
<td>4</td>
<td>Total mass per wing</td>
<td>$m_{w_{max}}$</td>
<td>Confidential</td>
<td>ton</td>
</tr>
<tr>
<td>5</td>
<td>Total mass per mast</td>
<td>$m_{m_{max}}$</td>
<td>Confidential</td>
<td>ton</td>
</tr>
<tr>
<td>6</td>
<td>Total mass section 2,3 and 4</td>
<td>$m_{sec_{234}}$</td>
<td>Confidential</td>
<td>ton</td>
</tr>
<tr>
<td>7</td>
<td>Vert. center of gravity wing</td>
<td>$VCG$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>8</td>
<td>Center of effort</td>
<td>$CEA$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>9</td>
<td>Total height of the mast</td>
<td>$h_{mast}$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>10</td>
<td>Total height mast section 2</td>
<td>$h_{sec2}$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>11</td>
<td>Maximum mast tip deflection</td>
<td>$v_{max}$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>12</td>
<td>Wall length along x-axis</td>
<td>$d_{xi}$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>13</td>
<td>Wall length along y-axis</td>
<td>$d_{yi}$</td>
<td>Confidential</td>
<td>m</td>
</tr>
<tr>
<td>14</td>
<td>Wall thickness ID10</td>
<td>$t_{thick}$</td>
<td>Confidential</td>
<td>mm</td>
</tr>
<tr>
<td>15</td>
<td>Wall thickness ID11</td>
<td>$t_{thin}$</td>
<td>Confidential</td>
<td>mm</td>
</tr>
</tbody>
</table>
## A.2 Material data

The materials that have been used throughout the project are characterized in table A.2

<table>
<thead>
<tr>
<th>Material</th>
<th>Stiffness GPa</th>
<th>Yield limit MPa</th>
<th>Density kg/m$^3$</th>
<th>Price SEK/kg</th>
<th>Remark/USN</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-Glass fibers</td>
<td>80</td>
<td>2000</td>
<td>2600</td>
<td>30</td>
<td>Fibers only</td>
</tr>
<tr>
<td>C-Glass fibers</td>
<td>70</td>
<td>3250</td>
<td>2600</td>
<td>190</td>
<td>Fibers only</td>
</tr>
<tr>
<td>S-Glass fibers</td>
<td>90</td>
<td>3900</td>
<td>2500</td>
<td>195</td>
<td>Fibers only</td>
</tr>
<tr>
<td>Carbon fibers</td>
<td>250</td>
<td>3900</td>
<td>1820</td>
<td>270</td>
<td>Fibers only</td>
</tr>
<tr>
<td>Carbon steel</td>
<td>210</td>
<td>315</td>
<td>7800</td>
<td>8</td>
<td>K02803</td>
</tr>
<tr>
<td>HSLA steel</td>
<td>205</td>
<td>1500</td>
<td>7850</td>
<td>12</td>
<td>K44220</td>
</tr>
<tr>
<td>Aluminum</td>
<td>72</td>
<td>550</td>
<td>2800</td>
<td>50</td>
<td>A97075</td>
</tr>
</tbody>
</table>
A.3 Truss structure: Axial forces

The axial forces in the braces are derived in ANSYS R02. The side pillars are modelled as discontinuous beam188, the braces in link180 elements. The individual components are then joined at the ends as fixed supports. In this way the side pillars carry the global bending moment and the braces only axial loads.

To model the difference in stiffness of the CFRP braces and HSLA steel side pillars without excessive computational effort the braces are modelled in a titanium alloy which possesses a similar stiffness than a 0-dominated CFRP. The loads and BCs are applied in the same fashion as described in section A.5. Consequently, the axial forces are extracted from the solutions as indicated in figures A.1 and A.2 and the final results reported in table 5.3.

