Long span high strength steel trusses

Authors: Vaidas Alechnavičius

József Bálint

Supervisor: Panagiotis Manoleas

University: Luleå University of Technology

Luleå University of Technology

2014/01/17

LULEÅ UNIVERSITY OF TECHNOLOGY

MASTER THESIS

Long span high strength steel trusses

Vaidas Alechnavičius

József Bálint

Luleå University of Technology

Department of Civil, Environmental and Natural Resources Engineering

Division of Structural and Construction Engineering - Steel Structures

SE - 971 87 LULEÅ

www.ltu.se/sbn

PREFACE

This thesis represents our work for the final semester of the Erasmus Mundus SUSCOS Master Programme of which the first semester we spent in University of Coimbra, Portugal, whilst the second semester was held in Czech Technical University of Prague. Therefore, we would like to thank all the staff members with whom we worked with in these two Universities.

Our research was performed at the Department of Civil, Environmental and Natural Resources Engineering of Luleå University of Technology at the Division of Structural and Construction Engineering in the Steel Structures Research Group.

We would like to thank our coordinator Milan Veljkovic for his guidance throughout our stay in Luleå and present our gratitude to our supervisor Panagiotis Manoleas for all his guidance and assistance on our thesis.

Moreover, we want to acknowledge Ulf Stenman and the rest of the COMPLAB staff members for their support during the experiments we carried out in the laboratory.

We thank our families and friends for all their support during the study period.

On behalf of József Bálint, special thanks are addressed to Zsolt Nagy, from Technical University of Cluj-Napoca for all his assistance and help.

ABSTRACT

Long span truss elements are required in modern applications like stadiums, show-rooms, exhibition halls, airports. Problems may be arise due to the dimensions and weight of the truss itself. Instability, buckling and finally collapse of the structure may be caused, if not installed and braced properly.

A convenient solution to this may be the use of HSS, since this will lead to a reduction in weight of the steel structure and implicitly in the foundation area (not to be analysed in this work). Also, smaller amounts of CO_2 emissions would be emitted (due to the reduced material and less transportation needs), which makes the study of the HSS usage an appealing one.

For an even greater weight reduction and a more sustainable solution, a truss constructed of built-up polygonal cross-sections is analysed and proposed, instead of the classic circular hollow sections approach for the chord and diagonal members. U shaped profile will be considered for the lower chord, which is subjected to tensional forces, a shape that optimally uses the necessary area for undertaking the tensional stresses.

Computational analysis will be performed in Abaqus Finite Element software. The main concern is the behaviour of the connections in the tension chord under various load levels and the buckling analysis of polygonal members.

For understanding deeper the advantages of using HSS, 3 steel grades (S355, S500 and S650) are compared from an economical and environmental point of view, both for the CHS and the built-up polygonal section trusses.

NOTATIONS

Latin capital letters

A	area of a cross section
A_{eff}	effective cross sectional area
A_g	gross cross sectional area
E	modulus of elasticity
F_y	tensile strength of steel
F_{ya}	average yield stress in cold-formed section
F_{yf}	yield stress at flat part of section
F_u	tensile strength of steel
$F_{w.Rd}$	design value of the weld force per unit length
G	shear modulus
I	second moment of inertia
L	full length of member
L_{cr}	critical length of member
$N_{b.Rk}$	characteristic buckling resistance
$N_{c.Rd}$	design resistance to normal forces of the cross-section for uniform compression
$N_{c.Rk}$	characteristic value of resistance to compression
N_{cr}	elastic critical force for the relevant buckling mode based on the gross cross sectional properties
$N_{t.Rd}$	design values of the resistance to tension forces

Latin small letters

a	weld throat thickness
b	flat width of plate
b_p	notional width of plate
b_{eff}	effective width of plate
f_y	yield strength
f_{ya}	average yield strength
f_{yb}	basic yield strength
f_u	ultimate tensile strength
$f_{vw.d}$	design shear strength of the weld
i	radius of gyration
k	effective length factor
k_σ	plate buckling coefficient
t	element thickness

Greekcapital letters

ΔP	hardening constant
Θ	degree of bent corners

Greeksmall letters

α	imperfection factor
β_w	correlation factor for fillet welds
γ_{M0}	partial factor for resistance of cross-sections
γ_{M2}	partial factor for resistance of cross-sections in tension to fracture
ε_{true}	true strain

ε_{xx}	strain in x direction
ε_{yy}	strain in y direction
$\bar{\lambda}$	relative slenderness
$\bar{\lambda}_p$	plate slenderness
ν	Poisson's ratio in elastic stage
ρ	reduction factor to determine the effective width of the plate
σ_{true}	true normal stress
σ_{11}	maximum principal stress
σ_{22}	minimum principal stress
σ_{Y}	yield stress of material
σ_{xx}	normal stress x direction
σ_{yy}	normal stress y direction
σ_{zz}	normal stress z direction
σ_{cr}	critical buckling stress
$\sigma_{\nu m}$	von Mises stress
τ_{xy}	tangent (shear) stress xy direction
Φ	value to determine the reduction factor χ
χ	reduction factor for buckling resistance
ψ	ratio of moments in segment

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

1 INTRODUCTION

Long span trusses are eligible solutions for a variety of engineering structures which require wide open spaces: stadiums, show-rooms, exhibition halls, airports, museums.
The more common they are used, the more it is of interest to reduce the cost of these type of structures. One way to do this is by using HSS elements, instead of the regular steel grades. Any grade higher than S355 will be considered HSS. For further reducing the weight of the structure and achieve better results, a truss constructed by built-up polygonal sections is proposed.

1.1 Background

Modern steel mills are able to produce high [HSS] strength and ultrahigh strength steel [UHSS] of tensile strength up to 1400MPa, thanks to continuous annealing[1]. Use of higher steel grades has been well established and documented in several production applications, especially the automotive industry [2]. In construction though, its use is not extended mainly due to serviceability, ductility and cost issues. Documentation around cost is not absent [3] but scarce none the less.

An effort is put during recent years to overcome the previous obstacles and increase the interest of engineers. The possibility to use such types of steel, provides an excellent solution for long span truss applications. The biggest advantages are the total reduction of the final structure weight and cost, which has been speculated for long [4] as well as the reduction of carbon footprint[5].
Apart from that, other applications have been considered in construction like bridges [6][7][8], hybrid girders [9] and other special applications like cranes which are proved to be economical [10].

Sport arenas, show-rooms, airports, stadiums demand large, column-less open areas in order to maximize the available free space. The need for such civil engineering structures requires a special attention to be given to the elements sustaining the roof.

Two popular solutions are used to undertake loads from the roof. Either beam elements are used, or a truss is designed. In the case of long spans (assumed to be anything higher or equal to 12 m), the governing limit state would most probably be the serviceability one, due to the large deformations that are to be dealt with in the case of big openings. Simple beams are not stiff enough to meet the deflection requirements for a long span, except large sections are designed. This is an uneconomical approach, the best solution being the design of a large truss, with slender elements which provides both the required strength and stiffness. Therefore, a big amount of steel which is not necessary to undertake the ultimate limit states load can be saved using HSS.

Even though it is still not widely used, HSS has already been implemented in structures as the recently built Friends Arena in Stockholm (Fig.1.1), indicating interesting engineering and business opportunities [11]. By using HSS instead of the regular S355, a reduction of 17% in the total weight of the main body of the roof was achieved[12], [13].

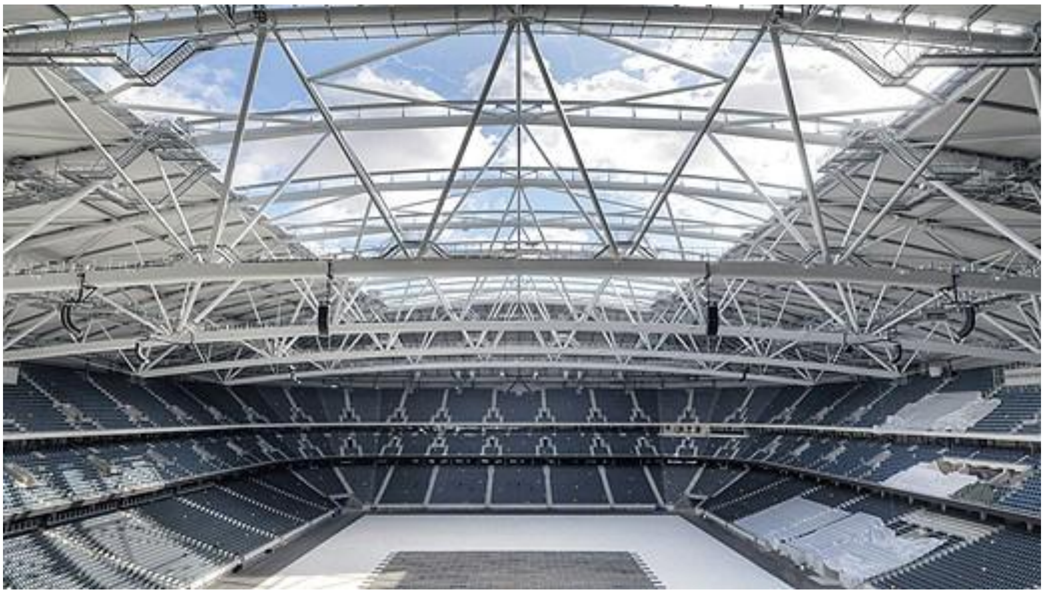


Figure 1.1 *Friends Arena, Stockholm*

In order to design competitive and architecturally appealing HSS trusses by maximizing their benefit, an innovative solution consisting of semi-closed polygonal truss members and a U-shaped profile tension chord will be investigated. Through the work presented herein a CHS truss is compared to a built-up section structure and the advantages of the latter one will be highlighted.

Studies on the behaviour of the CHS have been performed while working on different aspects of the hollow sections. Interest has been shown on the out-of-plane buckling length for truss girders with K-joints [14], or buckling of thin-walled long steel cylinders[15]. Attention has been paid to joints design in [16] where over 100 tests were performed in order to establish a formulation of the multi-planar joints strength, or in [17] where multi-planar K joints made of RHS have been investigated.

Literature on the built-up type of cross-sections is short, therefore a lot of questions and unknowns arise when using them. In this work, numerous questions are addressed regarding the design of the two types of trusses: the buckling analysis of the compression chord, the numerical analysis of the joint between the tension U-channel and the two polygonal diagonals and finally, the evaluation of cost and CO_2 emissions.

For the conventional truss structures made from hot-rolled sections, the design was optimized over the past decades, but in the case of trusses made from cold formed sections, new cross-sectional shapes and joint details still need to be developed. In [18] a HSS pentagon shaped cross-section is proposed and investigated through a calculation method based on the Generalized Beam Theory (GBT), which was compared to numerical calculations and experimental data.

The main concepts and steps that need to be followed when developing the numerical implementation of a GBT formulation aimed to perform first-order elastic–plastic analyses of thin-walled members have been presented in [19]. All the GBT results were compared to Abaqus shell finite element value, very good agreement between the two being obtained.

However, it is shown that FEM analysis provides better and more precise results than the GBT procedure and therefore, in this work an Abaqus approach is preferred.

1.2 Hollow section trusses

Nature provides us with several examples of the tubular shape behaviour when subjected to compression, torsion or bending.

These advantages of the circular hollow sections have been recognised and exploited even from ancient times. A good example of such application is the Firth of Forth bridge in Scotland (1890), seen in Figure1.2. [20]

Figure 1.2 *Firth of Forth bridge (Scotland)*
Source: http://infohost.nmt.edu

It is in that century that the manufacturers developed the first production methods for seamless and welded circular hollow sections. In 1886, the Mannesmann brothers developed the skew roll piercing process which made it possible to roll short thick walled tubes [20].

The most common hollow sections available on the market and the ones that are mostly used in design are the circular, square and rectangular ones, but there are special shapes available as well: triangular, hexagonal, octagonal, flat-oval, elliptical or half-elliptical.

Today, hollow steel sections can be rolled in various processes. Most common are hot and cold rolled steel tubes. Hot rolled sections are predominantly used for structural purposes while tubes rolled from cold rolled steel have better bending ability and give a better aesthetic appearance after being powder coated.

The tubular members have excellent mechanical properties. They present a high bending and torsional rigidity in comparison with I-shaped beams with the same mass, since the material is distributed further away from the section's centroid and they behave excellently under compression actions. Moreover, it represents a great shape against wind, water or wave loading, combined with the fact that it behaves perfectly against compression, bending and torsion, and having an architecturally attractive shape[21], the CHS is frequently chosen for structural elements in today's modern architecture. The exterior surface of the hollow sections is reduced compared to open cross-section, thus reducing also the cost for painting and fire protection solutions. The paint thickness is easy to achieve due to the big enough rounding of the elements. Applications may vary as follows: buildings, halls, bridges, barriers, masts, towers, offshore and special applications, such as glass houses, radio telescopes, sign gantries, parapets, cranes, jibs, sculptures, etc.[21].

1.3 Semi-closed polygonal section trusses

The idea of using HSS in polygonal sections for the truss elements is an innovative and emerging one, therefore the research and previous studies about this matter are still scarce. Nevertheless, the polygonal shapes have been implemented already by Ruukki in the construction of lattice towers for wind turbines.

A detailed research study about this was performed by Olga Garzon [22], who investigated the resistance of the polygonal cross-sections. The focus the thesis is the use of thinner walls on bolted elements in wind tower applications and the assessment of the design methods according to Eurocodes in comparison with FEM analysis.
The results of the study show that the Eurocode 3 part 1-3 and part 1-6 are in a good agreement when compared to the laboratory tests and FEM analysis performed, whenever the axial resistance was done on the folded plates. A smaller difference between numerical and analytical results was obtained when calculating the critical load with part 1-5, rather than with part 1-6 [22]. Therefore, in this thesis also part 1-5 is

used in order to determine the critical load of plates. It is also shown that the strength of the folded plate, even with less material used in the cross-section has a higher efficiency than the plates with circular cross-sections [22].

The use of HSS in polygonal shaped plates is further researched by Ruukki in a program called HISTWIN II, which desires to develop high wind turbine towers, based on a cylindrical tower concept. The project is being coordinated at LTU.

The proposed solution of a truss with built-up polygonal profile members, is an innovative idea. The objective is to maximize the efficiency of the cross-section by its geometry, while minimizing the quantity of steel used. This can lead to great economical and environmental benefits, as it will be shown later in this work.
There is no extended research and literature behind this type of built-up hollow sections so far and therefore there are many uncertainties on how these elements would behave under loads.

The main advantage of semi-closed polygonal profiles made from galvanised steel is that they facilitate simpler connections with minimum welding. Figure 1.3 shows possible polygonal profiles for compression chords and diagonals in a truss. The gusset plates required for the connections are inserted into the polygonal profile and secured with pretension bolts.

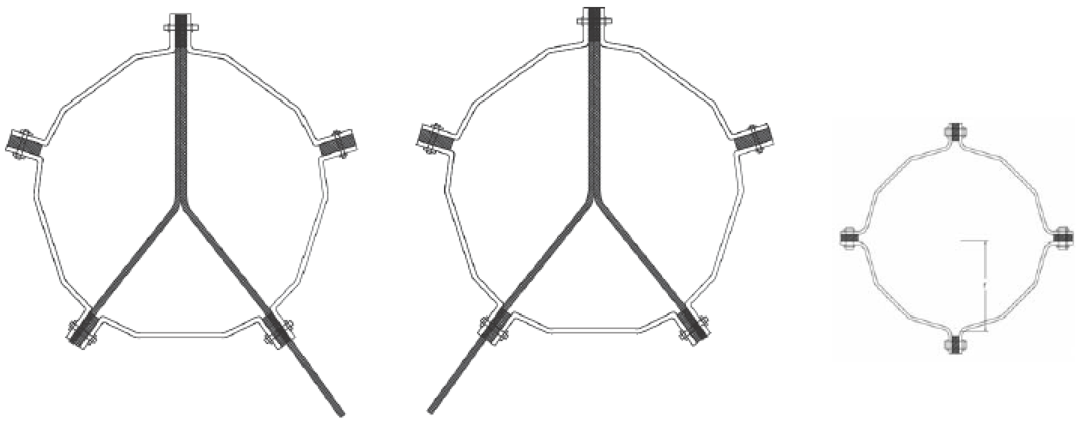


Figure 1.3 *Connection of semi-closed polygonal cross sections chords to diagonals*

The bottom chord is in tension and therefore an optimum shape could be in form of an open U-section as shown in Figure 1.4.

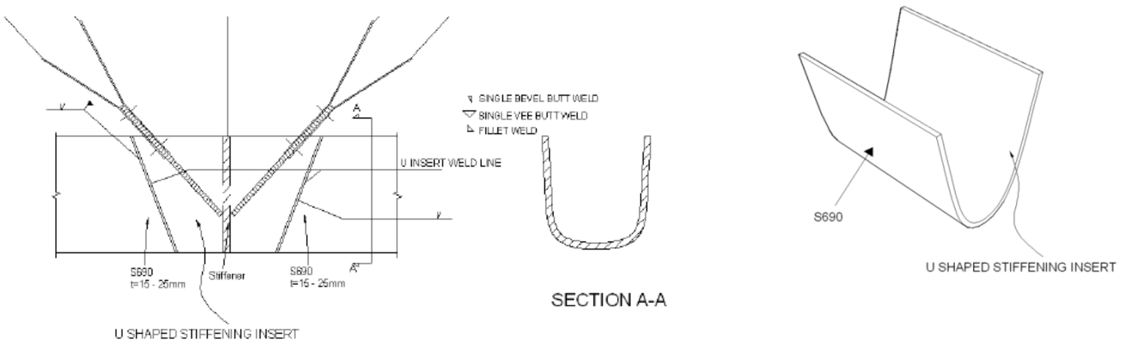


Figure 1.4 *U-shaped tension chord joint detail with diagonal; an alternative with tubular diagonals is shown for the sake of illustration*

1.4 Objectives and research questions

The main objective of our thesis is to investigate whether the usage of HSS semi-closed polygonal sections represents a viable and advantageous solution for long span truss elements, in comparison to the classic circular hollow sections, commonly used nowadays.

For this purpose, the influence of cold forming in the corners of the polygonal steel plates in compression is analysed, as given in design codes and in comparison with Finite Element Method.

Special attention is paid to the joint of the bottom chord to the two intersecting diagonals. The sides of the U channel are exposed to a biaxial stress state. The main characteristics of the stress field of that joint is analysed for various load levels.
Due to the concentration of stresses, 3 different displays of the joint will investigated in Abaqus:

- diagonals' gusset plates welded to the U-channel, without any stiffener
- diagonals' gusset plates welded to the U-channel with the presence of an extra plate as stiffener between the gusset plates[Fig.1.4]
- diagonals' gusset plates welded to the U-channel with a U-shaped welded stiffening insert

Buckling analysis of the compressed upper chords of the truss will be performed according to Eurocode and numerical methods (Abaqus).

A total of six cost and carbon footprint estimations will be performed in order to better emphasize the advantages of the use of HSS polygonal cross-sections:

- S355, S500, S650 for CHS truss
- S355, S500, S650 for the polygonal section truss

1.5 Limitations

There is a lack of literature regarding the use and behaviour of polygonal cross-sections in structural engineering.
Even though HSS structures have started to be more and more widely used, EN1993-1-12 still does not provide a lot of information about steel grades higher than S355.

Tests were performed in COMPLAB, at LTU, but the geometry of the compressed single plates (circular and polygonal) differ than those used in this work. There are plans to conduct experiments on complete scaled-down truss in the future at the COMPLAB.

Because of the high complexity of an entire truss model, we have analysed in Abaqus just a segment of the tension chord, to a distance from the connection area and a single compressed polygonal chord.

Due to difficulties in obtaining the price cost, several manufacturing processes were neglected from calculations, leading to an approximate estimation of the truss costs. Nevertheless, this should not affect the comparative study, since the processes were neglected on both types of trusses.

1.6 Scientific approach

In order to address these research questions the following approach was carried out:

1. Experimental laboratory tests were conducted on single plate circular and polygonal cross-sections made of S650 steel. The steel specimens were provided by Ruukki and the compression tests took place at COMPLAB Luleå University of Technology.
2. CHS and semi-closed trusses were designed according to EN 1993 part 1-1, EN 1993 part 1-3, EN 1993 part 1-5, EN 1993 part 1-8, EN 1993 part 1-12.
3. Connections and chords numerically analysed using Finite Element models. Results compared with hand calculations and theoretical values.
4. Cost and environmental evaluation, after obtaining the final cross-sections of the trusses' members.

1.7 Structure of the thesis

The first part consists of an introduction and background presentation, which is meant to present to the reader the scope of the project and the current situation of the studied subject. A short briefing of the details and outlines of the thesis should provide the reader with a generalized idea about the work performed.

The thesis will be structured on different chapters, each one treating different aspects, as follows:

Chapter 1 provides the reader with the first impression of the studied problem, the limitations encountered and the scientific approach that is used to answer the research questions.

Chapter 2 layout of the trusses and the design of CHS and polygonal elements, according to Eurocodes and CIDECT recommendations.

Chapter 3 gives the background of the numerical modelling of the connection in the tension chord.

Chapter 4 buckling analysis of the compressed cold-formed polygonal chord.

Chapter 5 results regarding the comparison of different types of steel and different cross-sections, with respect to the cost and carbon footprint evaluations.

Vaidas Alechnavicius worked on *Chapter 3* and *Chapter 4*, whilst Jozsef Balint created *Chapter 2* and *Chapter 5*. *Chapter 1* was written by both students, on a common agreement.

2 Structural Design of the Circular Hollow Section and Polygonal Section Truss Elements, According to Eurocodes

2.1 Truss layout and geometry

As mentioned in the previous chapter, CHS truss elements combine excellent structural behaviour with appealing shapes from architectural perspective.

Moreover, trusses are pleasant, modern and light structures, which require a relatively simple design and a small number of joints. The suggested layout for the studied truss is shown in the figure below. The same layout is applied to both the CHS and the polygonal cross-section trusses.

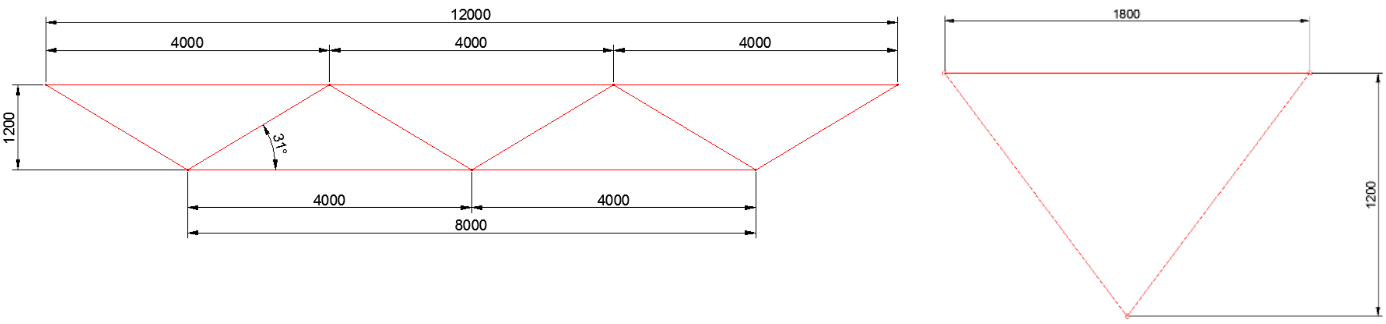


Figure 2.1 *Layout of the truss*

This layout represents a scaled-down truss, with a total length of 12m. The reason for this limitation is that in the near future laboratory testing of the truss will be made at LTU.

The truss is spatial triangular Warren type, a minimum eccentricity (e=0) was considered, in order to avoid the creation of additional moment around the joint area (Figure 2.2).

The final arrangement of chords and diagonals will be made after completing the design of members and joints.

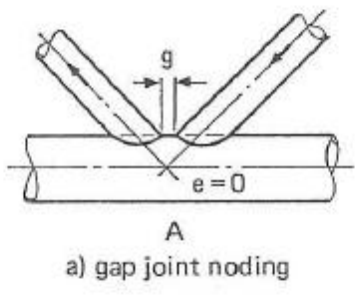


Figure 2.2 *Gap joint node*

2.2 Truss structural analysis

Structural analysis of the truss was performed with the help of "Autodesk Robot" software.

According to recommendations [21], the upper and lower chord of the truss are modelled as one continuous chord, whilst the horizontal bracings and diagonals are pinned at both ends. The truss is considered as simply supported.

In order to analyse the behaviour of the structure, a **2MN** load is applied evenly on the top chords over 4 points, **500kN** each.

A 3D model of the truss and the applied load is shown below.

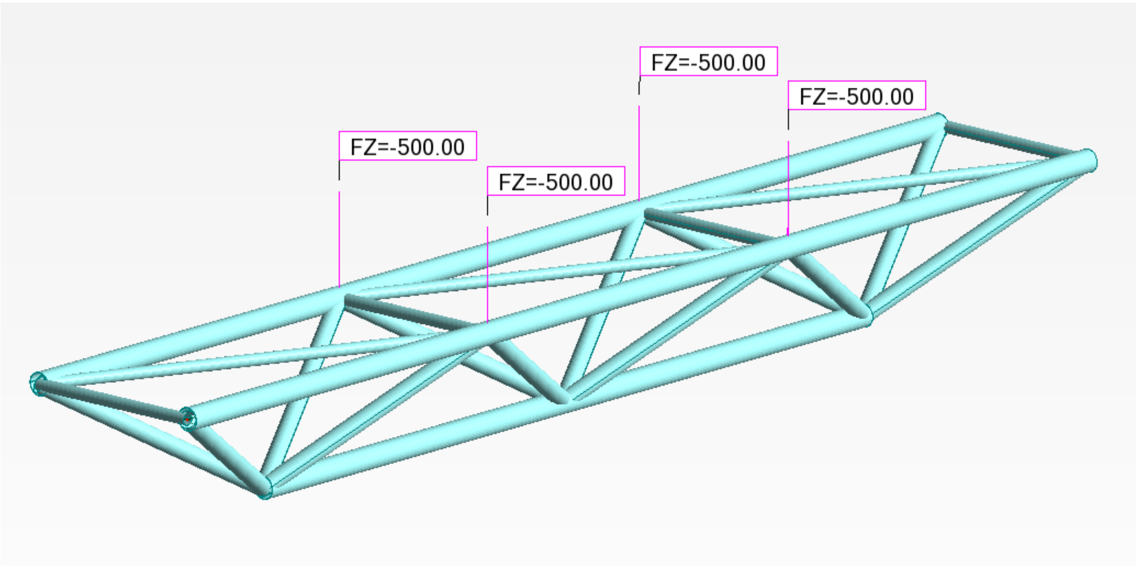


Figure 2.3 *3D model of the truss*

Internal forces, as obtained from the structural analysis software, are presented below (all values expressed in kN and kNm) :

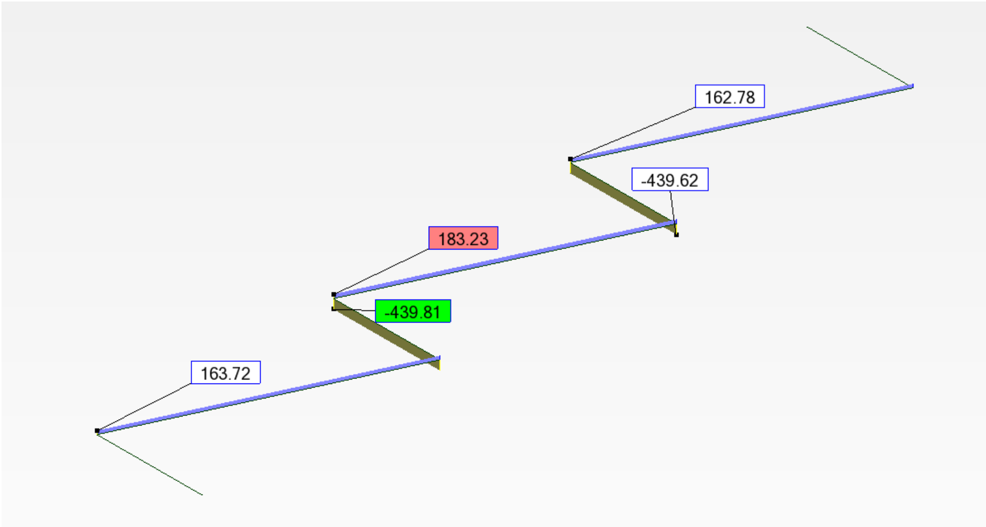


Figure 2.4 *Axial force in the horizontal bracings*

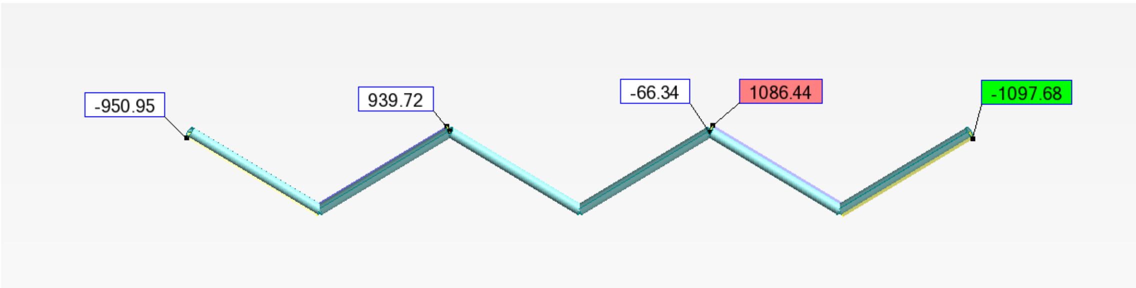


Figure 2.5 *Axial force in the diagonals*

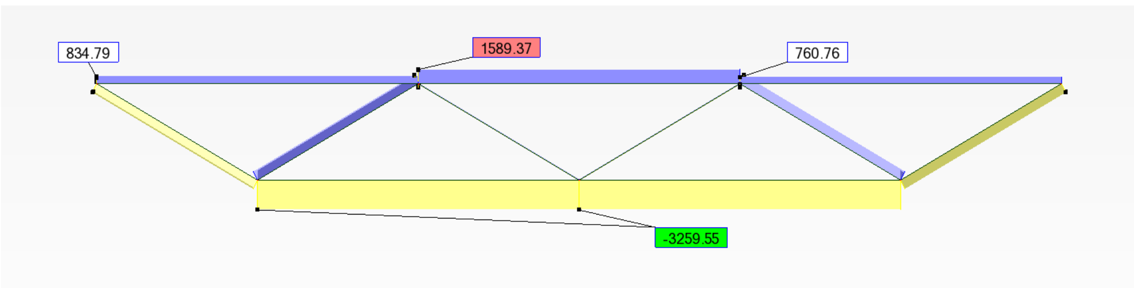


Figure 2.6 *Axial force diagram*

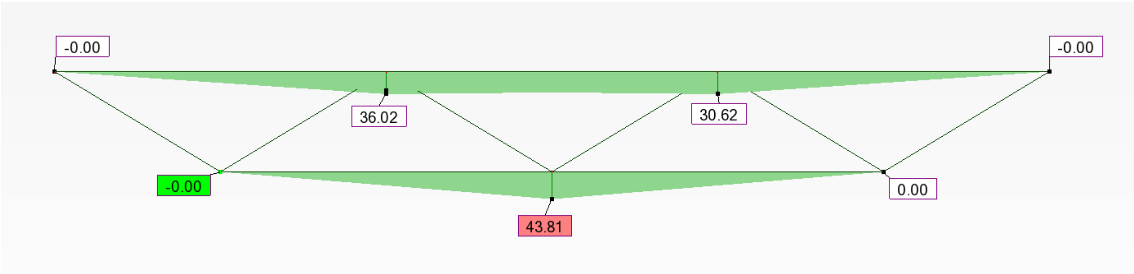


Figure 2.7 *Bending moment diagram*

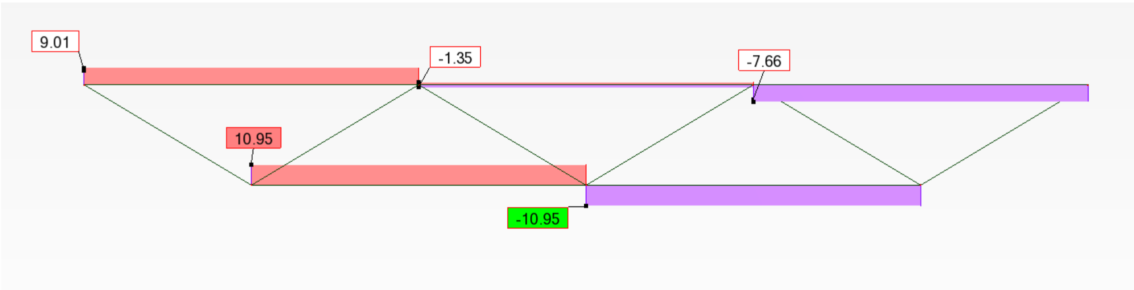


Figure 2.8 *Shear force diagram*

2.3 Design of the truss with Circular Hollow Sections

2.3.1 Member design according to Eurocodes

Three steel grades are used in this thesis, presented in the Table 2.1. An individual design for each grade will be performed.

The design codes used for the CHS truss are EN 1993 part 1-1 and EN 1993 part 1-12 (for HSS regulations in the case of S650 type of steel).

Table 2.1 *Steel grades used for trusses*

Steel	f_y(MPa)	f_u (MPa)
S355	355	470
S500	500	550
S650	650	700

Cross section classification.

The section class gives the extent to which resistance and rotation capacity of a cross section are limited by local buckling. In EC3 there are 4 classes given for circular hollow sections (CHS), but the design rules for joints are restricted only to class 1 and class 2. The class limits of the section according EC3 is given in the table below [21]:

Table 2.2 *Cross sectional classification limits*

$\varepsilon = \sqrt{235/f_y}$ and f_y in N/mm^2					
Limits	CHS in compression: d_i/t_i	RHS in compression (hot-finished and cold-formed): $(b_i - 2r_o)/t_i$ (*)		I sections in compression	
				Flange: $(b_i - t_w - 2r)/t_i$	Web: $(h_i - 2t_i - 2r)/t_w$
Class 1	$50\varepsilon^2$	33ε		18ε	33ε
Class 2	$70\varepsilon^2$	38ε		20ε	38ε
Reduction factor ε for various steel grades					
f_y (N/mm^2)	235	275	355	420	460
ε	1.00	0.92	0.81	0.75	0.71

Determination of member size.

Members' size is determined to undertake the axial load.

For tensile members like the bottom chord and the braces in tension, the area of the member should be sufficient to resist the tensile force.

The design resistance of a net section is taken as [23]:

$$N_{t.Rd} = \frac{A * f_y}{\gamma_{M0}}$$

The design resistance of cross section in compression is determined as follows [23]:

$$N_{c.Rd} = \frac{A * f_y}{\gamma_{M0}}$$

In addition, buckling resistance of the compressed members must be checked. According the design recommendations for CHS [21], an effective length factor of **K=0.9** can be used for the design of the compression chord. The effective length factor for the compression brace members can initially be assumed to be **K=0.75** [21].

The resistance reduction factor for compressed CHS members is obtained by buckling curve a [23], as a function of the slenderness of the member.

$$\bar{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}} = \frac{k * L}{i} * \frac{1}{\lambda_1}$$

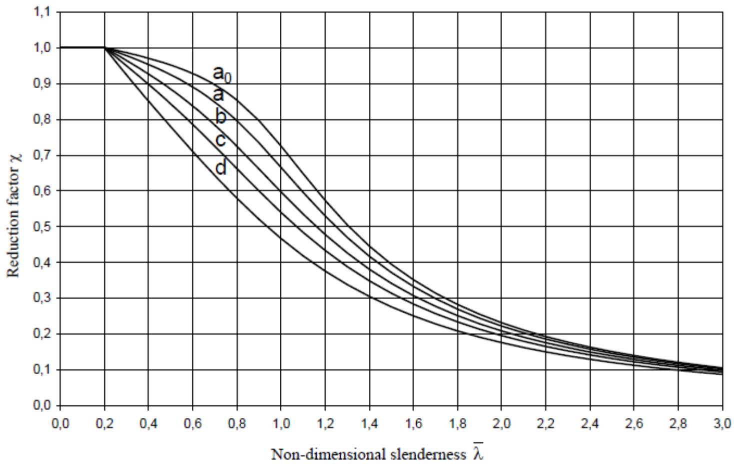


Figure 2.9 *Buckling curves[23]*

Complete member design procedure is described in Annex A and provided in MathCAD files.

2.3.2 Joint design (according to Eurocode & CIDECT)

Design of welded joints in truss with tubular elements is made according Eurocode 1993-1-8 [24] and recommendations provided by the Committee of Construction with Hollow Steel Sections [21].

Warren trusses provide great opportunity to use gap joints. This type of joints allows for a more convenient and simpler welding solution.

General joint considerations in the design [21]**:**

1. Chords should have thicker walls than braces do. Stronger walls in chords should collect the forces from brace members more effectively. Joint resistance increases while thickness to diameter ratio decrease;

2. Diagonals should have thin walls rather than thick. For that reason larger but thinner sections will provide sufficient buckling capacity in the compressed member. Moreover, thinner walls require smaller fillet welds;

3. CHS diagonals should have a smaller diameter comparing to CHS chord members;

4. Gap joints are preferred to the overlapped joints. The minimum gap should be $g \leq t_1 + t_2$ to provide enough space for welds (t_1, t_2 are the thicknesses of the 2 diagonals);

5. The angle between chord and braces should be more than 30 degrees.

All of the above recommendations were considered throughout the joint design.

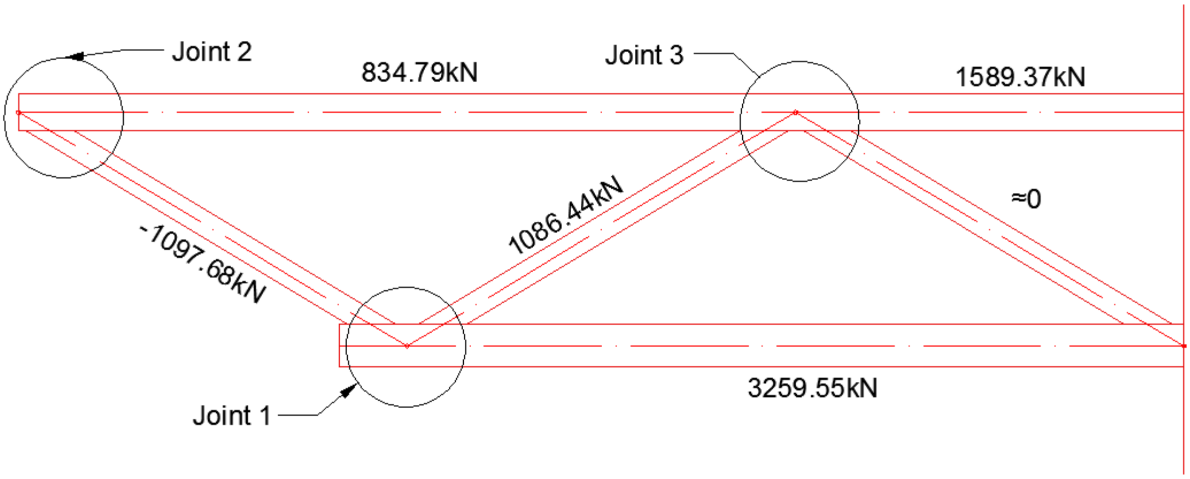


Figure 2.10 *Types of joints designed*

Three types of joints are designed in the truss (see figure 2.10. above):

Joint 1 – Multi-planar KK joint in the bottom chord (Figure 2.11 a);

Joint 2 – Uni-planar Y joint in the top chord (Figure 2.11 b);

Joint 3 – Uni-planar Y joint in the top chord (Figure 2.11 c).

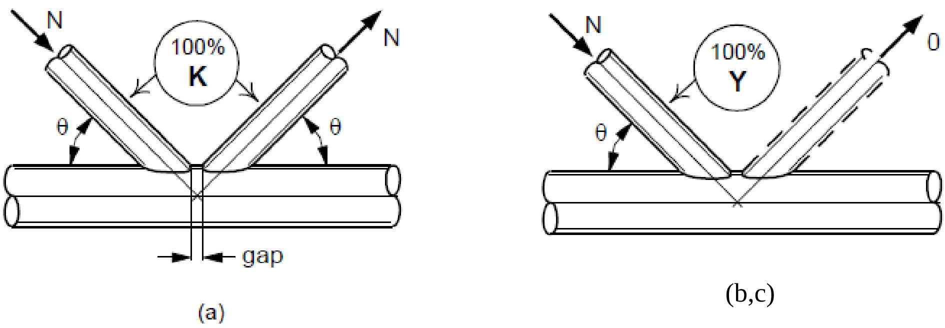


Figure 2.11 *Joint types in truss[21]*

Limitations due to material.

There is quite a big reduction in the design resistance of the joint provided by Eurocode for HSS members. For steel grades higher than S460 the joint resistance reduction factor is **0,8** [25].

2 main failure modes are considered in welded joint design [24]:

a) **Chord face failure** (plastic failure the chord face) or chord plastification;

b) **Punching shear failure** of a hollow section chord wall.

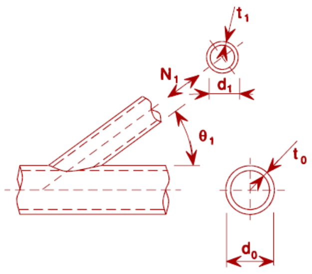
This is valid for joints that meet the requirements described in the table below [24]. The design resistance of a connection should be taken as the minimum resistance value obtained from these two criteria.

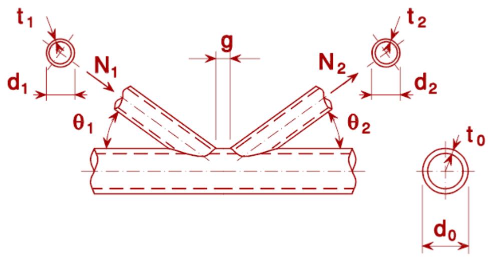
Table 2.3 *Range of validity for welded joints between CHS brace members and CHS chords.*

$0{,}2 \leq d_i/d_0 \leq 1{,}0$						
Class 2 and but	10 10	$\leq$ $\leq$	d_0/t_0 d_0/t_0	$\leq$ $\leq$	50 40	generally for X joints
Class 2 and	10	$\leq$	d_i/t_i	$\leq$	50	
$\lambda_{ov} \geq 25\%$						
$g \geq t_1 + t_2$						

The design resistance for uni-planar K and Y joints is given in the table below:

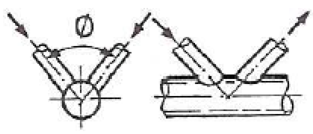
Table 2.4 *K and Y joint resistance formulae*

Chord face failure - T and Y joints	
	$N_{1,Rd} = \dfrac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2{,}8 + 14{,}2\beta^2)/\gamma_{M5}$

Chord face failure - K and N gap or overlap joints	
	$N_{1,\mathrm{Rd}} = \frac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1{,}8 + 10{,}2\frac{d_1}{d_0}\right) / \gamma_{M5}$ $N_{2,\mathrm{Rd}} = \frac{\sin\theta_1}{\sin\theta_2} N_{1,Rd}$
Punching shear failure - K, N and KT gap joints and all T, Y and X joints [i = 1, 2 or 3]	
When $d_i \le d_0 - 2t_0$: $N_{i,\mathrm{Rd}} = \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_i \frac{1+\sin\theta_i}{2\sin^2\theta_i} / \gamma_{M5}$	
Factors k_g and k_p	
$k_g = \gamma^{0,2}\left(1 + \frac{0{,}024\gamma^{1,2}}{1+\exp(0{,}5g/t_0 - 1{,}33)}\right)$	(see Figure 7.6)
For $n_p > 0$ (compression): $k_p = 1 - 0{,}3\, n_p(1 + n_p)$ but $k_p \le 1{,}0$ For $n_p \le 0$ (tension): $k_p = 1{,}0$	

Multi-planar joints, like the one in the bottom chord, require some additional strength checks. Correction factors are proposed for different kind of joints [21]. For K joints in triangular girders, as in this case, correction factor 1 can be used for uni-planar joint resistance formulae. Resistance of axial force and shear force in the gap zone must be checked for multi-planar KK joints. The recommended procedure is shown in table below [21]:

Table 2.5 *KK type of joint checking procedure*

KK gap joints	$\mu = 1.0$ Note: In a gap joint, the cross section in the gap has to be checked for shear failure: $\left(\frac{N_{gap,0}}{N_{pl,0}}\right)^2 + \left(\frac{V_{gap,0}}{V_{pl,0}}\right)^2 \le 1.0$ eq. 6.2 where: $N_{gap,0}$ = axial force in gap $N_{pl,0} = A_0 f_{y0}$ $V_{gap,0}$ = shear force in gap $V_{pl,0} = 0.58 f_{y0} \frac{2A_0}{\pi}$
	
Range of validity	Same as table 4.1 $60° \le \phi \le 90°$

Complete design of joints for all the steel grades is given in Annex A.

