DIMENSIONING AND SERIALIZATION OF INTERNAL WORKING PLATFORM SUPPORT BEAMS

Lappeenranta–Lahti University of Technology LUT

Master’s Programme in Mechanical Engineering, Master's thesis

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ABSTRACT

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Dimensioning and serialization of internal working platform support beams

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Keywords: working platform, steel structure, serialization

The goal of this master's thesis was to develop the design for the supporting beams of internal service platforms of certain equipment using serialization. The goal was to serialize the beams to fit the needs of a certain product family. The serialization was done by finding the most critical load cases in the product family and dimensioning the supporting beams and their joints to withstand the load cases in question. ISO 14122-2 standard, which guides the design of working platforms, and source material from the manufacturer of the equipment were used to determine the load cases. The work ended up looking at seven different cases.

The structures were dimensioned mainly in accordance with the Eurocode standards EN 1993 and EN 1990. For the service limit state examination, the deflection limit was determined according to the standard ISO 14122-2. Commonly available profile sizes were selected for investigation, from which the most suitable options were selected for different cases. The examination was done with analytical methods and using FE analysis. The most important goal of the structure analysis was to identify the most critical boundary conditions and ensure that the designed structure meets the requirements set by the boundary conditions.

As a result of the work, it was possible to produce structures that meet the requirements for seven load cases. Consequently, the variation in the structures of the working platforms of the considered product family was significantly reduced.

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Sisäisten hoitotasojen mitoitus ja sarjoitus

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Tarkastajat: Professori Timo Björk ja DI Antti Matikainen

Ohjaaja: Antti Nyyssönen, M.Sc. (Tech.)

Avainsanat: hoitotaso, teräsrakenteet, sarjoitus

Tämän diplomityön tavoitteena oli kehittää erään prosessilaitteen sisäistenhoitotasojen palkiston suunnittelua hyödyntäen sarjoitusta ja näin vähentää variaatiota rakenteissa. Tavoitteena oli sarjoittaa palkistot sopimaan tietyn tuoteperheen tarpeisiin. Sarjoitus tehtiin etsimällä tuoteperheestä kriittisimmät kuormitustapaukset ja mitoittamalla palkistot ja näiden liitokset kestämään kyseiset kuormitustapaukset. Kuormitustapausten määrittämiseen hyödynnettiin hoitotasojen suunnittelua ohjailevaa standardia ISO 14122-2 sekä lähdeaineistoa prosessilaitteen valmistajalta. Työssä päädyttiin tarkastelemaan seitsemää eri tapausta.

Rakenteiden mitoitus tehtiin pääasiassa Eurokoodistandardien EN 1993 ja EN 1990 mukaisesti. Käyttörajatilatarkastelua varten taipumarajoitus määritettiin standardin ISO 14122-2 mukaisesti. Tarkasteluun valittiin yleisesti saatavilla olevia profiilikokoja, joista valittiin sopivimmat vaihtoehdot eri tapauksiin. Tarkastelu tehtiin analyyttisin keinoin sekä FE-analyysiä hyödyntäen. Rakenteen analysoinnin tärkeimpänä tavoitteena oli tunnistaa kriittisimmät rajatilat ja varmistaa, että suunniteltu rakenne täyttää rajatilojen asettamat vaatimukset.

Työn tuloksena saatiin tuotettua suunnitelma vaatimukset täyttäviin rakenteisiin seitsemään kuormitustapaukseen. Näin ollen pystyttiin vähentämään variaatiota tarkastellun tuoteperheen hoitotasojen rakenteissa huomattavasti.

SYMBOLS AND ABBREVIATIONS

Roman characters

A	area [mm^2]
A_{eff}	effective cross-sectional area [mm2]
A_f	area of a flange [mm^2]
A_s	tensile area of a bolt [mm^2]
A_w	area of a web [mm^2]
a	throat thickness of the weld [mm]
b	beam flange width (mm)
E	Young's modulus [N/mm^2]
E_d	design value of the effect of actions
F_{k}	characteristic value of action [N]
$F_{tb,Ed}$	design tensile force per bolt [N]
$F_{tb,Rd}$	design tensile resistance per bolt [N]
$F_{w,Ed}$	design value of the weld force [N]
F_{wRd}	design weld resistance per unit length [N]
$F_{vb,Ed}$	design shear force per bolt [N]
$F_{vb,Rd}$	design shear resistance per bolt [N]
$f_{cw.d}$	design shear strength per unit length of the weld [N]
f_{u}	ultimate strength [N/mm^2]
f_{y}	yield strength [N/mm^2]
f_{yd}	design value of yield strength [N/mm^2]
G	shear modulus [N/mm^2]

h	height [mm]
I	area moment of inertia [mm^4]
I_w	warping constant [mm^4]
I_t	St. Venant's torsional constant [mm^4]
k	effective length factor referring to end rotation on plain
k_w	effective length factor referring to end warping
L	length [mm]
L_w	effective length of the weld [mm]
M	bending moment [Nmm]
$M_{b,Rd}$	lateral torsional buckling resistance [Nmm]
$M_{x,Ed}$	design value of bending moment about x axis [Nmm]
$M_{x,Rd}$	bending moment resistance about x axis [Nmm]
$M_{y,Ed}$	design value of bending moment about y axis [Nmm]
$M_{y,Rd}$	bending moment resistance y axis [Nmm]
m	mass per unit length [kg/m]
R_d	design value of the corresponding resistance
s	web thickness [mm]
t	thickness [mm]
W	section modulus [mm^3]
W_y	section modulus about y axis [mm^3]
W_z	section modulus about z axis [mm^3]
z_g	distance between the point of load application and the shear center [mm]

Greek characters

α_{LT}	imperfection factor for lateral torsional buckling
β_w	fillet weld correction factor
γ_f	partial factor that takes account of unfavourable deviation of an action from its representative value
γ_M	material safety factor
γ_{Sd}	partial factor associated with the uncertainty of action and/or action effect model
Δ_{LT}	Lateral displacement
δ	deformation [mm]
$\delta_{y,max}$	maximum allowed vertical deformation [mm]
ε	material parameter depending on f_y
ε_t	tensile strain [mm]
θ	angle of bending
θ_{LT}	angle of translation in lateral torsional buckling
λ_{LT}	relative slenderness for lateral torsional buckling
ρ	radius of bending [mm]
σ_t	tensile stress [N/mm^2]
$\sigma_{x,Ed}$	design value of longitudinal normal stress
$\sigma_{z,Ed}$	design value of transverse normal stress
τ_{Ed}	design value of shear stress
Φ_{LT}	lateral torsional buckling auxiliary variable
χ_{LT}	relative slenderness for lateral torsional buckling
ψ	combination factor

Constants

g	gravitational acceleration 9810 mm/s^2
v	Poisson's ratio for steel in elastic range 0,3

Abbreviations

BLF	buckling load factor
CAD	computer aided design
FEA	finite element analysis
HEA/HEB	European wide flange beams
IPE	European I beams
RHS	rectangular hollow section beams
SFS	Finnish standards association
SLS	service limit state
ULS	ultimate limit state

Table of contents

Appendices

1 Introduction

This master’s thesis was done for a certain international equipment supplier. The supplier’s industry is a highly competitive field and thus parts of this master’s thesis are explained vaguely to protect the products of the supplier. The focus of master’s thesis is on design and serialization of working platform support beams that are mounted inside of a piece of equipment. Following introductory sections will briefly discuss backgrounds of the topic and objectives of this this master’s thesis.

1.1 Motivation and research problem

Motivation for this master’s thesis comes from the supplier’s need to improve delivery efficiency of the working platforms located inside of process equipment. The working platforms are a temporary structure whose primary function is to allow access to areas inside the equipment. The working platforms have special requirements due to unusual location and use case along with general requirements set by standards.

Research problem in this master’s thesis is the currently inefficient method for designing and manufacturing the supporting beams for the working platforms. The working platforms are designed and manufactured by different industrial working platform manufacturers, as designed-to-order products to fit the needs of each individual equipment. As a tentative solution to the problem the design of the support beams could be standardized and serialized to be suitable for use in all equipment of a certain product family.

1.2 Research questions

This master's thesis is going to answer following research questions:

1. What are the expected load cases for the structure?
2. What is the minimum number of different support beam variations?
3. What profile shape and material are optimal for the support beams
4. What needs to be considered when designing serialized support beams for working platforms?
    a. What is the expected critical failure mode limiting dimensioning?
    b. How does serialization affect steel structure design?

1.3 Research objectives

The objective of the master thesis is to improve the delivery efficiency of the support beams using methods of serialization. Goal is to investigate range of equipment within a product family and find critical loading cases that can be used as the baseline for design of serialized support structures. The goal is to minimize variation in the support structures and find readily available solutions for all equipment in the product series to improve the efficiency of the delivery process. Goal of serialization is to also ensure safety by defining sufficient loading cases considering most unfavourable case of loading. Manufacturing friendly viewpoints will be considered where possible to further optimize the structure. The end goal of the thesis is to select suitable profiles, dimension welds and bolted connections and validate the structure with Finite Element Analysis (FEA) and analytical calculations.

1.4 Research methods

This master's thesis will combine methods of reviewing relevant standards, analytical calculations, and finite element analysis. Theoretical base for the master's thesis comes from the field of strength of materials and research on steel structures. Databases provided by LUT-University will be used for gathering information about the subject. Main tools for finding relevant literature are LUT Primo for books, e-books, and scientific publications and Finnish Standards Association's (SFS) webservice SFS Online for standards. FEA and analytical calculation are used to validate the structure. Models for FEA are made in Solidworks Computer Aided Design (CAD) software and analysis is done in Simcenter Femap with NASTRAN solver.

1.5 Scope

This master's thesis focuses only on the design of the support structures of the platforms and not the flooring that forms the platforms. Thus, due to unknown properties of the flooring, some assumptions are necessary. Designing of the support structures is limited to dimensioning the support beams, and weld and bolted connections. The general solution will remain similar to the current solution due to the supporting points being locked. Therefore, focus will be more in redefining the current design to allow serialization. The structure will be designed according to provisions of Eurocode standards SFS-EN 1990 and SFS-EN 1993. Safety of machinery standard SFS-EN ISO 14122-2 will be considered where applicable.

2 Structure of internal supports

Following chapter will briefly give insight to the structure to be studied and highlight important aspects to be considered in designing phase of this master's thesis. The structure consists of supporting beams, flooring, and standard temporary construction scaffolding. The structure is designed to support the weight of personnel doing work inside the equipment as well as weight of tools and spare parts.

2.1 Arrangement of supporting beams

The current solution for the platform supports is an array of beams that are pushed through the wall of the equipment and connected to a connection nozzle outside of the equipment. Two different supporting types are used: radial and crossing. Supporting beams that cross the entire equipment are used with smaller sized equipment where resulting support beam length is also shorter. Radial beams that extend slightly from the inside walls of the equipment are used with larger equipment. Installation of the crossing beams is more difficult due to their length. Installation of crossing beams also requires a counterweight for balancing. Radial beams are easier to install and do not require a counterweight. Radial beams are in larger equipment avoid the installation difficulty and to prevent excessive deformation of the supporting beams. This work does not investigate the installation of the supporting beams. Downside of radial beams is that they do not allow access to the middle of the equipment. Flooring plates are attached on top of the supporting beams to form the platform. Example setups of crossing beams is shown in left and centre of figure 1 and an example of radial beams setup is shown on right of figure 1.

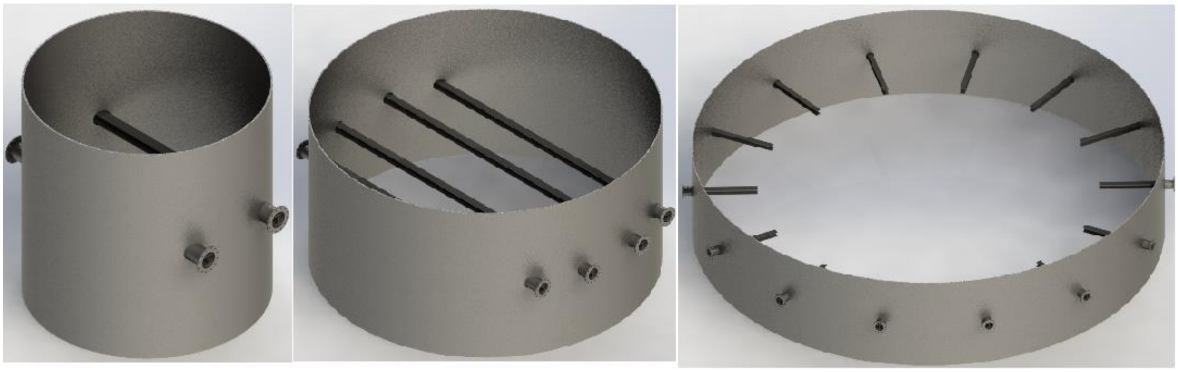


Figure 1. Supporting types.

Welded beam assemblies consist of beam and a beam end flange. Once the beam is in place, the beam end flange is connected to the outside nozzle flange by bolts. In cases with crossing beams, additional supporting piece is attached to the floating end of the beam as shown in figure 2. For taking a portion of the load off from the beam end flange weld and bolted connections, lower support pieces are attached between the beam and the inner wall of the nozzle on both ends of the beam. Additionally upper similar upper support pieces are possible to further aid with rigidity of the connection as shown in figure 3.

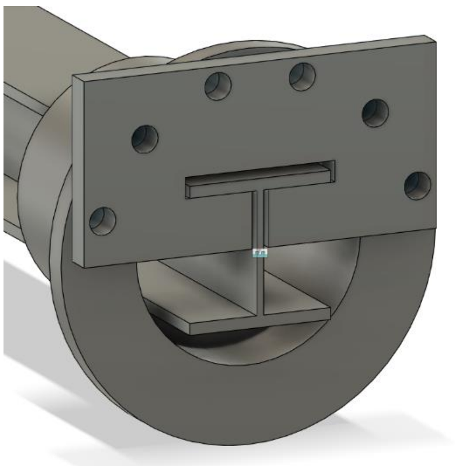


Figure 2. Detail of crossing beam floating end support piece.

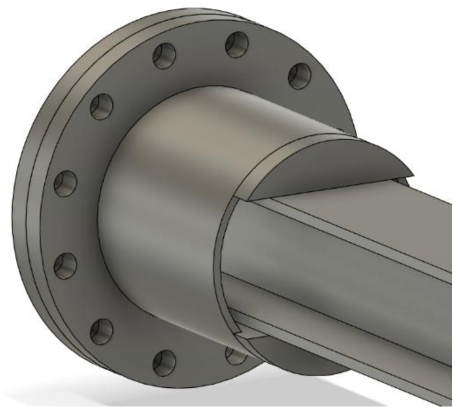


Figure 3. Detail of fixed nozzle support with nozzle wall support pieces.