Figure A.1: Truss structure: Axial forces due to a maximum bending moment in x-direction under ULS loads.
Figure A.2: Truss structure: Axial forces due to a maximum bending moment in y-direction under ULS loads
A.4 Sensitivity study of the effect of boundary conditions for sandwich panel instability

For the evaluation the critical buckling load according to equation 3.11 is compared to a simulation in ANSYS ACP for a panel of 2.29m width for three different values for the length. For both approaches the same parameters \( t_f = 10.32\, \text{mm}, \, t_c = 10.00\, \text{mm}, \, E_f = 79631\, \text{MPa}, \, E_c = 102\, \text{MPa}, \, G_c = 39\, \text{MPa} \) are used. In the FE analysis the hard BCs are modelled by the use of nodal rotations to lock the rotations around the supported edges. The above described material properties are achieved by the modelling a quasi-isotropic laminate in the form \([0\, 45\, -45\, 90]s\) from CFRP plys. The mesh density is controlled by the element size which is set to 90mm. Figure A.3 describes the BCs applied in the FEA.

Table A.3 displays the results of the comparison between the FEA and the analytical solution of equation 3.11.

<table>
<thead>
<tr>
<th>Panel length [m]</th>
<th>Panel width [m]</th>
<th>Analytical [N/mm]</th>
<th>ANSYS ACP [N/mm]</th>
<th>Delta* [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.56</td>
<td>2.29</td>
<td>879.8</td>
<td>947.3</td>
<td>7.7</td>
</tr>
<tr>
<td>8.67</td>
<td>2.29</td>
<td>879.8</td>
<td>955.8</td>
<td>8.6</td>
</tr>
<tr>
<td>5.78</td>
<td>2.29</td>
<td>879.8</td>
<td>981.2</td>
<td>11.5</td>
</tr>
</tbody>
</table>
It is assumed, that the difference between the analytical and the simulated solution roots in the boundary conditions that are applied in the simulation. To test this hypothesis the length of the panel is varied. For the analytical solution the magnitude of the critical load remains the same but is found at different mode numbers. For the FEA the results vary with the length of the panel. The percentual difference increases with decreasing length of the panel. It is therefore assumed, that the error roots in the BCs that are applied in the FEA. For a length of 11.56m the error is around 8%, which is judged acceptable, especially since the actual length of the panel is 23.13m and the error is expected to decrease further.

The BCs in the mast design are not hard simply supported on all sides. It is assumed, that the connections between two adjacent walls can be seen as such but on the upper and lower side of the masts sections the shear deformation of the core is not prevented, hence the BC has to be seen as soft simply supported. To evaluate the influence of these circumstances the FEA results presented in Table A.3 are compared to a setup, that represents the mixed types of BC. In table A.4 HBC represents the hard, MBC the mixed BCs.

Table A.4: Verification of the FEM model: Critical buckling loads for a sandwich panel

<table>
<thead>
<tr>
<th>Panel length [m]</th>
<th>Panel width [m]</th>
<th>ANSYS HBC [N/mm]</th>
<th>ANSYS MBC [N/mm]</th>
<th>Delta* [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.56</td>
<td>2.29</td>
<td>947.3</td>
<td>933.9</td>
<td>1.4</td>
</tr>
<tr>
<td>8.67</td>
<td>2.29</td>
<td>955.8</td>
<td>933.8</td>
<td>2.3</td>
</tr>
<tr>
<td>5.78</td>
<td>2.29</td>
<td>981.2</td>
<td>933.6</td>
<td>4.9</td>
</tr>
</tbody>
</table>

*Referencing the hard simply supported results.

The results seem to indicate, that the effect of modelling the upper and lower edge of the wall panel as hard BCs has a marginal effect for long panels. As the connection to the adjacent walls prevent relative motion along the z-axis between both face sheets the upper and lower edge can be seen as quasi-hard simply supported for long panels. As the lowest critical loads are given by the analytical solutions, equation 3.11 is used throughout the design work of the sandwich concepts.
A.5 FEA of the sandwich concept

The desired sandwich construction is modelled in ANSYS R02 using the static structural cell in conjunction with the two ACP modules. Figure A.4 gives inside into the way the model is built.