2.3.3 Weld design

K or Y joints require a welding around the entire perimeter of the connected member by means of butt weld, fillet weld or by combination of both. Fillet welds are designed to resist higher load than the brace member capacity. According to Eurocode 3 [24], the following minimal throat thickness *a* can be calculated. The higher steel class, the larger throat thickness is required:

$$\begin{cases} \text{For S355}: a \geq 1.10t \\ \text{For S460}: a \geq 1.48t \end{cases}$$

For steel grades greater than S460 and up to S700 the filler metal may have lower strength than the base.

The design shear strength of the weld is determined from [[24]. (4.4)]:

$$f_{vw.d} = \frac{f_u/\sqrt{3}}{\beta_w * \gamma_{M2}};$$

The design resistance per unit length is determined from [[24]. (4.3)]:

$$F_{w.Rd} = f_{vw.d} * a;$$

Design calculations of the welds are given in Annex A.

2.3.4 Results and conclusions

Results from the design calculations for different members and different steel grades are given in following tables:

Table 2.6 *CHS truss made out of S650 steel*

Steel - S650	Profile	Area of cross section (mm^2)	Length of members (m)	Weight per meter (kg/m)	Weight (kg)
Top chord	CHS 193.7x10	5771	24.00	45.30	1087.2
Diagonals	CHS 127x6	2281	27.52	17.90	492.608
Top braces	CHS 114.3x3	1049	21.00	8.23	172.83
Bottom chord	CHS 219.1x10	6569	8.70	51.60	448.92
				Total weight:	**2201.558**

Table 2.7 *CHS truss made out of S500 steel*

Steel - S500	Profile	Area of cross section (mm^2)	Length of members (m)	Weight per meter (kg/m)	Weight (kg)
Top chord	CHS 193.7x10	5771	24.00	45.30	1087.2
Diagonals	CHS 168.3x6	3059	27.20	24.00	652.8
Top braces	CHS 114.3x3	1049	21.00	8.23	172.83
Bottom chord	CHS 273.0x10	8262	8.80	64.90	571.12
				Total weight:	**2483.95**

Table 2.8 *CHS truss made out of S355 steel*

Steel - S355	Profile	Area of cross section (mm^2)	Length of members (m)	Weight per meter (kg/m)	Weight (kg)
Top chord	CHS 219.1x12.5	8113	24.00	63.70	1528.8
Diagonals	CHS 168.3x6.3	3206	26.74	25.20	673.848
Top braces	CHS108x4	1307	21.00	10.30	216.3
Bottom chord	CHS 323.9x12.5	9600	9.00	96.00	864
				Total weight:	**3282.948**

As expected, a decreased weight of approximately 1 tonne is observed by increasing the steel grade from S355 to S650.

Even smaller sections for HHS could be obtained. This is not permitted since there are restrictions in Eurocode stating that design resistance in joints of HSS members should be reduced by a factor of **0,8** [25]. In many cases the chord face failure was the main factor influencing the size of the cross section.

Economical and environmental assessment between different types of truss (circular and polygonal sections) and between different steel grades is made in the last chapter.

2.4 Semi-closed polygonal sections truss design

The idea is to design the truss using built-up polygonal cross sections made from cold formed plate elements. The bottom chord is designed as a U-shaped profile. Moreover, bracings between the top chords of the truss are circular hollow sections. Due to the small diameter needed for these braces, the usage of polygonal sections is not beneficial.

This type of structure requires some advanced solutions in joint detailing and the design process is not as straight forward as for CHS since there is not much research and experience about this type of profiles.

Design of the members is performed according to the given rules in Eurocodes 1993-1-1 [23], 1993-1-3 [26] for cold formed members and 1993-1-5 [27] for plate elements. Additional rules from Eurocode 1993-1-12 [25] for HSS are applied.

2.4.1 Layout and assembly of truss members

The layout of the truss is the same as in the case of CHS truss (see Figure 2.1.). Four different types of members can be distinguished in the structure of the truss:

1) Top chord – element in compression, built from 5 cold formed thin plates.

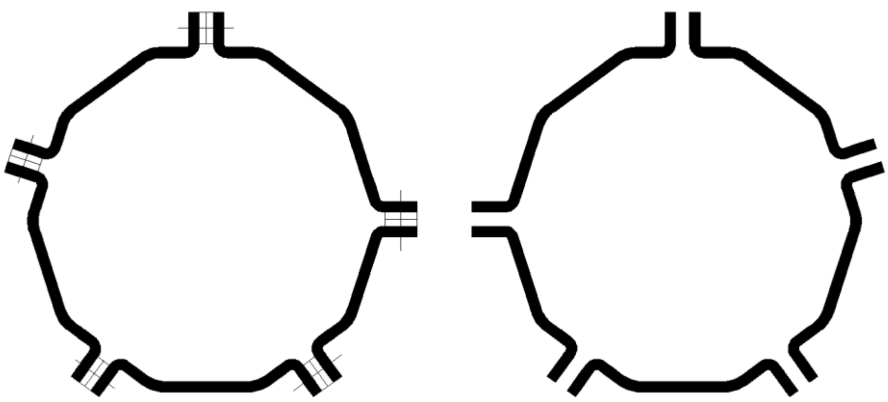


Figure 2.12 *Truss top chord built-up sections*

2) Diagonals – elements in tension or compression, built from 4 cold formed thin plates.

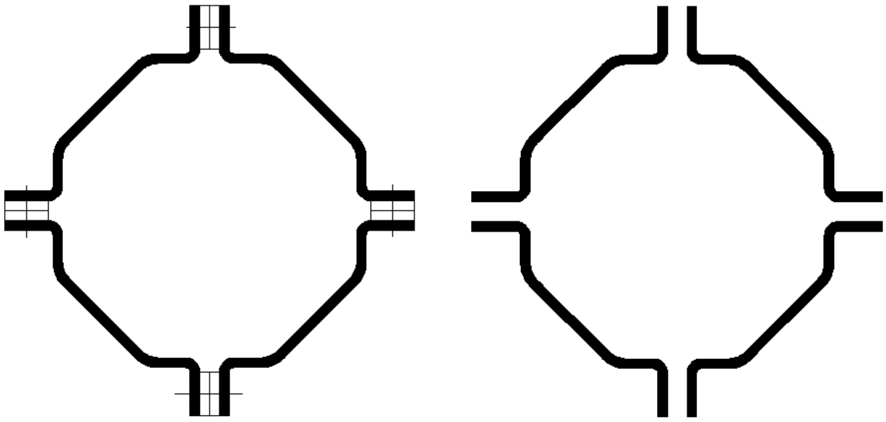


Figure 2.13 *Truss diagonals built-up sections*

3) Bottom chord – element in tension, made from cold formed (bended) thin plate.

Figure 2.14 *Open U-shaped profile*

4) Horizontal bracings – the same elements as in the CHS truss will be used (CHS 114.3X3).

By using these types of elements we expect to achieve a weight reduction of the total truss structure, whilst the stability of the elements will be equal or greater than the ones used in the CHS truss.

2.4.2 Design of thin walled members

Design of plate members is made according to the regulations of Eurocodes. The internal forces are considered the same as in the design of tubular truss (see previous chapter).

- **Material properties of cold formed sections**

The strength of the cold formed section is influenced by the number and size of the corners of the member. The increased average yield strength can be determined as proposed by 3.2.2 (3) in [26]:

$$f_{ya} = f_{yb} + (f_u - f_{yb})\frac{knt^2}{A_g} \text{ but } f_{ya} \leq \frac{(f_u + f_{yb})}{2}$$

k – a numerical coefficient that depends on the type of forming (7 for roll forming, 5 for other methods of forming)

n – the number of 90 degree bends in the cross-section with an internal radius $r \leq 5t$ (fractions of 90 degree bends should be counted as fractions of n)

- **Classification of cross sections**

Classification of the cross sections is made according to the table 5.2 in Eurocode 1993-1-1 for internal and external compressed parts. For class 4 cross section, effective area and widths should be used for calculations.

- **Influence of rounded corners**

In cross sections with rounded corners, the notional flat widths b_p of the plane elements should be measured from the mid-points of the adjacent corner, as it is shown in figure below:

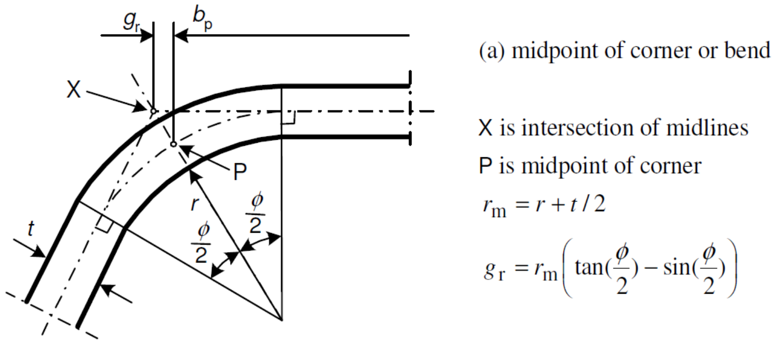


***Figure 2.15** Notional flat width[26]*

- **Local buckling**

Critical compressive stress for buckling plate element is defined by [28]:

$$\sigma_{cr} = k_\sigma * \frac{\pi^2 * E}{12 * (1 - \nu^2)} * \frac{t^2}{b^2}$$

Where k_σ is the plate buckling coefficient depending on the support conditions of the plate.

This equation does not include the influence of the rounded corner described by Eurocode 1993-1-3 (see figure 2.15 above). The critical stress with included notional flat width according EN 1993-1-3 becomes:

$$\sigma_{cr} = k_\sigma * \frac{\pi^2 * E}{12 * (1 - \nu^2)} * \frac{t^2}{{b_p}^2}$$

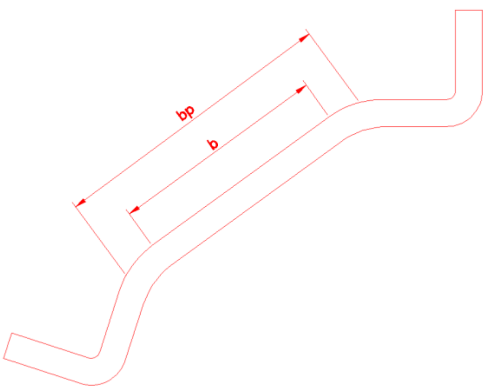


Figure 2.16 *Flat width b and notional flat width bp in bended plate*

- **Ultimate load for plates**

Plate elements in compression are subjected to post-buckling behaviour. That means that the stresses are redistributed in the cross-section area. To simplify this behaviour, it is assumed that in a simply supported plate loaded axially from both sides, the two stress blocks with constant stress over the total width appear [27]. The effective width is determined by using a reduction factor ρ and it is obtained by:

$$b_{eff} = \rho * b$$

In this case, the width b should be replaced by the notional flat width b_p [26] (see above):

$$b_{eff} = \rho * b_p$$

In those equations:

$$\rho = 1 \text{ if } \bar{\lambda}_p \leq 0{,}673$$

<u>For double supported elements in compression:</u>

$$\rho = \frac{1-0{,}055*(3+\psi)}{\bar{\lambda}_p^2} \text{ if } \bar{\lambda}_p \geq 0{,}673 \quad \text{but } \rho \leq 1{,}0$$

<u>For outstand compression element:</u>

$$\rho = \frac{1-0{,}188}{\bar{\lambda}_p^2}$$

Two expressions for cross section area are obtained:

$$A_g = \sum_{i=1}^{n} b_p * t$$

$$A_{eff} = \sum_{i=1}^{n} b_{eff} * t$$

Characteristic resistance for compressed member is obtained by EN 1993-1-3 chapter 6.1.3(1):

$N_{c,Rk} = A_{eff} * f_{yb}$ if $A_{eff} < A_g$

$N_{c,Rk} = A_g \left[f_{yb} + \left(f_{ya} - f_{yb}\right) * 4 * \left(1 - \frac{\lambda}{\lambda_{el}}\right)\right] \leq A_g * f_{ya}$ if $A_{eff} = A_g$

Buckling resistance for the flexural buckling of a compressed member made of a plate is based on the relative slenderness $\bar{\lambda}$ [23]:

$\bar{\lambda} = \sqrt{\frac{A * f_y}{N_{cr}}}$ for class 1,2,3 of cross sections;

$\bar{\lambda} = \sqrt{\frac{A_{eff} * f_y}{N_{cr}}} = \frac{L_{cr}}{i} * \frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1}$ for class 4 of cross sections;

$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}}$; $i = \sqrt{\frac{I}{A}}$

Reduction factor χ is calculated using relative slenderness and the imperfection factor of $\alpha = 0.49$ (Eurocode 1993-1-1, buckling curve "c"):

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0.5}}$$

$$\phi = 0.5 * (1 + \alpha * \left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2)$$

Characteristic buckling resistance of the member:

$N_{b,Rk} = \chi * A * f_y$ for class 1, 2, 3 of cross sections;

$N_{b,Rk} = \chi * A_{eff} * f_y$ for class 4 of cross sections;

For tensile members, like the U-shaped bottom chord, the area of the member should be sufficient to resist the tensile force. Design resistance of a net section is taken as [23]:

$$N_{t.Rd} = \frac{A * f_y}{\gamma_{M0}}$$

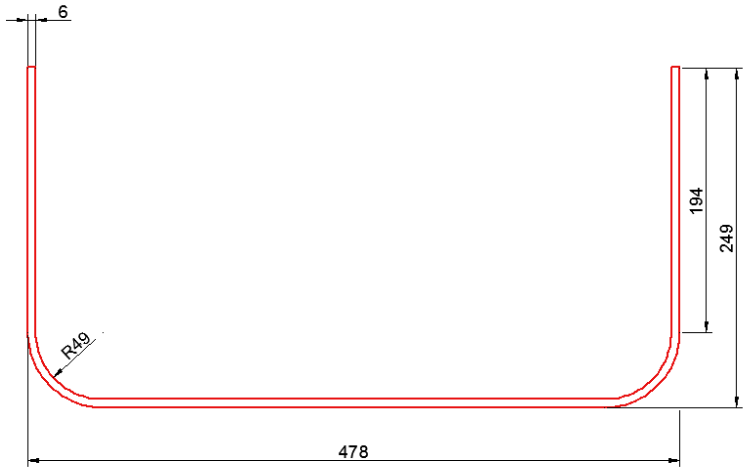


Figure 2.17 *U-shaped bottom chord (S650)*

Complete design of the plate members is presented in Annex B.

2.4.3 Results

In the following tables are shown the results obtained by calculations for different steel grades.

Table 2.9 *Built-up polygonal section truss made out of S650 steel*

Steel - S650	Profile*	Area of cross section (mm^2)	Length of members (m)	Weight per meter (kg/m)	Weight (kg)
Top chord	Pol 200x6	4539.8	24.00	35.63743	855.29832
Diagonals	Pol 140x4	2102	27.20	16.5007	448.81904
Top braces	CHS 114.3x3	1049	21.00	8.23	172.83
Bottom chord	Ux6	5517	8.70	43.30845	376.783515
				Total weight:	1853.730875

Table 2.10 *Built-up polygonal section truss made out of S500 steel*

Steel - S500	Profile*	Area of cross section (mm^2)	Length of members (m)	Weight per meter (kg/m)	Weight (kg)
Top chord	Pol 220x6	4853.5	24.00	38.099975	914.3994
Diagonals	Pol 160x4	2668	27.20	20.9438	569.67136
Top braces	CHS 114.3x3	1049	21.00	8.23	172.83
Bottom chord	Ux6	6600	8.80	51.81	455.928
				Total weight:	**2112.82876**

Table 2.11 *Built-up polygonal section truss made out of S355 steel*

Steel - S355	Profile*	Area of cross section (mm^2)	Length of members (m)	Weight per meter (kg/m)	Weight (kg)
Top chord	Pol 240x6	5165	24.00	40.54525	973.086
Diagonals	Pol 190x4	2986.86	26.74	23.446851	626.9687957
Top braces	CHS108x4	1307	21.00	10.30	216.3
Bottom chord	Ux6	9300	9.00	73.005	657.045
				Total weight:	**2473.399796**

* Pol "A"x"B" stands for a 10 sided polygon inscribed in a circle with the diameter "A" mm, with the thickness of the polygon being "B" mm.

Even though not as radically as in the case of the CHS truss, the same tendency of the total weight to decrease can be observed with the increase of the steel grade (620 kg difference between S650 and S355).

2.5 Global buckling verification of the entire truss

Due to the absence of any lateral constraints on the truss during the laboratory testing phase, a global buckling check of the entire truss is proposed.
This is done by considering the truss to be a built-up member in compression and by following the indications in EN 1993-1-1, part 6.4.

For steel structures under compression, it is very common to design built-up members, made by coupling two or more members in order to obtain stronger and stiffer sections.

The connection of the members can be done either by lacings or battening. The former method is used for this truss.

The truss of the two top chords may be considered to be a column with an initial imperfection of $e_0 = \frac{L}{500}$.

Verification is performed using the design chord forces $N_{ch.Ed}$ from compression forces N_{Ed} and moments M_{Ed} at mid span of the built-up member.

The design chord force is given by:

$$N_{ch.Ed} = 0.5 \times N_{Ed} + \frac{M_{Ed} h_0 A_{ch}}{2I_{eff}} \quad \text{where} \quad M_{Ed} = \frac{N_{Ed} \times e_0 + M_{Ed}^{I}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_V}}$$

N_{Ed} is the design value of the compression force to the built-up member

N_{cr} is the effective critical force of the built-up member

M_{Ed} is the design value of the maximum moment in the middle of the built-up member considering second order effects

M_{Ed}^{I} is the design value of the maximum moment in the middle of the built-up member without second order effects

h_0 is the distance between the centroids of chords

A_{ch} is the cross-sectional area of one chord

I_{eff} is the effective second moment of area of the built-up member

S_V is the shear stiffness of the lacings or battened panel

In this case M_{Ed}^{I} is zero, since in the truss there is no moment on this direction.

$I_{eff} = 0.5 \times h_0^2 \times A_{ch}$, as given in formula (6.72) of EN 1993-1-1.

S_V is taken from Figure 6.9 in EN 1993-1-1, according to the situation that suits the case (3rd case for this truss).

The buckling verification for the chords should be performed as:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \leq 1$$

where $N_{b.Rd}$ is the design value of the buckling resistance of the chord taking the buckling length L_{ch}.

The buckling length L_{ch} suggested by the Eurocode is equal to the length of one truss member in this case (fig. 6.8 from EN 1993-1-1).

Except that aspect, the determination of the design value of the buckling resistance is done according to the usual rules of Eurocode 3, part 1-1.

The detailed calculations are presented in Annex C.

3 CONNECTION BEHAVIOUR ANALYSIS AT THE TENSION CHORD

In this chapter are described the methods and numerical analysis for the bottom chord connection, followed by the obtained results and conclusions.

3.1 Introduction

The bottom chords in truss structures are subjected to tension force. The type of U-shape tension chord designed herein (see figure 3.1) is not a commonly used section in truss design. That requires an extra analysis for the behaviour of the chord at the connection zone.

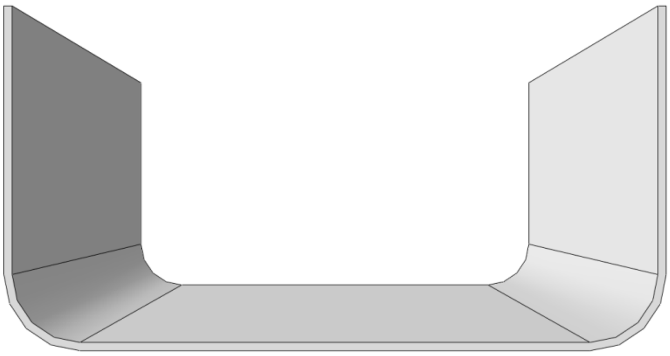


Figure 3.1 *U-section*

U channel is in an optimum profile shape for a bottom chord. The manufacturing of this element is relatively easy (cold-formed from thin plate). The main limitation in this task, is the lack of literature regarding this type of profiles in contrast to the connection behaviour of circular hollow sections (see chapter 2 above) for which rich literature can be founded.

The main questions answered by the analysis of the connection are the following:

1. The sides of the tension chord are subjected to biaxial stresses. What are the stress fields under different load levels in the section?

2. The connection is assembled in three configurations: no stiffener between gusset plates - stiffener between the plates - U-shaped stiffener. How does the connection behave in each case and what is the necessity for a stiffener?
3. If a stiffener is necessary, which type performs better and for what thickness?

3.2 Stresses in the plates

Ultimate stress analysis of the connection of the tension chord to the attached diagonals is complicated due to biaxial stresses. In a biaxial stress system, stresses lie in one plane and can be expressed by a pair of normal stresses and a shear stress.

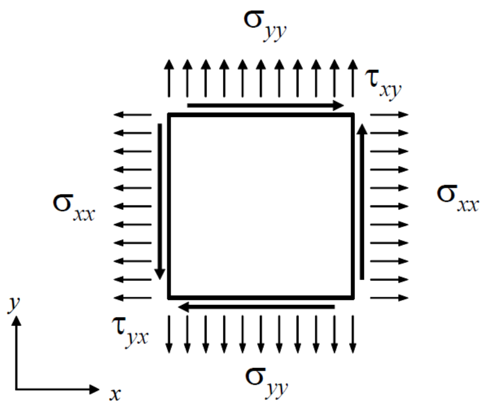


Figure 3.2 *Element of a structure in a biaxial stress state*

$$\sigma_{xx} \neq 0 \qquad \sigma_{yy} \neq 0 \qquad \sigma_{zz} = 0$$

In the case of biaxial stress, Hooke's law is written as follows:

$$\varepsilon_{xx} = (\sigma_{xx} - \nu\sigma_{yy})/E \qquad \varepsilon_{xx} = (\sigma_{xx} - \nu\sigma_{yy})/E$$

$$\gamma_{yy} = \frac{\sigma_{xy}}{G} = \frac{\tau_{xy}}{G}$$

By rearranging these equations to compute stresses for given strains, we get:

$$\sigma_{xx} = \frac{E}{(1-\nu^2)}\left[\varepsilon_{xx} + \nu\varepsilon_{yy}\right]$$

$$\sigma_{yy} = \frac{E}{(1-\nu^2)}\left[\varepsilon_{yy} + \nu\varepsilon_{xx}\right]$$

$$\tau_{xy} = G\gamma_{xy} = \frac{E}{2(1+\nu)}\gamma_{xy}$$

In a thin metal a biaxial stress state for all stresses lay within the plane of the material. Such a stress system is called Plane Stress.

In the bottom chord, where the member is subjected to tensile force, only uniaxial stress state exists. The maximum stress in this area is determined:

$$\sigma_{xx} = \frac{F}{A} = \frac{3259550N}{5516.1769mm^2} = 590.907MPa$$

F - axial force in the member;

A - area of the cross-section.

Biaxial stress state appears in the sides of the tension chord, in the area where truss diagonals are connected to the chord in tension.

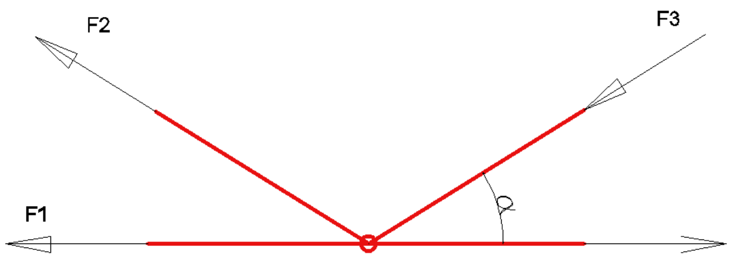


Figure 3.3 *Acting forces at the connection area*

3.3 Theories of failure

Having determined both axial and shear stresses in a biaxial stress system does not guarantee that these are maximum stresses of the actual member. The resultant force may lead to maximum value.

Figure 3.4 shows the typical stress-strain response of a ductile material such as mild steel. This type of stress-strain curve is obtained from tensile tests, where the material is exposed to uniaxial normal stress and have no shear stress. The material yields, at the yield stress σ_Y.

Usually the name of the steel grade (S355, S460 or S650) designates the material's yield stress, e.g.$\sigma_Y = 650MPa$ for S650 steel.

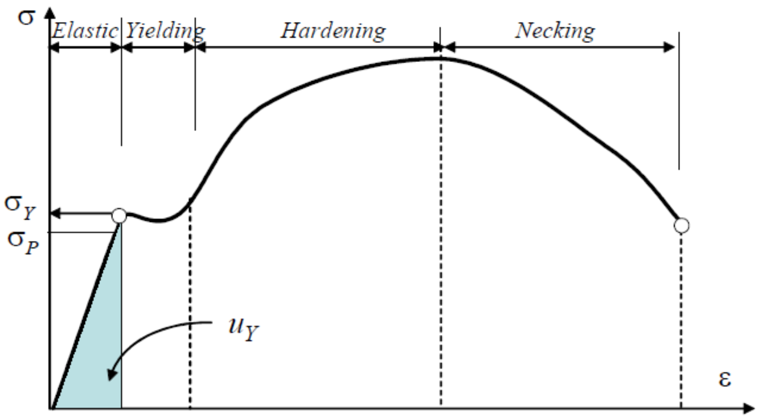


Figure 3.4 *Stress-strain diagram for ductile material*

Still, if a part is subjected to loads that lead to a combination of normal and shear stresses, some of them will cause the material to yield. These combinations of stresses that cause yielding are known as yield criterions. It is assumed that the material is ductile, isotropic and has the same behaviour both in tension and compression.

Several yield criterions can be found in literature.

Tresca's Yield Criterion (Maximum Shear Stress Theory): Yielding can be considered a shear phenomenon where layers of crystals or atoms slip relative to each other in shear. Tresca's criterion is based on the maximum shear stress reaching a critical level. For biaxial stress state it can be written:

$$\tau_{max} = \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \tau_{xy}^2} = \frac{\sigma_{11} - \sigma_{22}}{2} \leq \frac{\sigma_Y}{2}$$

Von Mises Yield Criterion (Maximum Distortion Energy Theory). It is possible to formulate criterion based on the distortions caused by strain energy. Von Mises yield criterion gives the equivalent stress at a point in a body acted upon by normal and shear stress in all direction. By applying Hooke's law, one can derive von Mises' criterion for biaxial stress state, as:

$$\sigma_{vm} = \sqrt{\sigma_{11}^2 - \sigma_{11}\sigma_{22} + \sigma_{22}^2} = \sqrt{\sigma_{xx}^2 - \sigma_{xx}\sigma_{yy} + \sigma_{yy}^2 + 3\tau_{xy}^2} \leq \sigma_Y$$

3.4 Method

Different types of bottom chord sections are numerically analysed by Finite Element Models in Abaqus to obtain stresses. The sections used are based on the design calculations of scaled down truss (see chapter 2 above). The main goal is to see the behaviour of the connections in different load levels, so for the numerical analysis only section made from S650 is used. The dimensions of the designed bottom chord are shown in the figure below.

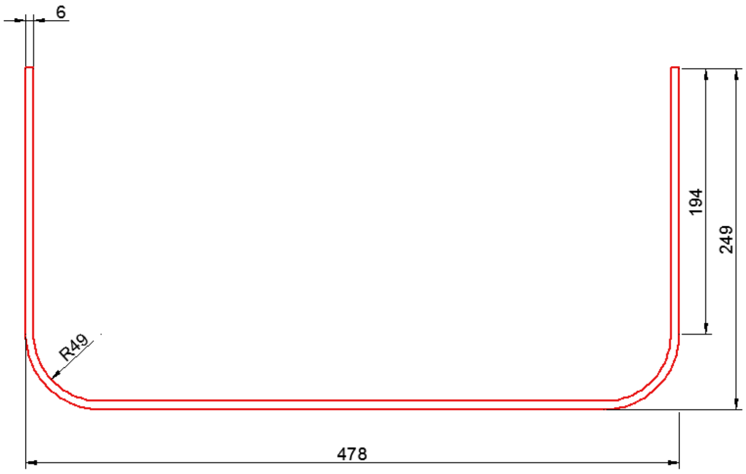


Figure 3.5 *Dimensions of U section (S650)*

The shape of the section is questionable, as there are no standard sections manufacturing at the moment. The section is made by cold bending of thin steel plate, the only limitation being the technology of the manufacturer where the section will be produced. For this thesis, the shape of the section is established in order to meet the design requirements (see chapter 2 above); also the assembly of truss elements is taken into account. The dimensions of U channel allow proper connections between the bottom chord and diagonals.

Two types of loading on the connection are investigated in this chapter. For one type the middle connection is taken into consideration, where the highest tension force occurs, but there are no (or there are very small) forces in diagonals, see figure 3.6.

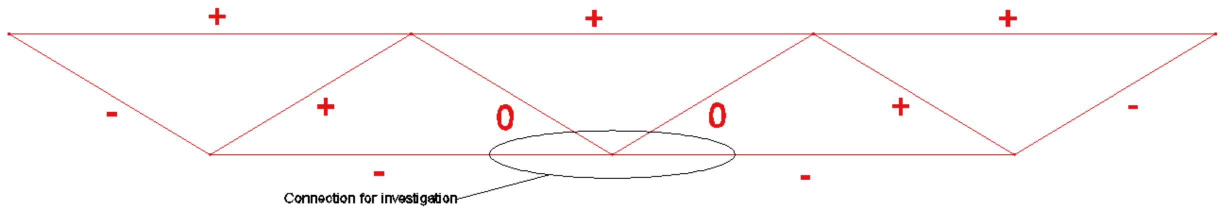


Figure 3.6 Compression (+) and tension (-) members in designed scaled down truss

The second type is loaded partially in tension, also in compression and tension from diagonals. The section is taken from a more regular truss arrangement, which is more often used in real truss designs, see figure 3.7.

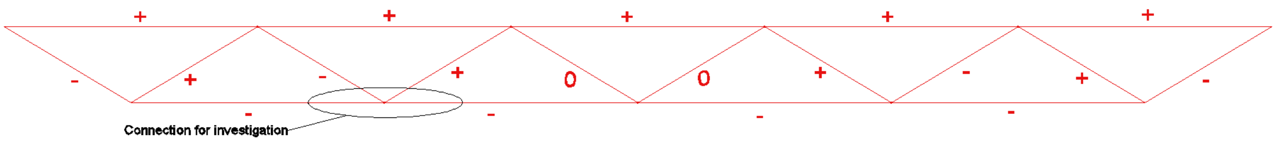


Figure 3.7 Compression (+) and tension (-) members in regular truss arrangement

Loads are taken from the design calculations of the scaled down truss (Chapter 2) and applied on the finite element model of the connection in the following way (see figure 3.8). The detailed description of Finite Element Modelling is given below in this chapter.

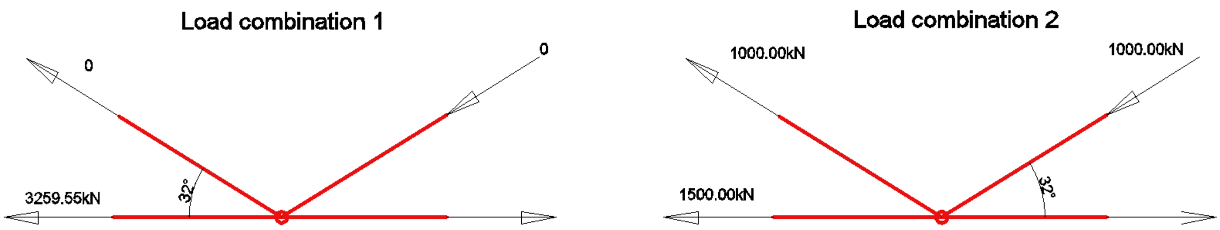


Figure 3.8 Loads applied on connection

The behaviour of three types of connections is analysed; they are shown in figure 3.9 below:

1. No stiffener between plates;
2. Stiffener between plates;
3. U shaped insert as a stiffener.

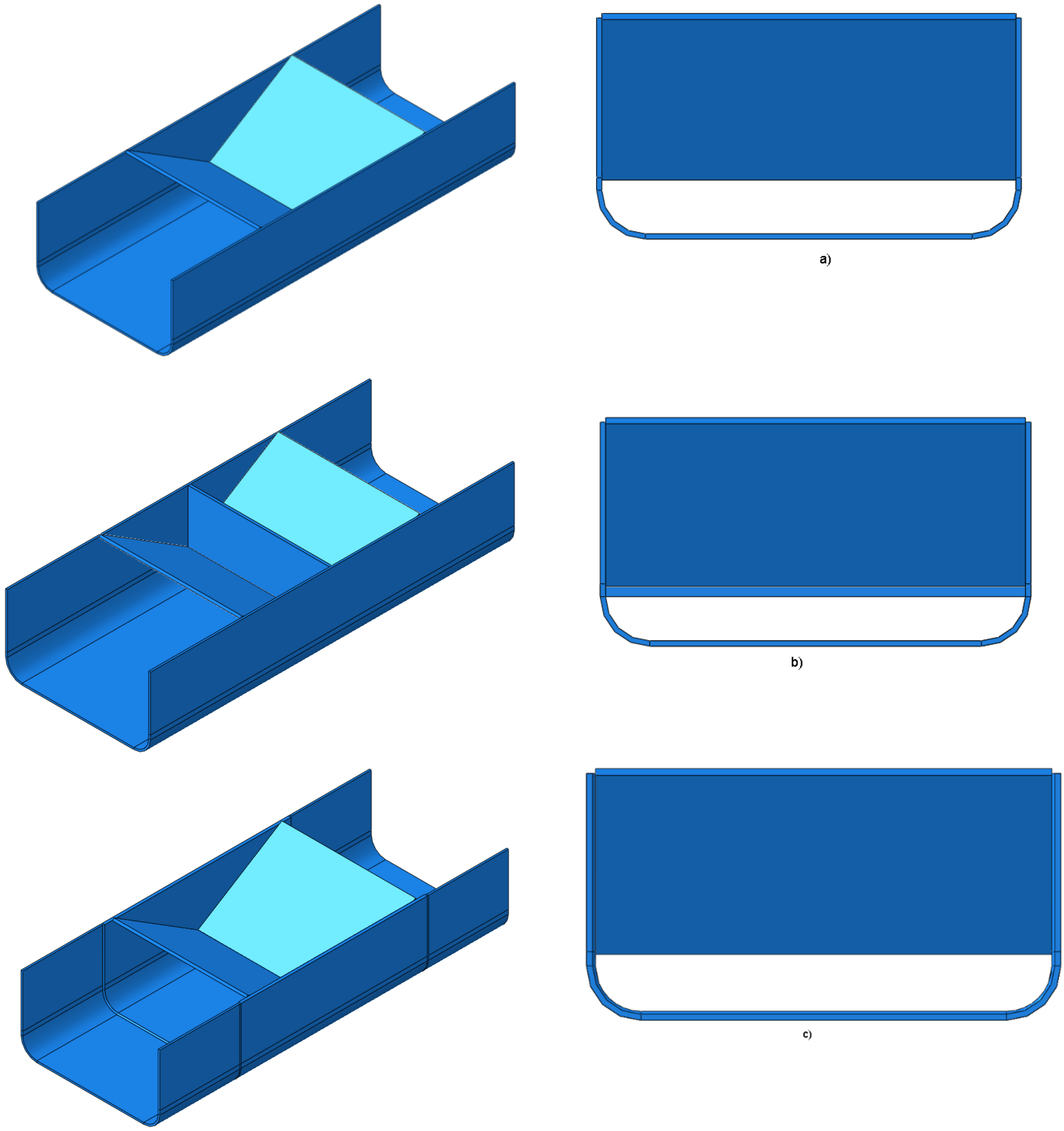


Figure 3.9 *Different types of connection under investigation*
a) without stiffener; b) with stiffener between plates, c) with U-shaped insert

All types of connections are numerically analysed in Abaqus. Both types of connections are analysed with welded plates together and without welding. The stress distribution in both normal directions is obtained from Finite Element Analysis, von Mises stresses being used to analyse the material's yielding.

3.5 Modelling in Abaqus

All models in Abaqus are made using solid elements. Firstly, parts are sketched in AutoCAD and imported into Abaqus to make more precise drawings and reduce time consumption, since realizing the shapes in Abaqus is quite demanding.

Length of the analysed U-section is **L=1200mm.** That allows enough flexibility for the joint area and boundary conditions not to affect the results too much.

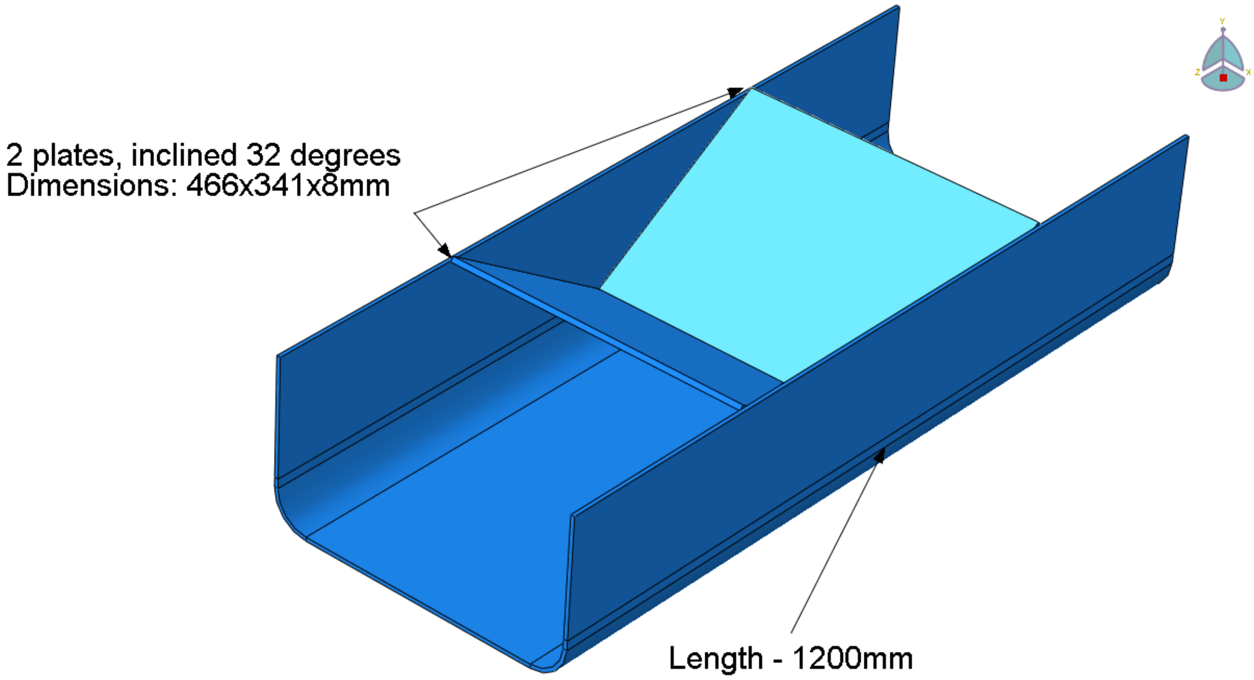


Figure 3.10 *Model of the connection without stiffeners*

Elastic material properties are applied for steel: Young's Modulus = 210 000 and Poisson's ratio = 0.3.

Proper material orientation is assigned along each plate, in order to obtain correct results in stress analysis.

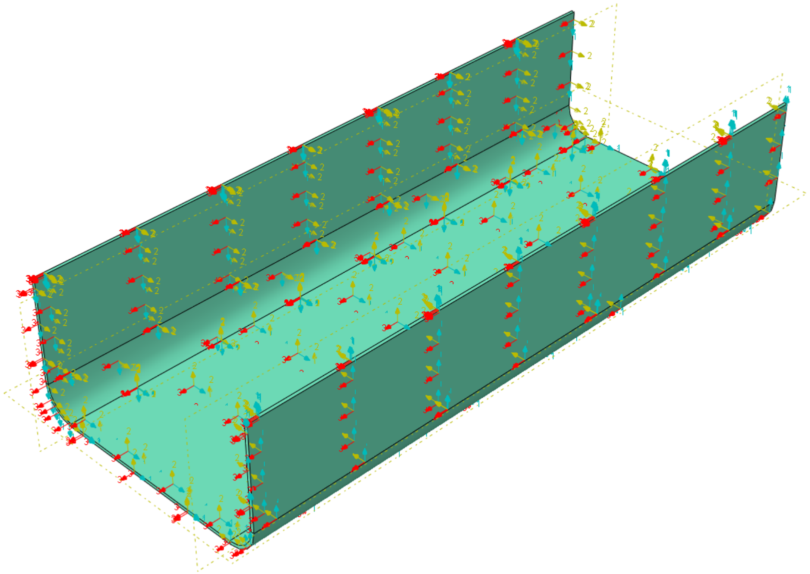


Figure 3.11 *Local material orientations in the U channel*

All instances: U channel and plates were assembled using translate and rotate feature tools. Plate and U-shape stiffeners are merged to the main U-chord to simulate homogeneous material behaviour. There were some difficulties to properly mesh the model if inclined plates are merged with the whole model, because there is a problem in generating proper mesh where the inclined plate and the U channel connect. The solution is to tie the plates to the main surface (U-channel). Tie constraint provides a simple way to bond surfaces together permanently, which allows easy mesh transition.

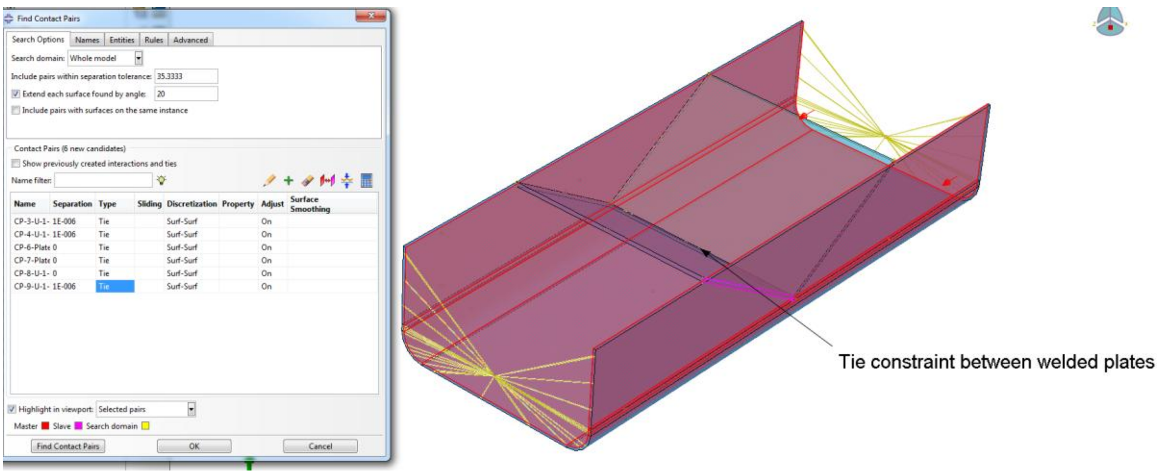


Figure 3.12 *Connecting surface ties*

For the model with the faces of the diagonals welded together, they have been constrained through a tie constrain as well.

As seen in Figure 3.12, two reference points are made at the centre of gravity. Points are coupled to the edge surfaces of the U segment by Distributing Coupling constraint. Loads and Boundary conditions are applied to those reference points. Both kinematic and distributing coupling is tested. Distributing coupling shows better results for this work. Kinematic coupling makes very stiff restrains all over the boundary area, which results in huge stresses at the support areas. The flanges of the chord should not be fixed because naturally, the cross-section is imposed to a partial rigidity.

Two types of loads are applied on the model (see figure 3.8). For the first case, the member in pure tension is analysed, where the influence of the plates and stiffeners is taken into account. The second case simulates the behaviour of the connection in biaxial stress state. The member is loaded both in tension in longitudinal direction, and forces from diagonals are introduced to the model. Forces from diagonals are applied as pressure load on the surface of the plate. As the scope of the work is to analyse stresses in the U-channel, the real load placement in the plate or at the bolt holes is not necessary.