Two different diameter nozzles are used in different cases. Nozzle size sets a geometrical limit for selecting a suitable profile shape and size as shown in figure 4 with example I-beam shapes representing IPE 200 profile in case of DN250 and IPE 240 in case of DN300.

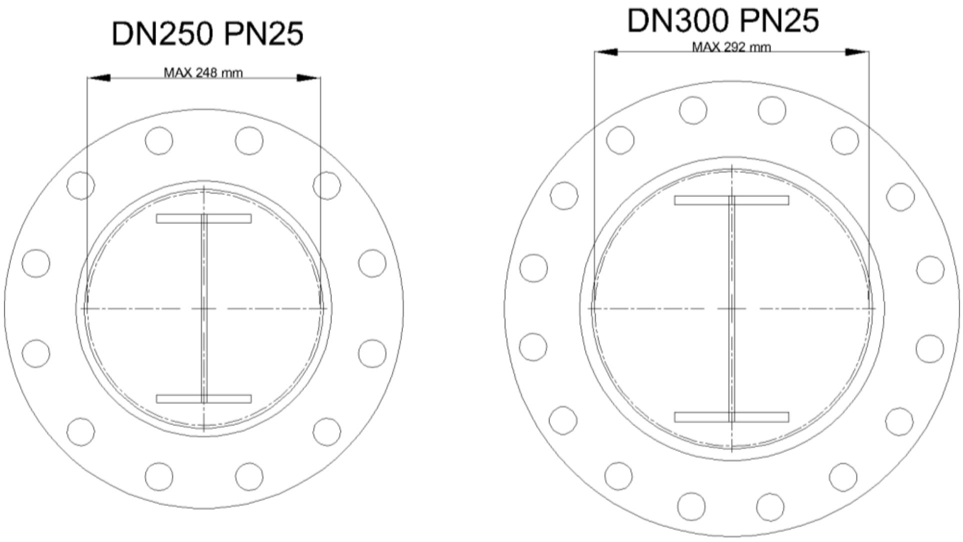


Figure 4. Nozzle sizes.

Dimensions of nozzle flanges are presented in SFS-EN 1092-1. Inner diameter is specified by the supplier. The profile must fit through the nozzle without contact to the inside walls of the nozzle. Also taking the size of weld bead between connection flange and beam into account a maximum diameter of 248 mm for DN250 PN25 nozzle and 292 mm for DN300

PN25 nozzle were defined as the limits. Goal was to avoid dimensioning beams too close to the defined maximum values to avoid any installation problems.

In this master's thesis the supports were divided into 7 different cases based on information gathered from the equipment product family as shown in table 1. These 7 cases are later used in defining loading cases for structural analysis. Larger size equipment has increasing number of supporting beams to cover a larger area and to support a larger weight of the internal scaffolding, increased number of personnel simultaneously working inside of the equipment and increased number of replacement parts, tooling etc.

Table 1. Support cases to be investigated.

Case	Support type	Number of support beams	Number of nozzles
1.	crossing	2	2 per side
2.	crossing	4	4 per side
3.	crossing	4	4 per side
4.	radial	8	8
5.	radial	10	10
6.	radial	12	12
7.	radial	14	14

2.1.1 Scaffolding

The working platforms need to support the weight of the working personnel, tools, parts, and scaffolding that is built on the platforms. Main function of the working platforms and scaffolding is allowing easy access to working areas shown in figure 5. The working areas are located at different heights inside the equipment and building scaffolds on top of intermediate service platforms allows easier access to these working areas. Following figure shows drawing of the scaffolding structures built on top of the platforms in the case of crossing beams. Different in the case of radial beams is that the scaffolding does not reach the middle part shown in yellow colour in figure 5.

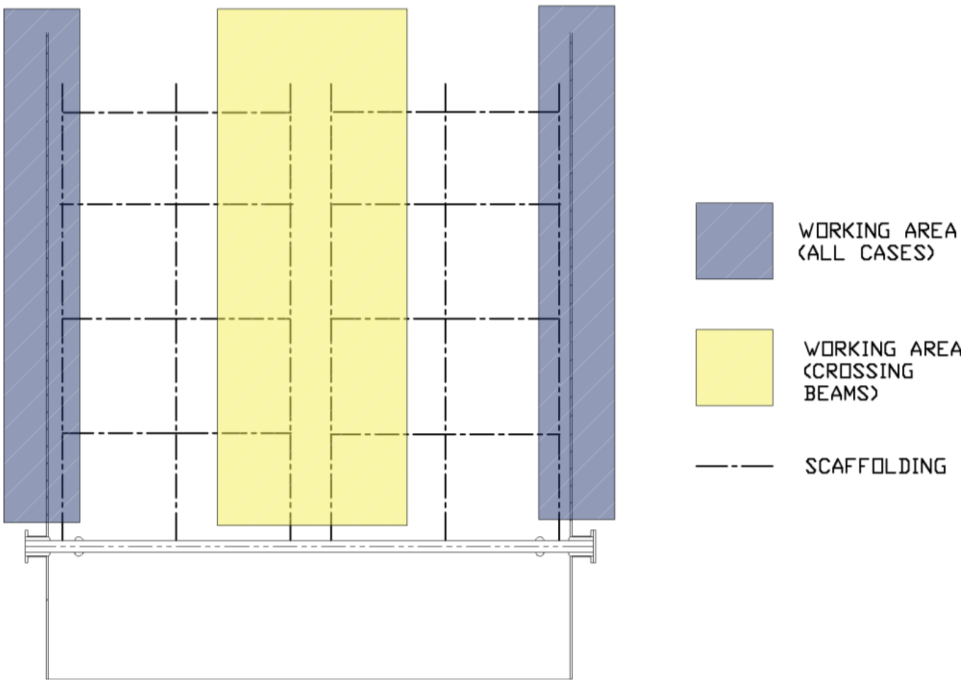


Figure 5. Scaffolding cross-section.

The scaffolding weight forms majority of the load imposed on the platforms and support beams. Position of the scaffold legs also affects how other imposed loads such as weight of working personnel and replacement parts are transferred to the platform structure since much of this load is carried through the scaffolding. Assumption was made that all imposed loads are carried through the scaffolding legs and thus loads were applied as combined point loads at assumed positions of scaffolding legs. Position of scaffold legs was assumed to be directly on top of the supporting beams.

A method for calculating example scaffolding load was acquired from a scaffold company. The method provided gives estimation of the total weight of the scaffolding based on the scaffold area and scaffold height. The height of the scaffold is divided into floors with estimated headroom of 2,25 m. The weight of a single intermediate floor is estimated to be 37,5 kg/m^2 and the topmost level is estimated to weigh 43,5 kg/m^2. To find out the total weight, the weight of all intermediate levels and topmost level weight is summed, and the result is multiplied by the area to find out the estimated total weight. (M.Mäläskä, 2023.)

3 Methods and theory behind design work

Following chapter describes methods of design based on applicable standards as well as theory behind design and dimensioning. The basis for analytical calculations and FEA will be defined in this chapter.

3.1 Materials of the support structure

Safety of machinery standard SFS-EN ISO 14122-2 that presents provisions for working platform design states only that the materials used in working platforms should be selected so that they withstand foreseeable conditions of use. Usually this means that working platforms are made from common carbon steel, stainless steel, or aluminum depending on the use case and environment of the platforms. The mechanical properties, manufacturability, cost, and global availability of common carbon steels are best suited for the application of this master’s thesis. Structural steels S235 and S355 were chosen as the baseline for the material selection due to their good global availability. S235 is typically the lowest strength structural steel that is used. Both S235 and S355 are widely used structural steels. Lower tensile strength steels can provide some advantages over higher strength steel one being usually a better availability of material and thus lower cost of structure. In some cases, and in regions where higher strength steel is available, higher strength steel can provide lower overall cost for the structure even if the material itself would be more expensive.

The structure under consideration will not presumably benefit from higher tensile strength as there is a vertical deformation limit set for the structure. Benefits of high strength steel can be better utilized in members whose deflections are not the limiting factor and where higher tensile strength can be used to create thin and lightweight structures. Vertical deflection cannot be prevented by use of higher strength steel but instead it can be effectively limited by increasing the height of the beam profile and thus increasing the area moment of inertia. Increasing the height of the profile in turn decreases the stability in bending which may become the limiting factor in some cases. Goal of the profile selection will be to find globally available cost-effective combination of the material and profile that will produce a structure that can withstands the loading cases that will be defined in this master's thesis. Standard I-profiles are favored, and their suitability will be evaluated.

3.1.1 Structural steel as material

Material properties for structural steel are defined in Eurocode 3 standard SFS-EN 1993-1-1. More specific information about the general technical delivery conditions of structural steels are defined in standards EN 10025, EN 10210 and EN 10219. These standards have multiple parts that each have technical specifications for different steel grades. The technical specification of common non-alloy structural steels can be found in EN 10025-2, which is the standard for hot rolled products of structural steels. For example, EN 10025-2 S235 has following nominal values for yield strength (f_y) and ultimate tensile strength (f_u) that are also listed in Eurocode standard (SFS-EN 1993-1-1 2022, p. 28):

$f_y = 235$ N/mm^2 when $t \leq 40$ mm

$f_u = 360$ N/mm^2 when $t \leq 40$ mm

$f_y = 215$ N/mm^2 when 40 mm $\leq t \leq 80$ mm

$f_u = 360$ N/mm^2 when 40 mm $\leq t \leq 80$ mm

where:

t is nominal thickness of the element

Yield strength is a key material property of steel, and it is defined as the point in stress strain curve where the behavior of the material turns from elastic behavior to plastic behavior meaning that deformations do not return completely after removing the load. Ultimate tensile strength is the maximum strength that the material has before necking and fracturing. Figure 6 shows an example of engineering stress-strain curve.

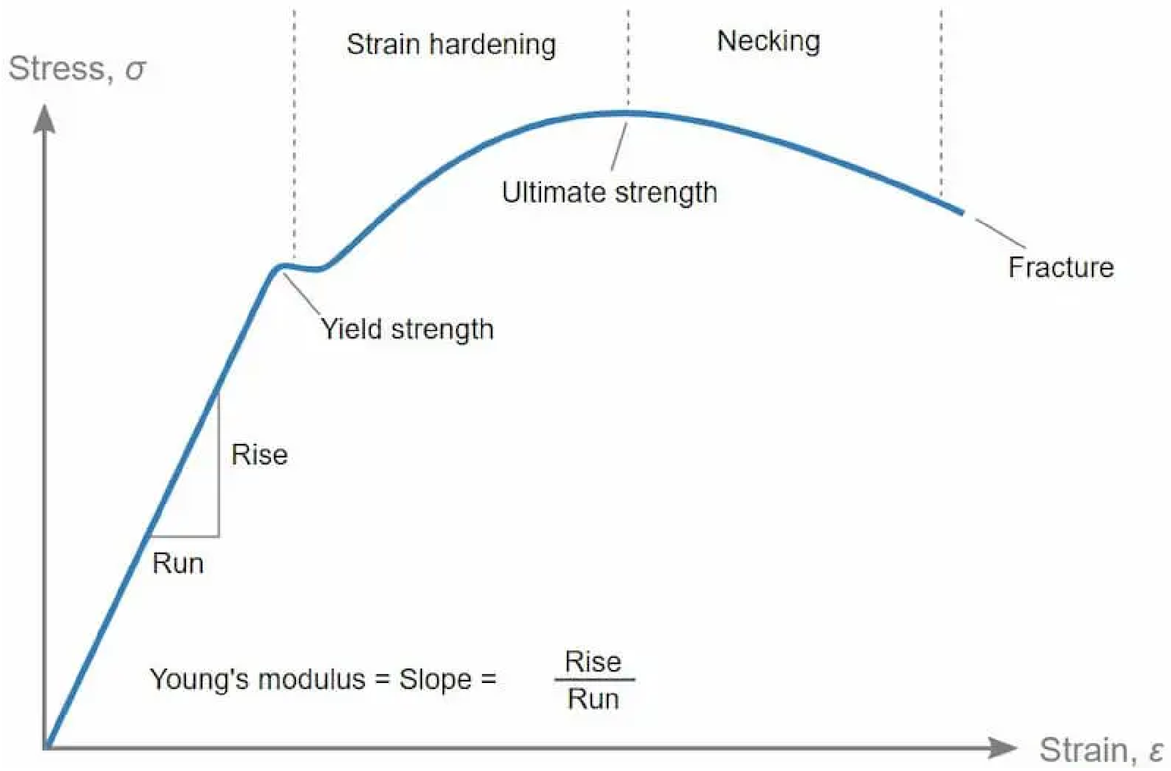


Figure 6. Example of a stress strain curve (A.Velling 2020).

A reduced design value for yield strength is used in Eurocode standards which is defined by following equation (E. Niemi 2003, p.15):

$$f_{yd} = \frac{f_y}{\gamma_M} \tag{1}$$

where:

f_{yd} is design value of yield strength.

f_y is nominal value of yield strength.

γ_M is material's partial factor.

According to Eurocode 3 following material properties of structural steels are used in this thesis (SFS-EN 1993-1-1 2022, p. 31):

Young’s modulus $E=210\ 000\,\frac{\mathrm{N}}{\mathrm{mm}^2}$

Shear modulus $G=\frac{E}{2(1+v)}\approx 81000\,\frac{\mathrm{N}}{\mathrm{mm}^2}$

Poisson’s ratio in elastic range $v=0{,}3$

A material parameter ε is used in EC3 that is used in assigning a cross-sectional class to a profile and it’s defined by following equation (SFS-EN 1993-1-1 2022, p. 31):

$$\varepsilon = \sqrt{\frac{235}{f_y}} \tag{2}$$

Material properties of steel and well-known behavior in the elastic range makes it a suitable choice in structure design. Hundreds of years of using and researching steel has provided designers tools to dimension steel structures to be tougher and lighter. Some of the oldest theories are still used to this day such as the Poisson’s ratio. Another important law in strength of materials is the Hooke’s law which gives the relation between applied load and strain as shown in equation:

$$\sigma_t=E\varepsilon_t \tag{3}$$

where:

σ_t is tensile stress

ε_t is tensile strain

3.1.2 Bending of a beam

Hooke’s law has important use in the Euler-Bernoulli beam theory which gives relation of beam’s deformation and applied load. Following derivation of the Euler-Bernoulli beam equation was done based on book “Strength of Materials Third Edition” by Ferninand L. Singer and Andrew Pytel. Two assumptions are made to form the base of this theory. Applied load is assumed to cause purely bending moment and no shear load and cross-section is assumed to stay undeformed under loading. These two assumptions make it possible to form equation for moment curvature. In practical cases with minimal shearing, these assumptions give realistic results. The Euler-Bernoulli theory looks at a section of a beam under loading show in following figure:

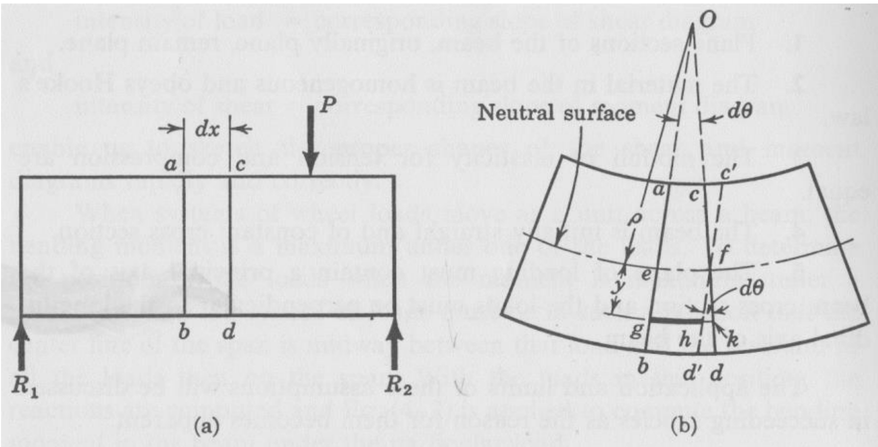


Figure 7. Beam section for Euler-Bernoulli theory (Singer & Pytel 1980, p. 154).