Figure A.4: Setup of the FEA for the sandwich structure: Figure (i) shows the thickness of the walls and edges mapped onto the mast structure. The edges are modelled in CFRP single skin, the wall sections in PVC foam core. Figure (ii) shows the applied forces and BC. The red arrows indicate the local loads as described in table 4.1. The grey boxes indicate the locked degrees of freedom. The green arrows denote an additional force component from the torsional moment. The local loads include the torsional components, however, only up to level where they are derived. Over the length of mast section 2 the torsional moment will increase and introduced into the lower side of the mast on the levels M12 and M11. To model this behaviour a conservative setup is selected. The delta between the torsional loads between the M21/M22 and M11/M12 is introduced into the upper side of the structure by four force components denoted $F_{\Delta T}$

Besides the result, that the overall configuration of face and core thicknesses and the design of the single skin edges performs as intended, points of interest are the face sheets, that are in contact with the edges behind a load point. Figure A.5 depicts the results for the inverse reserve factor for the failure criteria Maximum Stress, Maximum Strain and Tsai-Hill.
Figure A.5: Detailed results of the FEA described in figure A.4. The results of the inverse reserve factor for the composite failure criteria Maximum Stress, Maximum Strain and Tsai-Hill are plotted onto the structure. Usually the interpretation goes as such, that indices above 1 indicate failure. In this work the results are expected to stay below $1/\gamma_m = 0.42$ to not only satisfy strength concerns with regards to the FRP but to also comply with DNV rules for classification.

The results confirm that the introduction of monolithic edges into the sandwich concept can be a way to circumvent the challenges discussed in section 5.3.4.
A.6 Mid term status

To limit the effort along the project the most promising concepts are selected (truss structure, sandwich structure) to be designed in greater resolution. The base for the decision is presented in table A.5. It shall be made clear that the entries are outdated and are only reported for the sake of transparency in the decision.

Table A.5: Elimination of conceptual families

<table>
<thead>
<tr>
<th></th>
<th>BD</th>
<th>CD1a</th>
<th>CD1b</th>
<th>CD2a</th>
<th>CD2b</th>
<th>CD3</th>
<th>CD4a</th>
<th>CD4b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Price</td>
<td>+++</td>
<td>+++</td>
<td>+</td>
<td>+++</td>
<td>+++</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Complexity</td>
<td>+++</td>
<td>+++</td>
<td>+++</td>
<td>++</td>
<td>+++</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Mass [ton]</td>
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<td>23.2</td>
<td>12.4</td>
<td>14.4</td>
<td>21.2</td>
<td>9.7</td>
<td>13.7</td>
<td>10.2</td>
</tr>
</tbody>
</table>

The following codes apply: + Negative, ++ Neutral, +++ Positive. BM denotes the benchmark design, CD1a and b the attempts to increase the wall thickness, CD2a the strip reinforced conceptual design, CD2b the corrugated wall section, CD3 the truss structure and CD4a and b the sandwich designs in CFRP and GFRP respectively.

The benchmark design is judged unfit but reported to serve as a base line version.
A.7 CLT Code

Figure A.6 provides an overview of the code architecture for the tool described in section 3.5.1.

Figure A.6: CLT code architecture: At the beginning of the analysis the code imports material data stored in excel, consequently the user is asked to provide the data for the intended laminate as amount of laminae and respective orientation angles and materials. The ABD-matrix, describing the laminate itself, is computed and the laminate engineering constants are provided to the user. Now the user must input the loads as line loads and moment based on which the stresses and strains on laminae and laminate level can be computed. The relation of loads and strains is hereby done by the relation, depicted in equation 3.8. Once stresses and strains are known, the failure criteria Maximum Stress, Maximum Strain and Tsai-Hill are computed based on equations 3.5, 3.6, 3.7 and presented to the user for each layer of the laminate for judgement. If the criteria exceed 1 failure is to be expected and the laminate inputs should be revisited.