1st case: Maximum tensile load applied – 3259.55kN;

2nd case: Tensile load – 1500, tension and compression loads on plates, 1000kN each (load taken to simulate the maximum tensile load as resultant from applied loads).

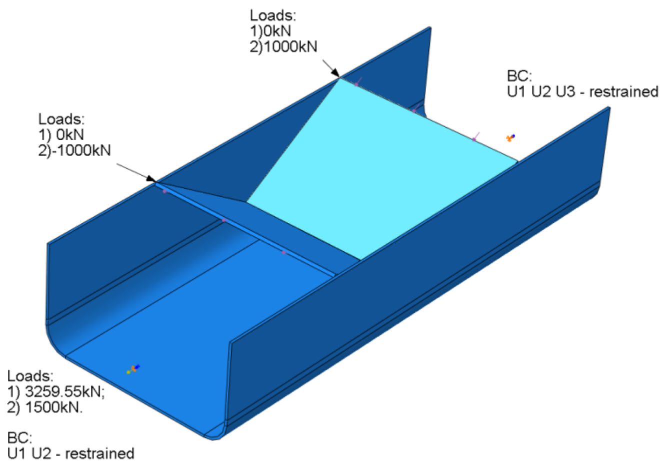


Figure 3.13 *Loads and boundary conditions on the model*

All parts are partitioned to make a structured mesh. Mesh global seed size – 5mm. All plates through thickness have at least 3 elements, which allows proper calculation results. C3D8R, 8-node linear brick elements are used.

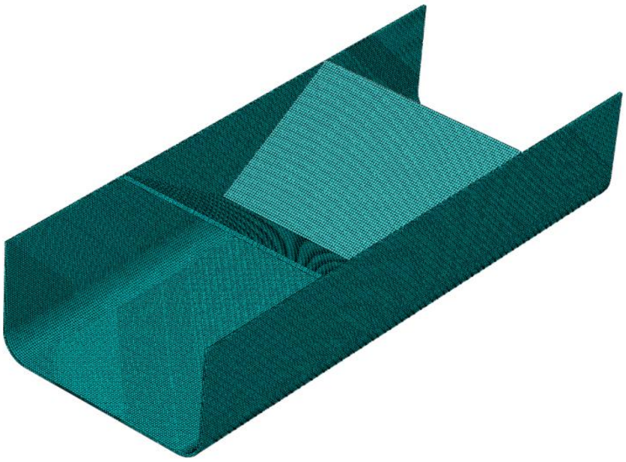


Figure 3.14 *Meshed FE model*

Separate jobs for static, general steps for each type of model are created. Detailed results obtained by FEM analysis are given in the next chapter.

3.6 Analysis and results

Table 3.1 *List of FE models*

Nr.	Model name	Model description	Load combination
1	NO_STIFF1	U channel without stiffeners (a)	LC1
2	NO_STIFF2	U channel without stiffeners (a)	LC2
3	NO_STIFF_WLD1	U channel without stiffeners, plates welded together (a)	LC1
4	NO_STIFF_WLD2	U channel without stiffeners, plates welded together (a)	LC2
5	MIDDLE_STIFF1	Stiffener between the plates (b)	LC1
6	MIDDLE_STIFF2	Stiffener between the plates (b)	LC2
7	MIDDLE_STIFF_WLD1	Stiffener between the plates, plates welded to stiffener (b)	LC1
8	MIDDLE_STIFF_WLD2	Stiffener between the plates, plates welded to stiffener (b)	LC2
9	U_INSERT_6MM_1	6mm thick U shaped stiffener (c)	LC1
10	U_INSERT_6MM_2	6mm thick U shaped stiffener (c)	LC2
11	U_INSERT_6MM_WLD_1	6mm thick U shaped stiffener, plates welded together (c)	LC1
12	U_INSERT_6MM_WLD_2	6mm thick U shaped stiffener, plates welded together (c)	LC2
13	U_INSERT_3MM_1	3mm thick U shaped stiffener (c)	LC1
14	U_INSERT_3MM_2	3mm thick U shaped stiffener (c)	LC2
15	U_INSERT_3MM_WLD_1	3mm thick U shaped stiffener, plates welded together (c)	LC1
16	U_INSERT_3MM_WLD_2	3mm thick U shaped stiffener, plates welded together (c)	LC2

16 different configuration models are analysed. Modelling procedure and load combinations are described above. For all models static, elastic analysis is performed. The goal of the analysis is to check the deformation behaviour and the stresses in the U shaped chord.

Two paths along the element in z and y directions are created in order to obtain the exact stresses in the analysed model.

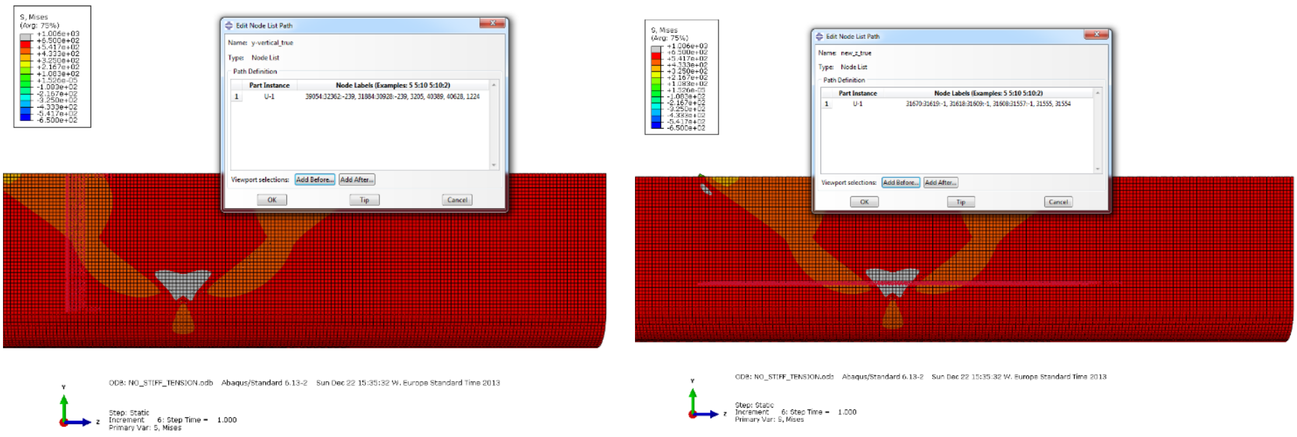


***Figure 3.15** Paths in z and y directions*

First horizontal path in z direction goes through the zone where the highest stress concentration occurs, where the 2 plates from the diagonals meet.

Second, vertical path in y direction passing through the middle point of the diagonal plate.

The stress fields shown in the figures have a yield stress limit of $\sigma_Y = 650MPa$. Grey zones in the figures show the zone where yielding occurs.

The following figures are showing the Von Mises stresses in the models.

Connection without stiffener

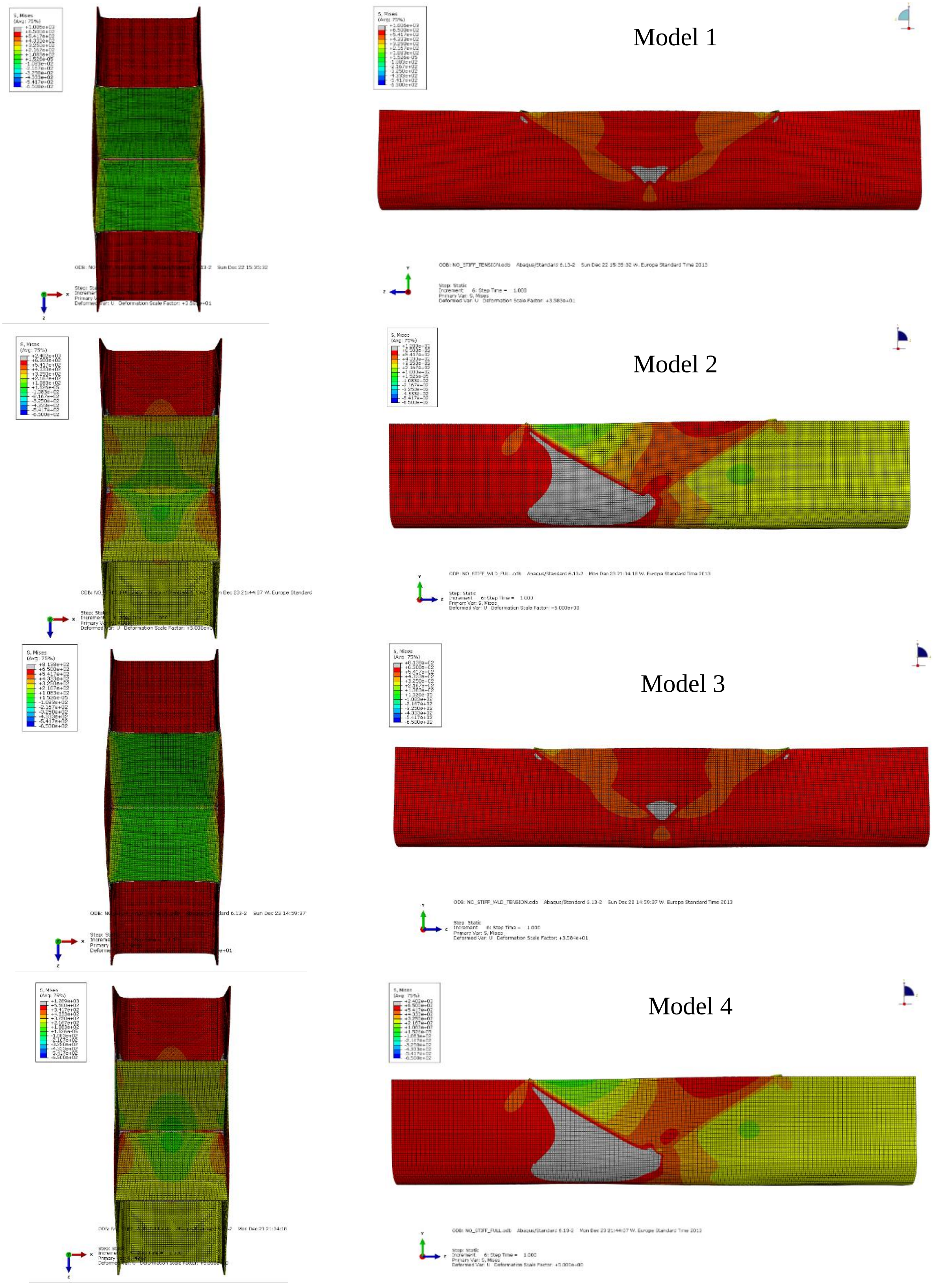


Figure 3.16 *Stress fields in joints without stiffener*

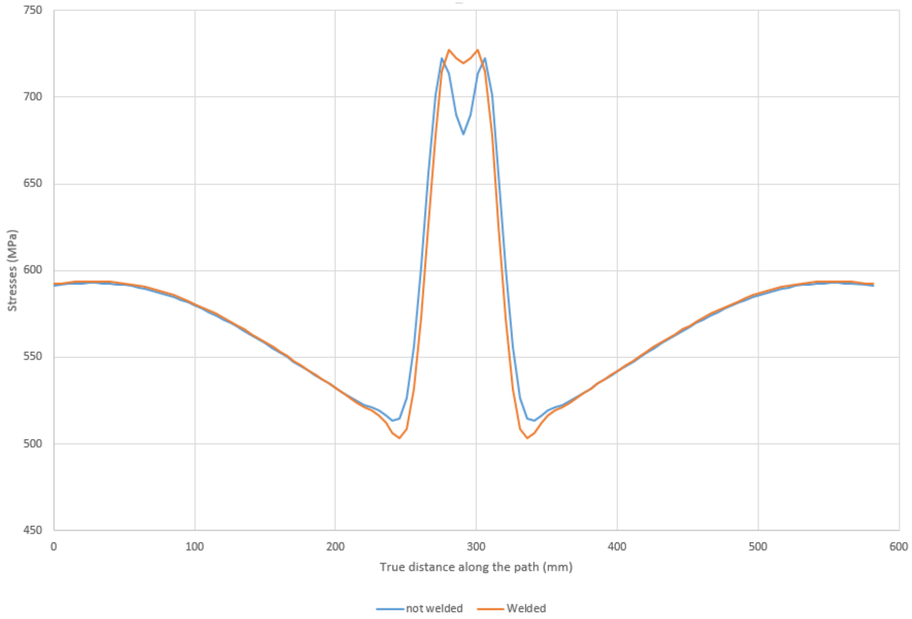


Figure 3.17 *Stress in member along horizontal direction. Tension only. (Connection without stiffener)*

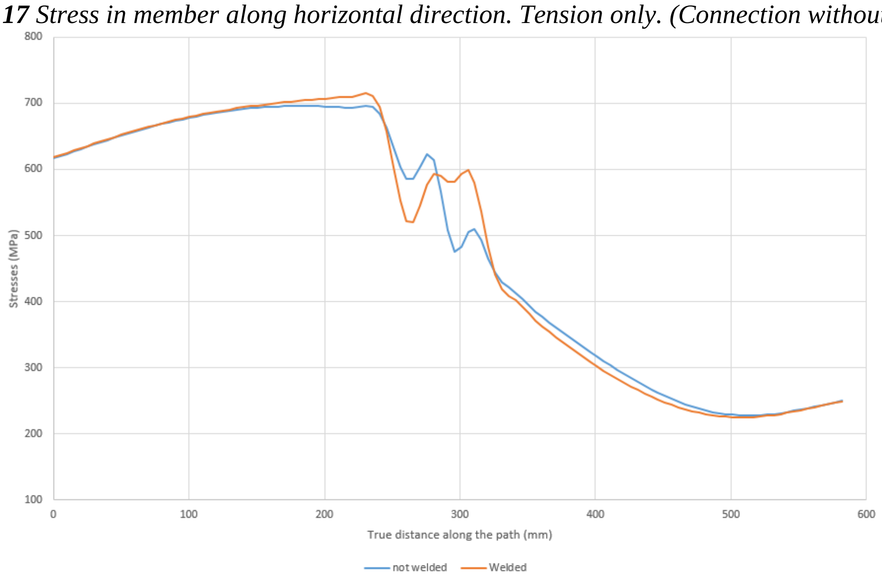


Figure 3.18 *Stress in member along horizontal direction. Full loading. (Connection without stiffener)*

Analysis shows that there is not a huge difference if the plates in the U chord are welded together or not. Still, the most critical area in the connection remains the corner where the two plates connect. Yielding starts in this zone.

When the elements are in the biaxial stress state (load combination 2) a huge part of the side of the tension chord yields. A stiffener is crucial in any load case for this connection.

Connection with middle stiffener

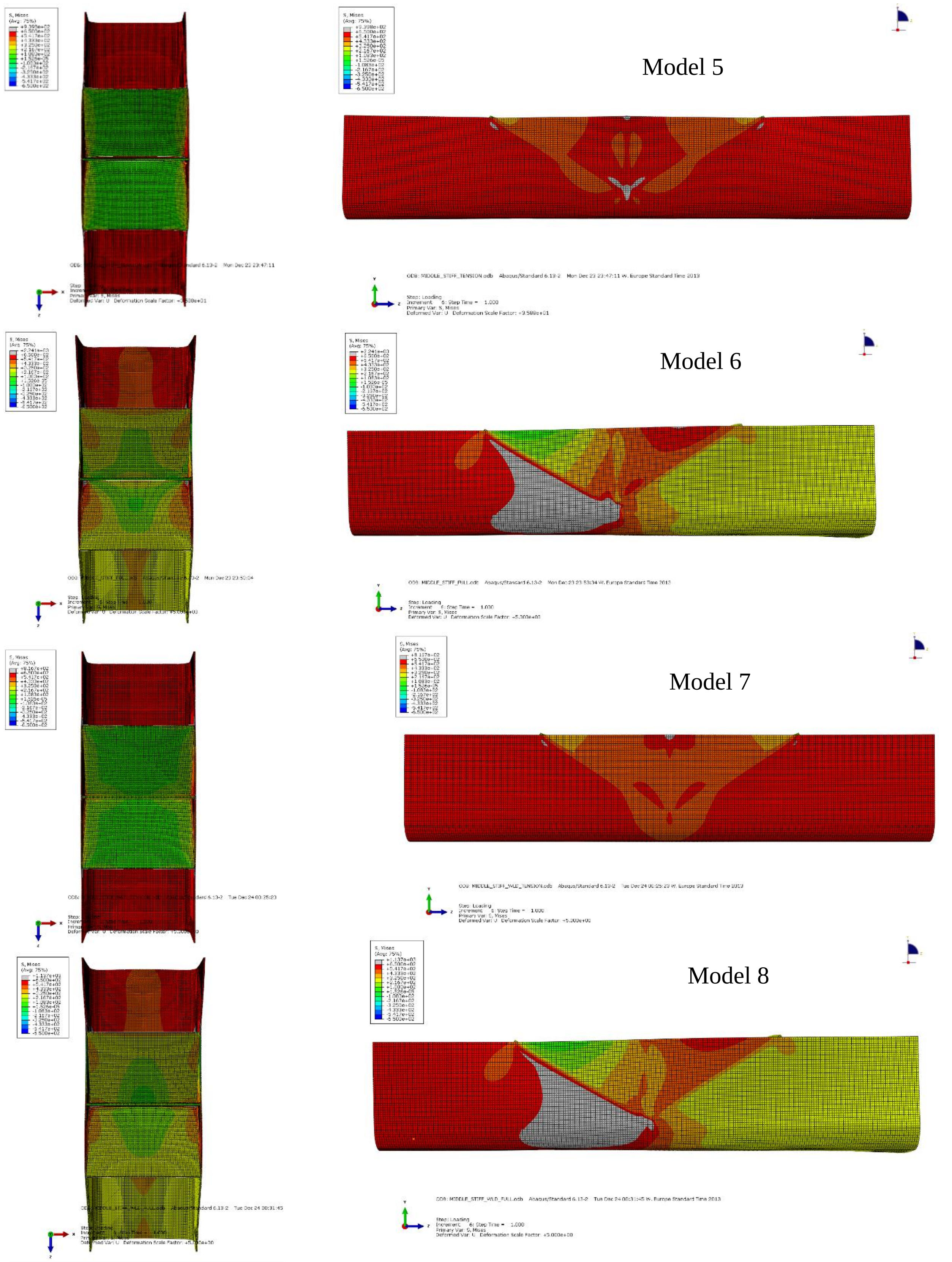


Figure 3.19 *Stress fields in joints with stiffener between the plates*

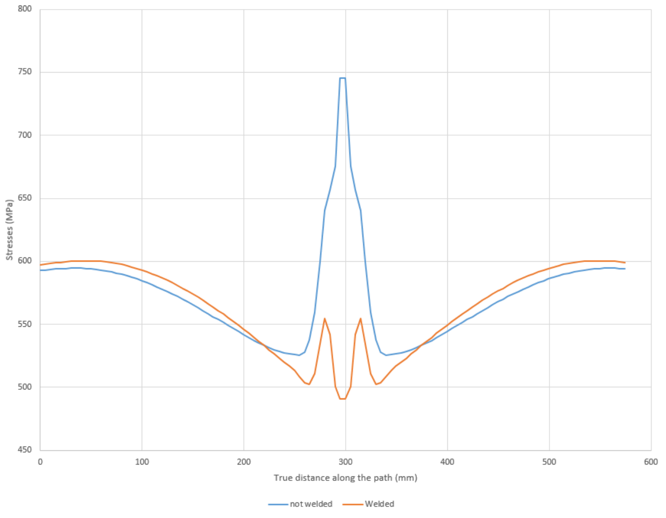


Figure 3.20 *Stress in member along horizontal direction. Tension only. (With middle stiffener)*

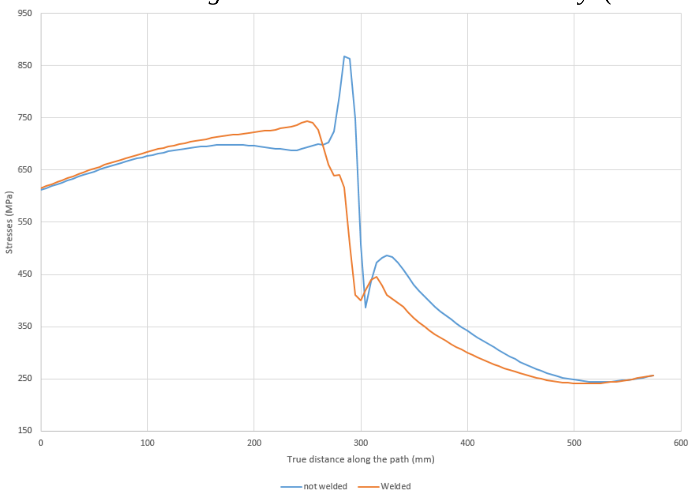


Figure 3.21 *Stress in member along horizontal direction. Full loading. (With middle stiffener)*

There is a big difference for stresses if plates are welded to the stiffener in this case. As the diagram shows, the peak of stress is at the corner zone, where 2 plates, stiffener and U channel intersect. Stresses are significantly reduced if the plates are welded together.

The middle stiffener is enough for the joint in tension only, as there is no yielding through the thickness of U channel. Small yielded zones are visible in the model, but they only appear on the outer surface of the plate and it does not affect the element highly. Middle stiffener can be applicable for joint in the middle of truss, where forces in diagonals are equal to 0 and the chord is subjected to maximum tensile force. Although the middle stiffener is suitable for tensile force only, there is a need to introduce a thicker plate along the side of tension chords. As the member is subjected to second load combination, the side plates start to yield.

Connection with 6mm U insert as stiffener

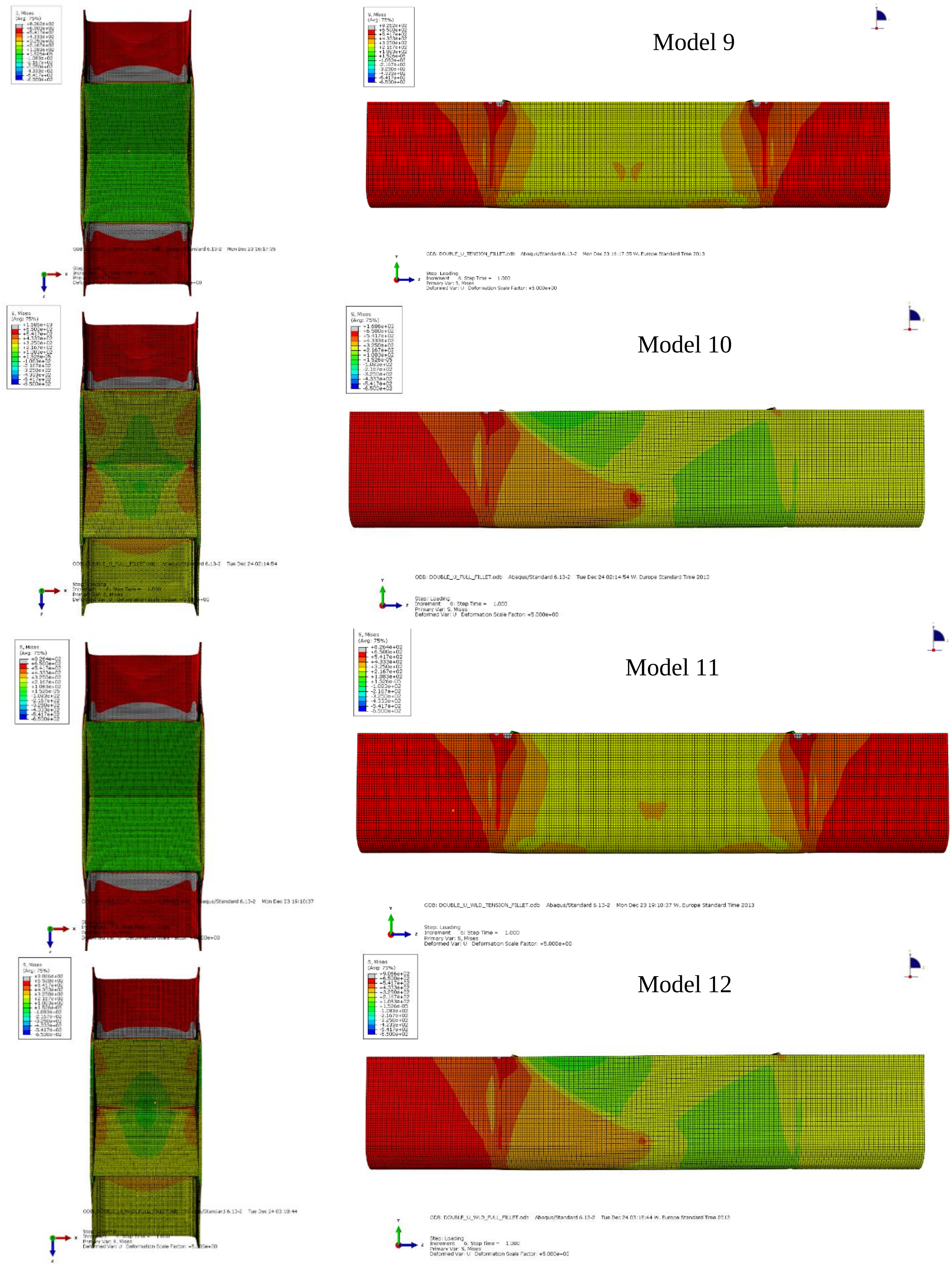


Figure 3.22 *Stress fields in joints with 6mm thick u-shaped stiffener*

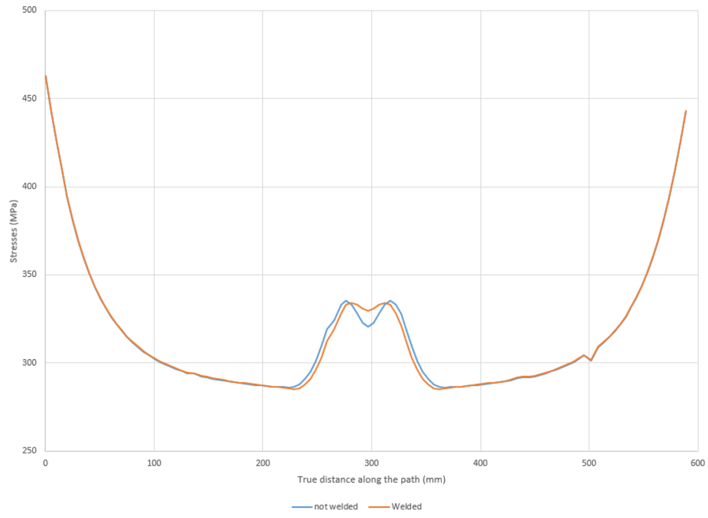


Figure 3.23 *Stress in member along horizontal direction. Tension only. (Connection with 6mm U insert)*

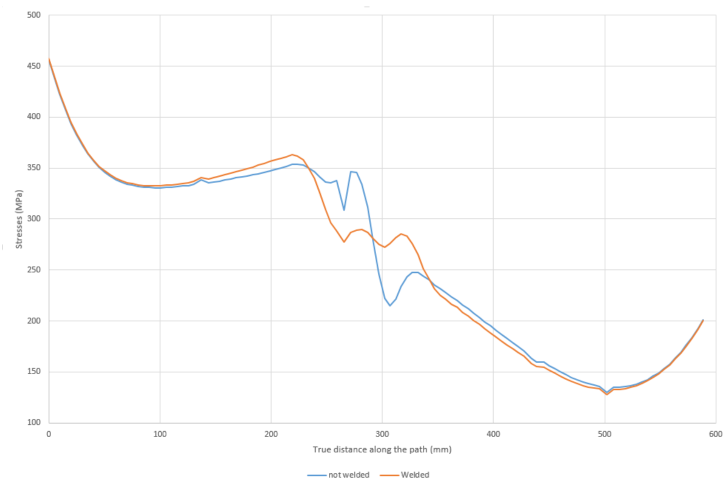


Figure 3.24 *Stress in member along horizontal direction. Full loading. (Connection with 6mm U insert)*

The U-shape stiffener reduces stresses in the chord a lot, but a few yield zones appear. One of them is the area in the U-chord where the stiffener is welded. There is a stress concentration zone along the path of the weld, but stresses exceed the limit value (650MPa) only in the outer surface (not through the thickness) and it is assumed that it does not affect the member strength.

A 6mm thickness stiffener is used for models 9, 10, 11 and 12. As the area of section doubled, there is no yielding in any of the two load combinations. The resistance of the member is sufficient, but this is a very conservative approach and the thickness of the U insert may be lowered.

Connection with 3mm U insert as stiffener

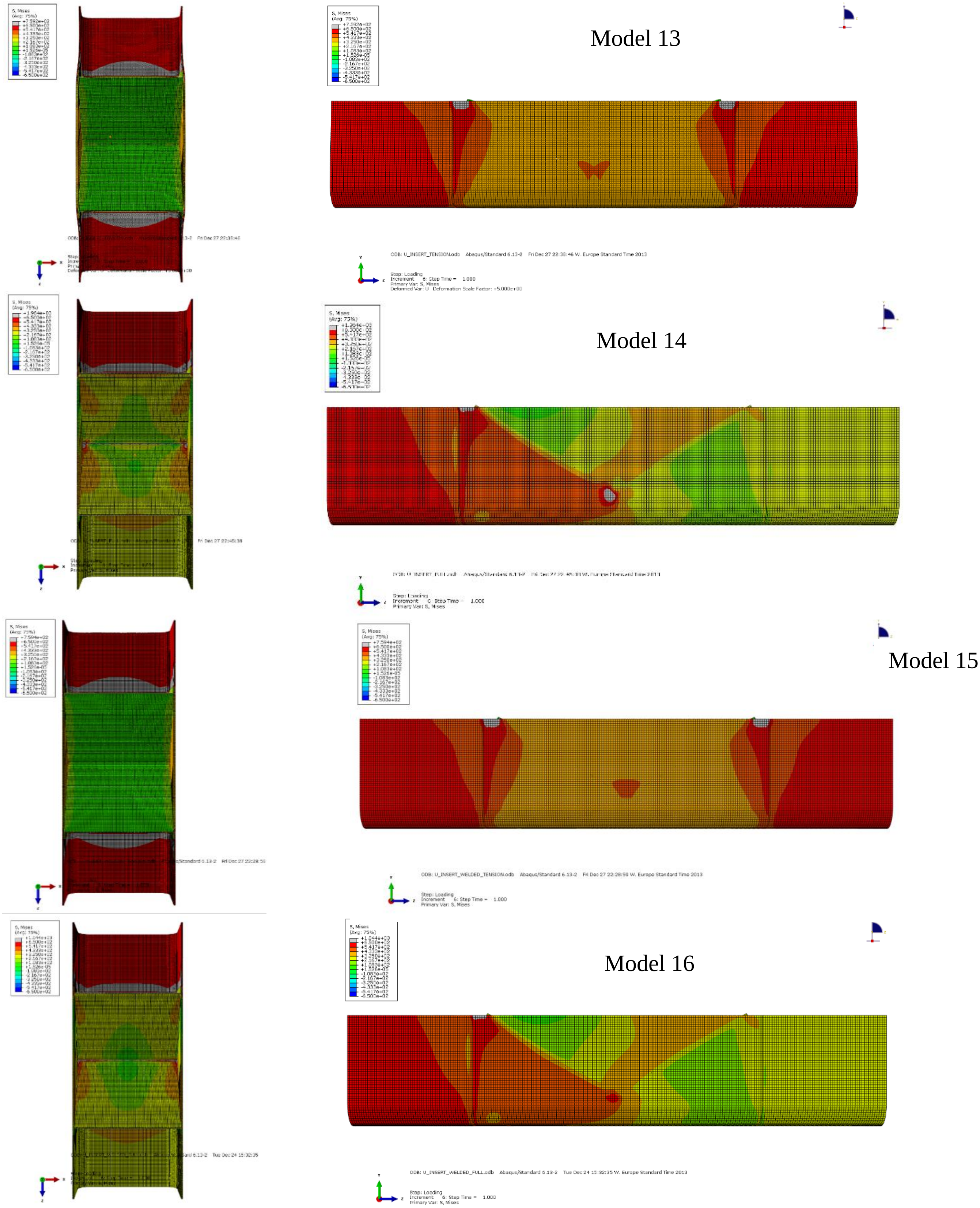


Figure 3.25 *Stress fields in joints with 3mm thick u-shaped stiffener*

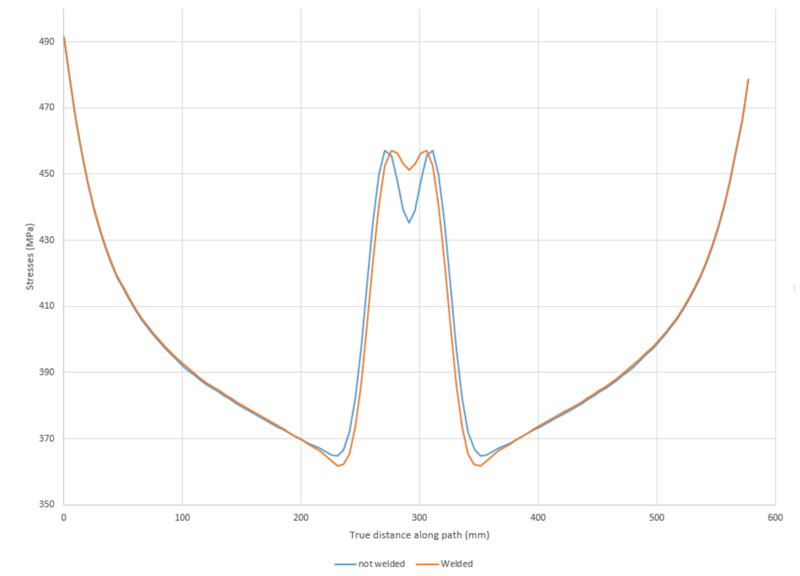


Figure 3.26 *Stress in member along horizontal direction. Tension only. (Connection with 3mm U insert)*

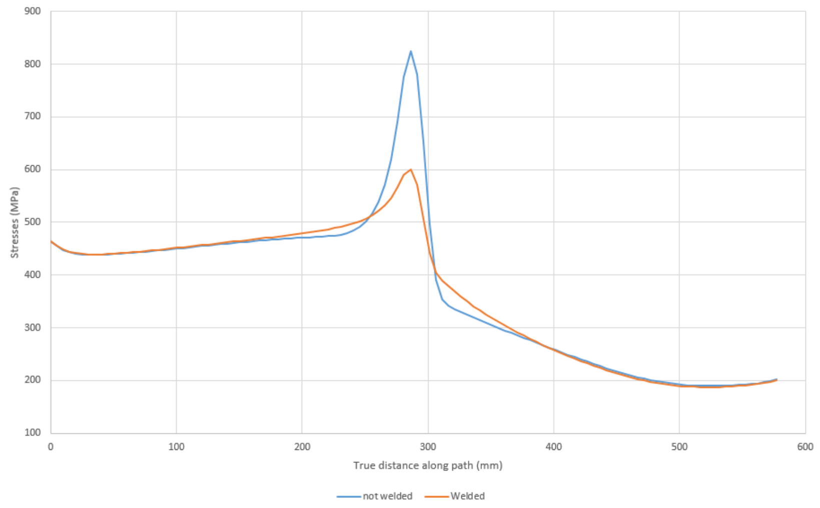


Figure 3.27 *Stress in member along horizontal direction. Tension only. (Connection with 3mm U insert)*

The 3mm stiffener also provides enough resistance for the chord. General behaviour of the connection remains the same as with the 6mm insert, but for the 3mm stiffener, stresses can exceed the yield limit if the plates are not welded together.

The conclusions and general comparisons of all types of connections are described in the following subchapter.

3.7 Conclusions

4 different connections (without stiffener - with stiffener between the plates - 6 and 3mm U shaped insert as stiffener) and several load variations, totally 16 models, were analysed in this chapter. The stress distributions in truss bottom chord both in z and y directions are given in figures below.

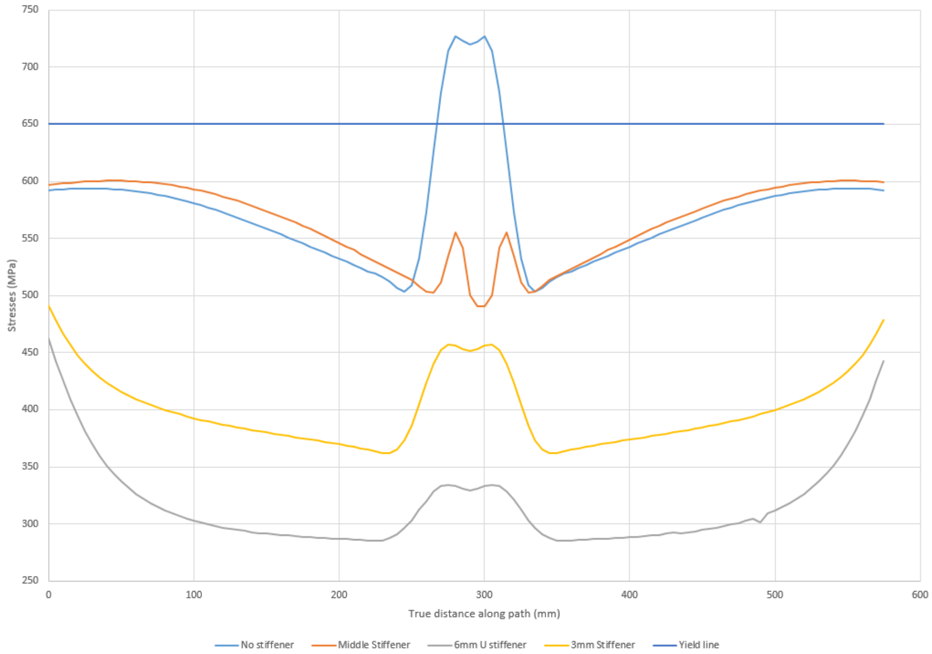


Figure 3.28 *Stress distribution in connections along longitudinal (z) axis. Tension only*

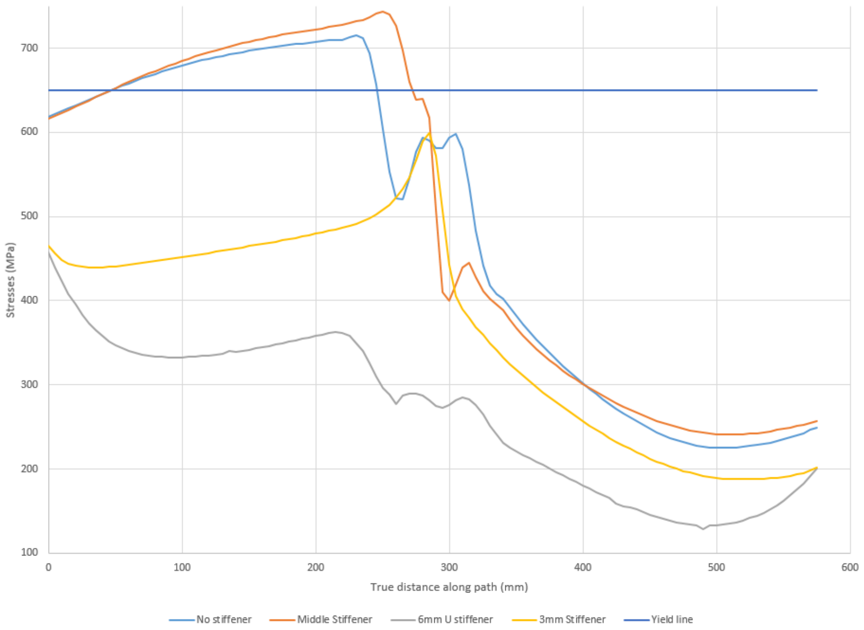


Figure 3.29 *Stress distribution in connections along longitudinal (z) axis. Full load*

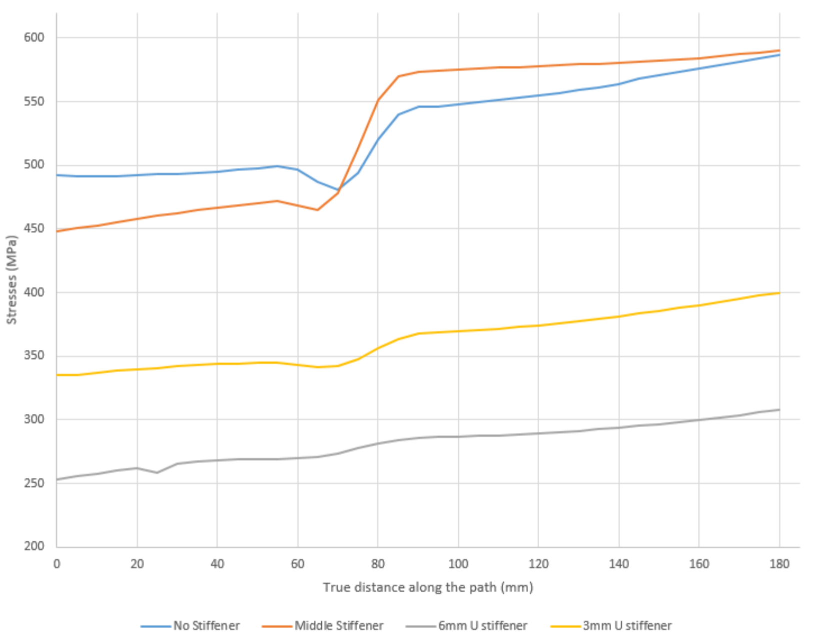


Figure 3.30 *Stress distribution in connections along vertical (y) axis. Tension only*

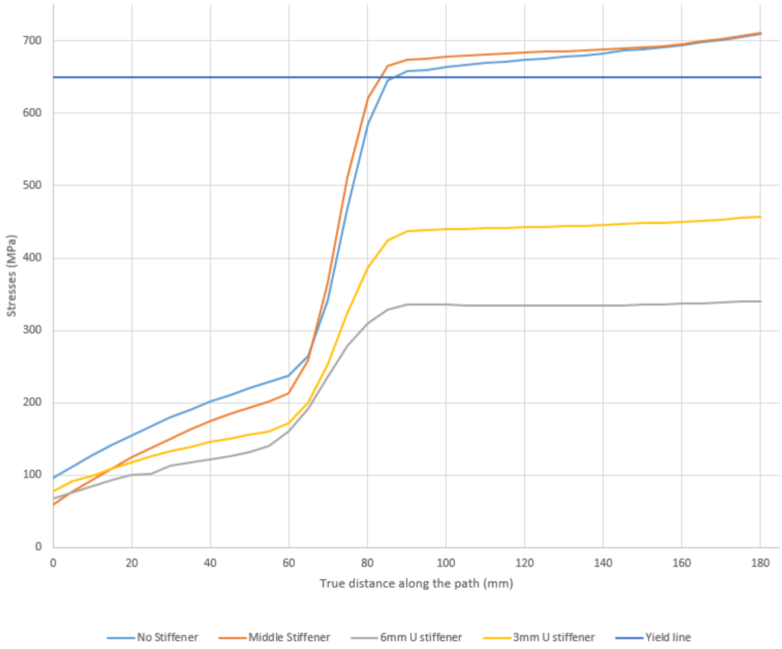


Figure 3.31 *Stress distribution in connections along vertical (y) axis. Full load*

Analysis shows that the connection behaves better and the stresses are distributed more evenly if the diagonal plates are welded together. This also stabilizes the connection.

As seen in the graphs, the connection without stiffener is not applicable in this design situation, since the stresses in the flanges of the chord exceed the yielding stress both if it is loaded in tension or fully loaded.

The middle stiffener can be used only if the section is in tension, as in the middle of the truss, where diagonals do not transfer loads. But if the connection is in biaxial stress state the U-shaped insert must be used, in order to increase the thickness of the chord.

Inserts made from 6 or 3mm plates were used for analysis. The 6mm plate gives huge safety for resistance and it can be a very conservative solution. On the other hand, the 3mm stiffener also provides the required resistance for the given loads and it would be the best solution.

There are limitations in the model with the U insert, because it is modeled by increasing the thickness of the U-profile, rather than a separate welded part.
In real life it would be very hard to achieve that and laboratory tests must be performed in order to see the real behaviour of the truss chord connection with diagonals.

4 BUCKLING ANALYSIS OF THE POLYGONAL CHORD

4.1 Introduction

Buckling behaviour of cold-formed semi-closed polygonal sections is analysed in this chapter. An alternative solution to designing top chords and diagonals for the truss is presented. Compressed members are built up sections from different cold-bended plates (see figure 4.1), which are connected by bolts along their length. This type of polygonal sections can be formed into different shapes and they are relatively inexpensive to produce in small series by brake forming, in order to meet any special design purposes.