Applied load P causes the beam to bend from state (a) to state (b). Flexure formula is used to get relation between the applied load and the deformation. Formation of the flexure formula begins by defining two lines a-b ja c-d that separated by distance of *dx*. Lines a-b ja c-d to rotate at an angle of $d\theta$ at the intersection of lines. Compared to undeformed beam the distance between a and c is shorter and between b and d is longer in the deformed state. At the neutral surface, the distance between e and f is unchanged. What can be interpreted from the changes in length is that between a and c there is compression and between b and d there is tension. Since the distance between e and f remains unchanged, it doesn’t have any normal stress. Next step in forming the moment curvature equation is defining some line g-h at distance of y from the neutral surface. The elongation of g-h in the deformed state is h-k. Length of h-k is equal to the arc of circle of radius y subtended by the angle $d\theta$ formed at intersection of lines ab and cd as shown in following equation:

$$\delta = hk = y\, d\theta \qquad (4)$$

where:

δ is the change in length

hk is the elongation of curve g-h

y is distance from the neutral surface

$d\theta$ is angle formed at the center of bending

Tensile strain resulting from bending can be found by dividing the deformation (δ) by the length (L) of neutral line e-f at undeformed neutral surface:

$$\varepsilon_t = \frac{\delta}{L} = \frac{y\, d\theta}{ef} \qquad (5)$$

Radius of the deformed curve at the neutral surface can be denoted as ρ. The curved length of neutral surface line e-f can is equal to $\rho\, d\theta$. This can be placed back to previous equation to form the following equation:

$$\varepsilon_t = \frac{y\, d\theta}{\rho\, d\theta} = \frac{y}{\rho} \qquad (6)$$

From here previously mentioned Hooke’s law can be used to find out what is the tensile stress of a fiber at distance of y from neutral surface:

$$\sigma_t = E\varepsilon_t = \left(\frac{E}{\rho}\right) y \qquad (7)$$

To derive the moment curvature equation from here, equilibrium conditions are applied. Equilibrium is considered on some arbitrary element dA as shown in following figure:

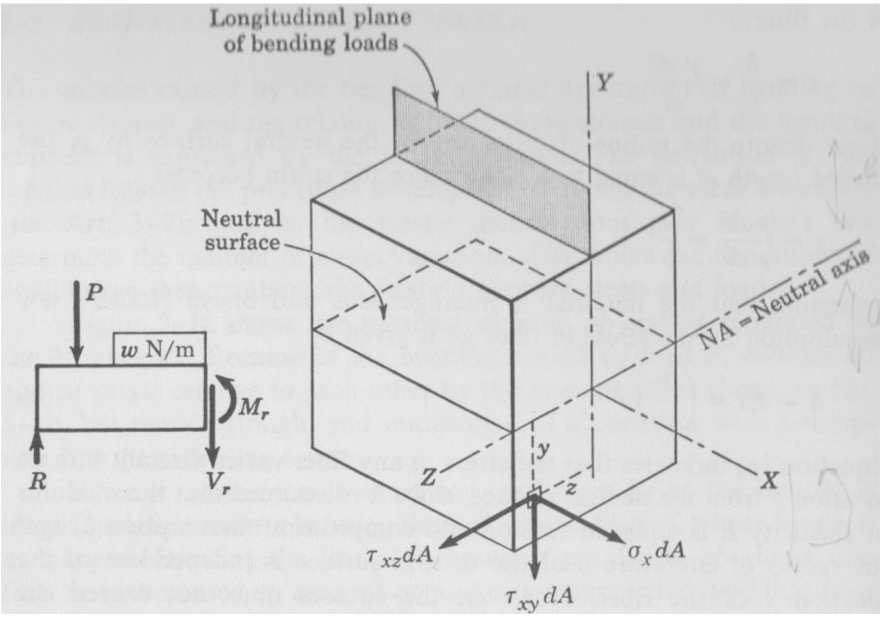


Figure 8. Investigated arbitrary element for deriving flexure formula (Singer & Pytel 1980, p. 154).

First condition is that external forces have no X components leading to condition:

$$[\sum X = 0] \text{ , } \int \sigma_x dA = 0$$

where, σ_x is equivalent to σ_t in equation (7) Therefore, following is also true:

$$\frac{E}{\rho} \int y dA = 0 \qquad (8)$$

$y\ dA$ is the moment of the differential area dA about the neutral axis. The integral $\int y dA$ is the total moment of area. Hence:

$$\frac{E}{\rho}A\bar{y}=0 \tag{9}$$

From this equation it can be noticed that all terms except $\bar{y}$ must be non-zero. Therefore, this condition tells that the distance to the centroid of the cross-sectional area must be zero meaning that the neutral axis must contain the centroid of the cross-sectional area.

Condition $[\sum Y=0]$ leads to the shear stress formula while condition $[\sum Z=0]$ leads to $\int \tau_{xz} y dA=0$ but since the loading in this case doesn't have a Z component, the system of τ_{xz} dA must be self-balancing.

Condition $\left[\sum M_y = 0\right]$ means that external and internal forces have no moment about the Y axis leading to condition:

$$\int z(\sigma_t)dA=0 \rightarrow \frac{E}{\rho}\int zy\ dA=0 \tag{10}$$

Final condition leading to moment curvature equation is $[\sum M_z=0]$ which requires a resisting moment M_r to be present. The resisting moment in the observed element can be written as y $(\sigma_x\ dA)$ leading to condition:

$$M=\int y(\sigma_t\ dA) \rightarrow \frac{E}{\rho}\int y^2 dA \tag{11}$$

In equation (11) term $\int y^2\, dA$ is the moment of inertia of the area about the neutral axis or other reference axis if so chosen. From here the moment curvature equation can be written as:

$$M = \frac{EI}{\rho} \rightarrow \frac{1}{\rho} = \frac{M}{EI} \qquad (12)$$

where:

M is the bending moment

I is the area moment of inertia

Processing the flexure formula further following equation can be formed by placing E/ρ from equation (7) giving:

$$\frac{E}{\rho} = \frac{M}{I} = \frac{\sigma_t}{y} \qquad (13)$$

By rearranging, the flexure formula can be written as follows:

$$\sigma_t = \frac{My}{I} \qquad (14)$$

This equation gives values of flexural stress in any distance from the neutral axis. As can be noticed the flexure stress is greatest at the furthest point from the neutral axis. The furthest point from the centroid is usually denoted as c in literature. A ratio of c/I is called the section modulus which is often denoted with S or W (later denoted with W). Final form for flexure formula is as follows (Singer & Pytel 1980, p. 154-158):

$$\sigma_{MAX}=\frac{M}{W} \tag{15}$$

Relation of section modulus and bending stress is used later when calculating resistances of the support beams in bending.

3.1.3 Deflection of a beam

The deflection of the beams can be evaluated through multiple ways by using the flexure formula in combination with geometry of the beam. Most used methods are double integration method, Macaulay's method, moment-area method, superposition method, and Castigliano's theorem. These methods work with relatively small deflection and in the linear elastic zone. To get understanding on how deflections are calculated, the double integration method is investigated by looking at derivation of the deflection curve by double integration method found on Ferninand L. Singer's and Andrew Pytel's book "Strength of Materials Third Edition". In double integration method the elastic curve of the beam subjected to bending is evaluated. The three important properties of the elastic curve are the distance along the axis of the undeformed beam, vertical deflection of the beam and the angle of rotation of the beam. First assumption is made that the deflection is so small that the angle of rotation can be approximated as the relation between the vertical deflection and the distance along the axis or in other words slope of the beam as shown in following equation (Singer & Pytel 1980, p. 214):

$$\theta=\frac{dy}{dx} \tag{16}$$

where:

θ is the angle of deflection

d_y is the vertical component of deflection curve slope

d_x is the horizontal component of deflection curve slope

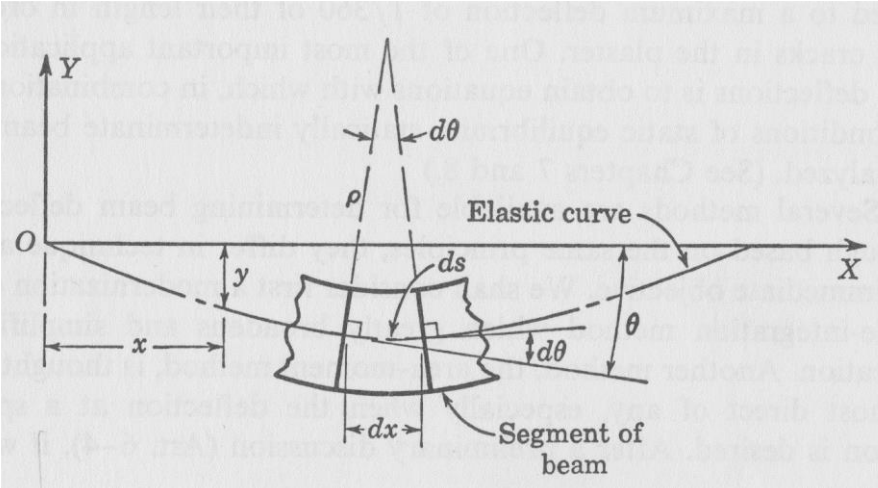


Figure 9. Elastic curve (Singer & Pytel 1980, p. 214).

Next, the moment-curvature equation (12) that was derived earlier is used to get relation between the deflection curve and the deformation of the beam subjected to bending. Radius of curvature ρ in the moment-curvature equation can be written as function of y and x coordinate as the following (Singer & Pytel 1980, p. 215):

$$\frac{1}{\rho} = \frac{\frac{d^2y}{dx^2}}{\left[1+\left(\frac{dy}{dx}\right)^2\right]^{3/2}} \tag{17}$$

Since the slope is small, its affect in the right-side expression's divider can be assumed as very small and the expression of the curvature can be placed back to moment curvature equation followingly:

$$\frac{1}{\rho}=\frac{d^2y}{dx^2}=\frac{M}{EI} \tag{18}$$

The vertical deflection can be calculated from this equation by integrating variable *M* twice with respect to x. First integration gives the equation for the slope of elastic curve (θ):

$$\theta=\frac{dy}{dx}=\frac{1}{EI}\int M(x)dx+C_1 \tag{19}$$

Second integration gives the equation for the vertical deflection (y):

$$y=\frac{1}{EI}\int\int M(x)dxdx+C_1x+C_2 \tag{20}$$

What can be noticed from the equation of beam deflection is that deflection can only be affected by changing the load and boundary case, selecting a material with different elastic modulus or by affecting the area moment of inertia by changing cross-sectional shape. Elastic curves for various load and boundary conditions can be solved from equation (20) by solving the integration constants C_1 and C_2.

For many common load and boundary conditions integration is not necessary for each new case but instead tables with already integrated deflection curves can be used. This is especially useful as the method of superposition can be used to combine different loading cases. Superposition method determines the total deflection as resultant of deflection caused by each load acting separately. Superposition method works with relatively small deflections so that each deflection caused by each load can be assumed to be independent. (Singer & Pytel 1980, p. 215) Appropriate deflection curve equations for analytical calculations were taken from engineering handbook “Tekniikan taulukkokirja” by Esko Valtanen.

3.1.4 Stability

Due to slenderness of structural parts loss of stability often happens before static yield strength is reached and stability verification becomes an important design step. Structural members that are under axial compression can lose stability through buckling. In buckling critical load causes the structure to rapidly deform. In columns loss of stability occurs most notably as Euler buckling and in plates as local buckling. Beams loaded with bending moment may lose stability by lateral torsional bulking. Lateral torsional buckling is caused by the compression stress in the top flange leading to destabilization. Methods for calculating critical bending moments for lateral torsional buckling can be found in Eurocode 3 standards which are later investigated. Slender cross-section beams may become susceptible to local plate buckling if some plate section of the beam reaches a critical plate buckling stress. This happens most notably in beams belonging in so called cross section class 4. Cross section classes are explained later in this master's thesis.

In this master's thesis the most important consideration in terms of stability is the lateral torsional buckling of the support beams. Following figure 10 shows an example of a beam in lateral torsional buckling along with the two important parameters of the phenomena being the lateral displacement (Δ_{LT}) and twisting of the beam (θ_{LT}). Actual buckling resistance will be calculated by using a method from Eurocode 3 standard. This method is presented in a later section of this chapter discussing design based on Eurocode standards.

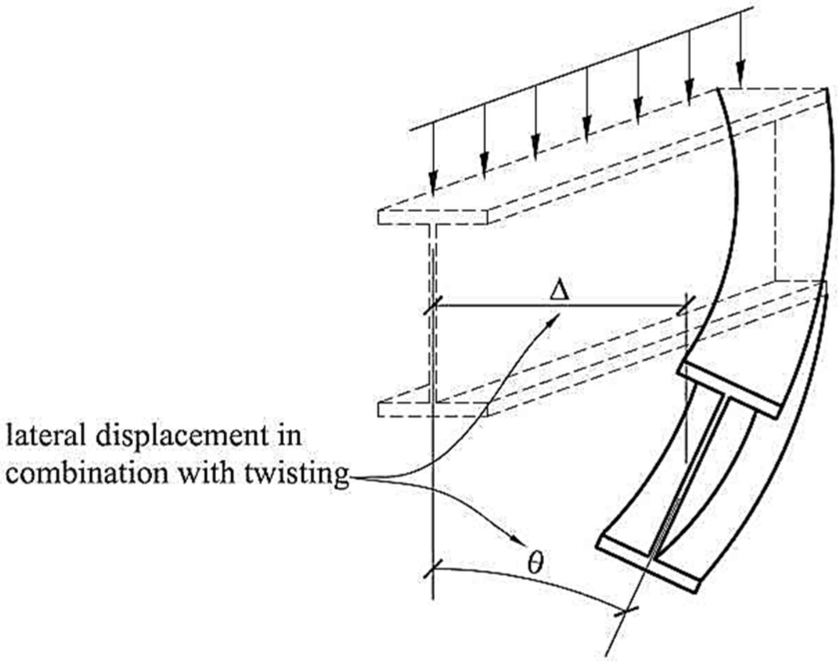


Figure 10. Example of lateral-torsional buckling (Al-Zaidee, Salah & Al-Hasany, Ehab 2017).

3.1.5 Profile

Material selection and profile selection work in combination to form a structure that withstands the foreseeable conditions. Profile selection affects the cross-sectional area and thus also affects the flexural rigidity, and the stability of the beam. Due to bending caused by the loads, flexural rigidity and stability are the driving factors in profile selection.

Optimal beam profile shape could be solved mathematically and customized based on previously defined maximum nozzle diameters or by choosing a standard profile that passes through the nozzle. In this master’s thesis, standard profiles with maximum allowable dimensions were selected for investigation by purely trial and error method. Following maximum profile sizes were selected to be investigated in cases with DN250 size flange: IPE200, HEA160, HEB160, RHS (200x120x8 mm) and RHS (200x120x12,5 mm). For DN300 nozzle additional bigger profile sizes were considered: IPE240, HEA180, HEB180.

Suitable profile satisfies the geometrical requirements but also has sufficient flexural rigidity and elastic strength based on theories defined in earlier sections.

Cross-section's design properties were applied according to standard SFS-EN 10365:2017 "Hot rolled steel channels, I and H sections. Dimensions and masses". Cross-section properties were calculated and used in calculations based on cross-section dimensions and weight per unit length presented in table 2. Symbols for IPE and HE dimensions are shown in figure 11. Same symbols were also applied for RHS cross sections.

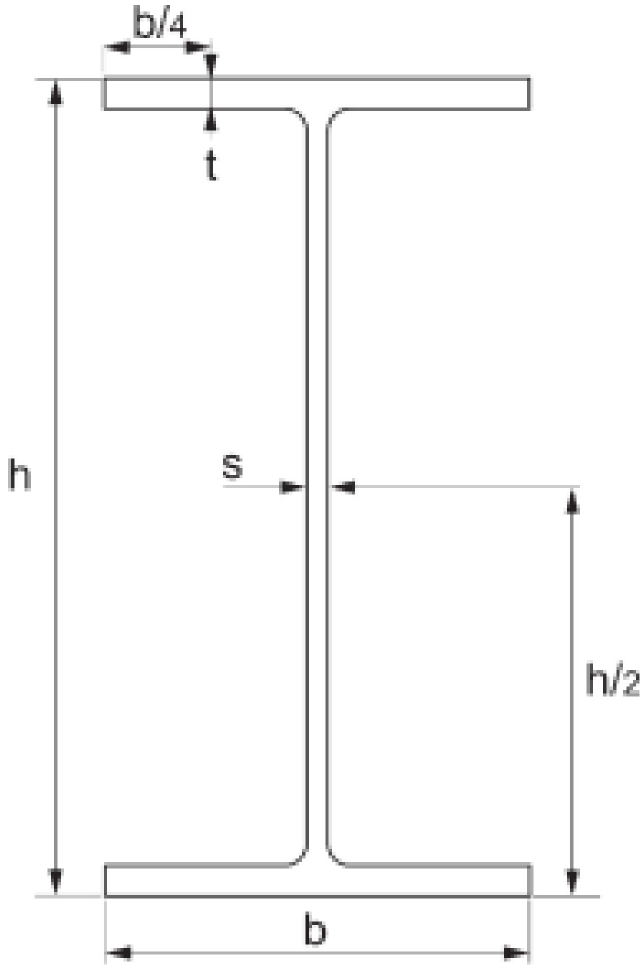


Figure 11. IPE and HE cross-section dimensions (SFS-EN 10365:2017, p. 5).

Table 2. Beam cross-section properties (SFS-EN 10365:2017 p. 7, p. 9).

Cross-section	*M* (kg/m)	*h* (mm)	*b* (mm)	*t* (mm)	*s* (mm)
IPE200	22,4	200	100	8,5	5,6
IPE240	30,7	240	120	9,8	6,2
HEA160	30,4	152	160	9	6
HEA180	35,5	171	180	9,5	6
HEB160	42,6	160	160	13	8
HEB180	51,2	180	180	14	8,5
RHS (200x120x8)	37,6	200	120	8	8
RHS (200x120x12,5)	56,6	200	120	12,5	12,5

3.2 Design based on Eurocode standards

This subsection describes methods of designing steel structures in accordance with Eurocode standards. Eurocodes are a collection of 10 standards developed to guide design work of buildings and other civil engineering works and construction products. Eurocode standards unify the design process between EU countries with a goal to make European companies more competitive as well as uniform safety of construction in Europe. Eurocodes include the following standards:

- basis of structural design (EN 1990);
- actions on structures (EN 1991);
- the design of concrete (EN 1992), steel (EN 1993), composite steel and concrete (EN1994), timber (EN 1995), masonry (EN 1996) and aluminium (EN 1999) structures; together with
- geotechnical design (EN 1997); and
- the design, assessment and retrofitting of structures for earthquake resistance (EN 1998)

(European commission, 2023.)

3.2.1 Design loads and limit state verification based on Eurocode standards

Load cases will be defined for the seven supporting cases based on product series specification of the supplier, scaffolding calculations according to previously mentioned method, and standards. Defining loads is crucial to the success of steel structure design since underestimated loads can lead to safety hazards and overestimations to increased costs and weight. Since the structure is not weight critical, limiting overestimations is not crucial to the success of the design while underestimations can lead to possible safety hazards. Design values of loads should always be chosen considering the most critical loading case i.e., where loads and consequences of failures are highest. In the case of this master's thesis the design loads to the platform and supporting beams are self-weight of the working platform structure, weight of the scaffold, personnel, and weight of replacement parts and equipment. Previous design iterations can give some insight to the design loads which have been assigned for some loading case. To get the reliable results loads are redefined by re-evaluating the different loading cases.

In SFS-EN 1990 "Eurocode. Basis of structural and geotechnical design" loads are categorized as actions which are described as "mechanical influence on a structure, or a structural member, exerted directly or indirectly from its environment". (SFS-EN 1990, p. 16) SFS-EN 1990 divides actions into four main categories. These are permanent actions (G), variable actions (Q), accidental actions (A) and seismic actions (A_E). Permanent actions include self-weight of the structure and self-weight of non-structural elements. Variable actions include loads that vary in time including imposed loads (Q), snow loads (S), wind loads (W), temperature variations (T). (SFS-EN 1990 2023, p. 38) Accidental actions act only a short duration but in significant magnitude and are statistically unlikely to happen during the design service life when proper care is taken in the operation. Accidental actions include for example impact loads from collisions or falls from height. (SFS-EN 1990 2023, p. 17) Seismic actions need to be considered in seismic zones where earthquakes are likely to happen, and they include actions caused by earthquakes. (SFS-EN 1998 2004, p. 15) This thesis is limited to investigating permanent loads and imposed loads. Since the structure under investigation is a working platform, guidelines for choosing appropriate loads can be found in standard SFS-EN ISO 14122-2 where following is stated:

“To determine the design load, at least the following shall be taken into account:

— the number of persons at work on the specific location;

— the mass of the tools, spare parts, and work equipment needed on location;

— the strength of impact on the structure due to a drop of tools and/or machine (replacement) parts;

— the possible concentrated loads due to the weight distribution (geometrics) of the used parts needed at the work;

— the weight due to environmental causes (e.g. fluids, water, snow, ice, spill, etc.) that can be deposited at the location of work on the platform.

The minimum loads to take into account for walkways and working platforms are as follows:

— 2 kN/m^2 uniformly distributed load to account for the structure;

— 1,5 kN concentrated load applied in the most unfavourable position over an area of 200 mm × 200 mm of the floor.” (SFS-EN ISO 14122-2, p. 13 – 14.)

With information from the standards, requirements from the supplier and known self-weight weight of the structures, a load case table can be formed as shown in table 3.

Table 3. Load component table.

Case	Permanent actions (G) [kg]		Total variable actions (Q) [kg]
	Beam weight (per beam) ($M_{b,RHS}$)	Beam weight (per beam) ($M_{b,HEA}$)	
1.	197	106	2987
2.	261	140	6148
3.	397	213	9338
4.	128	69	13104
5.	128	69	16684
6.	128	69	23447
7.	128	69	27707

Total variable actions presented in table 3 include loads of flooring weight, scaffolding weight, personnel, accessories, and additional concentrated loads. Weight of flooring that is included n the total variable actions was calculated according to DIN 24537-1 Gratings used as floor coverings – Part 1: Metal gratings, p. 6 and scaffolding weight was calculated by using the previously mentioned method. Other loads were estimated together with the supplier.

Eurocode standards utilizes multiplying partial factors in loads and reducing partial factors in material properties. (E. Niemi, p.14) Each so called limit state has their own associated partial factors. Generally, two limit states are considered which are the service limit state (SLS) and the ultimate limit state (ULS). SLS verification concerns normal use of the structure and ensures normal function and comfort. SLS verification usually includes defining limit states of deformation and of vibration. ULS is associated with safety of the structure. ULS verification includes for example analysis of plastic resistance, buckling and fracture forming phenomena and other cases where failure of the structure can happen, or people's safety might be at risk. (Silva, L. S. da., Simões, Rui., & Gervásio, Helena. 2010, p. 8-9) Structural analysis is based on the appropriate limit state under consideration. (SFS-EN 1993, p. 32) In this thesis the SLS under consideration is the vertical deflection of the beams while ULS is the yield limit or loss of stability of the structure.

In practice, ULS verification is done by checking the following condition shown in equation (SFS-EN 1990, p. 48):

$$E_d \leq R_{d,ULS} \qquad (30)$$

where:

$E_{d,ULS}$ is the ultimate limit state design value of the effect of actions.

R_d is the design value of the corresponding resistance.

Design values of the effects of actions E_d for ultimate limit state verification are defined using following equation (SFS-EN 1990, p. 48):

$$E_d = \gamma_{Sd} E\{\sum(\gamma_F \psi F_k); a_d; X_{Rd}\} \qquad (31)$$

where:

γ_{Sd} Is partial factor associated with the uncertainty of the action and/or action effect model.

$E\{\Sigma\}$ Denotes here the combined effect of the enclosed variables.

γ_f is partial factor that takes account of unfavourable deviation of an action from its representative value and its value which is specified in Eurocode standards is dependent on the category of action.

ψ is combination factor either equal to 1,0 for permanent actions or as defined in SFS-EN 1990 (p.69) for variable actions

F_k is characteristic value of action such as a load.

a_d denotes design values of geometrical properties which can be defined by adding a design value of imperfections to nominal geometrical properties if structure is sensitive to deviations in geometrical properties (SFS-EN 1990, p. 59-60)

X_{rd} denotes the values of material properties used in the assessment of E_d. For example, design value of yield strength F_{yd}.

SLS verification is done by checking the following condition similarly to ULS verification as shown in equation (SFS-EN 1990, p. 60):

$$E_d \leq C_{d,SLS} \tag{32}$$

where:

$C_{d,SLS}$ is the limiting design value of the relevant serviceability criterion

Design values are defined by using equation (SFS-EN 1990, p. 60):

$$E_d = E\{\sum(\gamma_F \psi F_k); a_d; X_{Rd}\} \tag{33}$$

where:

γ_F = 1,0

Design bending moments for limit state verifications are calculated by using the defined design loads according to Eurocode 3 and hand calculation tables found on “Tekniikan taulukkokirja” by Esko Valtanen.

3.2.2 Deflection

According to Eurocode standards serviceability criteria like maximum allowed vertical deflection should be specified by the relevant authority or by agreeing values with project's relevant parties. (SFS-EN 1990 p. 73) In this case the maximum vertical deflection limit was defined to be compliant with safety of machinery standard SFS-EN ISO 14122-2 which gives requirements for working platforms that are part of a stationary machine. Maximum deflection is defined by following equation:

$$\delta_{max} = \frac{L}{200} \tag{34}$$

Where:

L is the length of the span

δ_{max} is maximum allowed deflection.

Knowing the maximum allowed deflection a required area moment of inertia can be calculated using equation (20) by solving the equation in relation to area moment of inertia as shown in equation:

$$I = \frac{1}{Ey} \int \int M(x) dx dx + C_1 x + C_2 \tag{35}$$

Deflection of selected profiles with applied design loads will be evaluated using hand calculation tables and superposition theorem as well as by using FEA.

3.2.3 Cross-section design

Eurocode 3 states that “Cross-sections should be classified depending on the extent to which their resistance and rotation capacity is limited by their local buckling resistance.” Eurocode 3 divides cross-sections to 4 classes based on dimensions of compressed parts. The definitions of the cross-sectional classes from Eurocode 3 are following:

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

(SFS-EN 1993-1-1, p. 51.)

Following table shows limits of different cross-section classes according to Eurocode 3.

Table 4. Width to thickness limits for cross-section classification (Eurocode applied, 2023).

Width to thickness limits for cross-section classification according to EN1993-1-1 Table 5.2			
Class	Web		Outstand Flanges
	Web in pure compression	web in pure bending	Flanges in pure compression due to axial force or bending
Class 1	$c/t \leq 33\varepsilon$	$c/t \leq 72\varepsilon$	$c/t \leq 9\varepsilon$
Class 2	$c/t \leq 38\varepsilon$	$c/t \leq 83\varepsilon$	$c/t \leq 10\varepsilon$
Class 3	$c/t \leq 42\varepsilon$	$c/t \leq 124\varepsilon$	$c/t \leq 14\varepsilon$

Cross-sectional classes are determined by comparing the profile's plate sections' width-to-thickness ratio (*c/t*) to *ε*-parameter. Table 4 shows limits of different cross-section classes based on Eurocode 3. For calculations of bending resistance, the use of appropriate value for section modulus is checked based on the cross-section class. For cross section classes 1 and 2 the plastic section modulus can be used in determining bending resistance at full capacity. For cross-section class 3 the elastic section modulus must always be used. In the case of cross section class 4 the effective section modulus must be used to consider the partly ineffective utilization of material due to local buckling. (SFS-EN 1993-1-1, p. 53) Profiles with class 4 cross section are not considered in this investigation. Considered I-profiles such as HEA and IPE belong to either cross-section class 2 or 3 but because the structure is not allowed to yield, elastic section modulus will be used in all calculations. Thus, later mentions of section modulus will be the elastic section modulus.

If the loss of stability is prevented and deflection limit is not reached, the behaviour of the structure can become plastic i.e., stresses can exceed the yield strength of the material. In the case of this master's thesis stresses are not allowed to exceed the yield strength of the material and so stresses should be limited using von Mises yield criterion. The evaluation of static resistance is based on ULS load values meaning that additional partial factors are used.