Usually, cold formed profiles are open sections with very small torsional stiffness. This means that the resistance to global buckling is mostly governed by torsional or torsional flexural buckling. Since in this case the chords and diagonals are fully compressed, this would result in a very low resistance of the member.

Therefore, closed sections made from cold-formed members are designed, since it is a way of improving the resistance of the member by assembling them into one closed section. One of the aims of this analysis is to investigate the buckling shapes of the members in pure compression.

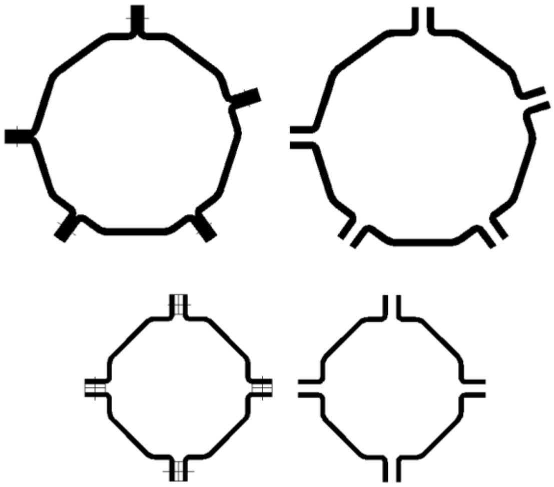


Figure 4.1 *Polygonal built-up sections for chords (top) and diagonals (bottom)*

Cold formed hollow sections are closed with longitudinal weld, but this solution is not feasible using coated or galvanised sheets. The main advantage of semi-closed polygonal profiles made from galvanised steel is that they facilitate easier connections with minimum welding. The section is called semi-closed because it is not continuously and rigidly connected. To connect bended polygonal plates pre-tensioned bolts are used along its length. Bolts should be distributed in a way that the whole member is working as a uniform element and not like single plates. In other words, the required spacing between the fasteners should be investigated.

Buckling and post-buckling behaviour of cold formed steel members are quite difficult to predict due to the material and geometrical non-linearity. Strength increase due to bending, geometrical imperfections and residual stresses are estimated and applied for non-linear analysis of the polygonal chord. Numerical analysis of the polygonal member (which is designed according to European standards in Chapter 2) was performed using ABAQUS software, for various parameters in the element. Results of the analysis are compared to the resistance obtained by Eurocode 3, parts 1-1 and 1-3 (see Chapter 2.)

4.2 Influence of cold work on mechanical properties of steel

The mechanical properties of cold-formed steel sections differs from those of the steel strip or plate before forming. Cold-forming operation of the steel section increases the yield stress and the tensile strength, but at the same time decreases the ductility of the material. The percentage increase of the yield stress is bigger than the increase of the ultimate strength. The strongest effect of increase of the material properties is at the corner level, where the effect of cold forming is the highest throughout the whole section. That means that the mechanical properties are different in various parts of the cross section, as in the corner parts the yield stress increases, while at the flat part it remains constant. Figure 4.2 shows the variations of mechanical properties at the specific locations in the channel section and a joist chord following the tests performed by Karren and Winter [29]. For that reason the buckling or yielding usually begins in the flat area of the section. Due to lower yield stress of the material, any other additional load will spread to the corners.

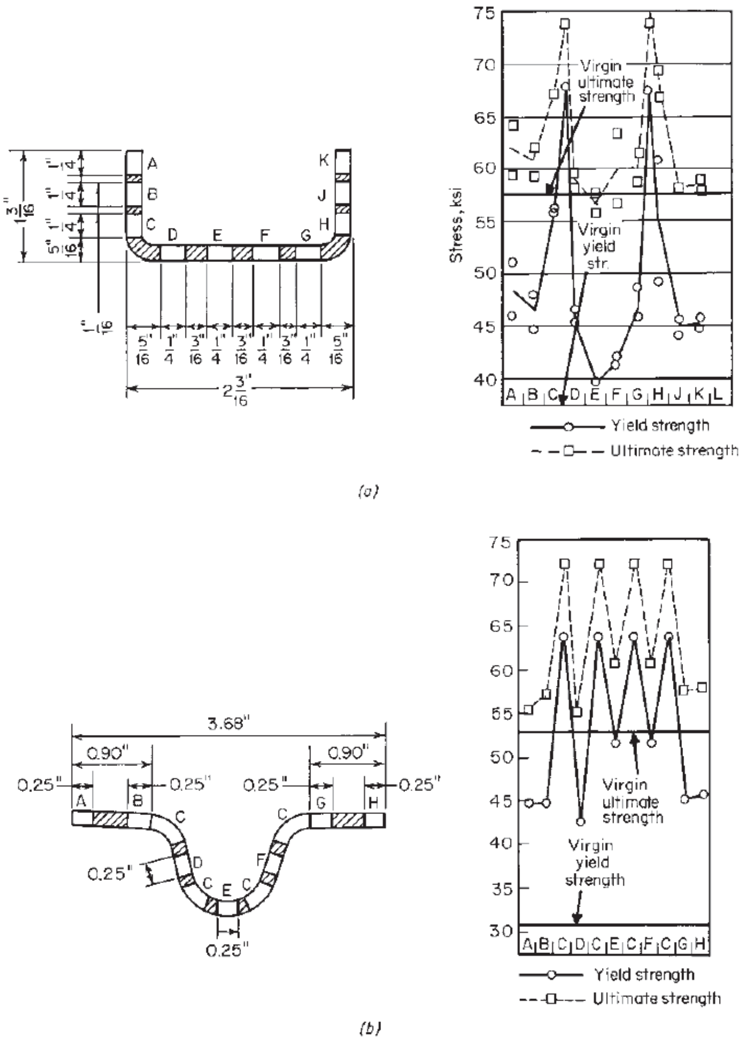


Figure 4.2 *Effect of cold work on mechanical properties in cold-formed steel sections*: a – channel section, b- joist chord *[29]*

The influence of bending was first-primarily investigated by Winter and Karren [29], and later on by Chajes, Britvec and Uribe [30]. After tests it was concluded that changes of the mechanical properties due to cold work are caused mainly by strain hardening and strain aging (see figure 4.3). Curve A represents the stress-strain curve of the base material, while curve B is due to unloading in the strain-hardening range, curve C shows immediate reloading and curve D is stress-strain curve of reloading after aging.

It is notable that the yield stresses of curves C and D are higher than the yield stress of the base material (flat part) and that the ductility decreases after strain hardening and strain aging.

Mainly, the effects of cold work on the mechanical properties of corners depend on:

- Type of steel;
- The types of stress (compression or tension);
- The direction of stress with respect to the direction of cold work (transverse or longitudinal);
- The F_u/F_y ratio;
- The inside radius-thickness ratio R/t;
- The amount of cold work.

From all the above factors, the F_u/F_y and R/t ratios are the most important factors that affect the change in mechanical properties of the sections. A material with big F_u/F_y ratio gives a large potential for strain-hardening. While the ratio increases, the yield stress of steel also increases. A small inside radius-thickness ratio R/t shows a higher level of cold work in the corner, what governs larger increase at the corner for smaller R/t ratios[31].

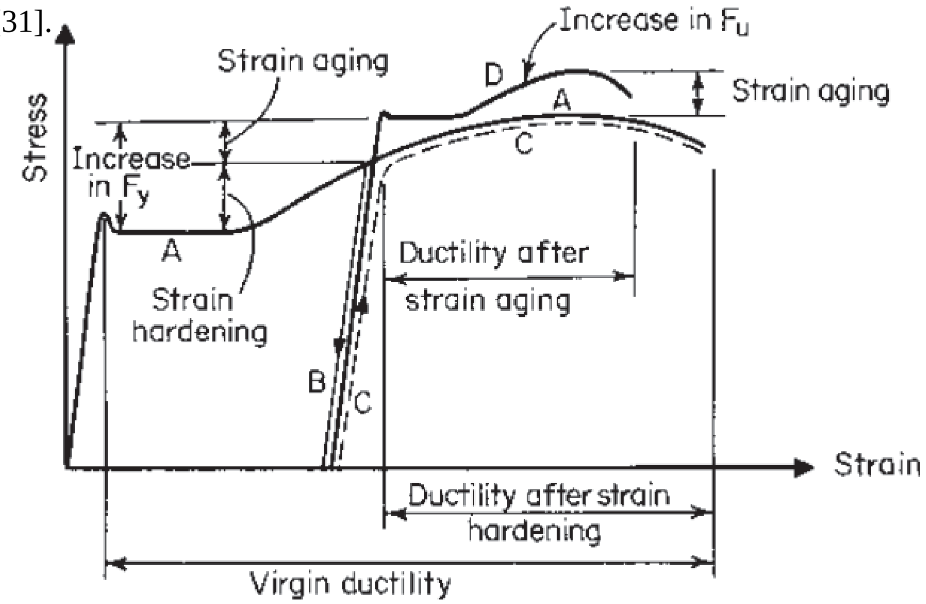


Figure 4.3 *Effects of strain hardening and strain aging on stress-strain characteristics[30]*

AISI approach. There are few propositions for calculating the increase of yield strength at corner zones. The research projects related to this matter began in the 1960's at Cornell University under the direction of Prof. G. Winter [30] with the assistance of other professors.

After experimental works Karren and Winter [29] proposed the following equation for the corner yield strength:

$$\frac{F_{yc}}{F_{yv}} = \frac{B_c}{(\frac{R}{t})^m}$$

Where the empirical coefficients:

$$B_c = 3{,}69 * \frac{F_{uv}}{F_{yv}} - 0.819 * \left(\frac{F_{uv}}{F_{yv}}\right) - 1.79$$

$$m = 0.192 * \frac{F_{uv}}{F_{yv}} - 0.068$$

where:

F_{yc} - yield stress at corner zone;

F_{yv} - yield stress for virgin material;

F_{uv} - tensile strength for virgin material;

R - inside bent radius;

t - thickness of the plate.

This equation was soon adopted by American Iron and Steel Institute (AISI) [32], whose specification gives the weighted average of yield stress in the whole section by the equation:

$$F_{ya} = CF_{yc} + (1 + C)F_{yf}$$

where:

$$F_{yc} = \frac{B_c F_{yv}}{(\frac{R}{t})^m}$$

C – ratio of corner area to total cross-sectional area.

Lind and Schroff [33] used the tests in [29] to develop a new expression for corner yield strength, as they said that the theory presented by Karren is "complicated and is not in a good agreement with the material behaviour". To develop less complicated models, they focused in analysing a linear strain hardening law and a simplified design rule based on hardening margin. For example, the difference between the virgin ultimate and yield strengths $(f_u - f_y)$ and strain hardening constant which would be the same for all materials[34]. Lind and Schroff explain their idea as: "The idea of the theory is simple. Whether a corner of a large or small radius is formed, the cold work, equal to the integral of the applied moment with respect to the angle of bend, should be about equal if strain hardening is linear. A small corner just concentrates the same work in a smaller volume of material. If the material hardens linearly, the work is independent of the radius, neglecting the elastic part. Further, if the increase in yield stress is a linear function of the work of forming, the increase in yield force for the corner will be a linear function of the work of forming". They did not carry any tests to prove this theory and just analysed and adapted previous test data. The hardening constant 5t was established and applied to the simple expression as follows:

$$\Delta P = 5t^2(f_u - f_y)(\frac{\Theta^\circ}{90^\circ})$$

The rule states that the yield strength is obtained by replacing the yield stress with the ultimate stress over an area of $5t^2$ at each 90° corner. Therefore, the corner yield strength can be calculated using equation:

$$F_{yc} = F_y + \frac{\Delta P}{area\ of\ corner}$$

S136 (Canadian norms) approach. Lind and Schroff [33] compared their calculations of the corner yield strength with the study of Karren and found good agreement. [33] shows that the increase of yield strength at a corner can be related to the strain hardening margin $(f_u - f_y)$ and the strain hardening constant *5t*. Canadian codes (S136) adopted Lind and Schroff expressions as a basic for calculating yield strength increase due to cold forming, as follows:

$$F_y' = F_y + 5D_A(F_u - F_y)/W^*$$

where:

F_y - virgin yield strength of steel;

F_u - ultimate yield strength of steel;

D_A - number of 90° corners or total number of degrees in the section divided by 90°

W^* - ratio of the centreline of a flange cross-section of a member in bending, or of the entire cross section of a tensile or compressive member, to the thickness (*w/t*).

Waterloo test program [35] was created in order to investigate the difference between AISI and S136 proposed methods. Tensile tests were performed with different bended and flat plates. Results were compared with theoretical calculations. Analysis showed that the results obtained by testing and by theoretical calculations (both by Karren and Lind/Schroff theories) were in good agreement and the results were almost identical. It was concluded that a simpler S136 method can be used in calculating the yield strength increase by cold bending.

Eurocode 3 approach. Eurocode 3 Part 1-3 [26] gives an equation to calculate the average yield strength f_{ya} for a full section. This equation is a modification of the formula used by AISI specifications, where a zone close to the corner is considered as fully plastified:

$$f_{ya} = f_{yb} + \left(C * n * \frac{t^2}{A_g}\right) * (f_u - f_{yb})$$

A_g is the gross cross sectional area and n is the number of 90° bends in the section, with internal radius $r < 5t$ and C is a factor depending on the steel forming method; $C = 7$ for cold-rolling and $C = 5$ for other methods of forming [26].

The average yield strength cannot exceed the boundaries:

$f_{ya} \leq 0{,}5 * (f_{yb} + f_u);$

or

$f_{ya} \leq 1{,}25 * f_{yb}$

The average yield strength f_{ya} can be used in numerical analysis. If test results are available, the input parameters to describe model should be used directly from tensile coupon tests.

High Strength Steel applications. As seen above, the current design codes of cold-formed steel structures have solutions for material properties change in the bended zones. However, all proposed methods are based on investigation of normal strength cold-formed steel. Nowadays, cold-formed high strength steel having yield strength greater than 450MPa is used more often in construction. This results in more researches towards cold-formed HSS to be carried out.

Chen and Young [36] tested a series of high strength cold-formed steel flat coupon specimens at normal and elevated room temperatures. Tests included flat coupon specimens having the normal (virgin) yield strength of G450MPa and G550MPa. In addition, two kinds of corner coupon specimens - inner and outer corners made of G450MPa were tested (see figure 4.4).

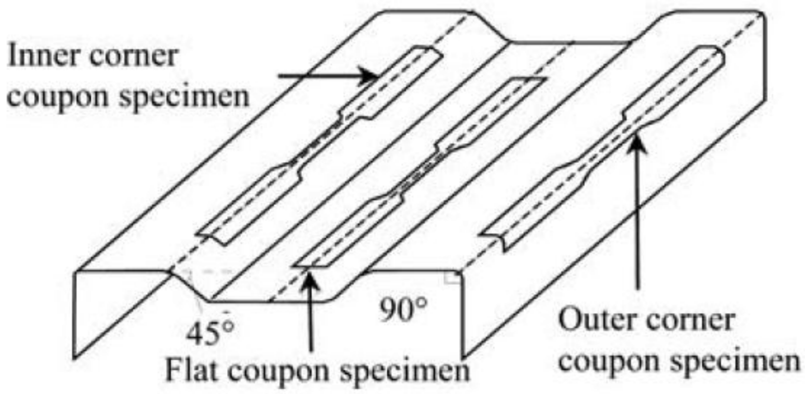


Figure 4.4 *Coupon specimen[36]*

Chen and Young used AISI method (calculation of yield strength increase at corner zones developed in [29]) as a base for their investigation. The method and empirical coefficients used by AISI are developed and based on normal strength cold-formed steel. [36] obtained the new strength coefficient k and strain-hardening exponent n, by measuring the stress-strain curves. They obtained new empirical coefficients:

For G550 steel:

$$B_c = 3{,}65\left(\frac{F_{uv}}{F_{yv}}\right) - 0{,}728\left(\frac{F_{uv}}{F_{yv}}\right)^2 - 1{,}75,$$

$$m = 0{,}171\left(\frac{F_{uv}}{F_{yv}}\right) - 0{,}073;$$

For G450 steel:

$$B_c = 3{,}70\left(\frac{F_{uv}}{F_{yv}}\right) - 0{,}728\left(\frac{F_{uv}}{F_{yv}}\right)^2 - 1{,}88,$$

$$m = 0{,}171\left(\frac{F_{uv}}{F_{yv}}\right) - 0{,}08;$$

The proposed model accurately predicted the corner strength enhancement in comparison with the experimental results.

4.3 Geometrical imperfections and residual stresses

To perform a proper geometrical non-linear analysis, some kind of disturbances in the member shape and material properties must be considered, in order to simulate a real behaviour of the member. Various characteristics should be analysed and taken into account if the strength of the member is studied. In the case of cold-formed sections, these characteristics are:

- geometrical imperfections, locally and along the member;
- residual stresses and change of yield strength due to cold forming effect.

The magnitude of the imperfection in the member, depends of the shape of the buckling mode, which can be obtained by eigenbuckling analysis of the compressed member. Usually, the geometrical imperfections are introduced in numerical models using equivalent sinusoidal shapes, with half-wavelength corresponding to the instability mode. Maximum measured imperfections can be conservatively used as an amplitude of the sinusoidal shape.

Imperfections of cold-formed steel members include bowing, warping and twisting, also local deviations and bar deflections. The polygonal sections designed in this work should be prevented from torsional buckling (bowing, warping, twisting) while the main issue remains the flexural buckling of the whole member and local buckling of the plate.

In the concern of sinusoidal imperfections (bar deflections), the magnitude of **1/1500** times of the member length L is proposed. This value corresponds to a statistical mean of imperfections of carbon steel columns, as Bjorhovde [37] suggests. Otherwise, the more conservative value of **L/1000** is proposed by ECCS recommendation [38]. For local imperfections in the plate, usually the value **b/200** is taken. *b* corresponds to the width of the plate.

Another effect which occurs in the member corners by cold-forming is residual stress. It is complicated to adequately model residual stresses in the analysis. Lack of data makes selecting an appropriate magnitude difficult. As a result, residual stresses are often excluded from the analysis or the stress-strain behaviour of the material is modified to approximate the effect of residual stresses.

Residual stresses in hot rolled members do not vary throughout the entire thickness – membrane residual stresses are dominant in this case. Flexural residual stresses dominate in the cold-formed members, through thickness variation. This variation of residual stresses can lead to early yielding on the faces of the cold-formed steel plates and can influence their local buckling strength [39].

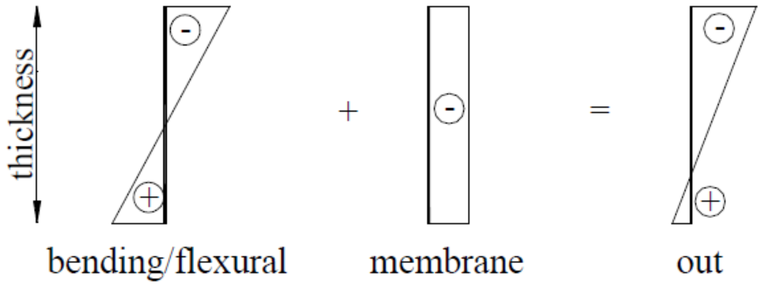


Figure 4.5 *Idealisation of residual stress[39]*

The idealised scheme of the residual stresses is shown in figure 4.5, but the experimental results show way more complex actual distribution of the residual stresses.

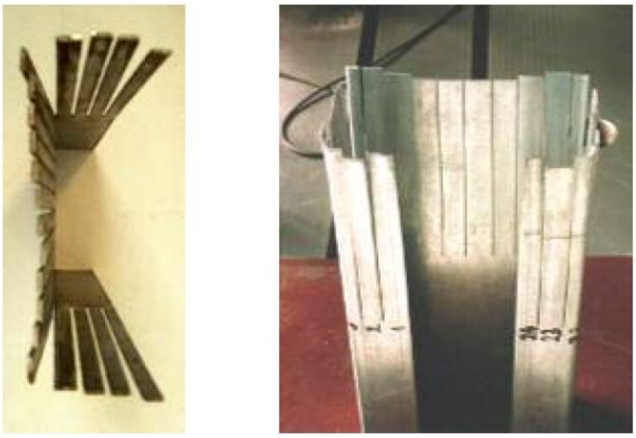


Figure 4.6 *Residual flower for plain and lipped channel sections[40]*

When residual stresses are applied in numerical analysis, an increase of the yield strength in the corner zones must be also included, because the change of yield strength has an opposite distribution compared to the residual stress and they compensate each other. For that reason, if the change of yield strength over the member is not considered in the analysis, residual stresses should also be neglected, so in the ULS analysis they can both be neglected and the approach will be safe. However, for thin walled sections, the effect of geometrical imperfections is far greater than the residual stresses [41].

4.4 Method

The main goal of this analysis is to obtain and compare the ultimate resistances of the truss polygonal chord subjected to compression force using different material properties. Stability of a member can be calculated by its critical load.

Eigenvalue linear buckling analysis is generally used to estimate critical buckling load of the ideal structures. For hinged bar, Euler formula for calculation of buckling load is:

$$F_{cr} = \frac{\pi^2 * E * I_{min}}{l^2}$$

where F_{cr} is critical (Euler) buckling load, E is the elastic modulus of material, I_{min} is the minimal moment of inertia of the cross-section and l is the buckling length of the structure.

Linear eigenvalue buckling analysis is performed in Abaqus software, where the critical buckling shapes of the member are obtained. It is ideal if the whole build up section is working as a solid member. This means that the bolts should be placed at proper distances in order to prevent the buckling of the single plates and keep the member working as one. The most crucial eigenshapes should be for the flexural buckling.

For the numerical plastic analysis, RIKS method is used. The RIKS method is generally used to predict the unstable, geometrically nonlinear collapse of a structure. Geometrical nonlinear static problems include buckling and collapse behaviour where the load-displacement response shows negative stiffness and the structure starts to release strain energy to stay in an equilibrium state. RIKS method uses load magnitude as additional unknown and it solves both for loads and displacements. Unstable problems can result in the load-displacement response as it is shown in figure 4.7. During the periods of response the load and displacement may decrease as the solution evolves. This behaviour can be caused by the start of material yielding [42].

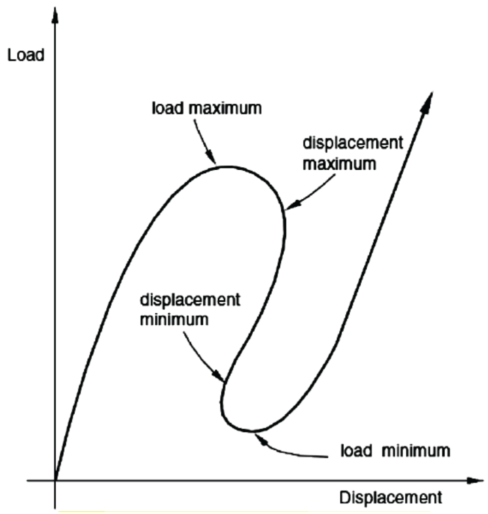


Figure 4.7 *Typical unstable static response[42]*

To trigger the buckling behaviour and simulate more realistic models, geometrical imperfections must be applied to the numerical model. The buckling shape for RIKS analysis is used from the previous linear buckling analysis. According to the most critical (the lowest critical load) eigenshape, the magnitude of imperfection is applied. The magnitude for different imperfection types (global, local) is described in codes and overviewed in the chapter above.

Nominal values are used for the elastic characteristics of steel – the Young's modulus is 210GPa and Poisson's coefficient is 0,3. Also, plastic material data should be introduced for the non-linear analysis in Abaqus. Using the characteristic yield strength and the characteristic ultimate strength of the material, the nominal strains can be calculated as shown in table 4.1.

Table 4.1 *Nominal Stress, Strains and Plastic Strains for S355 steel*

Stress (MPa)	Strains		Plastic strains
0	0	0	-
Fy = 355 MPa	$\varepsilon_1 = \frac{F_y}{E}$	0.00169	0
Fy = 355 MPa	$\varepsilon_2 = 0{,}025 - 5 \cdot \frac{F_u}{E}$	0.01286	0.0111
Fu = 510 MPa	$\varepsilon_2 = 0{,}02 + 50 \cdot \frac{F_u - F_y}{E}$	0.05690	0.0528
Fu = 510 MPa	∞	0.20	0.1794

Plasticity data in Abaqus should be defined as plastic true stress and plastic true strain. The following formulae are used to calculate input values for Aqabus simulations.

$$\varepsilon_{true} = \ln(1 + \varepsilon_{nom})$$

$$\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom})$$

$$\varepsilon_{true}^{pl} = \varepsilon_{true} - \frac{\sigma_{true}}{E}$$

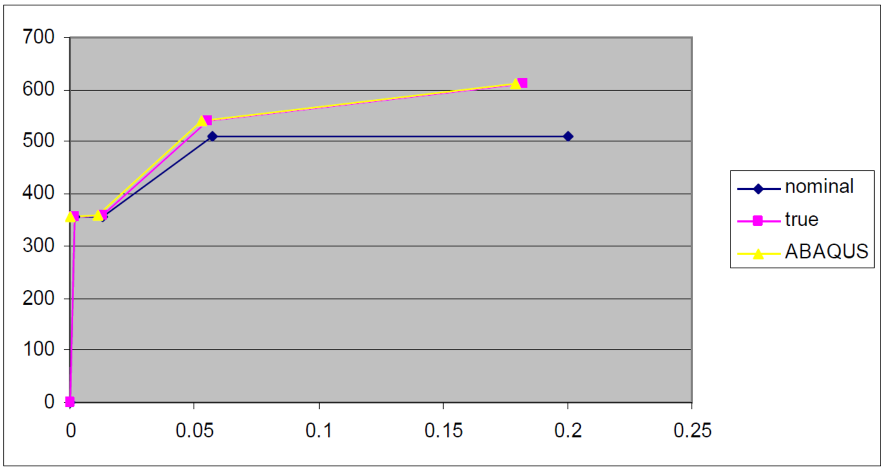


Figure 4.8 *The comparison between nominal data and true material data[43]*

True stresses and strains are calculated for the basic S650 steel, for increased yield strength average according EC3-1-5 and for increased yield strength at corner zones by improved AISI approach for HSS. The ultimate load by plastic analysis (RIKS method) for all 3 configurations is compared in this chapter.

Unfortunately, there are no real test results of analysed members to compare and verify the numerical model. However, buckling calculation of the designed compression chord according the Eurocodes is used to compare the obtained results by numerical simulations. Detailed design and description about compressed member is given in Chapter 2.

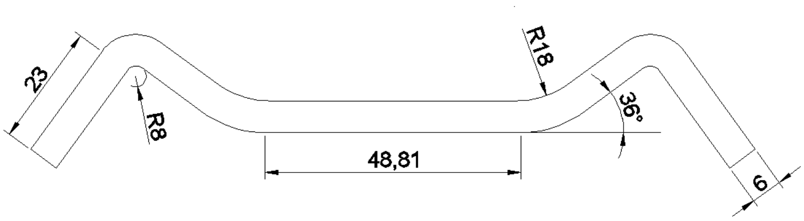


Figure 4.9 *Dimensions of the single plate*

The yield strength increase in the corner zones is calculated by Eurocodes and the method given in [29].

Eurocode [26] gives an equation for the average yield strength in cold-formed members:

$$f_{ya} = f_{yb} + \left(C * n * \frac{t^2}{A_g}\right) * (f_u - f_{yb})$$

$f_{yb} = 650MPa$ nominal yield strength of steel grade S650;

$f_u = 700MPa$ tensile strength of steel grade S650;

$C = 7$ numerical coefficient for roll forming;

$n = \frac{2*90+2*36}{90} = 2,8$ the number of 90 degree bends in the cross section with internal radius $r \leq 5t$;

$A_g = 9.069cm^2$ gross area;

$t = 6mm$ thickness of a plate.

$$f_{ya} = 650MPa + \left(7 * 2.8 * \frac{6^2mm}{906.9mm^2}\right) * (700MPa - 650MPa) = 688,902MPa$$

but

$$f_{ya} \leq \frac{f_u + f_{yb}}{2} = 675MPa$$

The average yield strength in cold formed members according to the Eurocode [26] is $f_{ya} = 675MPa$.

Karren and Winter [29] proposed a formula where it is possible to calculate strength increase in each corner of cold-formed section.

$$F_{yc} = \frac{B_c F_{yv}}{(\frac{R}{t})^m}$$

[36] proposed new empirical coefficients for this equation based on experiments performed for HSS. They are calculated as follows:

$$B_c = 3{,}65\left(\frac{F_{uv}}{F_{yv}}\right) - 0{,}728\left(\frac{F_{uv}}{F_{yv}}\right)^2 - 1{,}75$$

$$= 3{,}65\left(\frac{700MPa}{650MPa}\right)^2 - 0.728\left(\frac{700MPa}{650MPa}\right) - 1.75 = 1.336$$

$$m = 0{,}171\left(\frac{F_{uv}}{F_{yv}}\right) - 0{,}073 = 0{,}171\left(\frac{700MPa}{650MPa}\right) - 0{,}073 = 0.111$$

For 90° bent:

$$F_{yc\,90} = \frac{B_c F_{yv}}{(\frac{R}{t})^m} = \frac{1.336 * 650MPa}{(\frac{8mm}{6mm})^{0.111}} = 841.108MPa$$

For 36° bent:

$$F_{yc\,36} = \frac{B_c F_{yv}}{(\frac{R}{t})^m} = \frac{1.336 * 650MPa}{(\frac{18mm}{6mm})^{0.111}} = 768.704MPa$$

For the flat parts the nominal yield strength of $F_{yf} = 650MPa$ is used.

Nominal material parameters are recalculated to true stresses and true plastic strains for the numerical analysis.

Table 4.2 *Material properties for input to numerical model*

S650		EC3		Karren/Winter			
Average		Average		90 degree bent		36 degree bent	
True stress	True plastic strain	True stress	True plastic strain	True stress	True plastic strain	True stress	True plastic strain
652	0	677.2	0	844.4	0	770.8	0
655.4	0.0052	680	0.0041	845.2	0.001	771.8	0.0013
722.3	0.028	766.3	0.0312	930.6	0.029	871.2	0.0323
840	0.1783	888	0.1781	1080	0.1772	1008	0.1775

4.5 Modelling in Abaqus

The top chord is modelled in Abaqus by the procedure given in Chapter 2.

For the modelling, shell type elements are used. The model is assembled from 5 separate bended plates. The thickness of the plates is 6mm (to meet required resistance, see chapter 2.)

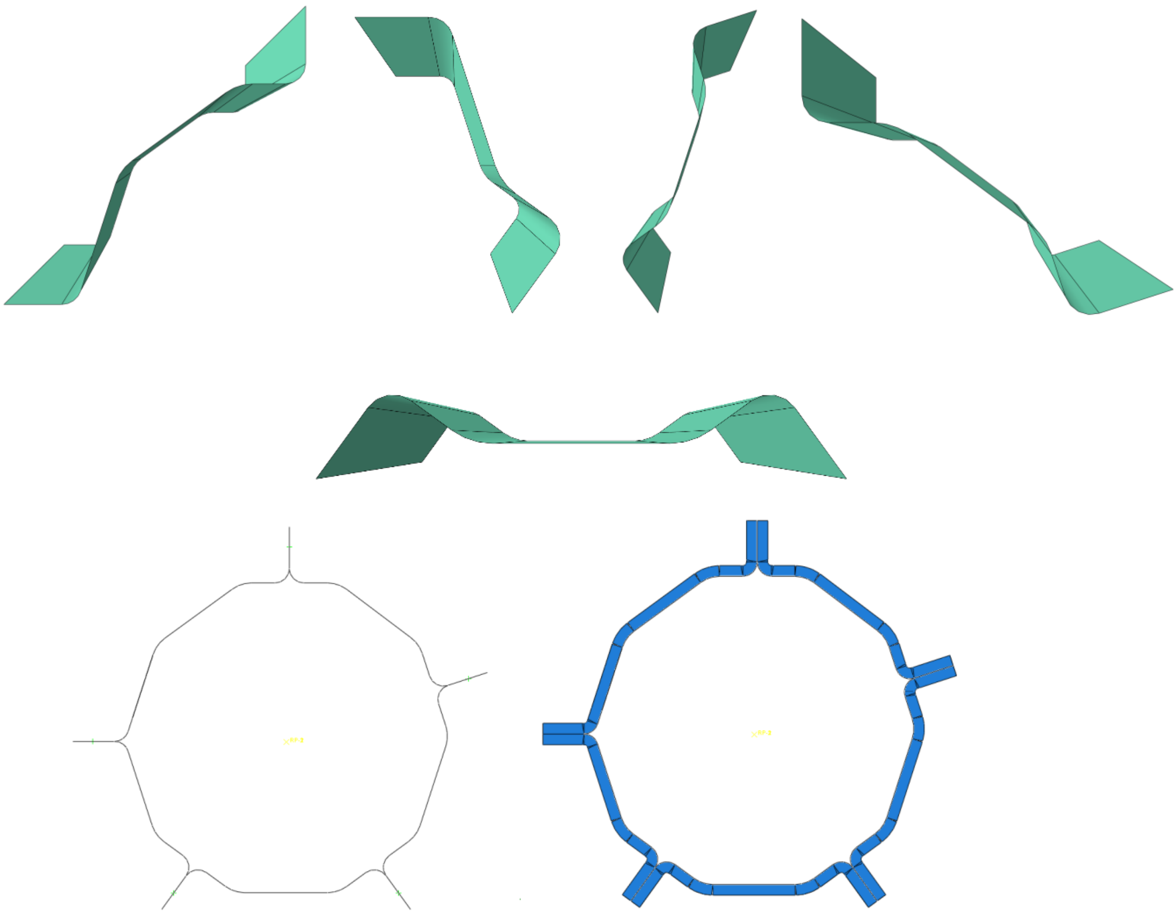


Figure 4.10 *Shell plates and assembled section*

The thickness of the shell element extrudes towards the outer side of the faces. This allows even assembly, without the intersection of the shell members. Rendered shell thickness in assembled members is shown in figure 4.9.

Plates are assembled using point based fasteners along the flanges of bended plates. Fasteners should provide the required stiffness to the element such that it would behave like a solid member, not as single plates. For this reason a proper spacing between the bolts should be established. The first test bolt spacing is taken as 500mm. Totally there are 9 bolts per one path in the member. If the selected spacing is not appropriate, for the next analysis the spacing is reduced. For more detailed results, see further chapter.

Figure 4.11 *Assembly by fasteners in Abaqus*

Sections are meshed by S4R elements. The seed sizing is 5mm, since a small mesh allows to obtain more accurate results.

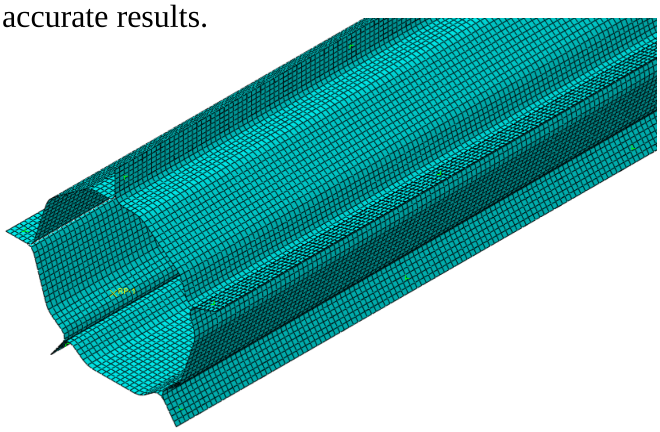


Figure 4.12 *Meshed section*

Boundary conditions are applied on two reference points. Points are placed at the centre of gravity of the build-up section and they are connected to the member using kinematic coupling constraint. The length of the element is 4000mm (length between two diagonals connected to the chord). Boundary conditions are pinned in one end (U1, U2, U3, UR3 restrained), and free in the longitudinal direction at other end (U1 and U2, UR3 restrained).

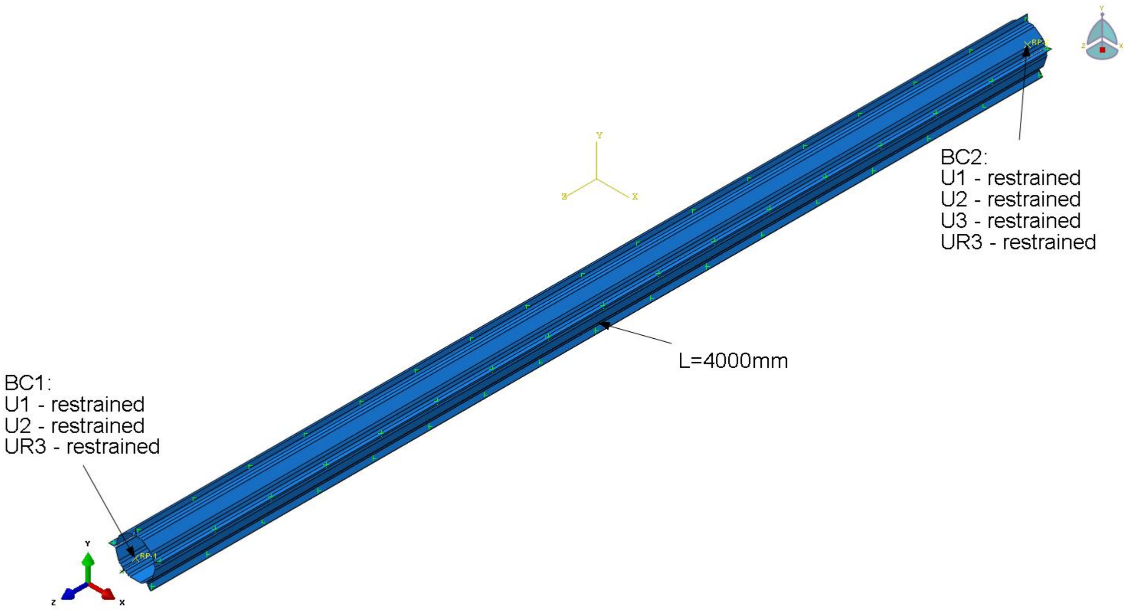


Figure 4.13 *Boundary conditions on the polygonal model*

Two types of analyses are performed for one model. First is the linear buckling analysis in order to obtain different buckling shapes and critical loads. For the linear buckling analysis 1kN load is applied. The obtained eigenvalue corresponds to the critical buckling load. The displacement data for different buckling shapes is written to the node file, which is used for RIKS analysis.

Secondly, non-linear analysis is performed in order to obtain the ultimate load of a member. Plastic material properties are introduced for this analysis. The true stress and strain values for different models are given in table 4.2.
In addition to the plastic material properties, buckling shape from the linear analysis and amplitude of the imperfections are introduced by editing the keywords file.

The outcome of RIKS analysis is a load-displacement curve, which shows the ultimate load of the member. The curve is obtained by generating analysis data and plotting the displacement of the free edge and the reaction force of the support in one graph.

4.6 Analysis and results

First of all, linear buckling analysis is performed for built-up sections where spacing between the bolts is 500mm. Buckling analysis shows (see figure 4.13) that the spacing between bolts is too long and the plates buckle as single elements, not as a solid member. Local buckling occurs in the first two buckling modes, while the flexural buckling mode is not governing.

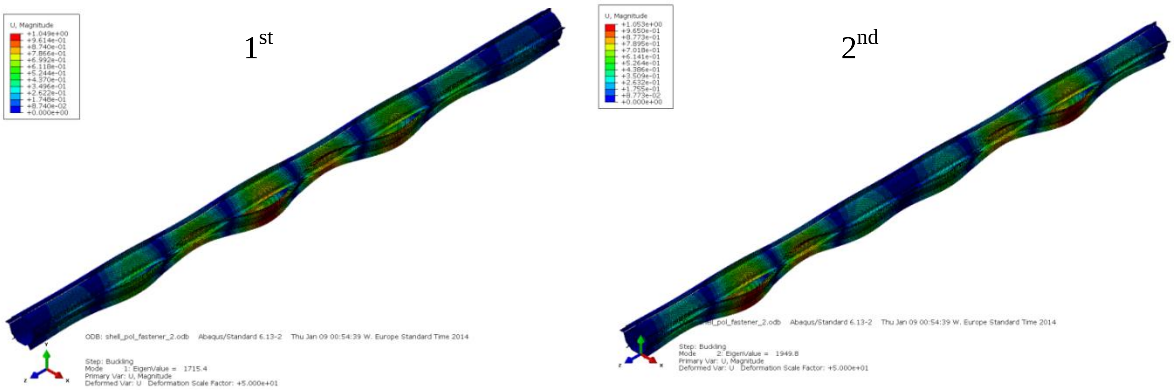


Figure 4.14 *First two buckling modes when spacing of the bolts is 500mm*

To achieve a more rigid body the spacing between bolts is reduced. At this model fasteners are placed at a 250mm distance from each other. Buckling analysis shows that the member is working as one solid built-up section. Flexural buckling firstly appears in the member. That results in a big increase of the critical buckling load of the member. The first four buckling shapes are shown in the figure below.

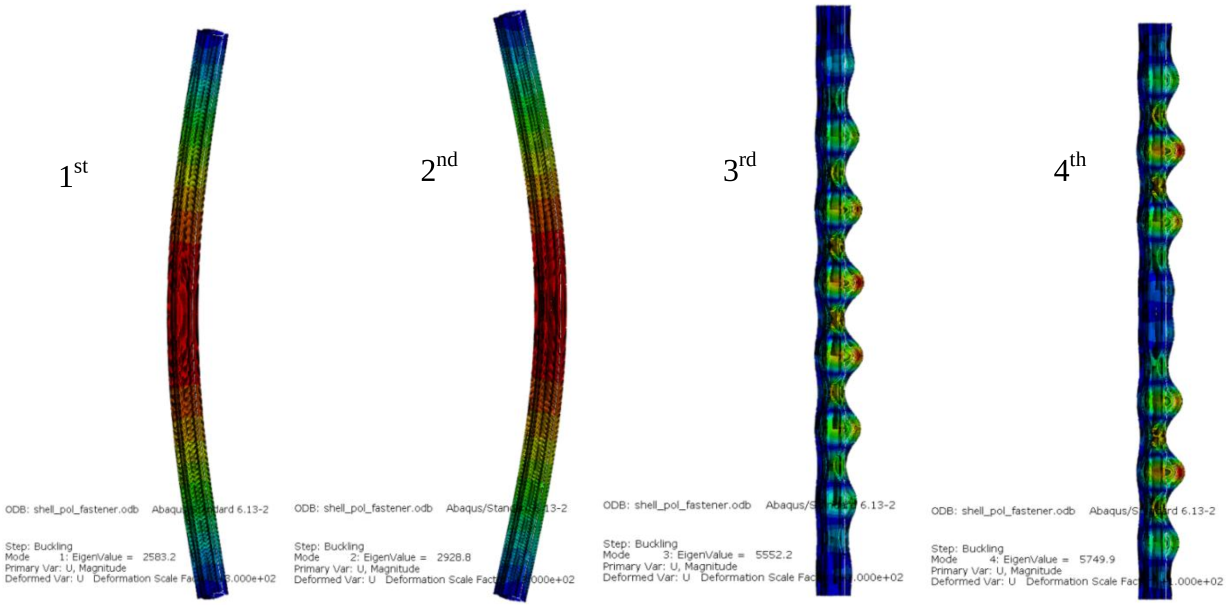


Figure 4.15 *Four buckling modes for polygonal section (spacing of bolts is 250mm)*

Non-linear RIKS analysis is performed for 3 models with different material properties:

1. Basic S650 steel material properties applied;
2. Average yield strength increased because of the cold-forming; calculation according to EC 1993-1-3;
3. Yield strength increased only in corner zones, calculation according to Karren/Winter method.

Plastic material properties for each model are given in table 4.2.

Shape of the imperfection is applied as 1st mode (flexural buckling) according to the linear buckling analysis. According to the ECCS recommendations, the amplitude of the imperfections is **l/1000=4mm**. In order to perform the analysis, 1mm displacement in longitudinal direction is applied as a force.