Design values of usual stress components affecting in the cross section are the following:

$\sigma_{x,Ed}$ Design value of longitudinal normal stress at the point of consideration

$\sigma_{z,Ed}$ Design value of transverse normal stress at the point of consideration

τ_{Ed} Design value of shear stress at the point of consideration

These values can be used to calculate von Mise's limit condition using equation:

$$\sqrt{\sigma_{x,Ed}{}^2+\sigma_{z,Ed}{}^2-\sigma_{x,Ed}\sigma_{z,Ed}+3\tau_{Ed}{}^2}\leq\frac{f_y}{\gamma_{M0}} \tag{36}$$

In practice it is efficient to check Von Mise’s stress is in FEA software as deriving different stress component is difficult for hand calculation. Additionally different stresses can be evaluated individually and compared to resistance of the cross-section giving the value of utilization factor. For members with bending moments and axial loads an approximation can be made about the combined stress by summing the utilization factors of each load component as shown in following equation (SFS-EN 1993-1-1, p. 57):

$$\frac{N_{Ed}}{N_{Rd}}+\frac{M_{y,Ed}}{M_{y,Rd}}+\frac{M_{z,Ed}}{M_{z,Rd}}\leq 1 \tag{37}$$

where:

N_{ed} is the design value of axial force

N_{rd} is the design value of axial force resistance

$M_{y,Ed}$ is the design value of bending moment about the y axis

$M_{y,Rd}$ is the design value of bending moment resistance about the y axis

$M_{z,Ed}$ is the design value of bending moment about the z axis

$M_{z,Rd}$ is the design value of bending moment resistance about the z axis

Axial force resistance can be evaluated using equation (SFS-EN 1993-1-1, p. 62-63):

$$N_{Rd}=\frac{Af_y}{\gamma_{M0}} \tag{38}$$

where:

A is area of the cross-section

Bending moment resistances about y and z axis can be evaluated using following equations (SFS-EN 1993-1-1, p. 63):

$$M_{yRd}=\frac{W_y f_y}{\gamma_{M0}} \tag{39}$$

$$M_{zRd}=\frac{W_z f_y}{\gamma_{M0}} \tag{40}$$

where:

W_y is the section modulus about y axis

W_z is the section modulus about z axis

Elastic shear resistance is also checked in critical parts using following criteria (SFS-EN 1993-1-1, p. 64-66):

$$\frac{\tau_{Ed}}{f_y/(\sqrt{3}\gamma_{M0})}\leq 1 \tag{41}$$

where:

τ_{ed} is shear stress which can be calculated for I- and H- cross-sections as following:

$$\tau_{Ed}=\frac{V_{Ed}}{A_w} \tag{42}$$

when $A_f / A_w > 0,6$

where:

A_w is the area of the web

A_f is the area of a single flange

Lateral torsional buckling resistance was checked for crossing beams case with the longest beam span as the long unsupported length of the beam was considered to have a critical effect in this failure mode. The process of verifying lateral torsional buckling resistance according to the Eurocode 3 has the following steps (SFS-EN 1993-1-1, p. 81-93):

- Calculating critical elastic buckling moment (M_{cr}) for doubly symmetric cross-sections by using the following equation (43) found in non-contradicting complementary information (NCCI) report "NCCI: Elastic critical moment for lateral torsional buckling SN003b-EN-EU" (2010):

$$M_{cr}=C_1\frac{\pi^2 EI_z}{(kL)^2}\left[\sqrt{\frac{k^2 I_w}{k_w{}^2 I_z}+\frac{(kL)^2 GI_t}{\pi^2 EI_z}+\left(C_2 z_g - C_3 z_j\right)^2}-C_2 z_g\right] \tag{43}$$

where:

k is effective length factor referring to end rotation on plain. k = 1 as conservative approach

k_w is effective length factor referring to end warping. k_w = as conservative approach

C_1 and C_2 are coefficients depending on the loading and end restraint conditions. NCCI report suggests values C_1 = 1,00 and C_2 = 0,00 for conservative approach.

L is the beam length between points which have lateral restraint.

I_w is the warping constant.

I_t is the St. Venant's torsional constant.

z_g is the distance between the point of load application and the shear centre.

- Calculating modified slenderness

$$\overline{\lambda_{LT}}=\sqrt{\frac{W_y f_y}{M_{cr}}} \tag{44}$$

where:

λ_{LT} is the modified slenderness

- Calculating the lateral torsional buckling auxiliary variable

$$\Phi_{LT}=0,5\left[1+\alpha_{LT}\left(\overline{\lambda_{LT}}-0,2\right)+\overline{\lambda_{LT}}^2\right] \tag{45}$$

where:

Φ_{LT} is the lateral torsional buckling auxiliary variable

α_{LT} is the imperfection factor for lateral torsional buckling

- Calculating reduction factor

$$\chi_{LT}=\frac{1}{\Phi_{LT}+\sqrt{\Phi_{LT}^2-\overline{\lambda_{LT}}^2}}\leq 1,0 \tag{46}$$

where:

χ_{LT} is the relative slenderness for lateral torsional buckling

- Calculating the design capacity and verifying design moment

$$M_{b,Rd} = \frac{\chi_{LT} W_y f_y}{\gamma_{M1}} \tag{47}$$

$$M_{Ed} \leq M_{b,Rd} \tag{48}$$

where:

$M_{b,Rd}$ is the lateral torsional buckling resistance

3.3 Dimensioning of welded connections

As a conservative approach the connections were analytically calculated to withstand loading conditions without the under-beam nozzle wall support. Methods for the design of joints are explained in Eurocode 3 standard SFS-EN 1993-1-8. Part 8 of the Eurocode 3 standard describes methods for designing connections with bolts rivets or pins and welded connections. Additional provisions are given for structural joints connection of H- or I-sections and hollow section joints. Provisions for welded connections are given in chapter 4 of SFS-EN 1993-1-8.

Type of weld that is used in connecting the support beams and end flanges is a simple fillet weld. Figure 12 shows an example of a fillet weld. The letter “a” in the figure stands for throat thickness which is key property in designing welded connections. According to SFS-EN 1993-1-8, effective throat thickness is the height of the largest triangle that is inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of the triangle. Minimum effective throat thickness is 3 mm. (SFS-EN 1993-1-8, p. 45)

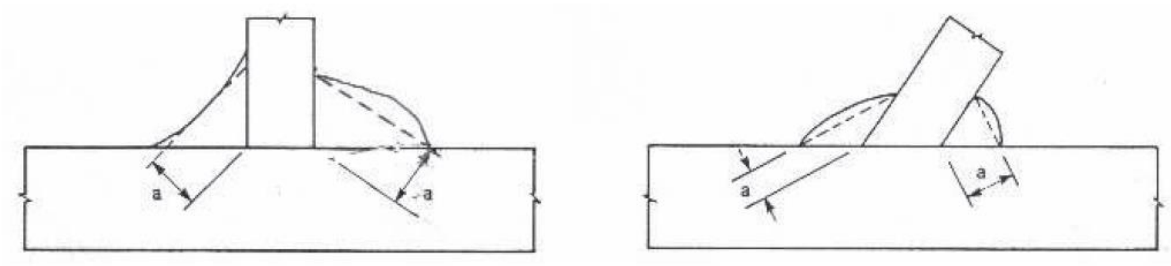


Figure 12. Effective throat thickness (a) in fillet weld (SFS-EN 1993-1-8, p. 45).

Two methods are explained in the Eurocode standard for defining the design resistance of the fillet weld. These methods are called the directional method and simplified method. In both methods the goal is to find a suitable throat thickness when the length of the weld is known. In directional method the forces acting on unit length of the weld are divided into stress components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its throat. In directional method the weld is sufficient when result of the stress components is below a certain criterion. In the simplified method the weld may be assumed to be adequate if at every point along its length, the resultant of all forces acting per unit length satisfy the following criteria as shown in equation (SFS-EN 1993-1-8, p. 44):

$$F_{wEd} \leq F_{wRd} \tag{49}$$

where:

F_{wEd} is the design value of the weld force per unit length.

F_{wRd} is the design weld resistance per unit length.

The design weld resistance is calculated from following equation (SFS-EN 1993-1-8, p. 44):

$$F_{w,Rd} = f_{cw.d} a \tag{50}$$

where:

$f_{cw.d}$ is the design shear strength per unit length of the weld

a is the throat thickness of the weld.

The design shear strength per unit length is calculated from the following equation:

$$f_{cw.d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} \qquad (51)$$

where:

β_w is the fillet weld correction factor based on relation of ultimate strength of base material and filler material.

For sufficient resistance an assumption is made that the applied forces act entirely as shear load. Combining the equations and dividing the load to unit length following equation can be derived to find out the required throat thickness.

$$a = \frac{F_{w,Ed} \beta_w \gamma_{M2} \sqrt{3}}{L_w f_u} \qquad (52)$$

where:

L_w is effective length of the weld

Additional rules for fillet welds from Eurocode 3 are that fillet welds are generally used only when connected parts form a 60 – 120 degree angle. Additional rules are applied when fillet welds are designed outside of this range. Eurocode 3 also states that “Fillet welds finishing at the ends or sides of parts should be returned continuously, full size, around the corner for a distance of at least twice the leg length of the weld, unless access or the configuration of the joint renders this impracticable.” In this case effective length of the weld is reduced by subtracting twice the effective throat thickness from overall length of the weld.

3.4 Dimensioning of bolted connections

The welded beam assemblies comprising of beam and flange are attached to connection nozzles by bolts. Provisions for design of bolted connections is given in Chapter 3 of Eurocode 3 SFS-EN 1993-1-8. Designers work is to select a suitable class of bolts, find a suitable number of bolts and assign their location to make a joint that withstands the designed conditions. Analysis of the bolts is done analytically as well as verified using FEA.

Eurocode 3 lists 7 classes of bolts based on their yield strength and ultimate strength as shown in table 5. Yield strength of the bolt is denoted with f_{yb} and ultimate strength is f_{ub}. Additionally, Eurocode 3 divide types of bolted connections to five categories as based on requirements for the joints as shown in table 6.

Table 5. Nominal values of the yield strength and the ultimate tensile strength for bolts (SFS-EN 1993-1-8, p. 20).

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Table 6. Categories of bolted connections (SFS-EN 1993-1-8, p. 22).

Category	Criteria	Remarks
Shear connections		
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistance at serviceability	$F_{v,Ed,ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ $F_{v,Ed} \leq N_{net,Rd}$	Preloaded 8.8 or 10.9 bolts should be used.
Tension connections		
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used.

In this master's thesis the bolts are dimensioned for shear and tension. Following criteria must be met for the bolts to be suitable as shown in following equations:

$$F_{v,Ed} \leq F_{v,Rd} \tag{53}$$

$$F_{t,Ed} \leq F_{t,Rd} \tag{54}$$

where:

$F_{vb,Ed}$ is the design shear force per bolt

$F_{vb,Rd}$ is the design shear resistance per bolt

$F_{tb,Ed}$ is the design tensile force per bolt

$F_{tb,Rd}$ is the design tension resistance per bolt

Based on SFS-EN 1993-1-8 shear and tension resistance are calculated by using following equations (SFS-EN 1993-1-8, p. 27):

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A_s}{\gamma_{M2}} \tag{55}$$

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} \tag{56}$$

where:

α_v = 0,6 for bolt classes 4.6, 5.6, 8.8

A_s is the tensile stress area of the bolt.

k_2 = 0,63 for countersunk bolts, otherwise k_2 = 0,9

3.5 FE-analysis

FE-analysis is a mathematical modelling method that can be used in variety of engineering problems like structural analysis, heat transfer and fluid dynamics. In this master's thesis structural FE-analysis is introduced in more detail by looking at typical workflow in FE-analysis software Femap.

The name finite element method comes from the idea that the problem is divided into finite number of elements that are easier to analyse than the original continuous problem. In structural FE-analysis the continuous behaviour of the structure is divided into finite elements through process of discretization. Due to discretization, even the best result of the analysis approximates the real continuous behaviour of the structure. The approximation can be made more accurate by dividing the problem into smaller parts, assuming the initial approximation approaches the real continuum solution. (Taylor, Zhu & Zienkiewiz 2013, p. 1-2) The elements have associated nodes that are located at boundaries or within the elements. Each node has their own equations representing the behaviour of that node. Nodes allow analysing the behaviour of the element and its relation to other elements. (Pavlou, D. G. (2015), p. 4)

In this master's thesis Siemens Femap software with NX Nastran solver is used for FE-analysis. Steps of modelling in Femap are the following:

- Model geometry/Import geometry from a CAD software
- Material model
- Element type
- Mesh size
- Meshing
- Setting applied loads and boundary conditions
- Model check and analysis set
- Analysis and post-processing of analysis results

Each step has a lot of options within the Femap software which will be covered in following sections in context of this work.

3.5.1 Geometry and mesh

Importing a ready or partly completed CAD file is usually the first step of making a FE-analysis in Femap. Femap allows importing geometries from CAD software in many industry-standard file types. For element properties and meshing Femap has a lot of options to choose from to form the most suitable model for analysis. Material model can be tuned down to fine details by choosing appropriate material type, stiffness properties, mass density and for example thermal properties. Material properties should be filled sufficiently depending on what type of analysis is run on the model. For static structural analysis, necessary properties are Young's modulus, Poisson's ratio, and mass density. Three main categories of element types in Femap are: line elements (1-dimensional), plane element (2-dimensional) and volume elements (3-dimensional). Some element types are that do not fall into any of the categories form a collection of other element types. Element types that were used in the modelling part of this work are following: beam element (1-dimensional), plate element (2-dimensional), rigid element and gap element (other).

In all seven cases under investigation, the geometries were first modelled in Solidworks CAD software and imported to Femap in Parasolid file type which is the native geometry file type for Femap. In cases equipped with crossing beams section of the equipment was modelled as shown in figure 13. Walls of the section, nozzles, beam flanges, and flooring plates were converted to mid-surfaces and meshed with 4-node plate elements with nominal size of 50 mm x 50 mm. Plate thickness was assigned to each part according to specification. Supporting beams were modelled as curves between the nozzles and meshed with 2-node beam elements with nominal size of 100 mm between nodes. Bolts that connect the beam end flanges to the nozzles were also modelled as beams.

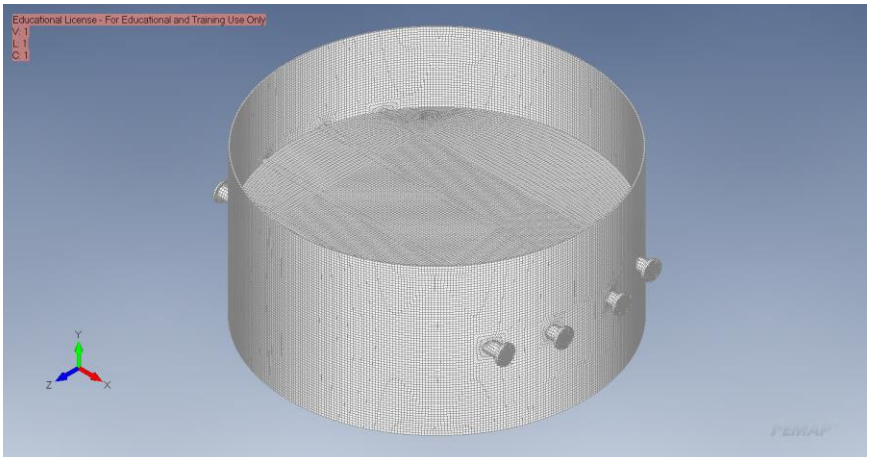


Figure 13. Mesh of crossing beams section.

Cases with radial beams were analysed separately as a plate model. The plate meshing was done like the previous example but with finer mesh size of 10 x 10 mm. Examples of radial beam plate mesh is provided in figure 14 and figure 15.

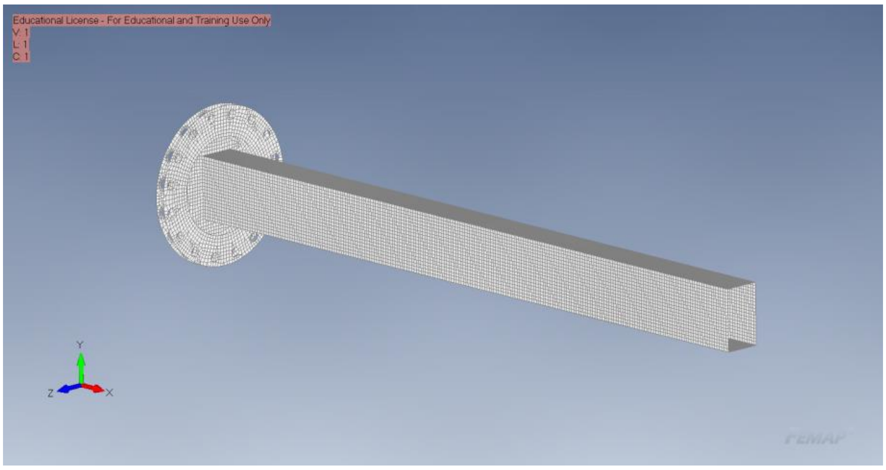


Figure 14. Mesh of rectangular hollow section radial beam.

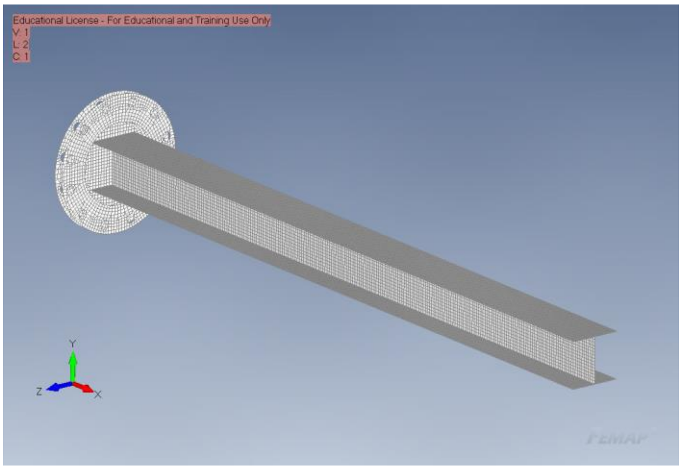


Figure 15. Mesh of HEB160 radial beam.

3.5.2 Constraints, loads, and setup of analysis.

In cases with radial beams setting up constraints started by applying rigid elements to transfer vertical translations between the flooring plate and the supporting beams. Rigid elements with all degrees of freedom constrained were applied to connect the supporting beams to the beam end flanges. Each bolt connection consisted of a beam element and two rigid elements that connect the bolt to the flange and the nozzle. Setup of rigid elements is shown in figure 16.

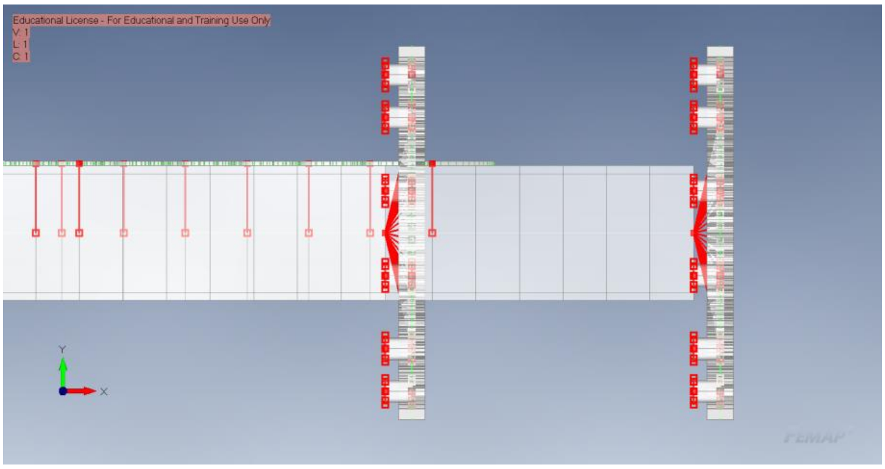


Figure 16. Rigid elements.

The bolts in radial beams were modelled similarly to the crossing beams example shown in figure 16 with the exception that the nozzle flange side of the bolt was instead constrained with a fixed constraint in space. Details of bolts modelled in cases with radial beams is provided in figure 17.

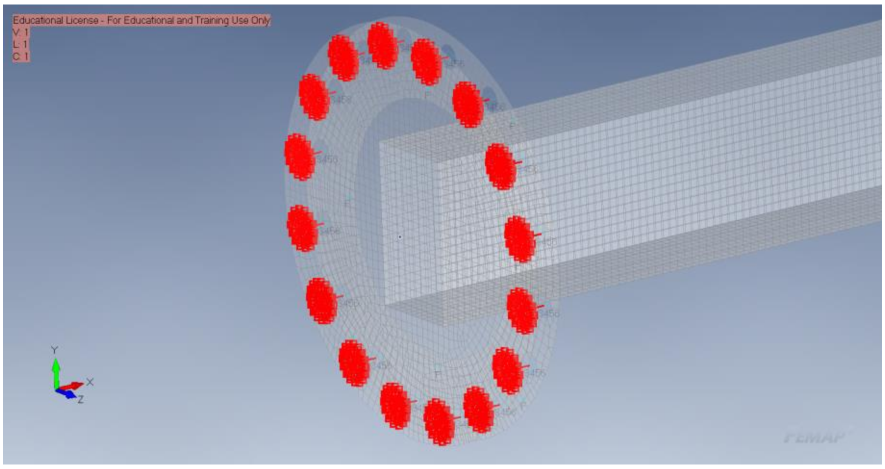


Figure 17. Detailed view of bolt model (radial beam).

Contacts were applied between the nozzles and the flanges with default frictionless contact property. Setup of contacts for crossing beams is depicted on figure 18 and for radial beams in figure 19.

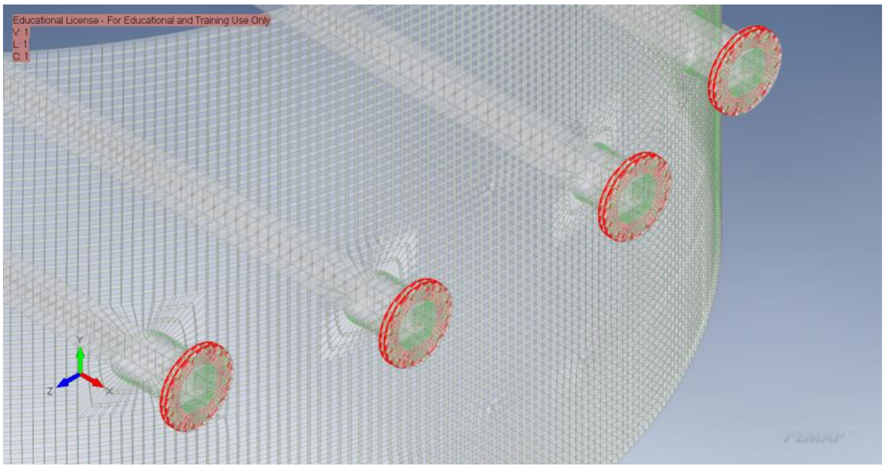


Figure 18. Beam end flange to nozzle flange contact in case of crossing supports.

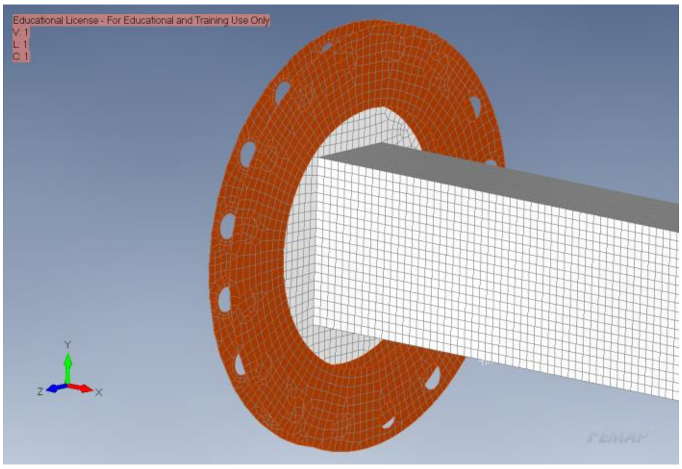


Figure 19. Beam end flange to nozzle flange contact in case of radial supports.

Gap elements were applied inside the nozzle as shown in figure 20 since the beams are supported at the nozzle walls by fitting a supporting block between nozzle and the beam. In cases with radial beams a simplified method was used where gaps were applied between the beam and set of nodes defined as the contact point as shown in figure 21. In cases with crossing supports the free ends of the beams were connected to the nozzles using only gap elements which allows the beams to slide at the contact point while transferring vertical translations. In linear static analysis gap elements work like contacts while nonlinear analysis allows setting up additional parameters for gaps such as initial gap where contact doesn't occur immediately. In this case the accuracy provided by linear static analysis and gaps as contact is sufficient as deformation limits limit the stresses well within the elastic range of the material.

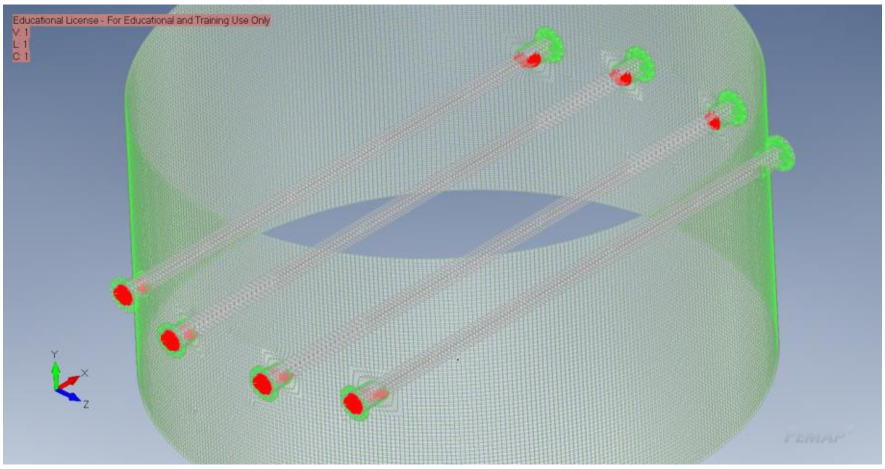


Figure 20. Gap elements in crossing beams support model.

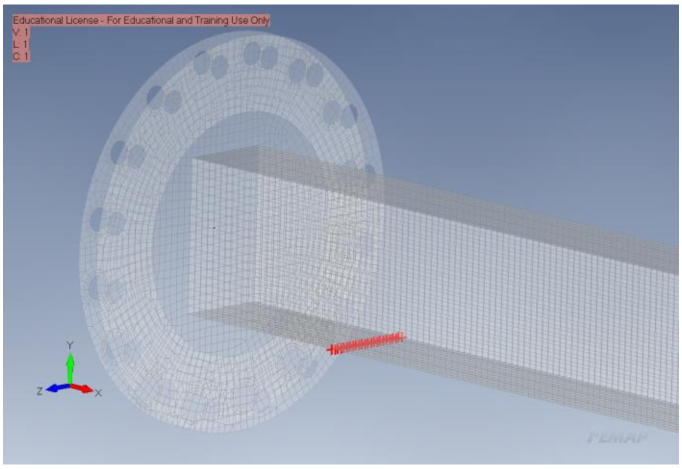


Figure 21. Gap elements in radial beam model.

In the case of crossing beam constraints were applied to the bottom edge of the observed section as curve constraints and on flooring plate on top of the supporting beams as surface constraint. All degrees of freedom were constrained at the bottom edge. Translation in xz-plane was constrained on the for the flooring plates. Constraints of crossing beams cases is presented in figure 22. Radial beams had fixed surface constraint on the nozzle flange and fixed nodal constraints at bolt ends and simplified nozzle wall support as shown in figure 23.

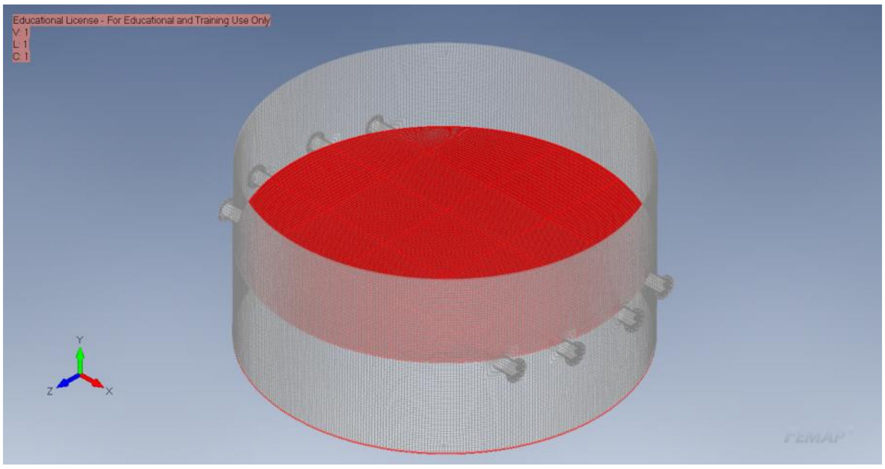


Figure 22. Constraint set for crossing beams section.