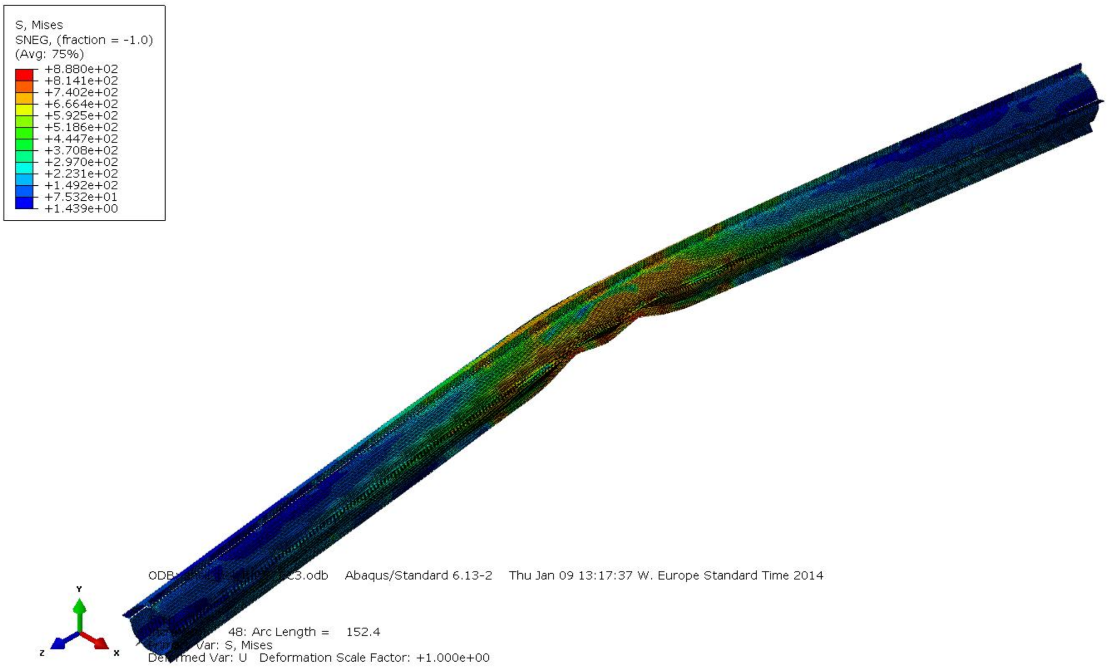


Figure 4.16 *The deformed shape of the polygonal section (plastic analysis)*

The load-displacement curves for the analysed models are given in figures below.

The ultimate resistance for S650 steel is **2077,99kN**.

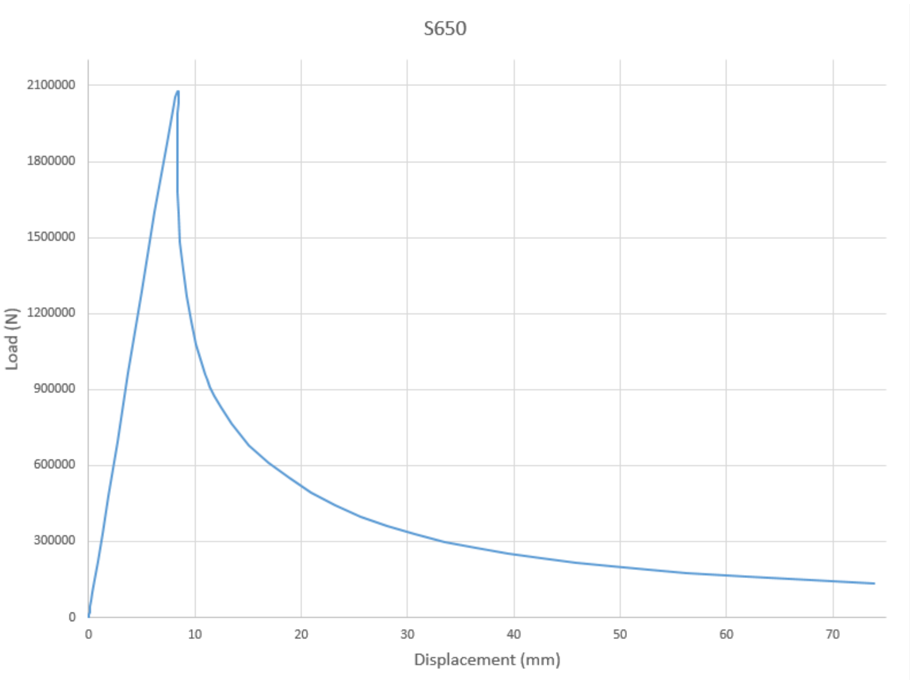


Figure 4.17 *Load-displacement curve (S650)*

The average yield increase given by Eurocode shows a bigger resistance – **2108,81kN.**

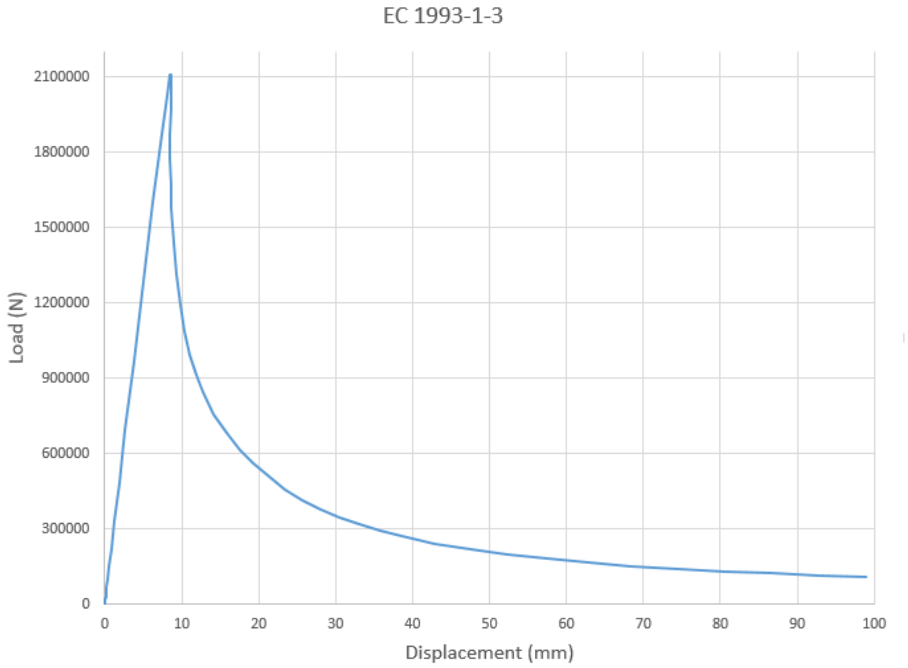


Figure 4.18 *Load-displacement curve (increased yield strength by EC 1993-1-3)*

When the yield strength increases only at corner zones, the maximum resistance of member is **2089,64kN**.

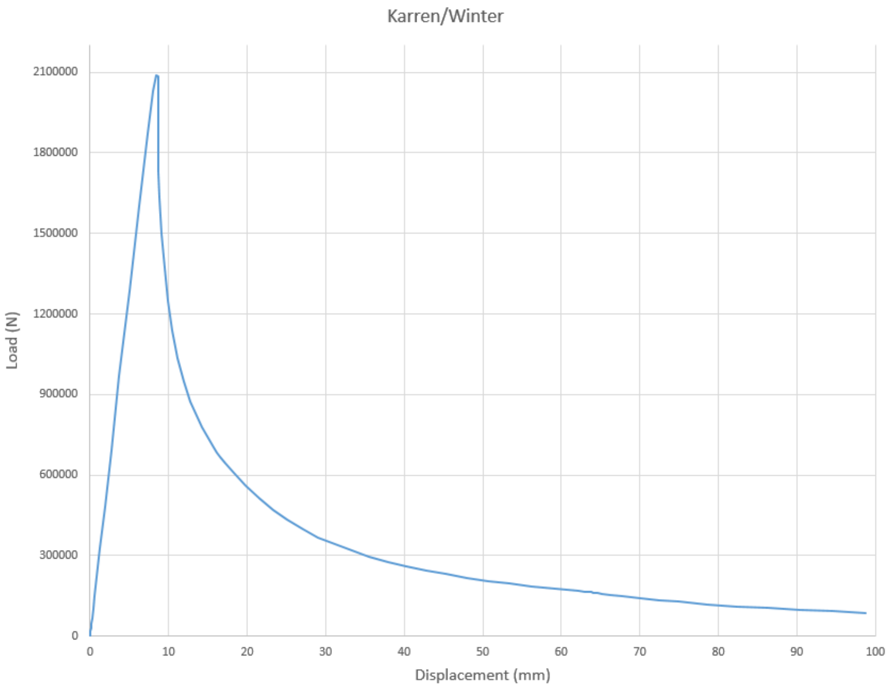


Figure 4.19 *Load-displacement curve (yield strength increased at corner zones only)*

Results gained by numerical analysis are compared to the hand calculations performed according to the Eurocode 1993 standards. The design procedure and description for polygonal chord are given in Chapter 2 and Annex B.

4.7 CONCLUSIONS

The linear buckling analysis and plastic RIKS analysis were performed for the built-up polygonal section. The main objective is to determine the ultimate buckling load for different material properties in the member, in order to investigate the effect of the yield strength increase that occurs in cold-bent sections corner areas. The different proposed methods of calculating yield strength increase are taken into account. Results obtained by numerical analysis are compared to the hand calculations according European design rules. The results are given in table 4.3.

Table 4.3 *Ultimate buckling loads*

Ultimate buckling load (kN)				
Numerical analysis			Design calculations by EC3	
S650	Average yield strength	Yield strength increased at corners	Buckling curve "c"	Buckling curve "a"
2077.99	**2108.81**	**2089.64**	**1805.721**	**2223.861**

Figure 4.20 shows the comparison between all numerical models and the ultimate resistance calculated according to the Eurocode.

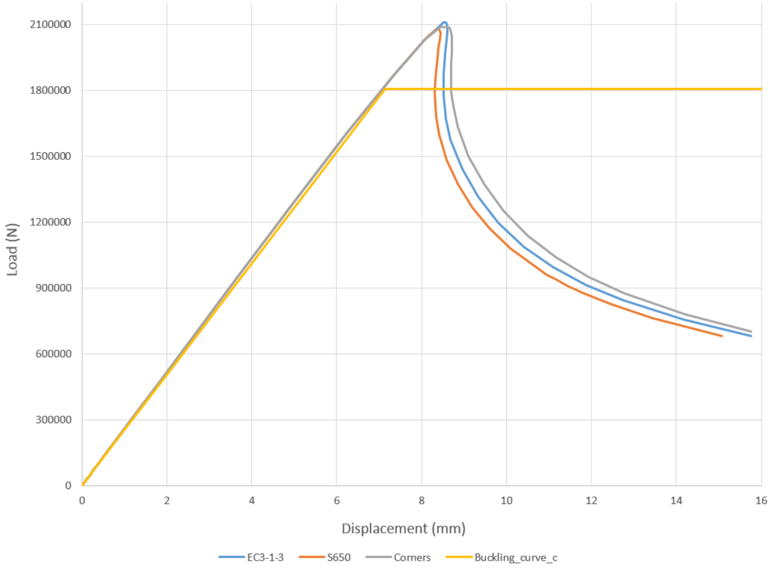


Figure 4.20 *Comparison of load-displacement curves*

Results show that the yield strength increase in the bent zones does not influence the buckling resistance of the member highly. There is just a slight difference between different resistances. The higher average yield strength calculated by Eurocode 3-1-3 gives the best resistance compared to the basic S650 steel properties and higher yield stresses at the corner zones. In conclusion, for the compressed member the variation of material properties does not matter so much as geometrical imperfections, boundary conditions or as the slenderness of member.

Hand calculations give lower results than FEM analysis. The main factor that influences this reduction is the buckling curve. For cold-formed members the Eurocode suggests to use buckling curve “c” which in the investigated case gives more conservative result when compared to the FEM analysis. Buckling curves “a” or “b” are more similar to the results obtained by FEM.

It is hard to predict the real behaviour of built-up polygonal cold-formed member, since no laboratory tests were performed for the analysed section. Therefore, the FEM model was not verified with any experimental test results. Compressed polygonal and circular plates were tested at the laboratory, but unfortunately they did not match the type of elements analysed in this thesis. Moreover, the yield strength increase at corner zones cannot be predicted only by calculations. Tensile coupon tests should be performed in order to get the real properties of different bent angles. There are plans to test built up sections and make coupon tests for bent plates in order to gain the real stress-strain relations for the material.

5 Comparative Study from an Economical and Environmental Point of View

A few decades ago, whenever designing a structure, the main concern of the civil engineering industry was to find the perfect equilibrium and balance between what were then considered 3 key factors: time, money (cost) and quality.

Figure 5.1 *Priority triangle*
(Image from oneresult.co.uk)

Nowadays, humanity is more and more concerned about how to provide a sustainable way of living and this decision reflects also on civil engineering and its way of thinking and designing. This is why an increasing amount of research and optimization is assigned to the sustainability aspect lately.
In this thesis, it will present that the solution with the built-up polygonal sections truss is not just more economical, but also more sustainable than the classical one.

5.1 Cost determination

5.1.1 Background

For the cost estimation of the trusses, the work of Haapio (2012) [44] and the paper of Kristo Mela (2013) [45] have been used.
HSS is more commonly used in other applications than constructions, but if properly used, it can represent a wise choice from economical point of view for buildings as well.

According to Evers & Maatje (2000), the cost breakdown of a steel structure, is as shown in the figure below:

1) Design process 13%
2) Material 38%
3) Production 27%
4) Coating 10%
5) Erection 12%

Figure 5.2 *Cost breakdown of steel structures*
source: Evers and Maatje (2000)

The cost of the design part was not covered in this thesis. For the rest of the processes, prices as given in Haapio(2012) [44] are used and correspond to the 2009 price level in Finland.

The method Haapio propose is a complex one, which is meant to cover the expenses of the structure in processes like manufacturing, transportation and erection. It includes configurable parameters such as labour, equipment and real estate, material and consumables unit costs.

A fixed unit cost, €/time unit was determined for a workshop activity. After an initial investment into the workshop, many related cost factors are fixed and those costs will run for their life time. Therefore, the time used to produce a feature is essential [44].

Calculations were performed for three types of steel (S355, S500 and S650) for both types of trusses. All calculation is presented in the Annex D.

5.1.2 Method

From all the processes presented in Haapio (2012), the following are used herein for the determination of the trusses cost:

- material cost
- blasting cost
- sawing cost
- painting cost

- transportation cost
- erecting cost

Therefore, the total costs are just approximate to the real values, but this should not affect the aim of the comparative study between the CHS truss and the polygonal cross-section truss, since both types of trusses are calculated using the same method and principle.

Next, details are presented about how each of the mentioned processes is calculated and what variables are taken into account.
Note should be taken that for some processes formulae from [45] are used, which may differ as form from the original ones, but they also have as basis formulae from Haapio (2012). That is the reason for which some calculation data is not matching [44].

Material cost.
The total cost of the steel elements is calculated using the following formula:

$$C_{SM} = W_{smpl} \times (C_{smbp} + C_{smg} + C_{smt} + C_{smq})$$

where
W_{smpl} [kg] is the steel weight
C_{smbp} [€/kg] is the basic cost
C_{smg} [€/kg] is the steel grade add-on
C_{smt} [€/kg] is the thickness add-on
C_{smq} [€/kg] is the quantity add-on

Blasting cost.
The elements are introduced into a shot blasting chamber at a constant speed. The total blasting cost can be calculated by the following formula:

$$C_B = T_{PB} \times (c_{LB} + c_{EqB} + c_{MB} + c_{REB} + c_{SeB} + c_{CB} + c_{EnB}) \times \frac{1}{u_B}$$

The time needed for processing the element (T_{PB}) is obtained by dividing the member's length (L_B [mm]) to the conveyor's speed, v_c [mm/min]. The conveyor's speed is considered to be 3000mm/min (Gietart).

c_{LB} is the labour unit cost of the blasting process. It's assumed that only one machine operator is used, at a rate of€0.46/min.

c_{EqB} represents the equipment cost, calculated at the value of €16050, or €0.13/min.

c_{MB} = €0.01/min is the cost of annual equipment maintenance.

c_{REB} = €0.16/min gives the real estate investment cost.

c_{SeB} = is €0.24/min the real estate maintenance cost.

c_{CB} represents the cost of consumables, steel shot in this case. The unit cost is of €0.02/min.

c_{EnB} is the cost of energy used during the blasting process. The given energy consumption unit cost is c_{EnB} =€0.07/min.

u_B is the utilisation ratio of the cost centre (set to 1).

Therefore, the final formula becomes:

$$C_B = \frac{L_B}{v_c} \times (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)/1$$

or

$C_B = L_B \times 0.000363$ [€/min]

Sawing cost.

The sawing cost is considered under the following form, as given by Mela(2013) [45]:

$$C_s = 1.2013 \times (T_{NS} + T_{PS}) + T_{PS} \times (c_{cs} + c_{ens})$$

T_{NS} is the non-productive time and it is equal to $T_{NS} = 4.5 + L/20000min$.

c_{ens} = 0.02€/min, represents the cost of energy.

The productive time depends on the position of the cross-section when the sawing is performed, but in this case this is not an issue, since the shapes of the elements are circular and polygonal.

It is determined by the following relation:

$$T_{PS} = \frac{A_h}{Q}$$

where

A_h [mm^2] is the cross-sectional area of the profile

Q [mm^2/min] is the sawing efficiency of the blade; this value changes according to the steel grades

c_{cs} is a factor which takes into account the steel grade

Painting cost.

The painting cost includes the cost needed for drying and is expressed as:

$$C_p = 4.17 \times 10^{-6} \times L \times A_u + 0.36L \times 10^{-3} \times W_{A\,min} \times 10^{-3}$$

(Mela (2013) [45]).

where

A_u represents the painted area per unit length

$W_{A\,min}$ is the smallest width dimension of the beam

Transportation cost.

Haapio (2012) proposes that the transportation is made with the help of a truck with an Euro trailer. Its dimensions give a total volume of 91m^3, whilst its maximum load cargo is limited to 24 tonnes. The maximum weight and maximum volume limitations lead to the limit ratio of 264kg/m^3. In case the ratio is below or equal to this limit, then the transportation cost is determined by the volume; otherwise it is determined by its weight.

The equations used for the two situations are the following:

$C_T = V_A \times (0.0106 \times d_{ws} + 1.2729)$ [€], for cost determined by volume

$C_T = W_A \times (0.00004 \times d_{ws} + 0.0048)$ [€], for cost determined by weight where

V_A [m^3] is the volume of the elements

W_A [kg] is the weight of the elements

d_{ws} [km] represents the distance between the workshop and the working site

Erecting cost.

According to Mela (2013), the erecting cost can be expressed as:

$$C_E = T_E \times \frac{C_{LE} + C_{EqE}}{u_E}$$

where

$C_{LE} = 3.1$ [€/min] is the cost of labour

C_{EqE}=1.3460 [€/min] represents the unit cost of the equipment

u_E= 0.36 is the efficiency factor

The time needed for erection is expressed as:

$$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$$

where

L_s=15m represents the distance from the lifting area to the final position

It is assumed that 5 workers are involved in the erecting process and a 25 tonnes lifting capacity crane is used.

5.1.3 Results

After the cost determination the following cost distributions are obtained:

For circular hollow sections truss

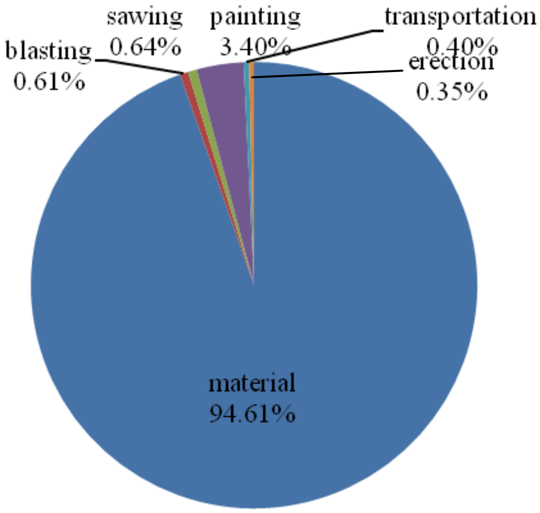


Figure 5.3 *Cost distribution for CHS S650*

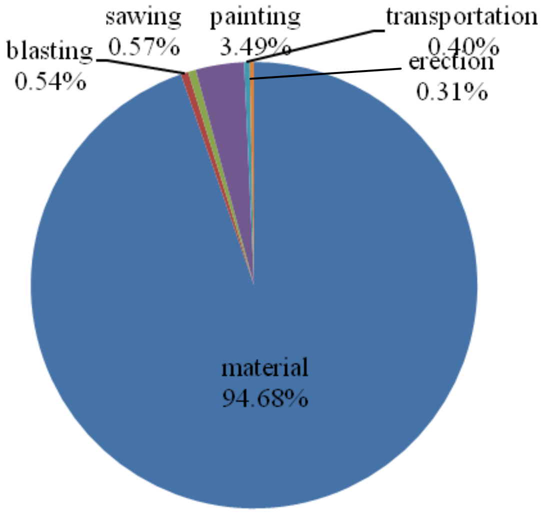


Figure 5.4 *Cost distribution for CHS S500*

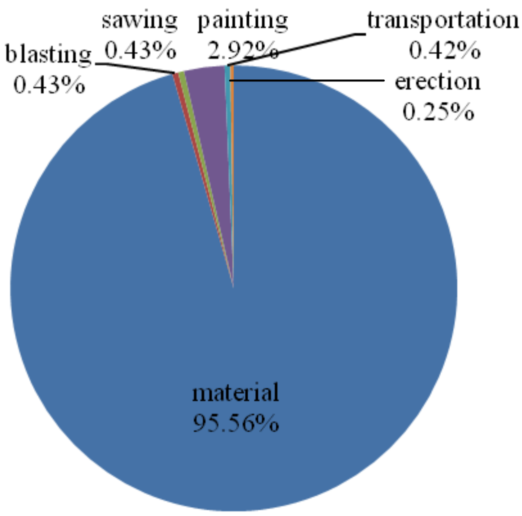


Figure 5.5 *Cost distribution for CHS S355*

<u>For Polygonal sections truss</u>

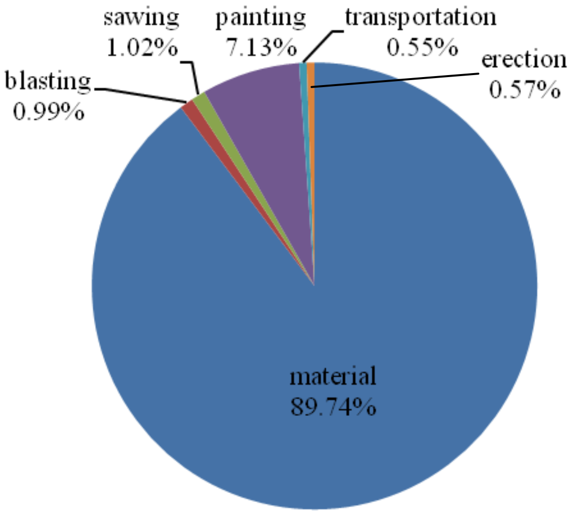


Figure 5.6 *Cost distribution for POL S650*

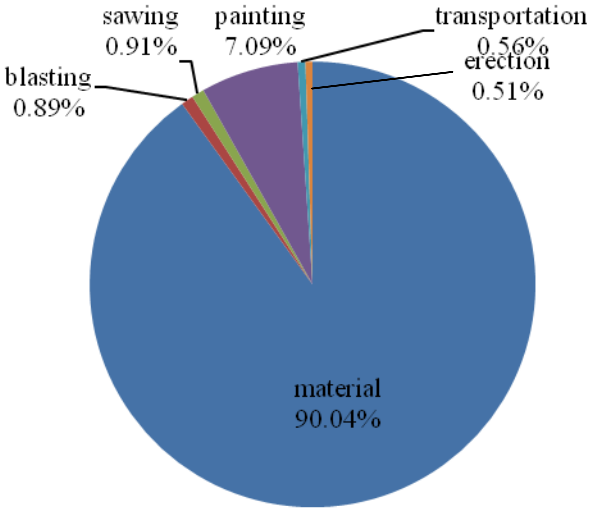


Figure 5.7 *Cost distribution for POL S500*

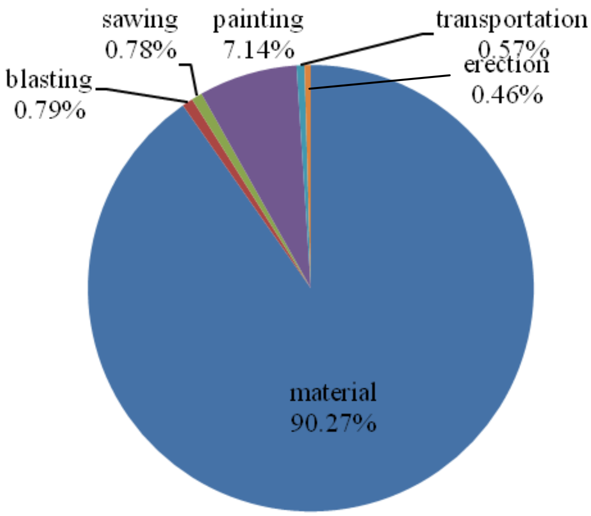


Figure 5.8 *Cost distribution for POL S355*

As seen in the charts, the price of the material is dominant and overwhelming in comparison with the costs of the other processes.

The only notable percentage difference between the CHS and POL trusses is regarding the painting cost. A jump from 1% to 7% is noticed when the change in cross-section is made. The cause of this is that the area that needs to be painted is approximately the same in both cases, but the total cost of the POL truss is highly reduced because of the material cost, as shown next.

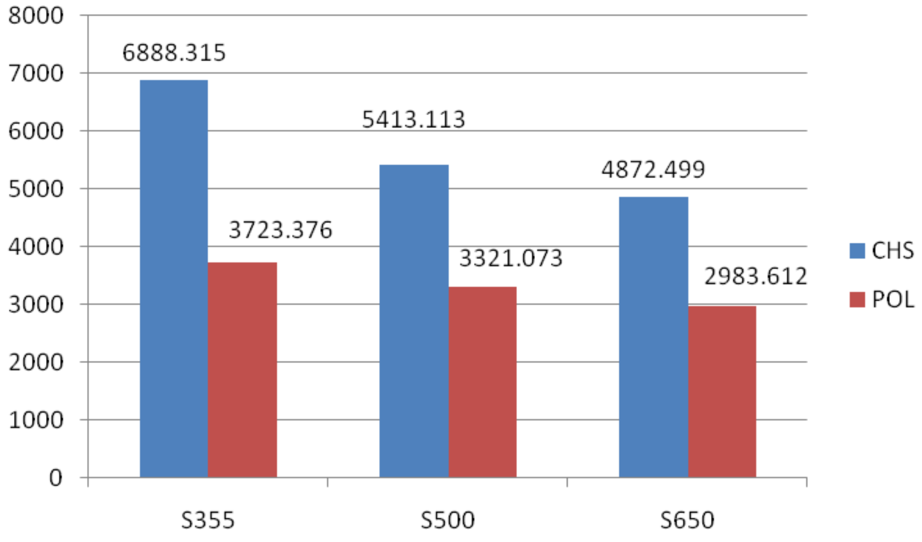


Figure 5.9 *Total cost for each type of truss*

A major difference can be seen in the total costs when comparing the CHS truss with the polygonal sections one. Considering the figures shown before (Fig. 5.3 to Fig 5.8) this difference is caused by the material cost difference.

There are two reasons for this material cost difference. One is the steel weight reduction itself when using the polygonal shapes (see Fig. 5.10 & Fig. 5.11) and the second one is the basic price of the steel: 1.88 €/kg for the CHS and 1.169 €/kg for the steel plates.

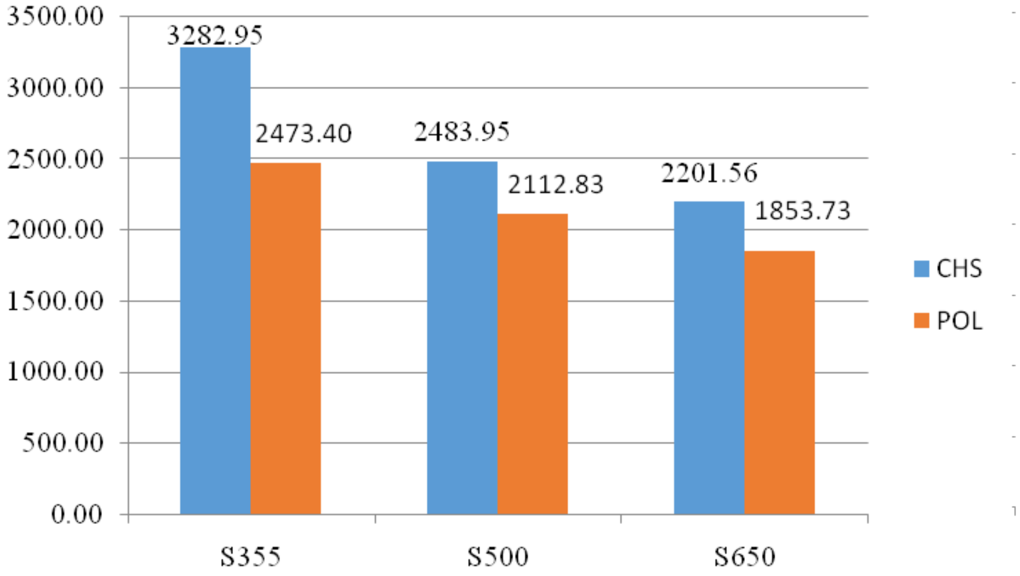


Figure 5.10 *Total weight for each type of truss (in kg)*

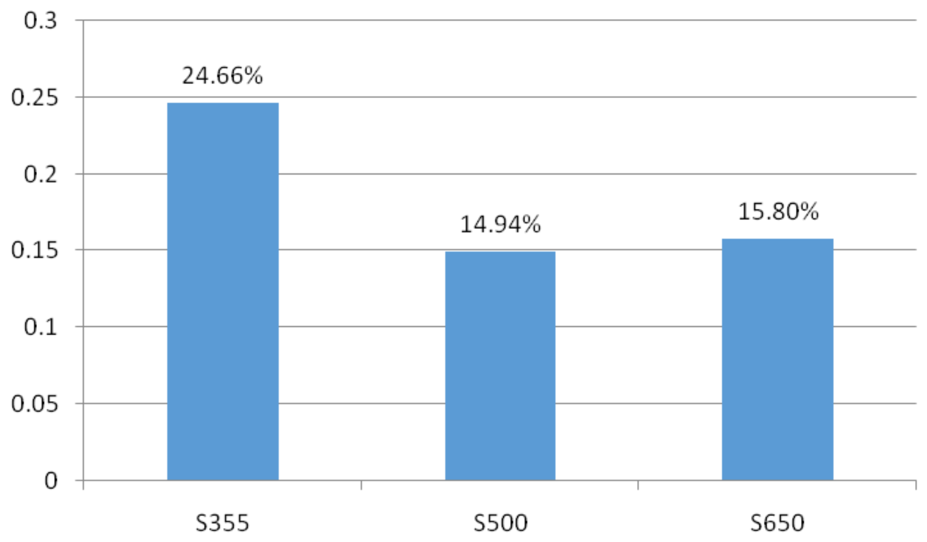


Figure 5.11 *Total weight reduction obtained when replacing the CHS with POL, expressed as percentage*

The cost reduction is significant overall (see Fig. 5.12), but it should be remembered that the values represent only an estimation since not all the processes needed for the fabrication and erection of the trusses are considered in the thesis. Also, certain coefficients and values may not reflect the reality due to the lack of information from the cost point of view.

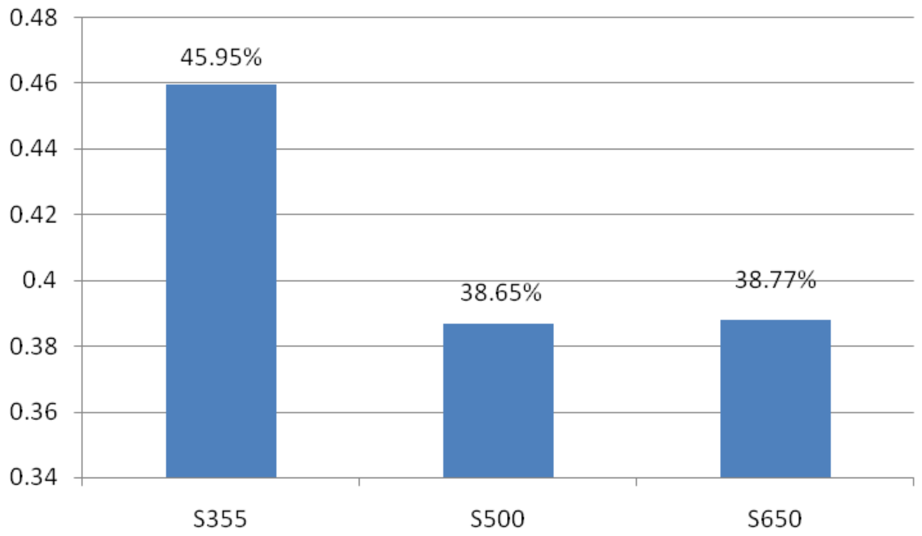


Figure 5.12 *Total cost reduction obtained when replacing the CHS with POL, expressed as percentage*

Moreover, it is interesting to analyse the cost fluctuation when HSS is used instead of a regular steel grade (S355).

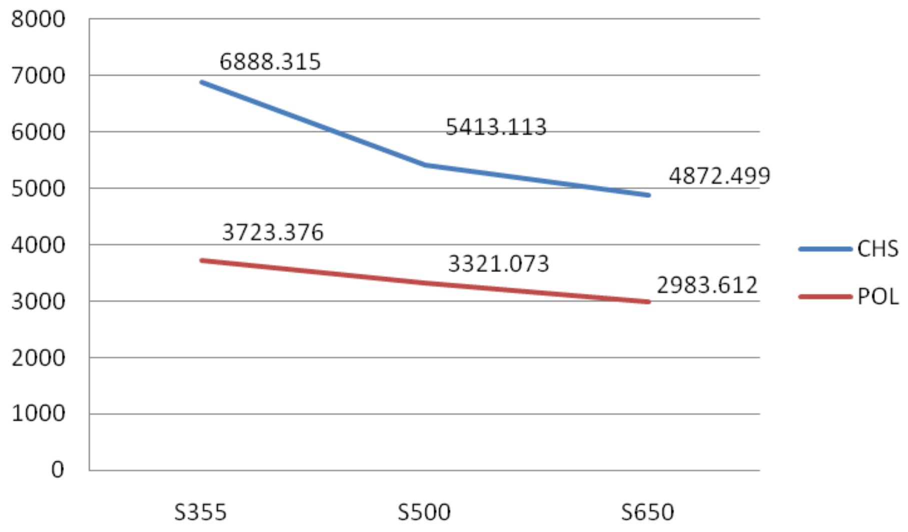


Figure 5.13 *Cost reduction when using HSS*

The tendency for both the CHS and polygonal trusses is for the total price to drop by using steel with higher yielding strength.

This may look like a paradox, since the price for fabricating the steel with a higher grade is more expensive, but as seen already the big benefit of using it leads to a big mass reduction of the structure (Fig. 5.11) and implicitly to a lower overall price of the truss.

From an economical point of view, the use of HSS on truss structures is beneficial since it significantly reduces the total cost, even though the percentage may not be as high as presented above. More complex and detailed analysis is recommended for a more precise value.

5.2 CO_2 footprint calculation

5.2.1 Background

Civil engineering is constantly developing and changing. This industry as well has to meet the requirements of the contemporary social life, by adapting to its needs.

Nowadays, there is a new balance that must be met and this is an equilibrium between the economic, social and environmental objectives. This is now known as Sustainable Development and is given an increasing attention, for the sake of future generations.

It is argued that Sustainable Development is now absolutely central to the practice of Civil Engineering [46].

The most comprehensive definition of sustainability comes from the Brundtland Commission Report of the United Nations in 1987 which states “sustainability is the development that meets the needs of the present without compromising the ability of future generations to meet their own needs.”

It was in 1969 that the concept was introduced to the public by the incorporation of National Environment Policy Act a.k.a (NEPA) [47].

The study of HSS is of great interest, since using this type of steel brings advantages environmentally wise.

HSS is known to provide major sustainable gains in active structures, since it provides material savings in production, it offers a bigger life span and reduces fuel consumption.

The difference between the active and passive structures is that the latter one is not influencing the environmental impact during its usage phase. Only production, transportation and erection are considered to have an impact.

Nevertheless, HSS can be a wise choice for passive, civil structures as well.

As shown by Jan-Olof Sperle, during the presentation "Environmental Advantages of using Advanced High Strength Steel in Steel Structures" in Oslo (2012) the high strength steel has the following advantages:

- the structure becomes lighter as the steel strength increases
- it is possible to obtain a weight reduction of 20 up to 40%
- there is less usage of natural resources involved
- as an outcome, there are environmental savings

The amount of CO_2 emissions can be considerably reduced by using HSS (see figure below).

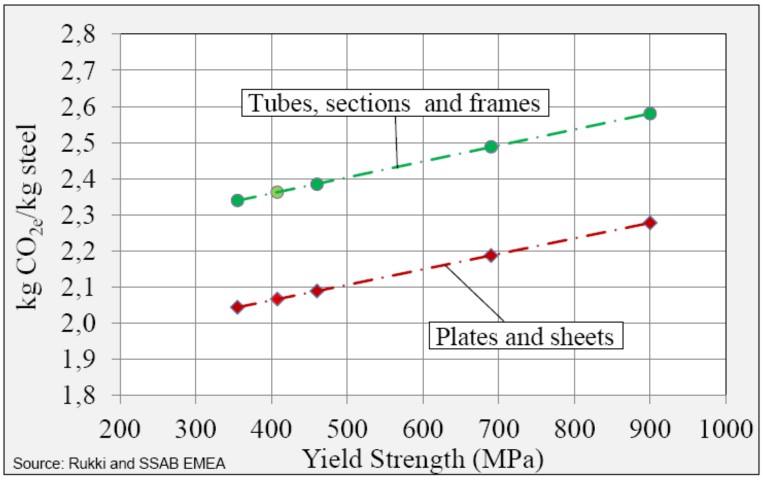


Figure 5.14 *CO2 emissions during production of steel, cradle to gate[12]*

This paper only considers the CO_2 footprint calculation and ignores the rest of gases producing greenhouse emissions, since the carbon dioxide is by far the governing one, with a 83.6% of the total emissions.

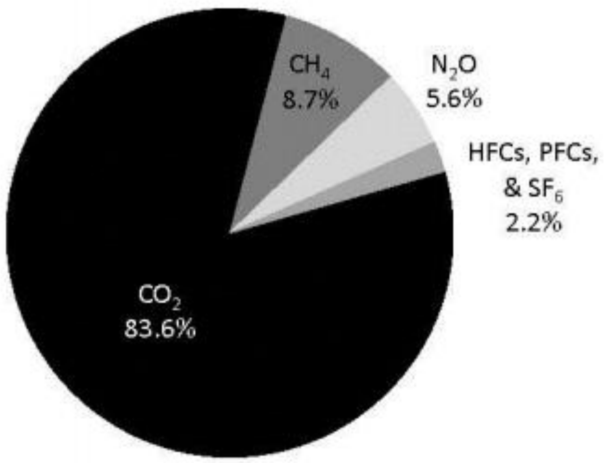


Figure 5.15 *Greenhouse Gas Emissions by Gas (2011)*
source: www.epa.gov

A description of the calculation method is given in the next subchapter.

5.2.2 Method

The goal is to study the environmental impact of HSS (S500 and S650) in comparison with a regular steel grade (S355), applied on the previously designed truss.
The CO_2 emissions are calculated both for the CHS truss and the polygonal shaped one.

A cradle-to-gate LCA is considered. Since the truss is a passive element, as mentioned in the background, the environmental impact of the steel production, painting and transportation to the site are highlighted. End-of-life credits are taken into account.

Environmental Product Declarations (EPD) from Ruukki [48] are used as Life Cycle Inventories (LCI). This steel producer is chosen because it will be the manufacturer of the truss, on which the full-scale experiments will be performed at COMPLAB, LTU.

Ruukki offers EPDs for both the production of steel plates and for tubular steel products. The End-of-Life recycling rate considered is of 90%. All production stages are taken into account in the provided values.
The values of carbon dioxide emitted to the air are the following:

- 710 g/kg for the **hot rolled steel plates**
- 1070 g/kg for the **tubular products**

The EPDs are dated back to 2011.
Next, a short description of the CO_2 emissions calculation for each of the three mentioned processes is presented.

Steel production.

The amount of CO_2 per kilogram for a higher yielding strength of the steel is determined with a formula presented in JouCO2&COSTi [49]:

$$CO_{2emissions\ .HSS} = \left[0.00018 \times (f_y - 355) + \frac{CO_{2emissions\ .350}}{1000}\right]$$

where

$CO_{2emissions\ .350}$ represents the quantity of CO_2 emissions given for the S355 steel type. Values given by Ruukki's EPD, as presented above, are used.

The total emissions for the steel production is obtained by multiplying the total mass of steel for a certain truss with the $CO_{2emissions\ .HSS}$ value.
For the regular S355 steel, for multiplication the value given in the EPD is directly used.

Painting.

An intumescent acrylic paint is used to offer the truss elements a fire resistance of R30. It is considered that all the elements are painted all around.
The thickness of the applied paint is of $t_{paint} = 1.5mm$.
The total volume of paint is calculated:

$$V_{paint} = A_{paint} \times t_{paint}$$

where A_{paint} is the total area that needs to be covered by paint.

JouCO2&COSTi is presenting an amount of CO_2 emissions of the acrylic paint equal to m_{acryl} =2.5kg/L.

Therefore, the total amount of carbon dioxide is determined by:

$$CO_{2.paint} = m_{acryl} \times V_{paint}$$

Transportation.

For the transportation a semi-track with a 25tonne load capacity is used.
Environmental data from *Lipasto.vtt.fi* from the year 2011 is used [50].

According to this source, the CO_2 emissions for the truck is the following:

$$CO_{2.track} = 41 \frac{g}{tonne \times kg}$$

A distance of 100 km between the manufacturer and the site is considered.
The transportation from the workshop to the working site is determined as:

$$CO_{2.transp} = m \times 100km \times CO_{2.track}$$

According to Lipasto [50] the empty track has the following emission:

$$CO_{2.track\ .empty} = 757 \frac{g}{km}$$

The environmental impact of the return of the track is easily determined by multiplying the above mentioned value to the distance the track covers.

$$CO_{2.transp\ .return} = CO_{2.track\ .empty} \times 100km$$

The total emissions for the transportation are obtained by summing the emissions for the round trip of the track.
Detailed calculation is provided in Annex E.

5.2.3 Results

Contribution of each of the three above mentioned processes to the total environmental impact is presented in the charts below.

For Circular hollow sections truss

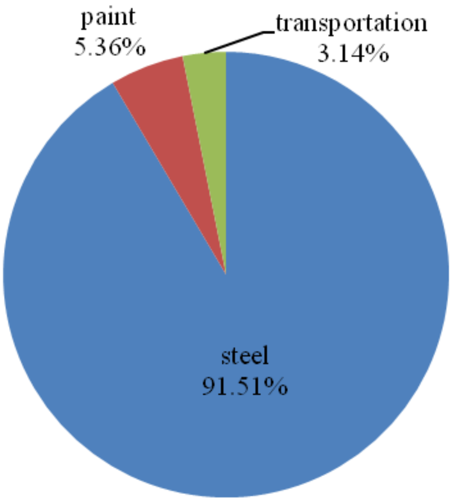


Figure 5.16 *CO2 emissions of different processes for CHS S650*

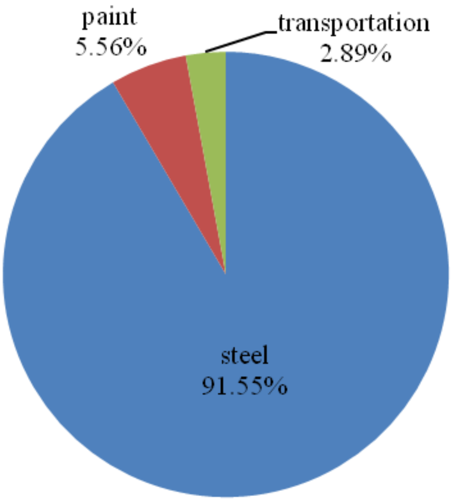


Figure 5.17 *CO2 emissions of different processes for CHS S500*

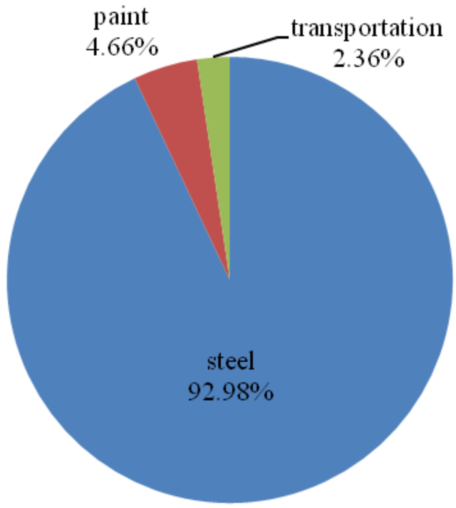


Figure 5.18 *CO2 emissions of different processes for CHS S355*

For Polygonal sections truss

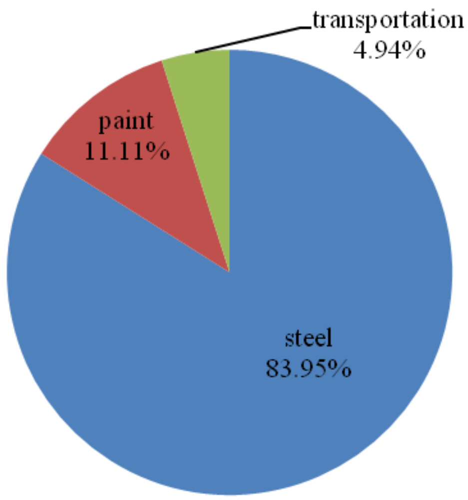


Figure 5.19 *CO2 emissions of different processes for POL S650*

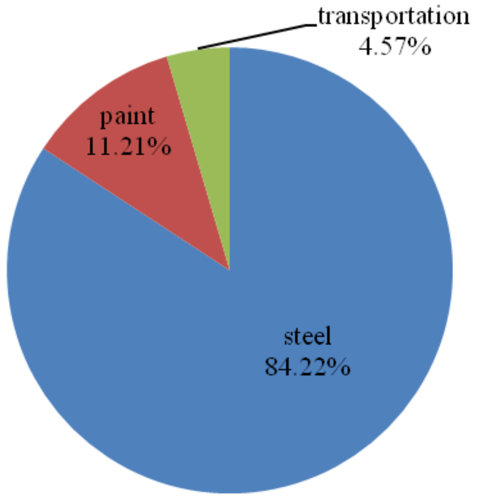


Figure 5.20 *CO2 emissions of different processes for POL S500*

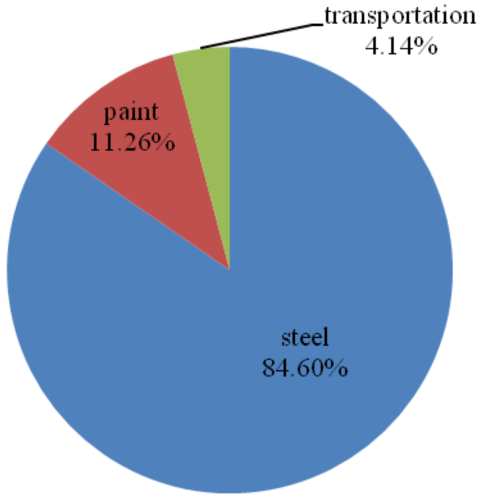


Figure 5.21 *CO2 emissions of different processes for POL S355*

Once more, as in the case of the cost determination, the amount of steel is the governing variable in the total CO_2 emissions. It shows an approximate value of 92% of the total for the CHS and 84% for the polygonal truss.

Again an increase in the percentage of the paint process is observed. There is an increase of about $10m^2$ of painted area when adopting the polygonal sections for the truss, which leads to higher emissions rate.
Nevertheless, the use of intumescent paint should be considered if fire resistance is required, since it is a better solution than increasing the element's size.

The transportation always produces the least significant impact, but it should be taken into account that a symbolic distance of just 100km is considered.
The decrease of total emissions for the case when CHS are used is more drastic than in the case of polygonal section truss, as shown in the figure below:

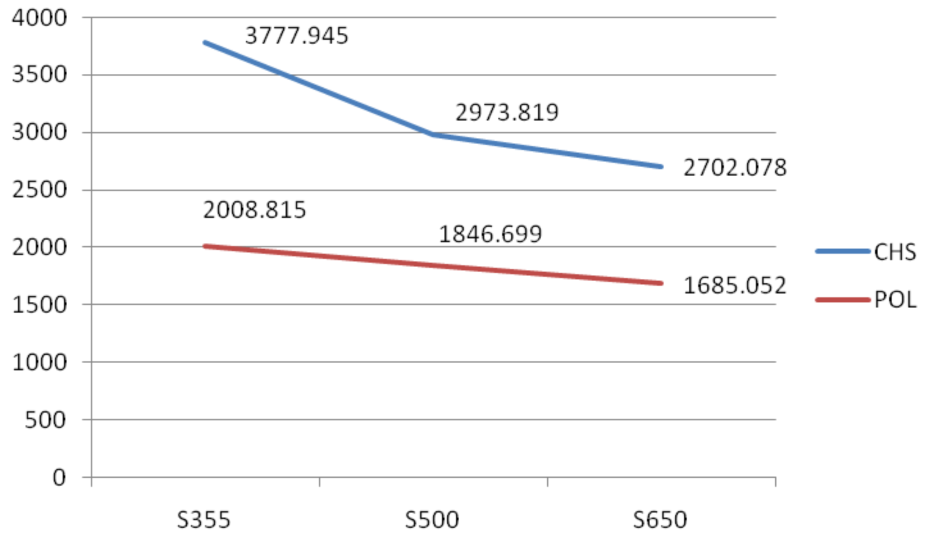


Figure 5.22 *CO2 emissions reduction when using HSS (in kg of CO2)*

Direct comparison between the CHS and polygonal trusses, for each type of steel separately, can be observed in Fig.5.23.

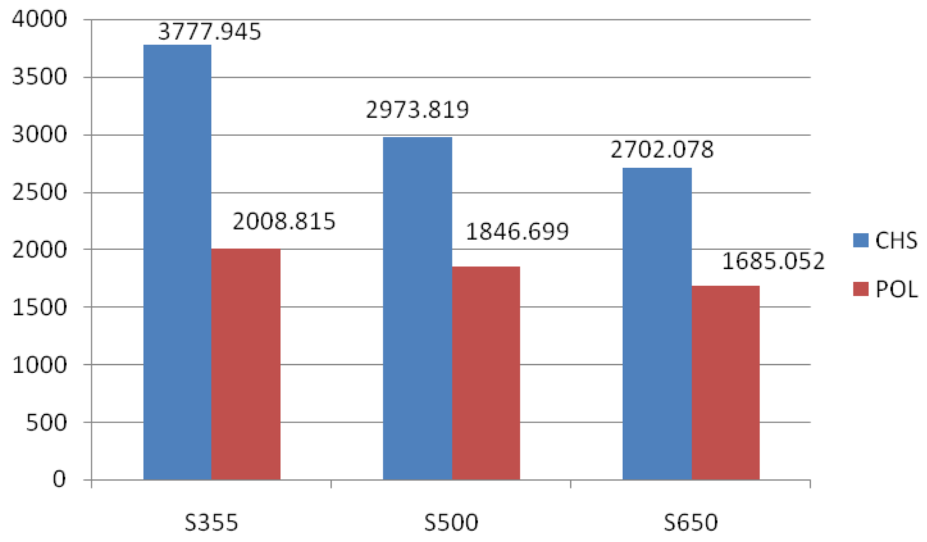


Figure 5.23 *Total CO2 emissions for each type of truss (in kg of CO2)*

Both the steel production and the transportation CO_2 emissions depend on the mass of the used steel for the trusses, thus explaining the overall difference in the carbon footprint results, when using the polygonal sections instead of the circular ones.

The importance of using HSS and a more innovative cross-section is summarized in the tables below where the carbon dioxide savings are shown.

Table 5.1 *CHS truss environmental savings*

Circular hollow section truss						
Steel type	Weight of the truss kg	Weight reduction kg	Weight reduction %	CO2 total emissions kg	Environmental savings kg CO2	Savings %CO2
S355	3282.95	—	—	3777.95	—	—
S500	2483.95	799.00	24.34	2973.82	804.13	21.28
S650	2201.56	1081.39	32.94	2702.08	1075.87	28.48

Table 5.2 *Built-up polygonal truss environmental savings*

Built-up polygonal section truss						
Steel type	Weight of the truss kg	Weight reduction kg	Weight reduction %	CO2 total emissions kg	Environmental savings kg CO2	Savings %CO2
S355	2473.40	—	—	2008.815	—	—
S500	2112.83	360.57	14.58	1846.699	162.12	8.07
S650	1853.73	619.67	25.05	1685.052	323.76	16.12

As expected, because of the weight reduction, the CO_2 emissions quantity is reduced with the increase of steel strength.
In the case of the CHS truss a reduction of 21% (for S500) up to 28% (S650) is seen. For the polygonal truss the reduction is not as significant, but savings of 8% for the S500 steel and 16% for S650 are still obtained. The cause of this is that when upgrading to a higher steel for the polygonal truss, smaller weight reductions are reached, which lead to a smaller CO_2 emissions saving. The manufacturing of steel is the dominant producer of carbon dioxide, as seen in Fig 5.19 to Fig 5.21.

Therefore, the use of HSS for the truss design is recommended, both from an economical point of view and from an environmental one.

CONCLUSIONS

This thesis represents the very beginning of the research which is planned by LTU to investigate new solutions for HSS truss design. The matters covered by this work can be used as first reference for future researchers who are going to work on this project.

Several innovative ideas in designing modern long span steel trusses were investigated in this thesis. The first one is to design the steel trusses made from high strength steel. The second one concerns new types of sections: semi-closed built-up polygonal and U-shaped profiles were used in design. It was expected that high strength steel and cold-formed profiles will provide higher strength along with a significant reduction of weight, but there were some additional unknowns that should have been answered by this work.

The main questions to be answered were mainly regarding the new proposed polygonal truss. One of the issues was the investigation of the behaviour of the joint in the bottom chord, where two diagonals meet. Another aspect that required special attention was the compressed polygonal chord, built up from cold-bent steel plates. The buckling resistance was analysed taking into account different material properties that are influenced by cold-bending. Moreover, it was worth to investigate the difference in cost and the environmental impact that could be achieved by designing HSS polygonal trusses instead of the more common tubular hollow section trusses.

To achieve these goals, two types of trusses were designed. The quite well known solution of tubular hollow section truss was compared to the innovative truss made from built-up polygonal and U-shaped sections. In order to achieve a more complex comparison study, each of the both types of trusses was designed to be made out of 3 types of steel: regular S355 and HSS S500 and S650.

The main aspects in the design procedures were buckling resistance of compressed chords and diagonals, tension resistance of bottom chord, lateral buckling of the entire truss. Design was made using current Eurocodes. CIDECT recommendations for circular hollow section truss design were used. Special attention was paid to the difference in the design rules for HSS. In some cases the reduction factor for the resistance of HSS is not necessary or is too conservative. For example the joint resistance for any HSS hollow section joint should be reduced by 20% (factor 0.8). The resistance of the joint was one of the governing factors in CHS truss design, which lead to bigger cross sections. Therefore, this reduction factor was significant and research should be done, in order to see if it is really needed.

The weight of each designed truss is given in table below:

	Weight (kg)		
	S355	**S500**	**S650**
CHS	3282.95	2483.95	2201.56
Polygonal	2473.40	2112.83	1853.73

The behaviour investigation of the bottom chord connection is broadly described in Chapter 3. Various models of connection were numerically investigated using *Abaqus* software. Two types of loading were analysed – tension only and tension load combined with load from diagonals. The objective of this research was to analyse the behaviour of different types of stiffeners in the connection. Models with no stiffener, plate stiffener between diagonals and U-shape insert as a stiffener with different thicknesses were analysed. Results showed that U stiffeners must be used almost in all cases, whilst there are too big stress concentrations in some zones with plate stiffener. Plate stiffener can be used while the connection is only subjected to tension force. The 3mm thickness U insert is enough for any case, while diagonal plates are welded together. Analysis showed that the connection with diagonal plates welded together performs better, since it reduces stress concentrations and stabilises the connection.

The mechanical properties of cold-formed steel sections differ from those of the steel strip or the plate before forming. The cold-forming operation of the steel section increases the yield stress and the tensile strength, but at the same time decreases the ductility of material. This phenomena was investigated in Chapter 4 by analysis of built up polygonal chord in compression. Firstly, linear buckling analysis was performed in order to obtain the critical buckling loads and buckling shapes for the member. Different spacing between bolts was investigated and the results showed that the limit of spacing in which the flexural buckling governs, rather than the local buckling, is 250-500mm. At 500mm spacing, the local buckling is the most crucial, every plate working as a single element. While spacing is reduced to 250mm, the member is working as one solid element and the flexural buckling is governing.

Plastic RIKS analysis was performed in order to obtain the ultimate load of the member, whilst introducing different plastic material properties proposed by various methods. Results showed that the yield strength increase in the bent zones does not influence the buckling resistance of the member. There is just a slight difference between the resistance values. FEM analysis was compared to hand calculations performed according to the Eurocodes. Hand calculations gave lower results than the FEM analysis, regarding the member resistance. The main factor that influences this reduction is the buckling curve. For cold-formed members the Eurocode suggests to use buckling curve “c”, which gives more conservative results in comparison with the ones obtained by FEM analysis, whilst curves “a” or “b” give more similar results to FEM analysis.

An approximate cost of the trusses was determined. The processes taken into account for this were the cost of material, blasting, sawing, painting, transportation and erecting. By far, the governing cost was determined by the material (around 90% of total cost), both in the case of CHS and polygonal shaped section trusses.

The high reduction in the total mass of the trusses led to a cost drop of up to 46% for S355 and around 38% for the HSS, when replacing CHS with built-up sections. Another reason for the lower price of the built-up section truss is that the basic price of steel differs a lot: 1.88 €/kg for the CHS and 1.169 €/kg for the steel plates.

An almost linear total cost reduction was obtained both for CHS and polygonal trusses, when increasing the steel grade.

A sustainability approach was taken into account as well for the 6 types of trusses investigated. For this, a cradle-to-gate life cycle analysis (LCA) was considered, using Environmental Product Declarations (EPD) from Ruukki.

This work only took into account the CO_2 footprint calculation and ignored the rest of the greenhouse effect producing gases, since the carbon dioxide accounts for 83.6% of the total emissions.

The steel production, the painting and the transportation of the elements were the processes investigated regarding the carbon footprint. For both CHS and polygonal section trusses, the biggest percentage was represented, once more, by the steel production with a 92% of the total for the CHS and 84% for the built-up cross-section truss.

The CO_2 emissions were reduced drastically with the increase of the steel grade. In the case of the CHS truss a reduction of 21% (for S500) up to 28% (S650) is seen. For the polygonal truss the reduction is not as significant, but savings of 8% for the S500 steel and 16% for S650 are still obtained.

It is safe to say that the use of HSS and of built-up cold formed sections for this type of long span trusses is beneficial both from an economical and environmental point of view.

FUTURE RESEARCH

There is a high need for future research, in order to verify the results obtained by calculations and numerical analyses.

First of all, the laboratory tests for the connection should be performed to investigate a real element behaviour of that area. The approximation of the connection model made by FEM may not lead to the real behaviour of truss joint, therefore it must be compared to experimental results.

True material properties should be obtained from tensile coupon tests, testing plates with different bent angles. This is already scheduled to be performed at LTU and it is just matter of time until they will be received.

Also, the compression test of built-up polygonal member could be performed, in order to obtain test data for the finite model verification.

As it was mentioned above, LTU is part of the project involving investigation of this type of trusses and there will be enough experimental tests conducted on this matter.

From the design point of view, serviceability limit states are very important for long span truss structures. As the scope of this thesis did not cover this aspect, the prevention of big deflection of long span trusses should be investigated in future research.

Moreover, different load combinations, such as reverse loading (suction force) should be investigated, since it produces compression forces in the bottom chord, which might lead to the buckling of the thin flanges of the U-shaped section.

Finally, a cost optimization is suggested, in order to obtain more realistic total cost values for the trusses.

REFERENCES

[1] J.-O. Sperle and K. Olsson, "High strength and ultra high strength steels for weight reduction in structural and safety-related applications," in *29 th International Symposium on Automotive Technology and Automation.*, 1996, vol. 1, pp. 115–125.
[2] WorldAutoSteel, "Advanced High Strength Steel (AHSS) Application Guidelines." World Steel Association, Jun-2009.
[3] B. Johansson, "Use and design of high strength steel structures," *Rev. Metall. Cah. Inf. Tech.*, vol. 100, no. 11, pp. 1115–1123+vi–ix, 2003.
[4] J. Farkas, "ECONOMY ACHIEVED BY USING HIGHER STRENGTH STEELS IN TRUSSES WELDED FROM SQUARE HOLLOW SECTIONS," in *Welding of Tubular Structures - Proceedings of the Second International Conference Held in Boston, Massachusetts, USA, 16 – 17 July 1984, Under the Auspices of the International Institute of Welding*, Boston, 1984, pp. 137–144.
[5] J.-O. Sperle, L. Hallberg, J. Larsson, and H. Groth, "THE ENVIRONMENTAL VALUE OF HIGH STRENGTH STEEL STRUCTURES (88044)."
[6] Lars Jensen, "Application of high strength steel in super long span modern suspension bridge design," Malmo, Sweden, 2009.
[7] C. Miki, K. Homma, and T. Tominaga, "High strength and high performance steels and their use in bridge structures," *J. Constr. Steel Res.*, vol. 58, no. 1, pp. 3–20, Jan. 2002.
[8] C. Shim, J. Whang, C. Chung, and P. Lee, "Design of Double Composite Bridges using High Strength Steel," *Procedia Eng.*, vol. 14, pp. 1825–1829, 2011.
[9] M. Veljkovic and B. Johansson, "Design of hybrid steel girders," *J. Constr. Steel Res.*, vol. 60, no. 3–5, pp. 535–547, Mar. 2004.
[10] J. Farkas, "Economy of higher-strength steels in overhead travelling cranes with double-box girders," *J. Constr. Steel Res.*, vol. 6, no. 4, pp. 285–301, 1986.
[11] "Ruukki to install EUR20m structures at Swedbank Arena.," *Nordic Business Report*, 19-Oct-2009.
[12] Jan-Olof Sperle, "HIGH STRENGTH STEEL IN THE ROOF OF SWEDBANK ARENA SAVINGS IN WEIGHT, COST AND ENVIRONMENTAL IMPACT," in *Nordic Steel Construction Conference Proceedings*, Oslo, 2012, p. 15.
[13] Innovation & Research FOCUS, "High strength long span structures," no. 92, pp. 4–5, Feb-2013.
[14] U. Hornung and H. Saal, "A method for calculating the out-of-plane buckling length of diagonals of truss girders with hollow sections and K-joints," *J. Constr. Steel Res.*, vol. 46, no. 1–3, p. 489, Apr. 1998.
[15] S. Houliara and S. A. Karamanos, "Buckling of Thin-Walled Long Steel Cylinders Subjected to Bending," *J. Press. Vessel Technol.*, vol. 133, no. 1, pp. 011201–011201, Dec. 2010.
[16] M. M. K. Lee and S. R. Wilmshurst, "Parametric study of strength of tubular multiplanar KK-joints," *J. Struct. Eng.*, vol. 122, no. 8, pp. 893–904, 1996.
[17] S. Herion and F. Mang, "Parametric Study on Multiplanas K-joints Made of RHS Regarding Axial Force, In-Plane Bending and Out-of-Plane Bending Moments," in *The Proceedings of the 4th (1994) International Offshore and Polar*

Engineering Conference, Osaka, Japan, April 10-15, 1994, vol. 4, International Society of Offshore and Polar Engineers, 1994.

[18] S. Blümel and M. Fontana, "Load-bearing and deformation behaviour of truss joints using thin-walled pentagon cross-sections," *Thin-Walled Struct.*, vol. 42, no. 2, pp. 295–307, Feb. 2004.

[19] M. Abambres, D. Camotim, and N. Silvestre, "Physically non-linear GBT analysis of thin-walled members," *Comput. Struct.*, vol. 129, pp. 148–165, Dec. 2013.

[20] J. Wardenier, *Hollow sections in structural applications*. Geneva; Zoetermeer, Netherlands: CIDECT ; Bouwen met Staal, 2010.

[21] Wardenier J, *Design guide for circular hollow section (CHS) joint under predominantly static loading*. 2008.

[22] Olga Garzon, "Resistance of polygonal cross-sections - Application on Steel Towers for Wind Turbine," Licentiate, Luleå University of Technology, Luleå, 2013.

[23] "Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings." May-2005.

[24] "Eurocode 3: Design of steel structures. Part 1-8: Design of joints." May-2005.

[25] "Eurocode 3 – Design of steel structures. Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S700." Feb-2007.

[26] "Eurocode 3: Design of steel structures. Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting." Oct-2006.

[27] "Eurocode 3 - Design of steel structures - Part 1-5: Plated structural elements." Oct-2006.

[28] S. Timoshenko, *Theory of elastic stability*. McGraw-Hill, 1961.

[29] Karren, K. W., and G. Winter, "Effects of Cold-Forming on Light-Gage Steel Members," *J. Struct. Div.*, vol. vol. 93, Feb. 1967.

[30] Chajes, A., S. J. Britvec, and G. Winter, "Effects of Cold- Straining on Structural Sheet Steels," *J. Struct. Div.*, vol. vol. 89, Apr. 1963.

[31] W. Yu and R. A. LaBoube, *Cold-formed steel design*. Hoboken, N.J.: Wiley, 2010.

[32] "North American Specification for the Design of Cold-Formed Steel Structural Members." Edition-2007.

[33] Lind, N.C., and Schroff, D.K., "Utilization of Cold Work in Light Gage Steel," *J. Struct. Div. ASCE*, vol. Vol. 101, Jan. 1975.

[34] P. A. Sloof and R. M. Schuster, "Yield strength increase of cold formed sections due to cold work of forming," in *Fifteenth international specialty conference on cold-formed steel structures.*, 2000.

[35] Sioof, P.A., *Yield Strength Increase of Cold Formed Sections Due to Cold Work of Forming*. Waterloo, Ontario, Canada, 1999.

[36] J. Chen and W. Jin, "Corner strength enhancement of high strength cold-formed steel at normal room and elevated temperatures," *J. Zhejiang Univ. Sci. A*, vol. 9, no. 9, pp. 1251–1257, Aug. 2008.

[37] Bjorhovde, R, "Deterministic and probabilistic approaches to the strength of steel columns," Lehigh University, PA, 1972.

[38] ECCS, "European Recommendations for the Design of Light Gauge Steel Members," p. Brussels, 1978.

[39] Schafer, B.W. – Peköz, T, "Computational modelling of cold-formed steel characterising geometric imperfections and residual stresses," *J. Constr. Steel Res.*, no. 47, pp. 193–210, 1998.
[40] Rondal, J. – Dubina, D. – Bivolaru, "Residual stresses and the behaviour of cold-formed steel structures," presented at the 17th Czech and Slovak International Conference on Steel Structures and Bridges, Bratislava, Slovakia, 1994, pp. 193–197.
[41] Rondal, J. - Dubina, D. - Ungueanu V., "Imperfections and computational modelling of cold-formed members."
[42] M. Życzkowski, "Post-buckling analysis of non-prismatic columns under general behaviour of loading," *Int. J. Non-Linear Mech.*, vol. 40, no. 4, pp. 445–463, May 2005.
[43] Tim Heistermann, Naveed Iqbal, Marouene Limam, and Christine Heistermann, Milan Veljkovic, "Compendium for the course S7001B Applied FEM Basics and at elevated temperatures." Department of Civil, Mining and Environmental Engineering Division of Structural Engineering – Steel Structures, Luleå, 2010.
[44] J. Haapio, "Feature-Based Costing Method for Skeletal Steel Structures based on the Process Approach," Mar. 2012.
[45] Kristo Mela, Markku Heinisuo, and Teemu Tiainen, "Deliverable D4.5," TUT Tampere University of Technology, RUOSTE Project Report D4.5, 2013.
[46] P. W. Jowitt, "Sustainability and the formation of the civil engineer," *Proc. ICE-Eng. Sustain.*, vol. 157, no. 2, pp. 79–88, 2004.
[47] "What is Sustainability and where did it come from????," *GREEN MOVEMENT: INNOVATIVE AND IDEAL*. .
[48] Ruukki, "Certificates, EC- and environmental declarations - Ruukki." [Online]. Available: http://www.ruukki.com/Products-and-solutions/Certificates-EC--and-environmental-declarations. [Accessed: 09-Jan-2014].
[49] TUT, Tampere University of Technology, "JouCO2 & COSTi," *Metallirakentamisen tutkimuskeskus*. [Online]. Available: http://metallirakentaminen.fi/jouco2-amp-costi. [Accessed: 09-Jan-2014].
[50] Kari Mäkelä, "Semi trailer combination Gross vehicle mass 40t, pay load capacity 25t Highway driving," 25-Apr-2012. [Online]. Available: http://lipasto.vtt.fi/yksikkopaastot/tavaraliikennee/tieliikennee/kapptiee.htm. [Accessed: 09-Jan-2014].

APPENDICES

Annex A - Design calculation of CHS truss elements, joints and welds for S355, S500, S650

Annex B - Design calculation of the built-up polygonal section truss elements for S355, S500, S650

Annex C - Global buckling design calculation of the CHS and polygonal built-up truss for S355, S500, S650

Annex D - Cost determination for CHS and polygonal trusses, using S355, S500 and S650

Annex E - CO_2 emissions calculation for CHS and polygonal trusses, using S355, S500 and S650

ANNEX A
Design of Circular Truss Chords and Braces S355

- according to EN 1993-1-1 & EN 1993-1-8

Chords checking

- **Tension:**

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the lower chord:

$$N_{Ed.t} := 3259.55kN$$

$$N_{t.Rd} := 3260kN$$

$$N_{t.Rd} = \frac{A_{net} \cdot f_y}{\gamma_{M0}} \qquad (6.6)\ \text{EN 1993-1-1}$$

$$\gamma_{M0} := 1.0$$

The type of steel used is S355, therefore: $f_y := 355 \frac{N}{mm^2}$

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 91.831 \cdot cm^2$$

We adopt the following **CHS: 323.9 X 12.5**, giving a gross area of **96.(** cm^2

- **Compression:**

$$\frac{N_{Ed}}{N_{c.Rd}} < 1 \qquad (6.9)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the upper chord:

$N_{Ed.c} := 1589.37kN$

<u>Cross-section classification:</u>

- according to Table 5.2 from EN 1993-1-1

$\varepsilon := \sqrt{\frac{235}{355}} = 0.814 \qquad \varepsilon^2 = 0.662$

$d := 219.1mm \qquad t := 10mm$

$\frac{d}{t} = 21.91 \quad < \; 50 \cdot \varepsilon^2 = 33.099$

Therefore, the Cross-section is **Class 1**.

$N_{c.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ (6.10) EN 1993-1-1

$N_{c.Rd} := 1590kN$

$\gamma_{M0} := 1$

$f_y := 355 \frac{N}{mm^2}$

$A := \frac{N_{c.Rd} \cdot \gamma_{M0}}{f_y} = 44.789 \cdot cm^2$

We adopt the following **CHS: 219.1 X 12.5**, giving a gross area of **81.** cm^2 (value adopted due to joint verification)

- <u>**Buckling resistance:**</u>

$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 81.13cm^2$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L$$

$E := 210GPa$ $\qquad I_y := 4344.58cm^4$ $\qquad L := 4m$ $\qquad k := 0.9$ (for chord)

$$L_{cr} := k \cdot L = 3.6\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 6948.032 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.644$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.816$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.759 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 2.186 \times 10^3 \cdot kN$$

$$N_{Ed} := 1590kN$$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.727 \qquad < 1$$

Diagonals checking

$$L_{br} := 2.5m - \frac{0.3239m}{2} - \frac{0.2191m}{2} = 2.228\,m$$

- **Diagonals in tension:**

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in diagonal:

$$N_{Ed.t} := 1097.68kN$$

$$N_{t.Rd} := 1098kN$$

$$N_{t.Rd} = \frac{A_{net} \cdot f_y}{\gamma_{M0}} \qquad (6.6)\ \text{EN 1993-1-1}$$

$$\gamma_{M0} := 1.0$$

The type of steel used is S355, therefore: $f_y := 355 \frac{N}{mm^2}$

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 30.93 \cdot cm^2$$

We adopt the following **CHS: 168.3 X 6.3**, giving a gross area of **32**$\cdot cm^2$

- **Compression:**

$$\frac{N_{Ed}}{N_{c.Rd}} < 1 \qquad (6.9)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in diagonal:

$$N_{Ed.c} := 1086.44kN$$

Cross-section classification:

- according to Table 5.2 from EN 1993-1-1

$\varepsilon := \sqrt{\frac{235}{355}} = 0.814 \qquad \varepsilon^2 = 0.662$

$d := 168.3mm \qquad t := 6.3mm$

$\frac{d}{t} = 26.714 \quad < \quad 50 \cdot \varepsilon^2 = 33.099$

Therefore, the Cross-section is **Class 1**.

$N_{c.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ (6.10) EN 1993-1-1

$N_{c.Rd} := 1087kN$

$\gamma_{M0} := 1$

$f_y := 355 \frac{N}{mm^2}$

$A := \frac{N_{c.Rd} \cdot \gamma_{M0}}{f_y} = 30.62 \cdot cm^2$

We adopt the following **CHS: 168.3 X 6.3**, giving a gross area of **32**. cm^2(value adopted due to the buckling verification)

- **Buckling resistance:**

$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 32.06cm^2$

$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$

$\gamma_{M1} := 1$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L_{br}$$

$E := 210GPa \qquad I_y := 1053.42cm^4 \qquad k := 0.75 \qquad$ (for brace)

$$L_{cr} := k \cdot L_{br} = 1.671\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 7815.788 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.382$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.617$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.907 \quad < 1 \qquad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1.032 \times 10^3 \cdot kN$$

$$N_{Ed} := 1025kN$$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.993 \qquad < 1$$

Design of top brace in tension:

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in brace:

$N_{Ed.t} := 439.81kN$

$N_{t.Rd} := 440kN$

$N_{t.Rd} = \frac{A_{net} \cdot f_y}{\gamma_{M0}}$ (6.6) EN 1993-1-1

$\gamma_{M0} := 1.0$

The type of steel used is S355, therefore: $f_y := 355 \frac{N}{mm^2}$

$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 12.394 \cdot cm^2$

We adopt the following **CHS: 108X4** , giving a gross area of **13.(** cm^2 .

Design of top diagonal brace:

- **Buckling resistance:**

$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 13.07cm^2$

$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$

$\gamma_{M1} := 1$

$L_{br.d} := 4700mm$

$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$ $L_{cr} = k \cdot L_{br.d}$

$E := 210GPa$ $I_y := 176.95cm^4$ $k := 0.75$ (for brace)

$$L_{cr} := k \cdot L_{br.d} = 3.525\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^{\ 2}} = 295.156 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 1.254$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.544$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.409 \quad < 1 \qquad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 189.725 \cdot kN$$

$$N_{Ed} := 182.23kN$$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.96 \qquad < 1$$

Resistance of joints:

Calculations made according EC 1993-1-8 and CIDECT recommendations.

Joint 1

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 355MPa$ steel strength

$d_0 := 323.9mm$ $t_0 := 12.5mm$ $d_1 := 168.3mm$ $t_1 := 6.3mm$ $d_2 := 168.3mm$ $t_2 := 6.3mm$

Stresses in the chord:

$N_{0.Ed} := -3259.55kN$ axial force in chord

$N_{1.Ed} := 1086.44kN$ axial force in left brace

$N_{2.Ed} := -1097.68kN$ axial force in right brace

$\Theta_{1.2} := 31 \cdot deg$ angle between chord and brace

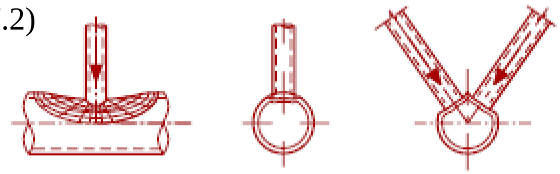
$$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos\left(\Theta_{1.2}\right) + N_{2.Ed} \cdot \cos\left(\Theta_{1.2}\right)\right) = -3.25 \times 10^{3} \cdot kN$$ (EC1993-1-8, 7.2)

$A_0 := 96.00cm^2$ area of the chord cross section

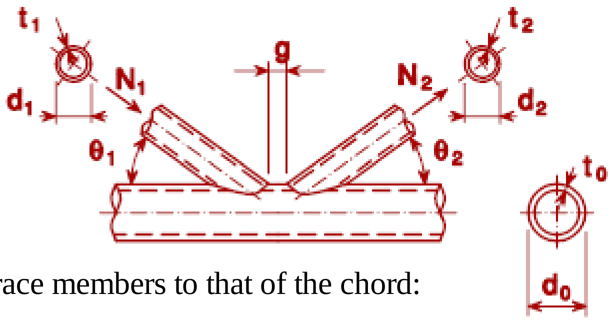
$$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} = -338.533 \cdot MPa$$ stress in the chord

<u>Chord face failure mode:</u>

For K and N gap joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = -0.954$$ (chord is in tension)



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_1 := \frac{d_0}{2 \cdot t_0} = 12.956$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_1 := \frac{d_1 + d_2}{2 \cdot d_0} = 0.52$$

$g_1 := 212.72mm$

$$k_g := \gamma_1^{0.2} \cdot \left[1 + \frac{0.024 \cdot \gamma^{1.2}}{1 + e^{\left(0.5 \cdot \frac{g_1}{t_0} - 1.33\right)}}\right] = 1.669$$

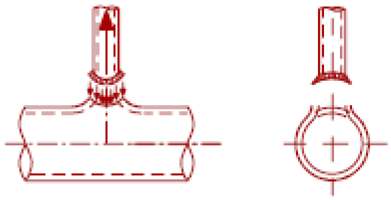
$k_p := 1.0$

Resistance of the joint:

$$N_{1.Rd} := \frac{\frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin(\Theta_{1.2})} \cdot \left(1.8 + 10.2 \cdot \frac{d_1}{d_0}\right)}{\gamma_{M5}} = 1.276 \times 10^3 \cdot kN$$

$$N_{2.Rd} := N_{1.Rd} = 1.276 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.Rd}} = 0.851$$

Punching shear failure mode:



When:

$$d_1 = 0.168\,m \quad < \quad d_0 - 2 \cdot t_0 = 0.299\,m$$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_{1.2})}{2 \cdot \sin(\Theta_{1.2})}}{\gamma_{M5}} = 1.992 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = 0.545$$

Multiplanar KK joints at gap should satisfy (EN1993-1-8, Table 7.7):

$N_{0.Ed} = -3.26 \times 10^3 \cdot kN$ axial force in the gap;

$N_{pl.0.Rd} := A_0 \cdot f_{y0} = 3.408 \times 10^3 \cdot kN$ resistance of the section;

$V_{0.Ed} := 10.95kN$

$$V_{pl.0.Rd} := 0.58 \cdot f_{y0} \cdot 2 \cdot \frac{A_0}{\pi} = 1.258 \times 10^3 \cdot kN$$

$$\left(\frac{N_{0.Ed}}{N_{pl.0.Rd}}\right)^2 + \left(\frac{V_{0.Ed}}{V_{pl.0.Rd}}\right)^2 = 0.915 \quad <1$$

Resistance of the joint 1 is sufficient.

Joint 2

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 355MPa$ steel strength

$d_0 := 219.1mm$ $t_0 := 12.5mm$ $d_1 := 168.3mm$ $t_1 := 6.3mm$

Stresses in the chord:

$N_{0.Ed} := 834.79kN$ axial force in chord

$N_{1.Ed} := -1097.68kN$ axial force in the brace

$\Theta_1 := 31 \cdot deg$ angle between chord and brace

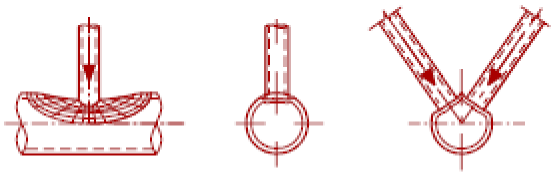
$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos\left(\Theta_1\right)\right) = 1.776 \times 10^3 \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 81.13 cm^2$ area of the chord cross section

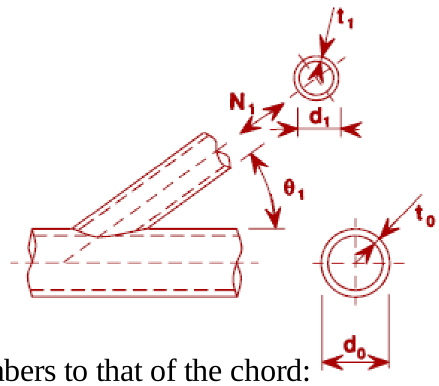
$\sigma_{p.Ed} := \dfrac{N_{p.Ed}}{A_0} = 218.869 \cdot MPa$ stress in the chord

<u>Chord face failure mode:</u>

For Y type joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\dfrac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = 0.617$$ (chord is in compression)



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_2 := \frac{d_0}{2 \cdot t_0} = 8.764$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_2 := \frac{d_1}{d_0} = 0.768$$

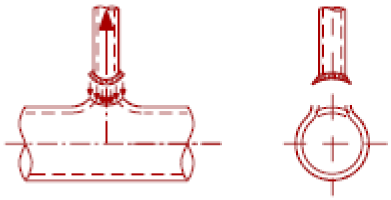
$$k_p := 1 - 0.3 \cdot n_p \cdot \left(1 + n_p\right) = 0.701$$

Resistance of the joint:

$$N_{1.Rd} := \frac{\gamma_2^{0.2} \cdot \dfrac{k_p \cdot f_{y0} \cdot t_0^2}{\sin\left(\Theta_1\right)} \cdot \left(2.8 + 14.2 \cdot \beta_2^2\right)}{\gamma_{M5}} = 1.303 \times 10^6 N$$

$$\frac{N_{1.Ed}}{N_{1.Rd}} = -0.843$$

Punching shear failure mode:



When:

$d_1 = 0.168\,m \quad <= \quad d_0 - 2 \cdot t_0 = 0.194\,m$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_1)}{2 \cdot \sin(\Theta_1)}}{\gamma_{M5}} = 1.992 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = -0.551$$

Resistance of the joint 2 is sufficient.

Joint 3

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 355MPa$ steel strength

$d_0 := 219.1mm \quad t_0 := 12.5mm \quad d_1 := 168.3mm \quad t_1 := 6.3mm$

Stresses in the chord:

$N_{0.Ed} := 1590kN$ axial force in chord

$N_{1.Ed} := 1087kN$ axial force in the brace

$M_{0.Ed} := 36.02kN \cdot m$ bending moment in the top chord

$W_{el.0} := 396580mm^3$

$\Theta_1 := 31 \cdot deg$ angle between chord and brace

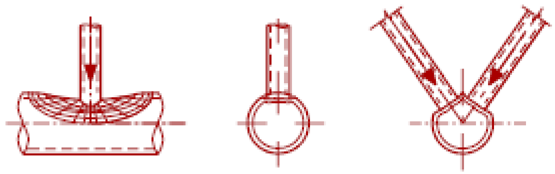
$N_{p.Ed} := N_{0.Ed} - (N_{1.Ed} \cdot \cos(\Theta_1)) = 658.259 \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 81.13cm^2$ area of the chord cross section

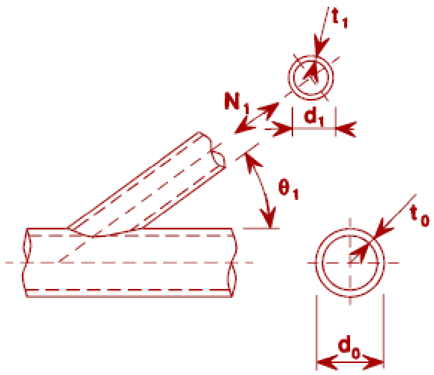
$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} + \frac{M_{0.Ed}}{W_{el.0}} = 171.963 \cdot MPa$ stress in the chord

Chord face failure mode:

For Y type joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = 0.484$$ (chord is in compression)



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_2 := \frac{d_0}{2 \cdot t_0} = 8.764$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_2 := \frac{d_1}{d_0} = 0.768$$

$$k_p := 1 - 0.3 \cdot n_p \cdot \left(1 + n_p\right) = 0.784$$

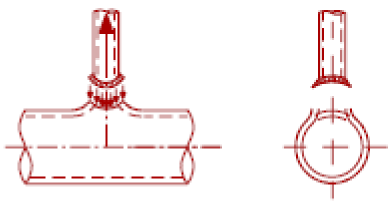
Resistance of the joint.

$$N_{1.Rd} := \frac{\gamma_2^{\,0.2} \cdot \frac{k_p \cdot f_{y0} \cdot t_0^{\,2}}{\sin\left(\Theta_1\right)} \cdot \left(2.8 + 14.2 \cdot \beta_2^{\,2}\right)}{\gamma_{M5}} = 1.458 \times 10^3 \cdot kN$$

$$\frac{N_{1.Ed}}{N_{1.Rd}} = 0.746$$

Punching shear failure mode:

When:



$d_1 = 0.168\,m$ <= $d_0 - 2 \cdot t_0 = 0.194\,m$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_1)}{2 \cdot \sin(\Theta_1)}}{\gamma_{M5}} = 1.992 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = 0.546$$

Resistance of the joint 3 is sufficient.

Design of welds

Minimum throat thickness (CIDECT 3.9, p. 24):

$a \geq 1.1 \cdot t_1$ for S355

$a := 1.1 \cdot t_1 = 6.93 \cdot mm$

$a := 7mm$

$f_u := 470MPa$

$\beta_w := 1.0$

$\gamma_{M2} := 1.25$

$l_{cir} := 750mm$

$$F_{w.Ed} := \frac{N_{1.Ed}}{l_{cir}} = 1.449 \times 10^3 \cdot \frac{kN}{m}$$

$$f_{vw.d} := \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = 2.171 \times 10^8 Pa$$

$$F_{w.Rd} := f_{vw.d} \cdot a = 1.52 \times 10^3 \cdot \frac{kN}{m}$$

$$\frac{F_{w.Ed}}{F_{w.Rd}} = 0.954$$

Fillet welds with throat thickness of 7mm is used for the joints. Fillet material is S355.

Design of Circular Truss Chords and Braces S500

- according to EN 1993-1-1 & EN 1993-1-12

Chords checking

- **Tension:**

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the lower chord:

$$N_{Ed.t} := 3259.55kN$$

$$N_{t.Rd} := 3260kN$$

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad \text{EN 1993-1-1}$$

$$\gamma_{M0} := 1.0$$

The type of steel used is S 500MC, therefore $f_y := 500 \frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 65.2 \cdot cm^2$$

We adopt the following **CHS: 273.0 X 10**, giving a gross area of **82**. cm^2 (adopted from joint verification)

- **Compression:**

$$\frac{N_{Ed}}{N_{c.Rd}} < 1 \qquad (6.9)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the upper chord:

$N_{Ed.c} := 1589.37kN$

Cross-section classification:

- according to Table 5.2 from EN 1993-1-1

$\varepsilon := \sqrt{\frac{235}{500}} = 0.686 \qquad \varepsilon^2 = 0.47$

$d := 193.7mm \qquad t := 10mm$

$\frac{d}{t} = 19.37 \quad < \; 50 \cdot \varepsilon^2 = 23.5$

Therefore, the Cross-section is **Class 1**.