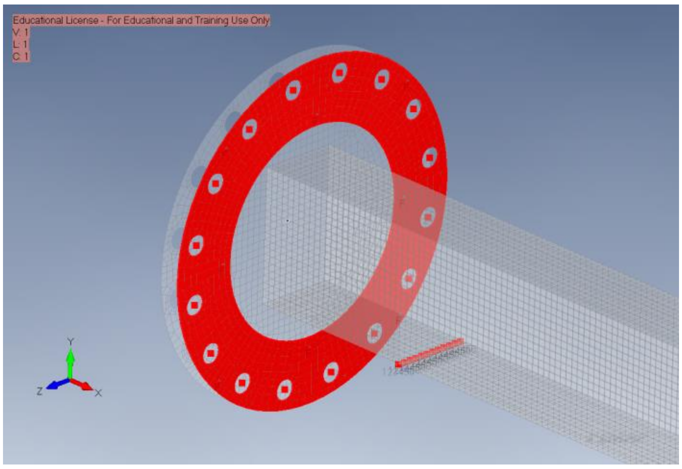


Figure 23. Constraint set for radial beam model.

Finally, before making the analysis, external loads were applied to the models. For cases with crossing supports loads were applied as nodal loads on the flooring plate on assumed locations of the scaffolding legs as shown in figure 28. Loads for radial beams were applied as loads on curve as shown in figure 29 at assumed positions of the scaffolding legs. Load on curve was chosen to distribute to load more evenly to the finer plate mesh and prevent excessive stress pikes. Body loads were applied to all models by setting a translational acceleration of 9810 mm/s^2 along negative y-axis representing gravity. For ULS values of body loads, value of gravity was multiplied by partial safety factor 1,35.

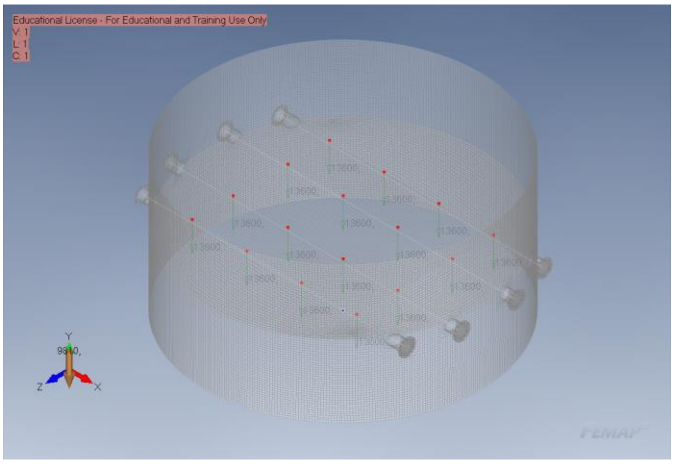


Figure 24. Loads on crossing beams model.

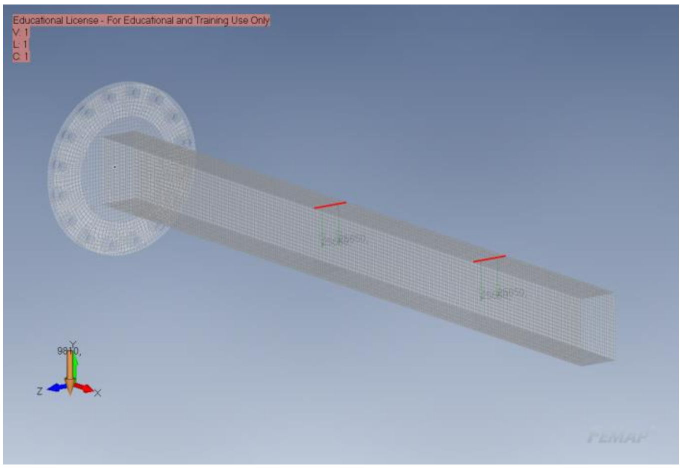


Figure 25. Loads on radial beam model.

Linear static analysis with program default settings was used in analysing stresses and deflection of the beams. Additionally, linear buckling analysis was run on case 3 to verify sufficient lateral torsional buckling resistance. Stresses and buckling were analysed by applying loads based on the ULS and deflections by applying loads based on SLS.

3.6 Serialization of working platform supports.

Goal of this thesis was to serialize working platform supports to suite to the requirements of range of different sized equipment. While currently the type of equipment and its various parameters is serialized in different size ranges, the working platforms are not. Size ranges are one primary way of making product families. Size ranges rationalizes the design and manufacturing work, and design can be done once for host of different products with minimal changes. In simplest form of serialization to size ranges may include only magnifying geometrical dimensions to form a product that is larger or smaller in dimensions. Manufacturing can be optimized when same methods that are already tried and tested are used for other products in the size ranges. Combined, simplifying design and manufacturing using size ranges can produce higher quality products, decrease delivery time, make acquisition of replacement parts and fittings easy and overall make the product more competitive. (Pahl & Beitz, 2007, s.465.)

According to (Yin, L., He, M.-L., Xie, W.-B., Yuan, F. 2018) standardization of products consist of three important metrics. These metrics represent three different design methodologies which are universalization, serialization, and modularization. Universalization aims for interchangeability of components in various contexts, serialization aims to optimize product architecture and modularization aims to split products into self-contained subcomponents. (Yin L et al. 2018, p. 359 – 360.) Practical examples of each standardization metric in the context of this master’s thesis could include following:

- Standard flanges – universal and applicable to various products
- Serialized process equipment – series of process equipment form a rationalized series of optimized sizes of vessels.

- Support beam modules – support beams under investigation form self-contained modules which can be used in various products of the product series. The support beam nozzles in the equipment could even be used as an interface for other functions allowing further modularization. Currently the nozzles are used only for attaching support beams.

Previously mentioned research by Yin L. et al. aims to find quantitative method for investigating level of standardization. In this master's thesis the assessment of standardization is purely qualitative as not enough data is available for quantitative assessment at this point. Some form of cost calculation could be used to see if there are benefits in increasing the level of standardization.

3.6.1 Designing steel structures for product series

The general approach to steel structure design remains the same when designing steel structures for product series since the structure needs to withstand the design conditions exactly like their design-to-order counterpart. What is different is that the same structure needs to fulfil wider range of requirements due to wider range of different conditions that needs to be considered to fulfil all requirements of the product series. So called "worst case" will be the basis on which the structure will be design for. Design of steel structures in this way may lead to suboptimal structures in cases with less load. This however, does not matter in non-weight critical cases and savings provided by sharing same parts or entire structures with different products may outweigh the disadvantages of suboptimal structure. Usually in cases where requirements differ largely for the parts of the structure, some division needs to be made to size ranges to optimize the structure. In this master's thesis the support structures were divided into 7 groups.

4 Results

This chapter showcases the most important results of the master's thesis. Majority of unsuitable beam profiles could be ruled out by looking at the deflection limit that was set according to ISO 14122-2. To minimize variation of beam profiles, HEB160 was selected for all crossing beams and RHS 200x120x12,5 for all radial beams. Analytical calculations and FEM results showcased in this chapter are based on these selected profiles. Calculations for welded and bolted joints are also presented later in this chapter.

4.1 Analytical evaluation

This section showcases important findings of analytical evaluation. Excel calculation tool was developed for easy evaluation of multiple cases with possibility to choose different beam cross-section and material properties. The calculation tool had checks for combined elastic stress, deflection, and lateral torsional buckling. With the help of the excel tool unsuitable profiles were ruled out of further investigation. List of analytical results is found in Appendix 1 where unsuitable design values are highlighted in red and yellow.

From the load component table presented in chapter 2.3 following table of imposed loads and resulting bending moments shown in table 7 were calculated. After calculating design values for forces and bending moments, verification of resistance could be done by using equations (37-48). Table 8 shows calculated resistances and utilization factors of the beams.

Table 7. Bending moments.

Case	Permanent loads per beam [N/m]	Point force [kN]	Number of point forces	Bending moment (at supports) [kNm]	Bending moment (center) [kNm]
1.	1087	10	2	10	8
2.	855	8	4	17	13
3.	1091	19	4	59	48
4.	1079	18	2	31	
5.	1101	20	2	35	
6.	1116	24	2	42	
7.	1126	26	2	45	

Table 8. Cross section resistance.

Case	Combined stress Resistance [MPa]	Combined Stress [Mpa]	Utilization Factor Combined Stress	Lateral torsional buckling resistance [kNm]	Bending moment (center) [kNm]	Utilization Factor Lateral torsional buckling
1.	355	25	0,11	87	8	0,09
2.	355	42	0,12	79	13	0,17
3.	355	149	0,42	63	48	0,74
4.	355	87	0,25			
5.	355	98	0,28			
6.	355	118	0,33			
7.	355	126	0,36			

Cases 1-3 had crossing beams with increasing span between beam support flanges. Due to deflection limits taller profiles such as IPE profile would generally be most suitable.

However, due to lateral torsional buckling limited the use of taller profiles in case 3. Due to long unsupported span of the beam in case 3, lateral torsional buckling resistance decreased to a point that it turned out to be the limiting factor in profile and material selection. Effect of lateral torsional buckling could be affected by supporting the beams diagonally. This could be done by designing diagonal supports between the beams. However, this investigation was left out of the scope of this work.

4.2 resistance FE-analysis

This section shows results from FE-analysis. Results of FE-analysis consists of total deformations, combined stress of beam elements in cases with crossing beams, and Von Mises stress of plate elements in cases with radial beams. Linear static buckling load factors were also checked for crossing beams cases.

4.2.1 Deformations

Deflection deformations of beams was limited to 1/200th of the total span of the beam. Results of FEA for deflections are presented in table 9.

Table 9. FEA Deflections.

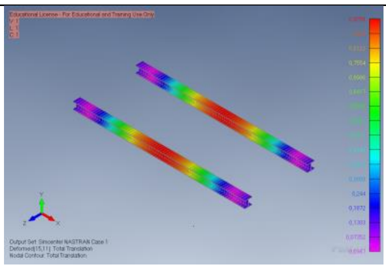
Case	Visual FEA results of deformations	Maximum allowed vertical deflection [mm]	Maximum deflection from FEA [mm]
1		17,4	0,9

Table 9 continues. FEA Deflections.

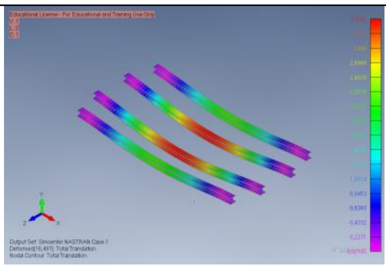
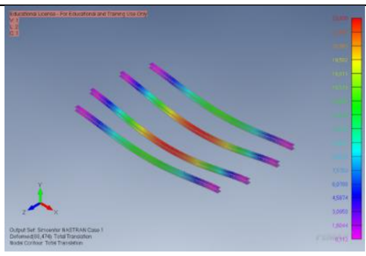
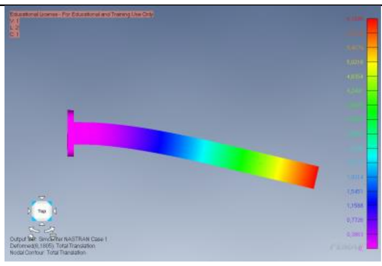
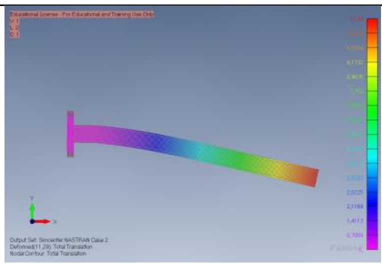
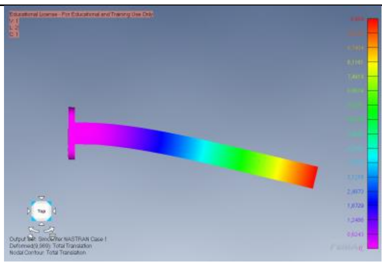
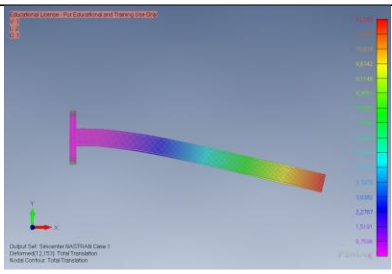
Case	Visual FEA results of deformations	Maximum allowed vertical deflection [mm]	Maximum deflection from FEA [mm]
2		23,0	3,3
3		35,0	24,0
6 (RHS)		11,3	6,2
6 (HEB)		11,3	11,3
7 (RHS)		11,3	10

Table 9 continues. FEA Deflections.

Case	Visual FEA results of deformations	Maximum allowed vertical deflection [mm]	Maximum deflection from FEA [mm]
7 (HEB)		11,3	12,2

4.2.2 Stresses

Results of stress analysis in FEA is shown in table 10. Colours of the visual results were set so that in cases with crossing beams purple represents lowest displayed stresses and red represents highest displayed stress in the beam elements. In cases with radial beams the scale of colours was same but instead the red colour represents the yield strength of the material.

Table 10. FEA Stresses.

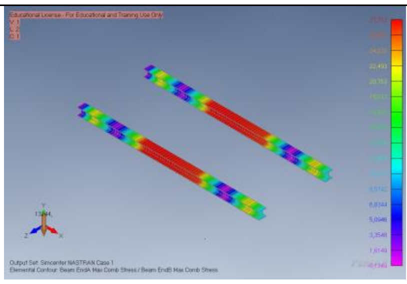
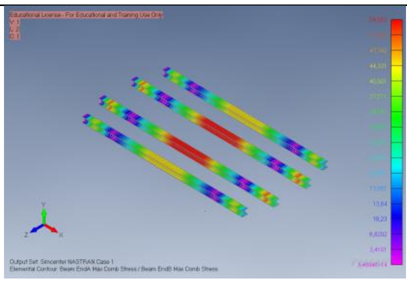
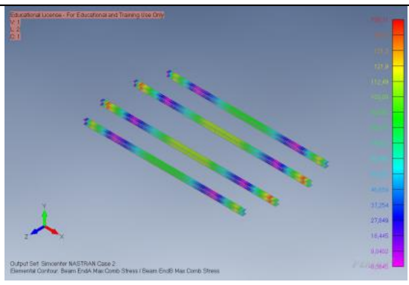
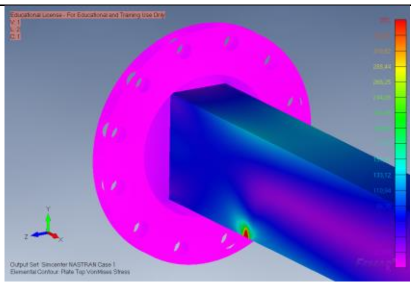
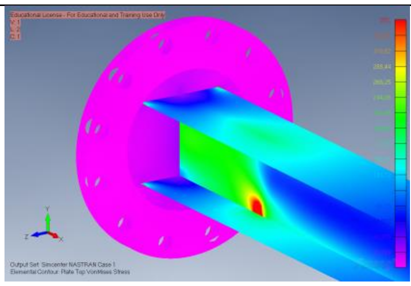
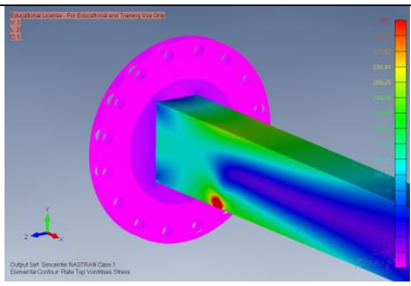
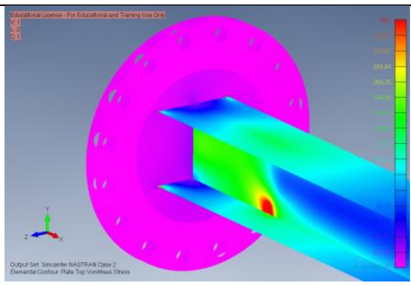
Case	Visual FEA results of combined stress	Maximum allowed stress [N/mm^2]	Maximum combined stress from FEA [N/mm^2]
1		355	28
2		355	55

Table 10 continues. FEA Stresses.