$N_{c.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ (6.10) EN 1993-1-1

$N_{c.Rd} := 1621kN$

$\gamma_{M0} := 1$

$f_y := 500 \frac{N}{mm^2}$ (Table 1, EN 1993-1-12)

$A := \frac{N_{c.Rd} \cdot \gamma_{M0}}{f_y} = 32.42 \cdot cm^2$

We adopt the following **CHS: 193.7 X 10**, giving a gross area of **57.7** cm^2 (value adopted due to joint verification)

- **Buckling resistance:**

$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 57.71cm^2$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L$$

$E := 210GPa$ $\quad I_y := 2441.59cm^4$ $\quad L := 4m$ $\quad k := 0.9$ (for chord)

$$L_{cr} := k \cdot L = 3.6\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3904.692 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.86$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.031$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.625 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1.803 \times 10^3 \cdot kN$$

$$N_{Ed} := 1589.37kN$$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.882 \quad < 1$$

Diagonals checking

$$L_{br} := 2.5m - \frac{0.273m}{2} - \frac{0.193m}{2} = 2.267\,m$$

- **Braces in tension:**

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in diagonal:

$$N_{Ed.t} := 1097.68kN$$

$$N_{t.Rd} := 1098kN$$

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad \text{EN 1993-1-1}$$

$$\gamma_{M12} := 1.0$$

The type of steel used is S 500MC, therefore $f_y := 500\frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 21.96 \cdot cm^2$$

We adopt the following **CHS: 168.3 X 6**, giving a gross area of **30**. cm^2

- **Compression:**

$$\frac{N_{Ed}}{N_{c.Rd}} < 1 \qquad (6.9)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the diagonal:

$$N_{Ed.c} := 1086.44kN$$

Cross-section classification:

- according to Table 5.2 from EN 1993-1-1

$$\varepsilon := \sqrt{\frac{235}{500}} = 0.686 \qquad \varepsilon^2 = 0.47$$

$$d := 168.3mm \qquad t := 6mm$$

$$\frac{d}{t} = 28.05 \quad < \; 70 \cdot \varepsilon^2 = 32.9$$

Therefore, the Cross-section is **Class 2**.

$$N_{c.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \quad (6.10) \text{ EN 1993-1-1}$$

$$N_{c.Rd} := 1086.44kN$$

$$\gamma_{M0} := 1$$

$$f_y := 500 \frac{N}{mm^2} \quad \text{(Table 1, EN 1993-1-12)}$$

$$A := \frac{N_{c.Rd} \cdot \gamma_{M0}}{f_y} = 21.729 \cdot cm^2$$

We adopt the following **CHS: 168.3 X 6**, giving a gross area of **30**. cm^2 (value adopted due to the joint verification)

- **Buckling resistance:**

$$\frac{N_{Ed}}{N_{b.Rd}} \leq 1 \quad (6.46) \text{ EN 1993-1-1}$$

$$A := 30.59cm^2$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L_{br}$$

$E := 210GPa \qquad I_y := 1008.69cm^4 \qquad k := 0.75$ (for brace)

$$L_{cr} := k \cdot L_{br} = 1.7\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 7231.88 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.46$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.669$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.865 \quad < 1 \qquad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1.323 \times 10^6 N$$

$$N_{Ed} := 1089.44kN$$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.823 \qquad < 1$$

Design of top brace in tension:

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in brace:

$N_{Ed.t} := 439.81kN$

$N_{t.Rd} := 440kN$

$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ EN 1993-1-1

$\gamma_{M12} := 1.0$

The type of steel used is S 500MC, therefore $f_u := 500 \frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 8.8 \cdot cm^2$

We adopt the following **CHS: 114.3x3**, giving a gross area of **10.4** cm^2.

Resistance of diagonal braces:

- **Buckling resistance:**

$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1 $L_{br.d} := 4700mm$

$A := 10.49cm^2$

$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$

$\gamma_{M1} := 1$

$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$ $L_{cr} = k \cdot L_{br}$

$E := 210GPa$ $I_y := 162.55cm^4$ $k := 0.75$ (for brace)

$L_{cr} := k \cdot L_{br.d} = 3.525\,m$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^{\ 2}} = 271.136 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 1.391$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$\alpha := 0.49$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.759$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.353 \quad < 1 \qquad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1.85 \times 10^5 N$$

$N_{Ed} := 182.23kN$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.985 \quad < 1$$

Resistance of joints:

Calculations made according EC 1993-1-8 and CIDECT recommendations.

Joint 1

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 500MPa$ steel strength

$d_0 := 273mm$ $t_0 := 10mm$ $d_1 := 168.3mm$ $t_1 := 6mm$ $d_2 := 168.3mm$ $t_2 := 6mm$

Stresses in the chord:

$N_{0.Ed} := -3259.55kN$ axial force in chord

$N_{1.Ed} := 1054kN$ axial force in left brace

$N_{2.Ed} := -1079kN$ axial force in right brace

$\Theta_{1.2} := 31 \cdot deg$ angle between chord and brace

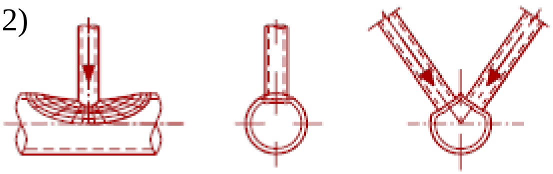
$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos\left(\Theta_{1.2}\right) + N_{2.Ed} \cdot \cos\left(\Theta_{1.2}\right)\right) = -3.238 \times 10^{3} \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 82.62cm^2$ area of the chord cross section

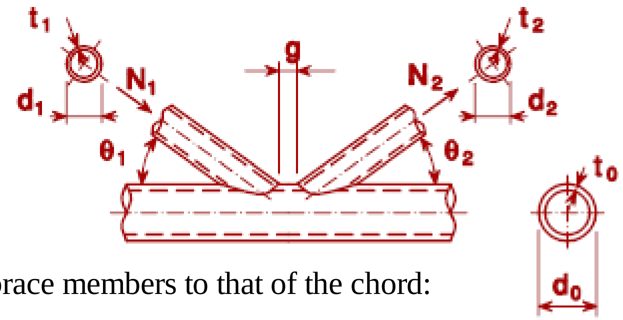
$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} = -391.929 \cdot MPa$ stress in the chord

Chord face failure mode:

For K and N gap joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = -0.784 \text{ (chord is in tension)}$$



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_1 := \frac{d_0}{2 \cdot t_0} = 13.65$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_1 := \frac{d_1 + d_2}{2 \cdot d_0} = 0.616$$

$g_1 := 127.88mm$

$$k_g := \gamma_1^{0.2} \cdot \left[1 + \frac{0.024 \cdot \gamma^{1.2}}{1 + e^{\left(0.5 \cdot \frac{g_1}{t_0} - 1.33\right)}}\right] = 1.687$$

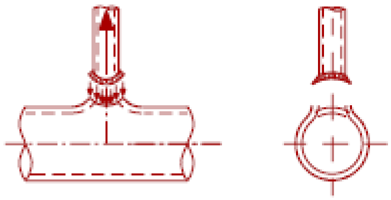
$k_p := 1.0$

Resistance reduction factor **0.8** is used for steel classes higher than S460.

$$N_{1.Rd} := 0.8 \cdot \frac{\frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin(\Theta_{1.2})} \cdot \left(1.8 + 10.2 \cdot \frac{d_1}{d_0}\right)}{\gamma_{M5}} = 1059.569 \cdot kN$$

$$N_{2.Rd} := N_{1.Rd} = 1059.569 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.Rd}} = 0.995$$

<u>Punching shear failure mode:</u>



When:

$$d_1 = 0.168\,m \quad < \quad d_0 - 2 \cdot t_0 = 0.253\,m$$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_{1.2})}{2 \cdot \sin(\Theta_{1.2})}}{\gamma_{M5}} \cdot 0.8 = 1.796 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = 0.587$$

Multiplanar KK joints at gap should satisfy (EN1993-1-8, Table 7.7):

$N_{0.Ed} = -3.26 \times 10^3 \cdot kN$ axial force in the gap;

$N_{pl.0.Rd} := A_0 \cdot f_{y0} = 4.131 \times 10^3 \cdot kN$ resistance of the section;

$$V_{0.Ed} := 10.95kN$$

$$V_{pl.0.Rd} := 0.58 \cdot f_{y0} \cdot 2 \cdot \frac{A_0}{\pi} = 1.525 \times 10^3 \cdot kN$$

$$\left(\frac{N_{0.Ed}}{N_{pl.0.Rd}}\right)^2 + \left(\frac{V_{0.Ed}}{V_{pl.0.Rd}}\right)^2 = 0.623 \quad <1$$

Resistance of the joint 1 is sufficient.

<u>Joint 2</u>

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 500MPa$ steel strength

$d_0 := 193.7mm \quad t_0 := 10mm \quad d_1 := 168mm \quad t_1 := 6mm$

<u>Stresses in the chord:</u>

$N_{0.Ed} := 834.79kN$ axial force in chord

$N_{1.Ed} := -1097.68kN$ axial force in the brace

$\Theta_1 := 31 \cdot deg$ angle between chord and brace

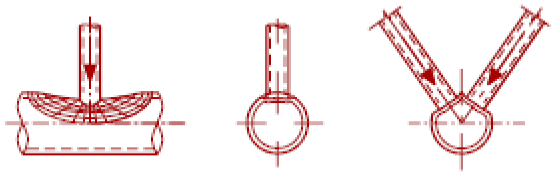
$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos\left(\Theta_1\right)\right) = 1.776 \times 10^3 \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 57.71cm^2$ area of the chord cross section

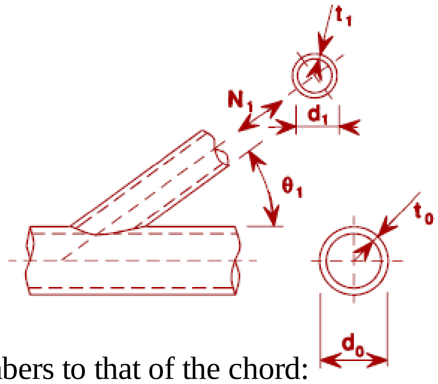
$\sigma_{p.Ed} := \dfrac{N_{p.Ed}}{A_0} = 307.691 \cdot MPa$ stress in the chord

<u>Chord face failure mode:</u>

For Y type joint. (EC1993-1-8, Table 7.2)



$n_p := \dfrac{\dfrac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = 0.615$ (chord is in compression)



The ratio of the chord width or diameter to twice it's wall thickness:

$\gamma_2 := \dfrac{d_0}{2 \cdot t_0} = 9.685$

The ratio of mean diameter or width of the brace members to that of the chord:

$\beta_2 := \dfrac{d_1}{d_0} = 0.867$

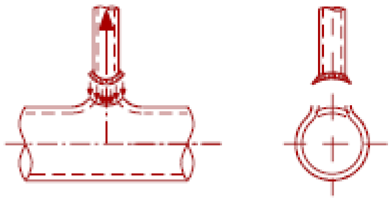
$k_p := 1 - 0.3 \cdot n_p \cdot \left(1 + n_p\right) = 0.702$

Resistance reduction factor **0.8** is used for steel classes higher than S460.

$$N_{1.Rd} := 0.8 \cdot \frac{\gamma_2^{0.2} \cdot \dfrac{k_p \cdot f_{y0} \cdot t_0^2}{\sin\left(\Theta_1\right)} \cdot \left(2.8 + 14.2 \cdot \beta_2^2\right)}{\gamma_{M5}} = 1.157 \times 10^3 \cdot kN$$

$\dfrac{N_{1.Ed}}{N_{1.Rd}} = -0.949$

Punching shear failure mode:



When:

$d_1 = 0.168\,m \quad < \quad d_0 - 2 \cdot t_0 = 0.174\,m$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_1)}{2 \cdot \sin(\Theta_1)}}{\gamma_{M5}} \cdot 0.8 = 1.793 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = -0.612$$

Resistance of the joint 2 is sufficient.

Joint 3

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 500MPa$ steel strength

$d_0 := 193.7mm \quad t_0 := 10mm \quad d_1 := 168mm \quad t_1 := 6mm$

Stresses in the chord:

$N_{0.Ed} := 1589.37kN$ axial force in chord

$N_{1.Ed} := 1086.44kN$ axial force in the brace

$M_{0.Ed} := 30.62kN \cdot m$ bending moment in the top chord

$W_{el.0} := 252100mm^3$

$\Theta_1 := 31 \cdot deg$ angle between chord and brace

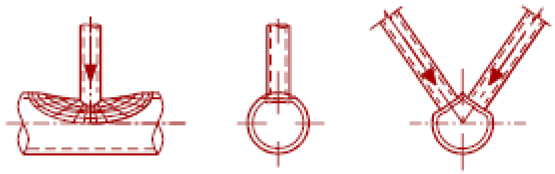
$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos(\Theta_1)\right) = 658.109 \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 57.71cm^2$ area of the chord cross section

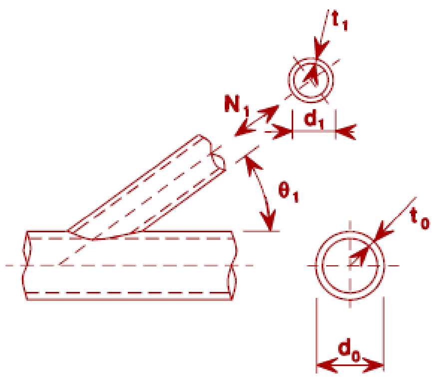
$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} + \frac{M_{0.Ed}}{W_{el.0}} = 235.497 \cdot MPa$ stress in the chord

Chord face failure mode:

For Y type joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = 0.471 \quad \text{(chord is in compression)}$$



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_2 := \frac{d_0}{2 \cdot t_0} = 9.685$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_2 := \frac{d_1}{d_0} = 0.867$$

$$k_p := 1 - 0.3 \cdot n_p \cdot \left(1 + n_p\right) = 0.792$$

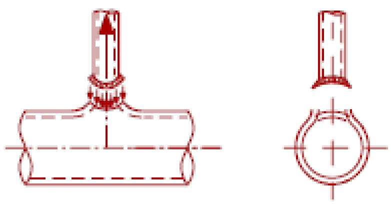
Resistance reduction factor **0.8** is used for steel classes higher than S460.

$$N_{1.Rd} := 0.8 \cdot \frac{\gamma_2^{0.2} \cdot \frac{k_p \cdot f_{y0} \cdot t_0^2}{\sin(\Theta_1)} \cdot \left(2.8 + 14.2 \cdot \beta_2^2\right)}{\gamma_{M5}} = 1.306 \times 10^3 \cdot kN$$

$$\frac{N_{1.Ed}}{N_{1.Rd}} = 0.832$$

Punching shear failure mode:

When:



$$d_1 = 0.168\,m \quad < \quad d_0 - 2 \cdot t_0 = 0.174\,m$$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1+\sin(\Theta_1)}{2\cdot\sin(\Theta_1)}}{\gamma_{M5}} \cdot 0.8 = 1.793 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = 0.606$$

Resistance of the joint 3 is sufficient.

Design of welds

Minimum throat thickness:

$a \geq 1.48 \cdot t_1$ for S460

$a := 1.48 \cdot t_1 = 8.88 \cdot mm$

$a := 8mm$

$f_u := 550MPa$

$\beta_w := 1.0$

$\gamma_{M2} := 1.25$

$l_{cir} := 850mm$

$$F_{w.Ed} := \frac{N_{1.Ed}}{l_{cir}} = 1.278 \times 10^3 \cdot \frac{kN}{m}$$

$$f_{vw.d} := \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = 2.54 \times 10^8 \, Pa$$

$$F_{w.Rd} := f_{vw.d} \cdot a = 2.032 \times 10^3 \cdot \frac{kN}{m}$$

$$\frac{F_{w.Ed}}{F_{w.Rd}} = 0.629$$

Fillet welds with throat thickness of 8mm is used for the joints. Fillet material is S460, as Eurocode suggests, fillet material can be different than base material.

Design of Circular Truss Chords and Braces S650

- according to EN 1993-1-1 & EN 1993-1-12

Chords checking

- **Tension:**

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the lower chord:

$N_{Ed.t} := 3259.55kN$

$N_{t.Rd} := 3260kN$

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad \text{EN 1993-1-1}$$

$\gamma_{M0} := 1.0$

The type of steel used is S 650MC, therefore $f_y := 650 \frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 50.154 \cdot cm^2$$

We adopt the following **CHS: 219.1 X 10**, giving a gross area of **65.6** cm^2 (adopted form joint verification)

- **Compression:**

$$\frac{N_{Ed}}{N_{c.Rd}} < 1 \qquad (6.9)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the upper chord:

$N_{Ed.c} := 1589.37kN$

Cross-section classification:

- according to Table 5.2 from EN 1993-1-1

$\varepsilon := \sqrt{\frac{235}{650}} = 0.601 \qquad \varepsilon^2 = 0.362$

$d := 193.7mm \qquad t := 10mm$

$\frac{d}{t} = 19.37 \quad < \ 70 \cdot \varepsilon^2 = 25.308$

Therefore, the Cross-section is **Class 2**.

$N_{c.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ (6.10) EN 1993-1-1

$N_{c.Rd} := 1621kN$

$\gamma_{M0} := 1$

$f_y := 650 \frac{N}{mm^2}$ (Table 1, EN 1993-1-12)

$$A := \frac{N_{c.Rd} \cdot \gamma_{M0}}{f_y} = 24.938 \cdot cm^2$$

We adopt the following **CHS: 193.7 X 10**, giving a gross area of **57.7** cm^2 (value adopted due to joint verification)

- **Buckling resistance:**

$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 57.71cm^2$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L$$

$E := 210GPa$ $\qquad I_y := 2441.59cm^4$ $\qquad L := 4m$ $\qquad k := 0.9$ (for chord)

$$L_{cr} := k \cdot L = 3.6\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3904.692 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.98$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.171$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.552 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 2.069 \times 10^3 \cdot kN$$

$$N_{Ed} := 1589.37kN$$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.768 \quad < 1$$

Diagonals checking

$$L_{br} := 2.5m - \frac{0.2191m}{2} - \frac{0.193m}{2} = 2.294\,m$$

- **Diagonals in tension:**

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in brace:

$$N_{Ed.t} := 1097.68kN$$

$$N_{t.Rd} := 1098kN$$

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad \text{EN 1993-1-1}$$

$$\gamma_{M0} := 1.0$$

The type of steel used is S 650MC, therefore $f_y := 650\frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 16.892 \cdot cm^2$$

We adopt the following **CHS: 127 X 6**, giving a gross area of **22.8** cm^2

- **Compression:**

$$\frac{N_{Ed}}{N_{c.Rd}} < 1 \qquad (6.9)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the upper chord:

$$N_{Ed.c} := 1086.44kN$$

<u>Cross-section classification:</u>

- according to Table 5.2 from EN 1993-1-1

$\varepsilon := \sqrt{\frac{235}{650}} = 0.601 \qquad \varepsilon^2 = 0.362$

$d := 127mm \qquad t := 6mm$

$\frac{d}{t} = 21.167 \quad < \quad 70 \cdot \varepsilon^2 = 25.308$

Therefore, the Cross-section is **Class 2**.

$N_{c.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ (6.10) EN 1993-1-1

$N_{c.Rd} := 1086.44kN$

$\gamma_{M0} := 1$

$f_y := 650 \frac{N}{mm^2}$ (Table 1, EN 1993-1-12)

$A := \frac{N_{c.Rd} \cdot \gamma_{M0}}{f_y} = 16.714 \cdot cm^2$

We adopt the following **CHS: 127 X 6**, giving a gross area of **22.8** cm^2 (value adopted due to the buckling verification)

- **<u>Buckling resistance:</u>**

$\frac{N_{Ed}}{N_{b.Rd}} \le 1$ (6.46) EN 1993-1-1

$A := 22.81cm^2$

$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$

$\gamma_{M1} := 1$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L_{br}$$

$E := 210GPa \qquad I_y := 458.44cm^4 \qquad k := 0.75 \quad$ (for brace)

$$L_{cr} := k \cdot L_{br} = 1.72\,m$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3210.045 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.68$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.848$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.737 \quad < 1 \qquad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1.093 \times 10^3 \cdot kN$$

$$N_{Ed} := 1086.44kN$$

$$\boxed{\frac{N_{Ed}}{N_{b.Rd}} = 0.994} \quad < 1$$

Design of top brace in tension:

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in brace:

$N_{Ed.t} := 439.81kN$

$N_{t.Rd} := 440kN$

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$$ EN 1993-1-12

$\gamma_{M12} := 1.0$

The type of steel used is S 650MC, therefore $f_y := 650 \frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 6.769 \cdot cm^2$$

We adopt the following **CHS: 114.3X3**, giving a gross area of **10.** cm^2

Resistance of top diagonl braces:

- **Buckling resistance:**

$$\frac{N_{Ed}}{N_{b.Rd}} \leq 1$$ (6.46) EN 1993-1-1 $L_{br.d} := 4700mm$

$A := 10.49cm^2$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$\gamma_{M1} := 1$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} \qquad L_{cr} = k \cdot L_{br.d}$$

$E := 210GPa$ $I_y := 162.55cm^4$ $k := 0.75$ (for brace)

$L_{cr} := k \cdot L_{br.d} = 3.525\,m$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 271.136 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 1.586$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$\alpha := 0.49$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 2.097$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.288 \quad < 1 \qquad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 196.562 \cdot kN$$

$N_{Ed} := 182.23kN$

$$\frac{N_{Ed}}{N_{b.Rd}} = 0.927 \qquad < 1$$

We adopt the following **CHS: 114.3X3**, giving a gross area of **10.** cm^2

Resistance of joints:

Calculations made according EC 1993-1-8 and CIDECT recommendations.

Joint 1

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 650MPa$ steel strength

$d_0 := 219.1mm$ $t_0 := 10mm$ $d_1 := 127mm$ $t_1 := 6mm$ $d_2 := 127mm$ $t_2 := 6mm$

Stresses in the chord:

$N_{0.Ed} := -3259.55kN$ axial force in chord

$N_{1.Ed} := 1086.44kN$ axial force in left brace

$N_{2.Ed} := -1097.68kN$ axial force in right brace

$\Theta_{1.2} := 31 \cdot deg$ angle between chord and brace

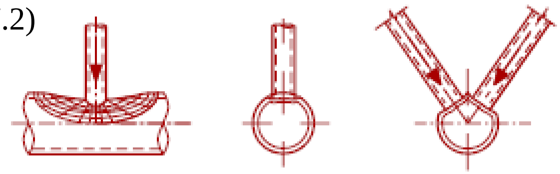
$$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos\left(\Theta_{1.2}\right) + N_{2.Ed} \cdot \cos\left(\Theta_{1.2}\right)\right) = -3.25 \times 10^{3} \cdot kN$$ (EC1993-1-8, 7.2)

$A_0 := 65.69cm^2$ area of the chord cross section

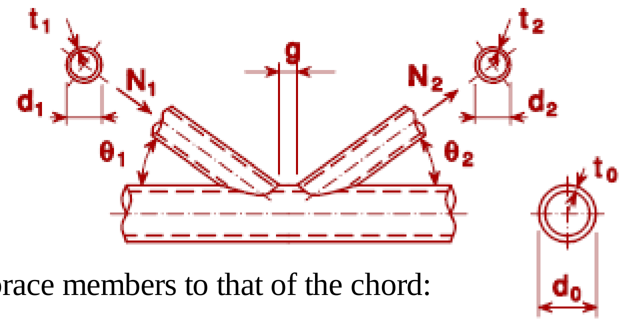
$$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} = -494.735 \cdot MPa$$ stress in the chord

<u>Chord face failure mode:</u>

For K and N gap joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = -0.761$$ (chord is in tension)



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_1 := \frac{d_0}{2 \cdot t_0} = 10.955$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_1 := \frac{d_1 + d_2}{2 \cdot d_0} = 0.58$$

$g_1 := 118.32mm$

$$k_g := \gamma_1^{0.2} \cdot \left[1 + \frac{0.024 \cdot \gamma^{1.2}}{1 + e^{\left(0.5 \cdot \frac{g_1}{t_0} - 1.33\right)}}\right] = 1.614$$

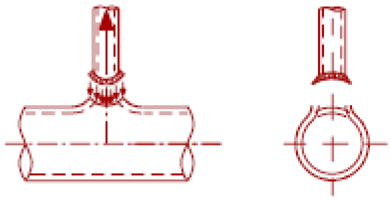
$k_p := 1.0$

Resistance reduction factor **0.8** is used for steel classes higher than S460.

$$N_{1.Rd} := 0.8 \cdot \frac{\frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin(\Theta_{1.2})} \cdot \left(1.8 + 10.2 \cdot \frac{d_1}{d_0}\right)}{\gamma_{M5}} = 1.257 \times 10^3 \cdot kN$$

$$N_{2.Rd} := N_{1.Rd} = 1.257 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.Rd}} = 0.864$$

Punching shear failure mode:



When:

$$d_1 = 0.127\,m \quad < \quad d_0 - 2 \cdot t_0 = 0.199\,m$$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_{1.2})}{2 \cdot \sin(\Theta_{1.2})}}{\gamma_{M5}} \cdot 0.8 = 1.762 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = 0.617$$

Multiplanar KK joints at gap should satisfy (EN1993-1-8, Table 7.7):

$N_{0.Ed} = -3.26 \times 10^3 \cdot kN$ axial force in the gap;

$N_{pl.0.Rd} := A_0 \cdot f_{y0} = 4.27 \times 10^3 \cdot kN$ resistance of the section;

$V_{0.Ed} := 10.95kN$

$$V_{pl.0.Rd} := 0.58 \cdot f_{y0} \cdot 2 \cdot \frac{A_0}{\pi} = 1.577 \times 10^3 \cdot kN$$

$$\left(\frac{N_{0.Ed}}{N_{pl.0.Rd}}\right)^2 + \left(\frac{V_{0.Ed}}{V_{pl.0.Rd}}\right)^2 = 0.583 \quad <1$$ **Resistance of the joint 1 is sufficient.**

Joint 2

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 650MPa$ steel strength

$d_0 := 193.7mm \quad t_0 := 10mm \quad d_1 := 127mm \quad t_1 := 6mm$

Stresses in the chord:

$N_{0.Ed} := 834.79kN$ axial force in chord

$N_{1.Ed} := -1097.68kN$ axial force in the brace

$\Theta_1 := 31 \cdot deg$ angle between chord and brace

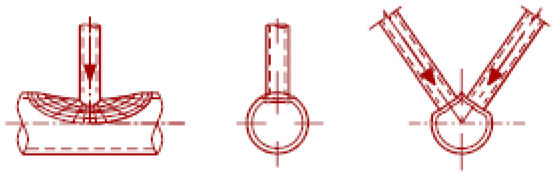
$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos(\Theta_1)\right) = 1.776 \times 10^3 \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 57.71cm^2$ area of the chord cross section

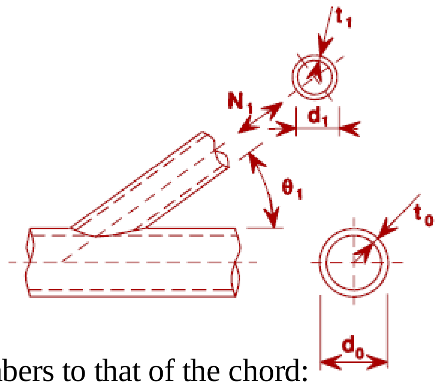
$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} = 307.691 \cdot MPa$ stress in the chord

Chord face failure mode:

For Y type joint. (EC1993-1-8, Table 7.2)



$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = 0.473$ (chord is in compression)



The ratio of the chord width or diameter to twice it's wall thickness:

$\gamma_2 := \frac{d_0}{2 \cdot t_0} = 9.685$

The ratio of mean diameter or width of the brace members to that of the chord:

$\beta_2 := \frac{d_1}{d_0} = 0.656$

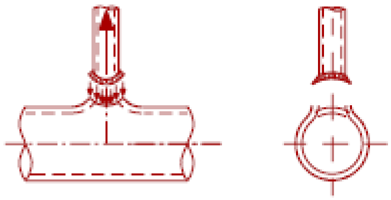
$k_p := 1 - 0.3 \cdot n_p \cdot \left(1 + n_p\right) = 0.791$

Resistance reduction factor **0.8** is used for steel classes higher than S460.

$$N_{1.Rd} := 0.8 \cdot \frac{\gamma_2^{0.2} \cdot \frac{k_p \cdot f_{y0} \cdot t_0^2}{\sin(\Theta_1)} \cdot \left(2.8 + 14.2 \cdot \beta_2^2\right)}{\gamma_{M5}} = 1.12 \times 10^3 \cdot kN$$

$$\frac{N_{1.Ed}}{N_{1.Rd}} = -0.98$$

Punching shear failure mode:



When:

$d_1 = 0.127\,m \quad < \quad d_0 - 2 \cdot t_0 = 0.174\,m$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_1)}{2 \cdot \sin(\Theta_1)}}{\gamma_{M5}} \cdot 0.8 = 1.762 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = -0.623$$

Resistance of the joint 2 is sufficient.

Joint 3

$\gamma_{M5} := 1.0$ partial safety factor for resistance of joints in HS girders

$f_{y0} := 650MPa$ steel strength

$d_0 := 193.7mm \quad t_0 := 10mm \quad d_1 := 127mm \quad t_1 := 6mm$

Stresses in the chord:

$N_{0.Ed} := 1589.37kN$ axial force in chord

$N_{1.Ed} := 1086.44kN$ axial force in the brace

$M_{0.Ed} := 30.62kN \cdot m$ bending moment in the top chord

$W_{el.0} := 252100mm^3$

$\Theta_1 := 31 \cdot deg$ angle between chord and brace

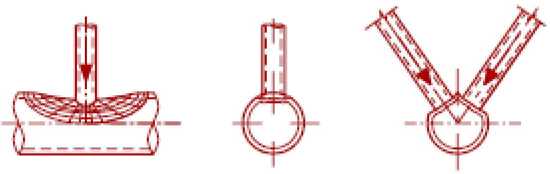
$N_{p.Ed} := N_{0.Ed} - \left(N_{1.Ed} \cdot \cos(\Theta_1)\right) = 658.109 \cdot kN$ (EC1993-1-8, 7.2)

$A_0 := 57.71cm^2$ area of the chord cross section

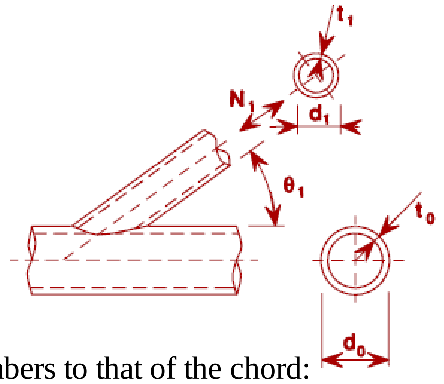
$\sigma_{p.Ed} := \frac{N_{p.Ed}}{A_0} + \frac{M_{0.Ed}}{W_{el.0}} = 235.497 \cdot MPa$ stress in the chord

Chord face failure mode:

For Y type joint. (EC1993-1-8, Table 7.2)



$$n_p := \frac{\frac{\sigma_{p.Ed}}{f_{y0}}}{\gamma_{M5}} = 0.362$$ (chord is in compression)



The ratio of the chord width or diameter to twice it's wall thickness:

$$\gamma_2 := \frac{d_0}{2 \cdot t_0} = 9.685$$

The ratio of mean diameter or width of the brace members to that of the chord:

$$\beta_2 := \frac{d_1}{d_0} = 0.656$$

$$k_p := 1 - 0.3 \cdot n_p \cdot (1 + n_p) = 0.852$$

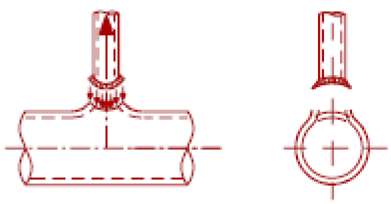
Resistance reduction factor **0.8** is used for steel classes higher than S460.

$$N_{1.Rd} := 0.8 \cdot \frac{\gamma_2^{0.2} \cdot \frac{k_p \cdot f_{y0} \cdot t_0^2}{\sin(\Theta_1)} \cdot \left(2.8 + 14.2 \cdot \beta_2^2\right)}{\gamma_{M5}} = 1.206 \times 10^3 \cdot kN$$

$$\frac{N_{1.Ed}}{N_{1.Rd}} = 0.901$$

Punching shear failure mode:

When:



$$d_1 = 0.127 m \quad < \quad d_0 - 2 \cdot t_0 = 0.174 m$$

$$N_{1.2.Rd} := \frac{\frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{1 + \sin(\Theta_1)}{2 \cdot \sin(\Theta_1)}}{\gamma_{M5}} \cdot 0.8 = 1.762 \times 10^3 \cdot kN \qquad \frac{N_{1.Ed}}{N_{1.2.Rd}} = 0.617$$

Resistance of the joint 3 is sufficient.

Design of welds

Minimum throat thickness:

$a \geq 1.48 \cdot t_1$ for S460

$a := 1.48 \cdot t_1 = 8.88 \cdot mm$

$a := 8mm$

$f_u := 700MPa$

$\beta_w := 1.0$

$\gamma_{M2} := 1.25$

$l_{cir} := 750mm$

$$F_{w.Ed} := \frac{N_{1.Ed}}{l_{cir}} = 1.449 \times 10^3 \cdot \frac{kN}{m}$$

$$f_{vw.d} := \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = 3.233 \times 10^8 Pa$$

$$F_{w.Rd} := f_{vw.d} \cdot a = 2.587 \times 10^3 \cdot \frac{kN}{m}$$

$$\frac{F_{w.Ed}}{F_{w.Rd}} = 0.56$$

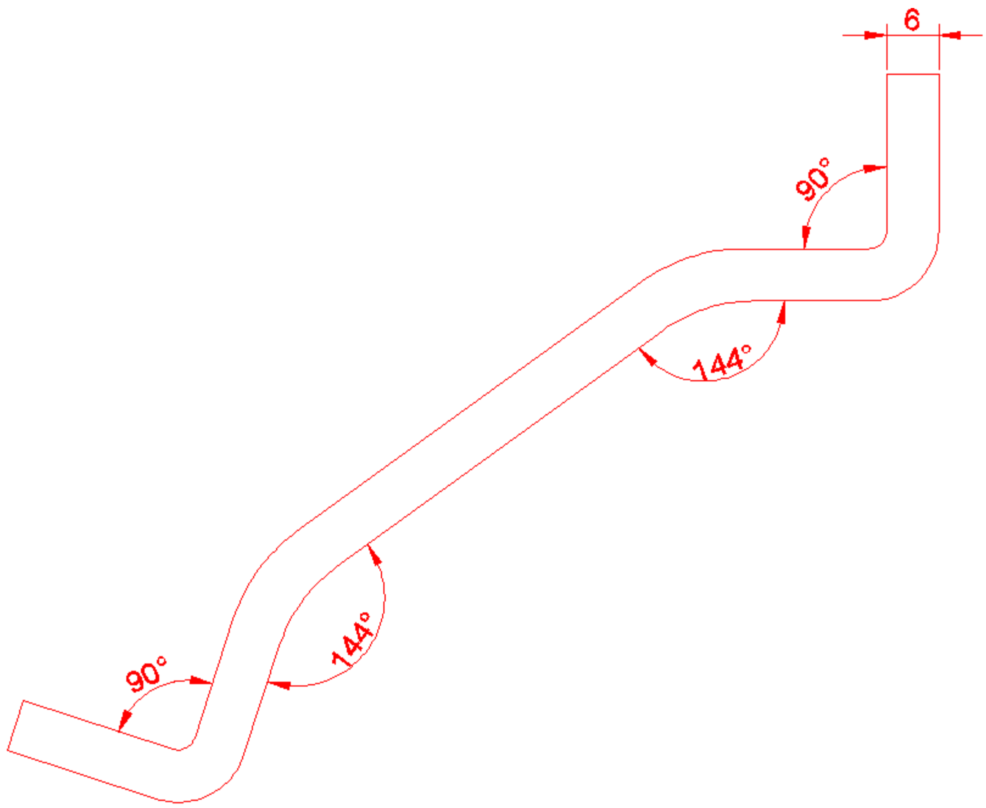
Fillet welds with throat thickness of 8mm is used for the joints. Fillet material is S460, as Eurocode suggests, fillet material can be different than base material.

ANNEX B

Design calculation of the folded plates of the polygonal cross-section S355

Design of the compressed top chord made from semi-closed polygonal sections:

-according to EN 1993-1-1, EN 1993-1-3, EN 1993-1-5



Section properties:

$t := 6mm$ - plate thickenss

$\nu := 0.3$ - Poisson's ration

$f_{yb} := 355MPa$ - basic yield strength

$E := 210GPa$ - Modulus of Elasticity

$r_1 := 2mm$

$r_2 := 20mm$ - bent corner radii

$r_{m1} := r_1 + 0.5 \cdot t = 5 \cdot mm$

$r_{m2} := r_2 + 0.5 \cdot t = 23 \cdot mm$ -radii at midpoint of corner

$f_u := 470MPa$ - ultimate yielding strength

$L_{plate} := 4m$ -length of plate

$$\varepsilon := \sqrt{\frac{235MPa}{f_{yb}}} = 0.814$$ -strain coefficient

Normal widths of the flat parts

$b_1 := 18mm$

$b_2 := 18.58mm$

$b_3 := 61.17mm$

$b_4 := b_2$

$b_5 := b_1$

Section classification

- *Class of b.1 part*

$$\frac{b_1}{t} = 3 \quad < \ 33 \cdot \varepsilon = 26.849$$

$Class_{b1} := 1$

- *Class of b.2 part*

$$\frac{b_2}{t} = 3.097 \quad < \ 33 \cdot \varepsilon = 26.849$$

$Class_{b2} := 1$

- *Class of b.3 part*

$$\frac{b_3}{t} = 10.195 \ < \ 33 \cdot \varepsilon = 26.849$$

$Class_{b3} := 1$

- *Class of b.4 part*

$Class_{b4} := Class_{b2}$

- *Class of b.5 part*

$Class_{b5} := Class_{b1}$

Influence of the corners (5.1) EN 1993-1-3

Corners' arch lengths:

$$\phi_1 := 90 \cdot \frac{\pi}{180} = 1.571$$

$$\phi_2 := 36 \cdot \frac{\pi}{180} = 0.628$$

$$u_1 := \phi_1 \cdot r_{m1} = 7.854 \cdot mm$$

$$u_2 := \phi_2 \cdot r_{m2} = 14.451 \cdot mm$$

Notional widths of plane cross section parts *b*p allowing for corner radii Fig. 5.1, EN 1993-1-3

$$g_{r1} := r_{m1} \cdot \left(\tan\left(\frac{\phi_1}{2}\right) - \sin\left(\frac{\phi_1}{2}\right) \right) = 1.464 \cdot mm$$

$$g_{r2} := r_{m2} \cdot \left(\tan\left(\frac{\phi_2}{2}\right) - \sin\left(\frac{\phi_2}{2}\right) \right) = 0.366 \cdot mm$$

$$b_{p1} := b_1 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) - g_{r1} = 21.536 \cdot mm$$

$$b_{p2} := b_2 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) + r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r1} - g_{r2} = 29.223 \cdot mm$$

$$b_{p3} := b_3 + 2 \cdot \left(r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r2} \right) = 75.385 \cdot mm$$

$$b_{p4} := b_{p2} = 29.223 \cdot mm$$

$$b_{p5} := b_{p1} = 21.536 \cdot mm$$

Maximum width to thickness ratios (Table 5.1, EN 1993-1-3)

$\phi := 36\text{deg}$

$\frac{b_{p1}}{t} \leq 50 = 1$

$\frac{b_{p2}}{t} \leq 500 \cdot \sin(\phi) = 1$

$\frac{b_{p3}}{t} \leq 50 = 1$

Since all the geometrical ratios are inside the limits, the provisions of EN 1993-1-3 may be applied.

Average yield strenght (3.2.2) EN 1993-1-3

$n := \frac{2 \cdot 90\text{deg} + 2 \cdot 36\text{deg}}{90\text{deg}} = 2.8$ the number of 90° bends in the cross-section with an internal radius <= 5*t*

$k := 7$ - numerical coefficient for roll forming

$A_g := 10.33\text{cm}^2$ - gross area (value taken from AutoCAD)

$f_{ya} := f_{yb} + \left(f_u - f_{yb}\right) \cdot \frac{k \cdot n \cdot t^2}{A_g} = 433.552 \cdot \text{MPa}$ (3.1) - average yield strength

$f_{ya} \leq \frac{f_u + f_{yb}}{2} = 0$

then:

$f_{ya} := 412.5\text{MPa}$

Determination of effective widths for a plane element without stiffeners (5.5.2 EN 1993-1-3)

The effective widths of the element will be equal to the calculated widths, since the class of the cross-section is Class 1.

Axial compression resistance (6.1.3 EN 1993-1-3)

$$A_{full} := 5 \cdot A_g = 5.165 \times 10^3 \cdot mm^2$$

$k_{\sigma1} := 0.43$ $k_{\sigma2} := 4$ $k_{\sigma3} := 4$ - buckling factors (Table 4.1 and 4.2, EN 1993-1-5)

$$\lambda_{p1} := \frac{\frac{b_{p1}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma1}}} = 0.237$$

$$\lambda_{p2} := \frac{\frac{b_{p2}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma2}}} = 0.105$$

- plate slendernesses (4.4 EN 1993-1-5)

$$\lambda_{p3} := \frac{\frac{b_{p3}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma3}}} = 0.272$$

$\lambda := \max(\lambda_{p1}, \lambda_{p2}, \lambda_{p3}) = 0.272$ - element slenderness

$$\lambda_{e0} := 0.673$$

$$\gamma_{M0} := 1$$

$$N_{c.Rd} := \frac{A_{full} \cdot \left[f_{yb} + (f_{ya} - f_{yb}) \cdot 4 \cdot \left(1 - \frac{\lambda}{\lambda_{e0}}\right)\right]}{\gamma_{M0}} = 2.542 \times 10^3 \cdot kN$$

but

$$N_{c.Rd} > \frac{A_{full} \cdot f_{ya}}{\gamma_{M0}} = 2130.563 \cdot kN$$

then:

$$N_{c.Rd} := 2130.563kN$$

Local buckling

Critical stress

$k_{\sigma 1} = 0.43$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.51} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_1^{\,2}} = 9068.226 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.31} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_{p1}^{\,2}} = 6335.138 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.51} := \sigma_{crit.1.51} \cdot A_{full} = 46837.389 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.31} := \sigma_{crit.1.31} \cdot A_{full} = 32720.987 \cdot kN$$

Critical stress

$k_{\sigma 3} = 4$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.53} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_3^{\,2}} = 7304.356 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.33} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_{p3}^{\,2}} = 4809.407 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.53} := \sigma_{crit.1.53} \cdot A_{full} = 37726.999 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.33} := \sigma_{crit.1.33} \cdot A_{full} = 24840.587 \cdot kN$$

Global buckling

$\gamma_{M1} := 1$

$\lambda_1 := 93.9 \cdot \varepsilon = 76.399$

$A_{full} = 51.65 \cdot cm^2$

$I := 34721185.8635mm^4$

$i := \sqrt{\frac{I}{A_{full}}} = 81.99 \cdot mm$ -radius of gyration

$L_{cr} := 0.9 \cdot L_{plate} = 3.6\,m$

$\lambda := \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1} = 0.575$ (6.50 EN 1993-1-1)

The buckling curve used will be curve "c".
Therefore, we adopt the following imperfection facto $\alpha := 0.49$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.757$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 0.8$ $\chi < 1$ - reduction factor

$N_{b.Rdglob} := \frac{\chi \cdot N_{c.Rd} \cdot \gamma_{M0}}{\gamma_{M1}} = 1705.017 \cdot kN$

Ultimate load resistance

$N_{b.Rd} := \min\left(N_{c.Rd}, N_{b.Rdglob}\right) = 1705.017 \cdot kN$

Axial load in the top cmpressed chord:

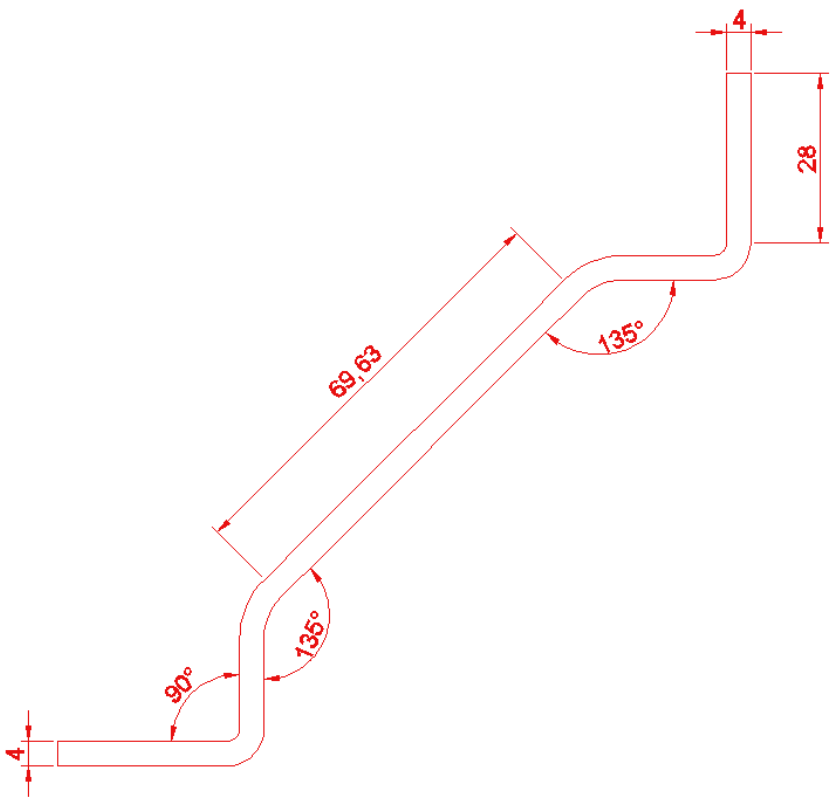
$N_{Ed} := 1589.37kN$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.932$ < 1

Top chord is resistant to compression force by applied load.