Case	Visual FEA results of combined stress	Maximum allowed stress [N/mm^2]	Maximum combined stress from FEA [N/mm^2]
3		355	150
6 (RHS)		355	155
6 (HEB)		355	244
7 (RHS)		355	244
7 (HEB)		355	244

In cases with radial beams local stress peaks seen as bright red colour come from simplified modelling of the lower nozzle wall support and thus don't represent a realistic stress distribution.

Additionally, the lateral torsional buckling load factor (BLF) was checked for case 3 where lateral torsional buckling was considered to have a possible effect on the ultimate resistance of the structure. Following figure shows the assumed buckling mode representing lateral displacement during lateral torsional buckling. BLF obtained from FEM was 1,614. To find out elastic buckling load magnitude, the loads are multiplied by the BLF. With known elastic buckling loads the critical elastic buckling moment can be calculated and same analytical procedure presented in equations 43 – 47 can be used to find out actual buckling resistance. Resulting buckling resistance calculated from BLF was 56,90 kNm and resulting utilization factor was 0,81.

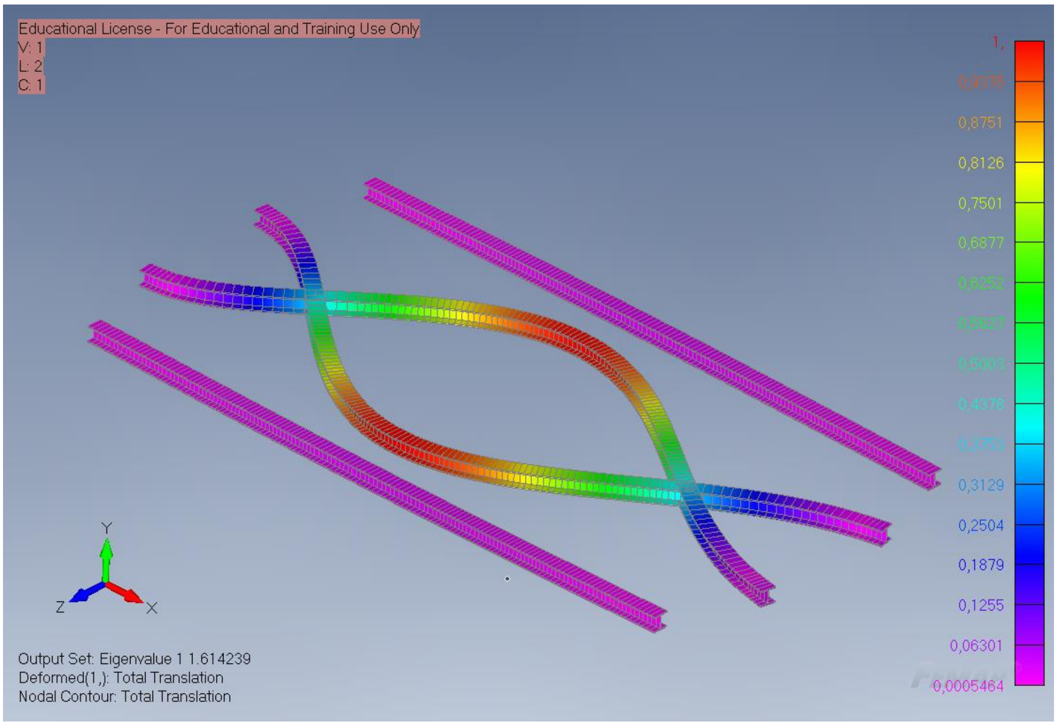


Figure 26. Assumed buckling mode in Femap.

4.2.3 Throat thickness of beam end welds

By using equation (51), minimum throat thickness calculated for the most critical welds as follows:

Crossing beams:

$$a=\frac{F_{Ed}\beta_w\gamma_{M2}\sqrt{3}}{L_w f_y}=\frac{75000*0,9*1,25*\sqrt{3}}{150*355}\approx 2,7\text{ mm}$$

Radial Beams:

$$a=\frac{F_{Ed}\beta_w\gamma_{M2}\sqrt{3}}{L_w f_y}=\frac{82000*0,9*1,25*\sqrt{3}}{110*355}\approx 4,1\text{ mm}$$

For additional safety the ultimate tensile strength was replaced with yield strength. The resulting throat thickness was rounded to a conservative value of 5 mm resulting in the following utilization factors:

Crossing beams:

$$U=\frac{F_{w,Ed}}{\frac{f_y/\sqrt{3}}{\beta_w\gamma_{M2}}aL_w}=\frac{75000}{\frac{355/\sqrt{3}}{0,9*1,25}*5*150}\approx 0,55$$

Radial Beams:

$$U=\frac{F_{w,Ed}}{\frac{f_y/\sqrt{3}}{\beta_w\gamma_{M2}}aL_w}=\frac{82000}{\frac{355/\sqrt{3}}{0,9*1,25}*5*110}\approx 0,82$$

4.2.4 Resistance of bolted connections

By using equations (54) and (55), resistance of bolted connections was calculated as follows:

$$F_{v,Rd}=\frac{\alpha_v f_{ub} A}{\gamma_{M2}}=\frac{0{,}6*800*353}{1{,}25}=135552\ \mathrm{N}$$

$$F_{t,Rd}=\frac{k_2 f_{ub} A}{\gamma_{M2}}=\frac{0{,}9*800*353}{1{,}25}=203328\ \mathrm{N}$$

Visual FEA results as well as values for bolt shear force and tensile force is shown in tables 11 and 12.

Table 11. FEA results of bolted joints.

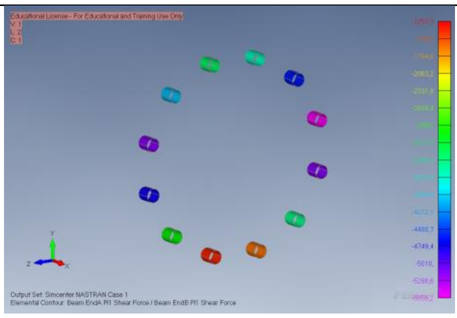
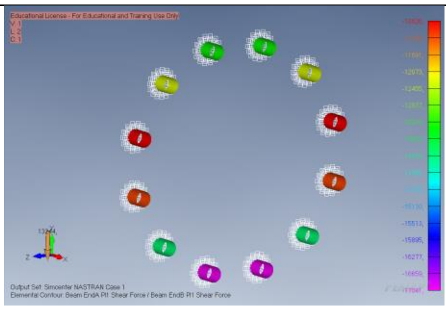
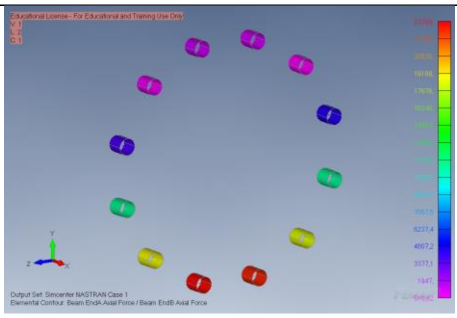
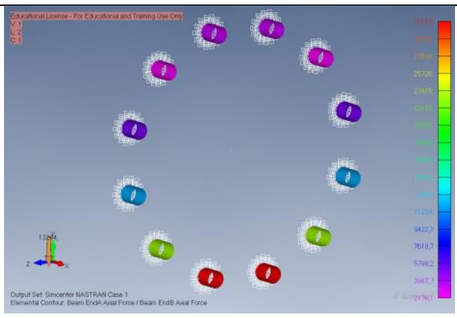
Highest shear force [kN]		
crossing beams:		17,1
radial beams:		5,6

Table 12. FEA results of tensile force in bolted joints.

Tensile force		
Crossing beams:		23,4
Radial beams:		31,2

Utilization ratio for both crossing beams case and radial beams can be calculated by comparing design resistances to highest shear and tensile force from FEA.

Bolt shear force utilization factors:

Crossing beams:

$$U = \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{17,1}{135,5} \approx 0,13$$

Radial Beams:

$$U = \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{5,6}{135,5} < 0,1$$

Bolt tensile force utilization factors:

Crossing beams: $$U = \frac{F_{t,Ed}}{F_{t,Rd}} = \frac{23,4}{203,3} \approx 0,12$$

Radial Beams: $$U = \frac{F_{t,Ed}}{F_{t,Rd}} = \frac{31,2}{203,3} \approx 0,16$$

5 Conclusions

This master's thesis is aimed to answer following research questions:

1. What are the expected load cases for the structure?
2. What is the minimum number of different support beam variations?
3. What needs to be considered when designing serialized support beams for working platforms?
    a. How does serialization affect steel structure design?
4. What is the expected limit in dimensioning?
    a. yielding?
    b. loss of stability?
    c. excessive deformation?
5. What profile shape and material are optimal for the support beams
    a. global availability?
    b. manufacturability?

This master's thesis was able to answer these research questions in sufficient detail. Results obtained by analytical calculation and FEM indicate that the structure fulfils the necessary criteria that was defined according to standards and requirements of the supplier. In the beginning the most important step was to define loading cases by identifying boundary conditions from the specification of the equipment. Seven loading cases were defined on which the support structures were design for. Out of the seven cases, five variations with different supporting types and beam lengths were analysed. Eurocode standards SFS-EN 1990 and SFS-EN 1993 as well as safety of machinery standard SFS-EN ISO 14122-2 were considered in the defining the loading cases and dimensioning the beams.

Results show that the expected limit in dimensioning the support beams is the serviceability deformation limit set by SFS-EN ISO 14122-2 standard for most of the cases. Exception was case 3 where long span between beam end supports led to loss of stability of the structure in form of lateral torsional buckling at ultimate limit state loads which set a limit for the material and profile selection.

Materials and profiles were selected by identifying the most critical loading cases for the two different supporting types that were the crossing type and radial type. Thus, narrowing the material and profile selection to just two different profile cross-sections and one material. Viewpoints of global availability and manufacturability were also considered. Standard profiles were selected for investigation. Final profile shapes were HEA 160 for crossing beams and RHS 200x120x12,5 for radial beams. First point for later improvement would be finding an alternative to the RHS beams used in radial beams to improve global availability. It was assumed that HE-type profiles would have had better global availability. With the loading cases defined in this master's thesis a conservative approach was taken leading to selection of RHS profile for radial beams. In terms of manufacturability no special requirements were considered as material was selected to be common structural steel which was assumed to be easily weldable for example. Resistance of bolted connections with class 8.8 bolts was found to be sufficient with very low utilization factors making it possible to reduce the number of bolts for optimized design. Similarly, a conservative approach was taken to design of welded connections, but the resulting 5 mm throat thickness is still reasonable.

For further investigation the cross-section design could be optimized by setting necessary requirements such as moment resistance and geometrical limitations and selecting profile this way rather than choosing the profiles from standard selection by solely looking at largest possible cross-sections. Optimizing the profiles this way could potentially provide cost savings compared to procedure used in this investigation. As was also mentioned in section 3.4 the improvements to cost efficiency are entirely qualitative and further investigation should include quantitative assessment.

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Appendix 1. Results of hand calculations

PROFILE SELECTION TABLE					
NEAR ALLOWED LIMIT STATE			OVER ALLOWED LIMIT STATE		
ID	Cross section	Allowed deflection [mm]	Deflection [mm]	Combined stress [MPa]	Lateral Torsional Buckling Capacity
CASE 1	IPE200	17,4	0,7	38,5	0,18
CASE 1	HEA160	17,4	0,8	34	0,13
CASE 1	HEB160	17,4	0,5	24	0,08
CASE 1	RHS(200x120x8)	17,4	0,5	30	-
CASE 1	RHS(200x120x12,5)	17,4	0,4	21	-
CASE 2	IPE200	23	2,6	41	0,24
CASE 2	HEA160	23	3	36	0,15
CASE 2	HEB160	23	2	26	0,1
CASE 2	RHS(200x120x8)	23	2	32	-
CASE 2	RHS(200x120x12,5)	23	1,4	22,4	-
CASE 3	IPE200	35	24	163	1,25
CASE 3	HEA160	35	27,8	144	0,68
CASE 3	HEB160	35	18,7	102	0,4
CASE 3	RHS(200x120x8)	35	18,4	125,5	-
CASE 3	RHS(200x120x12,5)	35	13	88,8	-
CASE 4	IPE200	11,25	10,2	159	-
CASE 4	HEA160	11,25	11,8	140,4	-
CASE 4	HEB160	11,25	8	99,8	-
CASE 4	RHS(200x120x8)	11,25	7,8	122,2	-
CASE 4	RHS(200x120x12,5)	11,25	5,6	86,4	-
CASE 5	IPE200	11,25	11,4	178,3	-
CASE 5	HEA160	11,25	13,3	157,4	-
CASE 5	HEB160	11,25	8,9	111,2	-
CASE 5	RHS(200x120x8)	11,25	8,8	137	-
CASE 5	RHS(200x120x12,5)	11,25	6,2	96,9	-
CASE 6	IPE200	11,25	13,6	212,7	-
CASE 6	HEA160	11,25	15,8	187,8	-
CASE 6	HEB160	11,25	10,6	132,7	-
CASE 6	RHS(200x120x8)	11,25	10,5	163,4	-
CASE 6	RHS(200x120x12,5)	11,25	7,4	115,6	-
CASE 7	IPE200	11,25	14,8	230,8	-
CASE 7	IPE240	11,25	7,4	138,3	-
CASE 7	HEA160	11,25	17,2	203,8	-
CASE 7	HEA180	11,25	11,4	152,8	-
CASE 7	HEB160	11,25	11,5	144	-
CASE 7	HEB180	11,25	7,5	105,4	-
CASE 7	RHS(200x120x8)	11,25	11,4	177,3	-
CASE 7	RHS(200x120x12,5)	11,25	8	125,4	-