Design of the compressed braces made from semi-closed polygonal sections:

-according to EN 1993-1-1, EN 1993-1-3, EN 1993-1-5



__Section properties:__

$t := 4mm$ - plate thickenss

$\nu := 0.3$ - Poisson's ration

$f_{yb} := 355MPa$ - basic yield strength

$E := 210GPa$ - Modulus of Elasticity

$r_1 := 2mm$

$r_2 := 10mm$ - bent corner radii

$r_{m1} := r_1 + 0.5 \cdot t = 4 \cdot mm$

$r_{m2} := r_2 + 0.5 \cdot t = 12 \cdot mm$ -radii at midpoint of corner

$f_u := 470MPa$ - ultimate yielding strength

$L_{plate} := 2.5m - 0.219m = 2.281\,m$ -length of plate

$\varepsilon := \sqrt{\frac{235MPa}{f_{yb}}} = 0.814$ -strain coefficient

Normal widths of the flat parts

$b_1 := 18mm$

$b_3 := 49.63mm$

$b_5 := b_1$

Section classification

- *Class of b.1 part*

$\frac{b_1}{t} = 4.5 \; < \; 33 \cdot \varepsilon = 26.849$

$Class_{b1} := 1$

- *Class of b.3 part*

$\frac{b_3}{t} = 12.408 \quad < 33 \cdot \varepsilon = 26.849$

$Class_{b3} := 1$

- *Class of b.5 part*

$Class_{b5} := Class_{b1}$

Influence of the corners (5.1) EN 1993-1-3

Corners' arch lengths:

$\phi_1 := 90 \cdot \frac{\pi}{180} = 1.571$

$\phi_2 := 45 \cdot \frac{\pi}{180} = 0.785$

$u_1 := \phi_1 \cdot r_{m1} = 6.283 \cdot mm$

$u_2 := \phi_2 \cdot r_{m2} = 9.425 \cdot mm$

Notional widths of plane cross section parts *b*p allowing for corner radii Fig. 5.1, EN 1993-1-3

$$g_{r1} := r_{m1} \cdot \left(\tan\left(\frac{\phi_1}{2}\right) - \sin\left(\frac{\phi_1}{2}\right) \right) = 1.172 \cdot mm$$

$$g_{r2} := r_{m2} \cdot \left(\tan\left(\frac{\phi_2}{2}\right) - \sin\left(\frac{\phi_2}{2}\right) \right) = 0.378 \cdot mm$$

$$b_{p1} := b_1 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) - g_{r1} = 20.828 \cdot mm$$

$$b_{p3} := b_3 + 2 \cdot \left(r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r2} \right) = 58.814 \cdot mm$$

$$b_{p5} := b_{p1} = 20.828 \cdot mm$$

Maximum width to thickness ratios (Table 5.1, EN 1993-1-3)

$\phi := 45deg$

$$\frac{b_{p1}}{t} \leq 50 = 1$$

$$\frac{b_{p3}}{t} \leq 50 = 1$$

Since all the geometrical ratios are inside the limits, the provisions of EN 1993-1-3 may be applied.

Average yield strenght (3.2.2) EN 1993-1-3

$$n := \frac{2 \cdot 90deg + 2 \cdot 45deg}{90deg} = 3$$ the number of 90° bends in the cross-section with an internal radius <= 5*t*

$k := 7$ - numerical coefficient for roll forming

$A_g := 7.47cm^2$ - gross area (value taken from AutoCAD)

$$f_{ya} := f_{yb} + \left(f_u - f_{yb}\right) \cdot \frac{k \cdot n \cdot t^2}{A_g} = 406.727 \cdot MPa$$ (3.1) - average yield strength

$$f_{ya} \leq \frac{f_u + f_{yb}}{2} = 1$$

Determination of effective widths for a plane element without stiffeners (5.5.2 EN 1993-1-3)

The effective widths of the element will be equal to the calculated widths, since the class of the cross-section is Class 1.

Axial compression resistance (6.1.3 EN 1993-1-3)

$$A_{full} := 4A_g = 2.988 \times 10^3 \cdot mm^2$$

$k_{\sigma 1} := 0.43$ $k_{\sigma 3} := 4$ - buckling factors (Table 4.1 and 4.2, EN 1993-1-5)

$$\lambda_{p1} := \frac{\frac{b_{p1}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 1}}} = 0.344$$

- plate slendernesses (4.4 EN 1993-1-5)

$$\lambda_{p3} := \frac{\frac{b_{p3}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 3}}} = 0.318$$

$\lambda := \max(\lambda_{p1}, \lambda_{p3}) = 0.344$ - element slenderness

$$\lambda_{e0} := 0.673$$

$$\gamma_{M0} := 1$$

$$N_{c.Rd} := \frac{A_{full} \cdot \left[f_{yb} + \left(f_{ya} - f_{yb} \right) \cdot 4 \cdot \left(1 - \frac{\lambda}{\lambda_{e0}} \right) \right]}{\gamma_{M0}} = 1.363 \times 10^3 \cdot kN$$

but

$$N_{c.Rd} > \frac{A_{full} \cdot f_{ya}}{\gamma_{M0}} = 1215.3 \cdot kN$$

then:

$N_{c.Rd} := 1215.3kN$

Local buckling

Critical stress

$k_{\sigma 1} = 0.43$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.51} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_1}^2} = 4030.323 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.31} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_{p1}}^2} = 3010.037 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.51} := \sigma_{crit.1.51} \cdot A_{full} = 12042.604 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.31} := \sigma_{crit.1.31} \cdot A_{full} = 8993.992 \cdot kN$$

Critical stress

$k_{\sigma 3} = 4$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.53} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_3}^2} = 4931.6 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.33} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_{p3}}^2} = 3511.632 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.53} := \sigma_{crit.1.53} \cdot A_{full} = 14735.62 \cdot kN$$

Critical load according to EN 1993-1-3:

$N_{crit.1.33} := \sigma_{crit.1.33} \cdot A_{full} = 10492.756 \cdot kN$

Global buckling

$\gamma_{M1} := 1$

$\lambda_1 := 93.9 \cdot \varepsilon = 76.399$

$A_{full} = 29.88 \cdot cm^2$

$I := 12682813.4639 mm^4$

$i := \sqrt{\frac{I}{A_{full}}} = 65.15 \cdot mm$ -radius of gyration

$L_{cr} := 0.75 \cdot L_{plate} = 1.711 m$

$\lambda := \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1} = 0.344$ (6.50 EN 1993-1-1)

The buckling curve used will be curve "c".
Therefore, we adopt the following imperfection facto $\alpha := 0.49$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.594$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 0.927$ $\chi < 1$ - reduction factor

$N_{b.Rdglob} := \frac{\chi \cdot N_{c.Rd} \cdot \gamma_{M0}}{\gamma_{M1}} = 1126.247 \cdot kN$

Ultimate load resistance

$N_{b.Rd} := \min\left(N_{c.Rd}, N_{b.Rdglob}\right) = 1126.247 \cdot kN$

Axial load in the diagonal:

$N_{Ed} := 1086.44 kN$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.965 < 1$ Brace is resistant to compression force by applied load.

Design of top braces:

Top braces are made from CHS, the design is given in previous calculations of CHS, as they are identical.

Desing of U-shaped bottom chord in tension:

Bottom chord in the truss must sustain the tensile force:

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the lower chord:

$$N_{Ed.t} := 3258.90\text{kN}$$

$$N_{t.Rd} := 3259\text{kN}$$

$$N_{t.Rd} = \frac{A_{net} \cdot f_y}{\gamma_{M0}} \qquad (6.6)\ \text{EN 1993-1-1}$$

$$\gamma_{M0} := 1.0$$

The type of steel used is S355, therefore: $f_y := 355 \frac{N}{mm^2}$

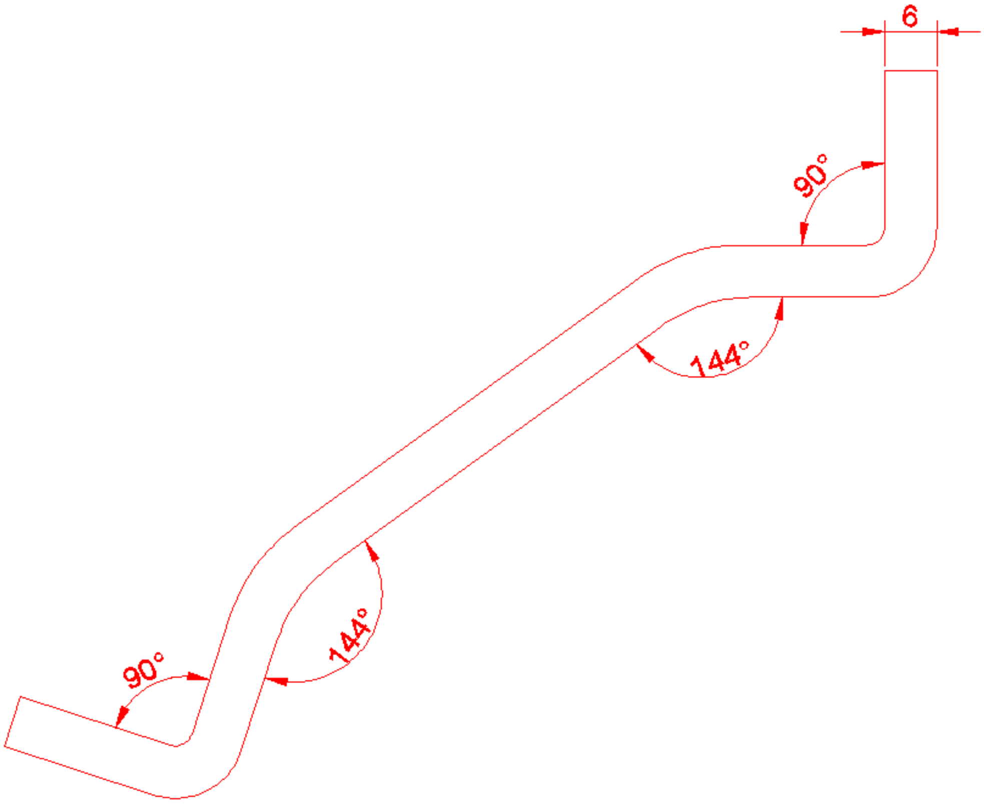
$$A_{net} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 91.803 \cdot cm^2$$

We adopt the following U-shape section, giving a gross area of **93.(** cm^2

Design calculation of the folded plates of the polygonal cross-section S500

Design of the compressed top chord made from semi-closed polygonal sections:

-according to EN 1993-1-1, EN 1993-1-3, EN 1993-1-5



Section properties:

$t := 6mm$ - plate thickenss

$\nu := 0.3$ - Poisson's ration

$f_{yb} := 500MPa$ - basic yield strength

$E := 210GPa$ - Modulus of Elasticity

$r_1 := 2mm$

$r_2 := 20mm$ - bent corner radii

$r_{m1} := r_1 + 0.5 \cdot t = 5 \cdot mm$

$r_{m2} := r_2 + 0.5 \cdot t = 23 \cdot mm$ -radii at midpoint of corner

$f_u := 550MPa$ - ultimate yielding strength

$L_{plate} := 4m$ -length of plate

$$\varepsilon := \sqrt{\frac{235MPa}{f_{yb}}} = 0.686$$ -strain coefficient

Normal widths of the flat parts

$b_1 := 18mm$

$b_2 := 15.49mm$

$b_3 := 54.99mm$

$b_4 := b_2$

$b_5 := b_1$

Section classification

- *Class of b.1 part*

$$\frac{b_1}{t} = 3 \quad < \; 33 \cdot \varepsilon = 22.624$$

$Class_{b1} := 1$

- *Class of b.2 part*

$$\frac{b_2}{t} = 2.582 < \; 33 \cdot \varepsilon = 22.624$$

$Class_{b2} := 1$

- *Class of b.3 part*

$$\frac{b_3}{t} = 9.165 < \; 33 \cdot \varepsilon = 22.624$$

$Class_{b3} := 1$

- *Class of b.4 part*

$Class_{b4} := Class_{b2}$

- *Class of b.5 part*

$Class_{b5} := Class_{b1}$

Influence of the corners (5.1) EN 1993-1-3

Corners' arch lengths:

$$\phi_1 := 90 \cdot \frac{\pi}{180} = 1.571$$

$$\phi_2 := 36 \cdot \frac{\pi}{180} = 0.628$$

$$u_1 := \phi_1 \cdot r_{m1} = 7.854 \cdot mm$$

$$u_2 := \phi_2 \cdot r_{m2} = 14.451 \cdot mm$$

Notional widths of plane cross section parts *b*p allowing for corner radii Fig. 5.1, EN 1993-1-3

$$g_{r1} := r_{m1} \cdot \left(\tan\left(\frac{\phi_1}{2}\right) - \sin\left(\frac{\phi_1}{2}\right) \right) = 1.464 \cdot mm$$

$$g_{r2} := r_{m2} \cdot \left(\tan\left(\frac{\phi_2}{2}\right) - \sin\left(\frac{\phi_2}{2}\right) \right) = 0.366 \cdot mm$$

$$b_{p1} := b_1 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) - g_{r1} = 21.536 \cdot mm$$

$$b_{p2} := b_2 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) + r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r1} - g_{r2} = 26.133 \cdot mm$$

$$b_{p3} := b_3 + 2 \cdot \left(r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r2} \right) = 69.205 \cdot mm$$

$$b_{p4} := b_{p2} = 26.133 \cdot mm$$

$$b_{p5} := b_{p1} = 21.536 \cdot mm$$

Maximum width to thickness ratios (Table 5.1, EN 1993-1-3)

$\phi := 36\text{deg}$

$$\frac{b_{p1}}{t} \le 50 = 1$$

$$\frac{b_{p2}}{t} \le 500 \cdot \sin(\phi) = 1$$

$$\frac{b_{p3}}{t} \le 50 = 1$$

Since all the geometrical ratios are inside the limits, the provisions of EN 1993-1-3 may be applied.

Average yield strenght (3.2.2) EN 1993-1-3

$$n := \frac{2 \cdot 90\text{deg} + 2 \cdot 36\text{deg}}{90\text{deg}} = 2.8$$ the number of 90° bends in the cross-section with an internal radius <= 5*t*

$k := 7$ - numerical coefficient for roll forming

$A_g := 9.707\text{cm}^2$ - gross area (value taken from AutoCAD)

$$f_{ya} := f_{yb} + \left(f_u - f_{yb}\right) \cdot \frac{k \cdot n \cdot t^2}{A_g} = 536.345 \cdot \text{MPa}$$ (3.1) - average yield strength

$$f_{ya} \le \frac{f_u + f_{yb}}{2} = 0$$

then:

$f_{ya} := 525\text{MPa}$

Determination of effective widths for a plane element without stiffeners (5.5.2 EN 1993-1-3)

The effective widths of the element will be equal to the calculated widths, since the class of the cross-section is Class 1.

Axial compression resistance (6.1.3 EN 1993-1-3)

$$A_{full} := 5 \cdot A_g = 4.854 \times 10^3 \cdot mm^2$$

$k_{\sigma1} := 0.43$ $k_{\sigma2} := 4$ $k_{\sigma3} := 4$ - buckling factors (Table 4.1 and 4.2, EN 1993-1-5)

$$\lambda_{p1} := \frac{\frac{b_{p1}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma1}}} = 0.281$$

$$\lambda_{p2} := \frac{\frac{b_{p2}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma2}}} = 0.112$$ - plate slendernesses (4.4 EN 1993-1-5)

$$\lambda_{p3} := \frac{\frac{b_{p3}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma3}}} = 0.296$$

$\lambda := \max(\lambda_{p1}, \lambda_{p2}, \lambda_{p3}) = 0.296$ - element slenderness

$$\lambda_{e0} := 0.673$$

$$\gamma_{M0} := 1$$

$$N_{c.Rd} := \frac{A_{full} \cdot \left[f_{yb} + (f_{ya} - f_{yb}) \cdot 4 \cdot \left(1 - \frac{\lambda}{\lambda_{e0}}\right) \right]}{\gamma_{M0}} = 2.698 \times 10^3 \cdot kN$$

but

$$N_{c.Rd} > \frac{A_{full} \cdot f_{ya}}{\gamma_{M0}} = 2548.088 \cdot kN$$

then:

$$N_{c.Rd} := 2548.088 kN$$

Local buckling

Critical stress

$k_{\sigma 1} = 0.43$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.51} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_1}^2} = 9068.226 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.31} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_{p1}}^2} = 6335.138 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.51} := \sigma_{crit.1.51} \cdot A_{full} = 44012.636 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.31} := \sigma_{crit.1.31} \cdot A_{full} = 30747.592 \cdot kN$$

Critical stress

$k_{\sigma 3} = 4$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.53} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_3}^2} = 9038.398 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.33} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_{p3}}^2} = 5706.722 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.53} := \sigma_{crit.1.53} \cdot A_{full} = 43867.864 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.33} := \sigma_{crit.1.33} \cdot A_{full} = 27697.573 \cdot kN$$

Global buckling

$\gamma_{M1} := 1$

$\lambda_1 := 93.9 \cdot \varepsilon = 64.375$

$A_{full} = 48.535 \cdot cm^2$

$I := 27519242.4093mm^4$

$i := \sqrt{\frac{I}{A_{full}}} = 75.299 \cdot mm$ -radius of gyration

$L_{cr} := 0.9 \cdot L_{plate} = 3.6\,m$

$\lambda := \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1} = 0.743$ (6.50 EN 1993-1-1)

The buckling curve used will be curve "c".
Therefore, we adopt the following imperfection facto $\alpha := 0.49$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.909$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 0.698$ $\chi < 1$ - reduction factor

$N_{b.Rdglob} := \frac{\chi \cdot N_{c.Rd} \cdot \gamma_{M0}}{\gamma_{M1}} = 1778.877 \cdot kN$

Ultimate load resistance

$N_{b.Rd} := \min\left(N_{c.Rd}, N_{b.Rdglob}\right) = 1778.877 \cdot kN$

Axial load in the top cmpressed chord:

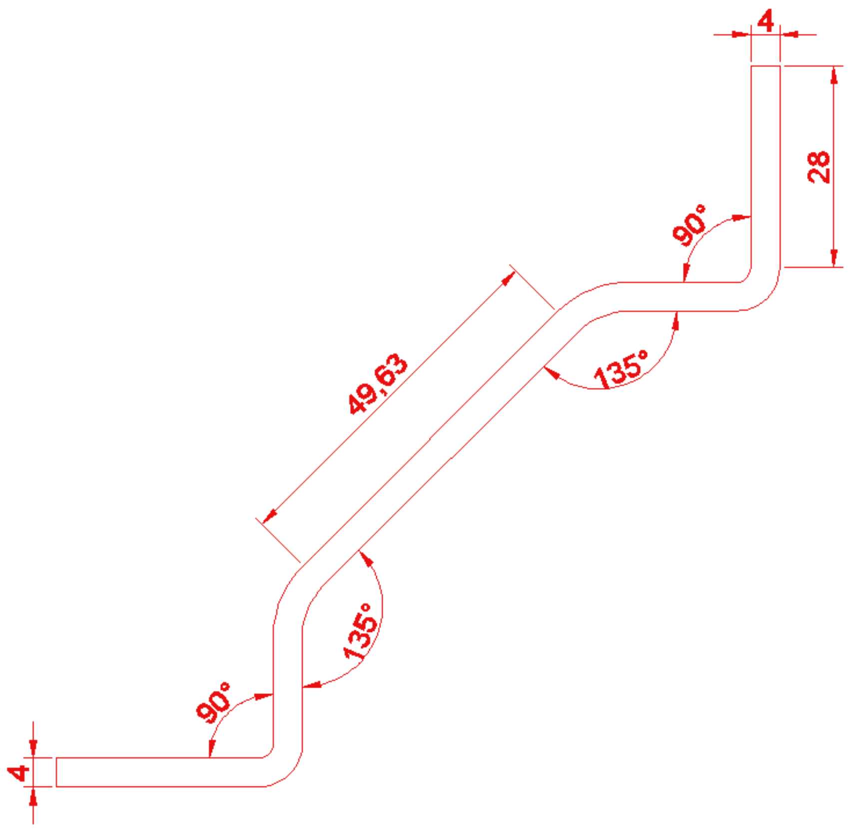
$N_{Ed} := 1589.37kN$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.893$ < 1

Top chord is resistant to compression force by applied load.

Design of the compressed braces made from semi-closed polygonal sections:

-according to EN 1993-1-1, EN 1993-1-3, EN 1993-1-5



Section properties:

$t := 4mm$ - plate thickenss

$\nu := 0.3$ - Poisson's ration

$f_{yb} := 500MPa$ - basic yield strength

$E := 210GPa$ - Modulus of Elasticity

$r_1 := 2mm$

$r_2 := 10mm$ - bent corner radii

$r_{m1} := r_1 + 0.5 \cdot t = 4 \cdot mm$

$r_{m2} := r_2 + 0.5 \cdot t = 12 \cdot mm$ -radii at midpoint of corner

$f_u := 550MPa$ - ultimate yielding strength

$L_{plate} := 2.5m - 0.219m = 2.281\,m$ -length of plate

$\varepsilon := \sqrt{\frac{235MPa}{f_{yb}}} = 0.686$ -strain coefficient

Normal widths of the flat parts

$b_1 := 28mm$

$b_3 := 49.63mm$

$b_5 := b_1$

Section classification

- *Class of b.1 part*

$\frac{b_1}{t} = 7 \quad < \; 33 \cdot \varepsilon = 22.624$

$Class_{b1} := 1$

- *Class of b.3 part*

$\frac{b_3}{t} = 12.408 \quad < 33 \cdot \varepsilon = 22.624$

$Class_{b3} := 1$

- *Class of b.5 part*

$Class_{b5} := Class_{b1}$

Influence of the corners (5.1) EN 1993-1-3

Corners' arch lengths:

$\phi_1 := 90 \cdot \frac{\pi}{180} = 1.571$

$\phi_2 := 45 \cdot \frac{\pi}{180} = 0.785$

$u_1 := \phi_1 \cdot r_{m1} = 6.283 \cdot mm$

$u_2 := \phi_2 \cdot r_{m2} = 9.425 \cdot mm$

Notional widths of plane cross section parts b_p allowing for corner radii Fig. 5.1, EN 1993-1-3

$$g_{r1} := r_{m1} \cdot \left(\tan\left(\frac{\phi_1}{2}\right) - \sin\left(\frac{\phi_1}{2}\right) \right) = 1.172 \cdot mm$$

$$g_{r2} := r_{m2} \cdot \left(\tan\left(\frac{\phi_2}{2}\right) - \sin\left(\frac{\phi_2}{2}\right) \right) = 0.378 \cdot mm$$

$$b_{p1} := b_1 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) - g_{r1} = 30.828 \cdot mm$$

$$b_{p3} := b_3 + 2 \cdot \left(r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r2} \right) = 58.814 \cdot mm$$

$$b_{p5} := b_{p1} = 30.828 \cdot mm$$

Maximum width to thickness ratios (Table 5.1, EN 1993-1-3)

$$\phi := 45deg$$

$$\frac{b_{p1}}{t} \le 50 = 1$$

$$\frac{b_{p3}}{t} \le 50 = 1$$

Since all the geometrical ratios are inside the limits, the provisions of EN 1993-1-3 may be applied.

Average yield strenght (3.2.2) EN 1993-1-3

$$n := \frac{2 \cdot 90deg + 2 \cdot 45deg}{90deg} = 3$$ the number of 90° bends in the cross-section with an internal radius <= 5*t*

$k := 7$ - numerical coefficient for roll forming

$A_g := 6.67cm^2$ - gross area (value taken from AutoCAD)

$$f_{ya} := f_{yb} + \left(f_u - f_{yb}\right) \cdot \frac{k \cdot n \cdot t^2}{A_g} = 525.187 \cdot MPa \quad (3.1)$$ - average yield strength

$$f_{ya} \le \frac{f_u + f_{yb}}{2} = 0$$

Determination of effective widths for a plane element without stiffeners (5.5.2 EN 1993-1-3)

The effective widths of the element will be equal to the calculated widths, since the class of the cross-section is Class 1.

Axial compression resistance (6.1.3 EN 1993-1-3)

$$A_{full} := 4A_g = 2.668 \times 10^3 \cdot mm^2$$

$k_{\sigma 1} := 0.43$ $k_{\sigma 3} := 4$ - buckling factors (Table 4.1 and 4.2, EN 1993-1-5)

$$\lambda_{p1} := \frac{\frac{b_{p1}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 1}}} = 0.604$$

- plate slendernesses (4.4 EN 1993-1-5)

$$\lambda_{p3} := \frac{\frac{b_{p3}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 3}}} = 0.378$$

$\lambda := \max(\lambda_{p1}, \lambda_{p3}) = 0.604$ - element slenderness

$$\lambda_{e0} := 0.673$$

$$\gamma_{M0} := 1$$

$$N_{c.Rd} := \frac{A_{full} \cdot \left[f_{yb} + (f_{ya} - f_{yb}) \cdot 4 \cdot \left(1 - \frac{\lambda}{\lambda_{e0}}\right)\right]}{\gamma_{M0}} = 1.362 \times 10^3 \cdot kN$$

but

$$N_{c.Rd} > \frac{A_{full} \cdot f_{ya}}{\gamma_{M0}} = 1401.2 \cdot kN$$

then:

$$N_{c.Rd} := 1401.2kN$$

Local buckling

Critical stress

$k_{\sigma 1} = 0.43$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.51} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_1^{\,2}} = 1665.593 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.31} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_{p1}^{\,2}} = 1373.985 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.51} := \sigma_{crit.1.51} \cdot A_{full} = 4443.801 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.31} := \sigma_{crit.1.31} \cdot A_{full} = 3665.793 \cdot kN$$

Critical stress

$k_{\sigma 3} = 4$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.53} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_3^{\,2}} = 4931.6 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.33} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_{p3}^{\,2}} = 3511.632 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.53} := \sigma_{crit.1.53} \cdot A_{full} = 13157.508 \cdot kN$$

Critical load according to EN 1993-1-3:

$N_{crit.1.33} := \sigma_{crit.1.33} \cdot A_{full} = 9369.034 \cdot kN$

Global buckling

$\gamma_{M1} := 1$

$\lambda_1 := 93.9 \cdot \varepsilon = 64.375$

$A_{full} = 26.68 \cdot cm^2$

$I := 8536050.5131mm^4$

$i := \sqrt{\frac{I}{A_{full}}} = 56.563 \cdot mm$ -radius of gyration

$L_{cr} := 0.75 \cdot L_{plate} = 1.711\,m$

$\lambda := \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1} = 0.47$ (6.50 EN 1993-1-1)

The buckling curve used will be curve "c".
Therefore, we adopt the following imperfection facto $\alpha := 0.49$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.676$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 0.86$ $\chi < 1$ - reduction factor

$N_{b.Rdglob} := \frac{\chi \cdot N_{c.Rd} \cdot \gamma_{M0}}{\gamma_{M1}} = 1204.627 \cdot kN$

Ultimate load resistance

$N_{b.Rd} := \min\left(N_{c.Rd}, N_{b.Rdglob}\right) = 1204.627 \cdot kN$

Axial load in the top cmpressed chord:

$N_{Ed} := 1086.4kN$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.902$ < 1 Brace is resistant to compression force by applied load.

Design of top braces:

Top braces are made from CHS, the design is given in previous calculations of CHS, as they are identical.

Desing of U-shaped bottom chord in tension:

Bottom chord in the truss must sustain the tensile force:

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the lower chord:

$N_{Ed.t} := 3259.55kN$

$N_{t.Rd} := 3260kN$

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad \text{EN 1993-1-1}$$

$\gamma_{M0} := 1.0$

The type of steel used is S 500MC, therefore $f_y := 500 \frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

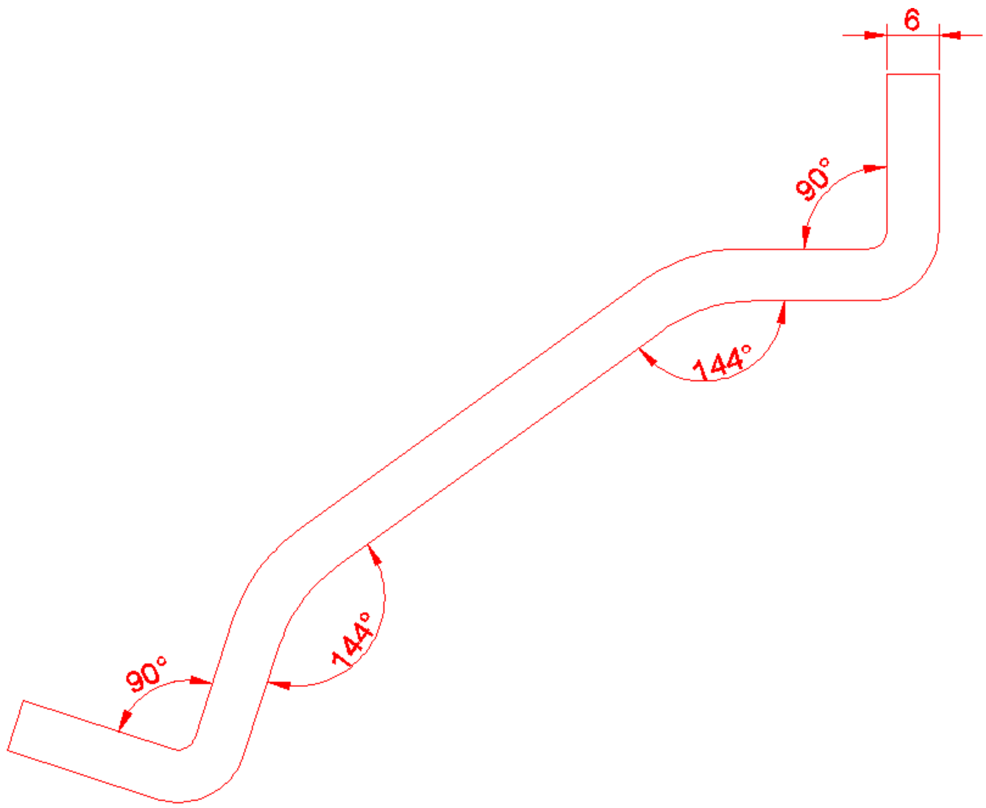
$$A := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 65.2 \cdot cm^2$$

We adopt u-shaped cross section, giving a gross area of **66.0(** cm^2

Design calculation of the folded plates of the polygonal cross-section S650

Design of the compressed top chord made from semi-closed polygonal sections:

-according to EN 1993-1-1, EN 1993-1-3, EN 1993-1-5



Section properties:

$t := 6mm$ - plate thickenss

$\nu := 0.3$ - Poisson's ration

$f_{yb} := 650MPa$ - basic yield strength

$E := 210GPa$ - Modulus of Elasticity

$r_1 := 2mm$

$r_2 := 20mm$

- bent corner radii

$r_{m1} := r_1 + 0.5 \cdot t = 5 \cdot mm$

$r_{m2} := r_2 + 0.5 \cdot t = 23 \cdot mm$

-radii at midpoint of corner

$f_u := 700MPa$ - ultimate yielding strength

$L_{plate} := 4m$ -length of plate

$\varepsilon := \sqrt{\frac{235MPa}{f_{yb}}} = 0.601$ -strain coefficient

Normal widths of the flat parts

$b_1 := 18mm$

$b_2 := 14.4mm$

$b_3 := 48.8mm$

$b_4 := b_2$

$b_5 := b_1$

Section classification

- *Class of b.1 part*

$\frac{b_1}{t} = 3 \quad < \; 33 \cdot \varepsilon = 19.842$

$Class_{b1} := 1$

- *Class of b.2 part*

$\frac{b_2}{t} = 2.4 \; < \; 33 \cdot \varepsilon = 19.842$

$Class_{b2} := 1$

- *Class of b.3 part*

$\frac{b_3}{t} = 8.133 < \; 33 \cdot \varepsilon = 19.842$

$Class_{b3} := 1$

- *Class of b.4 part*

$Class_{b4} := Class_{b2}$

- *Class of b.5 part*

$Class_{b5} := Class_{b1}$

Influence of the corners (5.1) EN 1993-1-3

Corners' arch lengths:

$$\phi_1 := 90 \cdot \frac{\pi}{180} = 1.571$$

$$\phi_2 := 36 \cdot \frac{\pi}{180} = 0.628$$

$$u_1 := \phi_1 \cdot r_{m1} = 7.854 \cdot mm$$

$$u_2 := \phi_2 \cdot r_{m2} = 14.451 \cdot mm$$

Notional widths of plane cross section parts *b*p allowing for corner radii Fig. 5.1, EN 1993-1-3

$$g_{r1} := r_{m1} \cdot \left(\tan\left(\frac{\phi_1}{2}\right) - \sin\left(\frac{\phi_1}{2}\right) \right) = 1.464 \cdot mm$$

$$g_{r2} := r_{m2} \cdot \left(\tan\left(\frac{\phi_2}{2}\right) - \sin\left(\frac{\phi_2}{2}\right) \right) = 0.366 \cdot mm$$

$$b_{p1} := b_1 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) - g_{r1} = 21.536 \cdot mm$$

$$b_{p2} := b_2 + r_{m1} \cdot \tan\left(\frac{\phi_1}{2}\right) + r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r1} - g_{r2} = 25.043 \cdot mm$$

$$b_{p3} := b_3 + 2 \cdot \left(r_{m2} \cdot \tan\left(\frac{\phi_2}{2}\right) - g_{r2} \right) = 63.015 \cdot mm$$

$$b_{p4} := b_{p2} = 25.043 \cdot mm$$

$$b_{p5} := b_{p1} = 21.536 \cdot mm$$

Maximum width to thickness ratios (Table 5.1, EN 1993-1-3)

$\phi := 36deg$

$$\frac{b_{p1}}{t} \leq 50 = 1$$

$$\frac{b_{p2}}{t} \leq 500 \cdot \sin(\phi) = 1$$

$$\frac{b_{p3}}{t} \leq 50 = 1$$

Since all the geometrical ratios are inside the limits, the provisions of EN 1993-1-3 may be applied.

Average yield strenght (3.2.2) EN 1993-1-3

$$n := \frac{2 \cdot 90deg + 2 \cdot 36deg}{90deg} = 2.8$$ the number of 90° bends in the cross-section with an internal radius <= 5*t*

$k := 7$ - numerical coefficient for roll forming

$A_g := 9.069cm^2$ - gross area (value taken from AutoCAD)

$$f_{ya} := f_{yb} + \left(f_u - f_{yb}\right) \cdot \frac{k \cdot n \cdot t^2}{A_g} = 688.902 \cdot MPa$$ (3.1) - average yield strength

$$f_{ya} \leq \frac{f_u + f_{yb}}{2} = 0$$

then:

$f_{ya} := 675MPa$

Determination of effective widths for a plane element without stiffeners (5.5.2 EN 1993-1-3)

The effective widths of the element will be equal to the calculated widths, since the class of the cross-section is Class 1.

Axial compression resistance (6.1.3 EN 1993-1-3)

$A_{full} := 48.19cm^2$

$k_{\sigma 1} := 0.43 \quad k_{\sigma 2} := 4 \quad k_{\sigma 3} := 4$ - buckling factors (Table 4.1 and 4.2, EN 1993-1-5)

$$\lambda_{p1} := \frac{\frac{b_{p1}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 1}}} = 0.321$$

$$\lambda_{p2} := \frac{\frac{b_{p2}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 2}}} = 0.122$$

- plate slendernesses (4.4 EN 1993-1-5)

$$\lambda_{p3} := \frac{\frac{b_{p3}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 3}}} = 0.308$$

$\lambda := \max(\lambda_{p1}, \lambda_{p2}, \lambda_{p3}) = 0.321$ - element slenderness

$\lambda_{e0} := 0.673$

$\gamma_{M0} := 1$

$$N_{c.Rd} := \frac{A_{full} \cdot \left[f_{yb} + (f_{ya} - f_{yb}) \cdot 4 \cdot \left(1 - \frac{\lambda}{\lambda_{e0}} \right) \right]}{\gamma_{M0}} = 3.385 \times 10^3 \cdot kN$$

but

$$N_{c.Rd} > \frac{A_{full} \cdot f_{ya}}{\gamma_{M0}} = 3252.825 \cdot kN$$

then:

$N_{c.Rd} := 3252.825kN$

Local buckling

Critical stress

$k_{\sigma 1} = 0.43$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.51} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_1}^2} = 9068.226 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.31} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_{p1}}^2} = 6335.138 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.51} := \sigma_{crit.1.51} \cdot A_{full} = 43699.782 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.31} := \sigma_{crit.1.31} \cdot A_{full} = 30529.029 \cdot kN$$

Critical stress

$k_{\sigma 3} = 4$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.53} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_3}^2} = 11476.759 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.33} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot {b_{p3}}^2} = 6882.941 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.53} := \sigma_{crit.1.53} \cdot A_{full} = 55306.5 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.33} := \sigma_{crit.1.33} \cdot A_{full} = 33168.891 \cdot kN$$

Global buckling

$\gamma_{M1} := 1$

$\lambda_1 := 93.9 \cdot \varepsilon = 56.46$

$A_{full} = 48.19 \cdot cm^2$

$I := 20648815.6419mm^4$

$i := \sqrt{\dfrac{I}{A_{full}}} = 65.459 \cdot mm$ -radius of gyration

$L_{cr} := 0.9 \cdot L_{plate} = 3.6\,m$

$\lambda := \dfrac{L_{cr}}{i} \cdot \dfrac{1}{\lambda_1} = 0.974$ (6.50 EN 1993-1-1)

The buckling curve used will be curve "c".
Therefore, we adopt the following imperfection facto $\alpha := 0.49$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.164$

$\chi := \dfrac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 0.555$ $\chi < 1$ - reduction factor

$N_{b.Rdglob} := \dfrac{\chi \cdot N_{c.Rd} \cdot \gamma_{M0}}{\gamma_{M1}} = 1805.721 \cdot kN$

Ultimate load resistance

$N_{b.Rd} := \min\left(N_{c.Rd}, N_{b.Rdglob}\right) = 1805.721 \cdot kN$

Axial load in the top cmpressed chord:

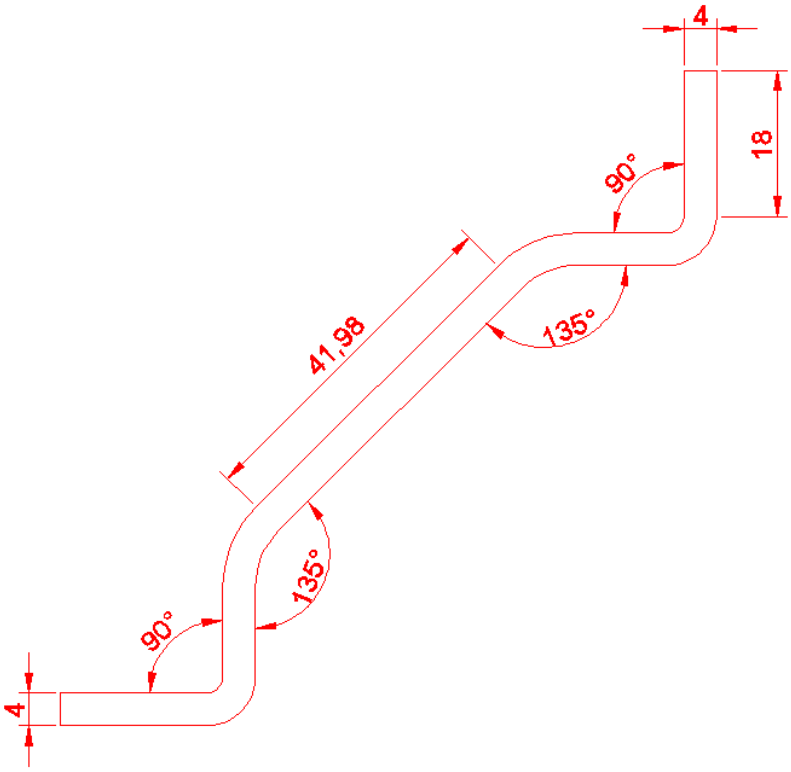
$N_{Ed} := 1589.37kN$

$\dfrac{N_{Ed}}{N_{b.Rd}} = 0.88$ < 1

Top chord is resistant to compression force by applied load.

Design of the compressed braces made from semi-closed polygonal sections:

-according to EN 1993-1-1, EN 1993-1-3, EN 1993-1-5



<u>Section properties:</u>

$t := 4mm$ - plate thickenss

$\nu := 0.3$ - Poisson's ration

$f_{yb} := 650MPa$ - basic yield strength

$E := 210GPa$ - Modulus of Elasticity

$r_1 := 2mm$

$r_2 := 10mm$ - bent corner radii

$r_{m1} := r_1 + 0.5 \cdot t = 4 \cdot mm$

$r_{m2} := r_2 + 0.5 \cdot t = 12 \cdot mm$ -radii at midpoint of corner

$f_u := 700MPa$ - ultimate yielding strength

$L_{plate} := 2.5m - 0.219m = 2.281\,m$ -length of plate

$\varepsilon := \sqrt{\frac{235MPa}{f_{yb}}} = 0.601$ -strain coefficient

Normal widths of the flat parts

$b_1 := 18mm$

$b_3 := 41.98mm$

$b_5 := b_1$

Section classification

- *Class of b.1 part*

$\frac{b_1}{t} = 4.5 \ < \ 33 \cdot \varepsilon = 19.842$

$Class_{b1} := 1$

- *Class of b.3 part*

$\frac{b_3}{t} = 10.495 \quad < \ 33 \cdot \varepsilon = 19.842$

$Class_{b3} := 1$

- *Class of b.5 part*

$Class_{b5} := Class_{b1}$

Influence of the corners (5.1) EN 1993-1-3

Corners' arch lengths:

$\phi_1 := 90 \cdot \frac{\pi}{180} = 1.571$

$\phi_2 := 45 \cdot \frac{\pi}{180} = 0.785$

$u_1 := \phi_1 \cdot r_{m1} = 6.283 \cdot mm$

$u_2 := \phi_2 \cdot r_{m2} = 9.425 \cdot mm$

Notional widths of plane cross section parts b_p allowing for corner radii Fig. 5.1, EN 1993-1-3

$$g_{r1} := r_{m1}\cdot\left(\tan\left(\frac{\phi_1}{2}\right) - \sin\left(\frac{\phi_1}{2}\right)\right) = 1.172\cdot mm$$

$$g_{r2} := r_{m2}\cdot\left(\tan\left(\frac{\phi_2}{2}\right) - \sin\left(\frac{\phi_2}{2}\right)\right) = 0.378\cdot mm$$

$$b_{p1} := b_1 + r_{m1}\cdot\tan\left(\frac{\phi_1}{2}\right) - g_{r1} = 20.828\cdot mm$$

$$b_{p3} := b_3 + 2\cdot\left(r_{m2}\cdot\tan\left(\frac{\phi_2}{2}\right) - g_{r2}\right) = 51.164\cdot mm$$

$$b_{p5} := b_{p1} = 20.828\cdot mm$$

Maximum width to thickness ratios (Table 5.1, EN 1993-1-3)

$$\phi := 45deg$$

$$\frac{b_{p1}}{t} \le 50 = 1$$

$$\frac{b_{p3}}{t} \le 50 = 1$$

Since all the geometrical ratios are inside the limits, the provisions of EN 1993-1-3 may be applied.

Average yield strenght (3.2.2) EN 1993-1-3

$$n := \frac{2\cdot 90deg + 2\cdot 45deg}{90deg} = 3$$ the number of 90° bends in the cross-section with an internal radius <= 5*t*

$k := 7$ - numerical coefficient for roll forming

$A_g := 5.255cm^2$ - gross area (value taken from AutoCAD)

$$f_{ya} := f_{yb} + \left(f_u - f_{yb}\right)\cdot\frac{k\cdot n\cdot t^2}{A_g} = 681.97\cdot MPa \qquad (3.1)$$ - average yield strength

$$f_{ya} \le \frac{f_u + f_{yb}}{2} = 0$$

then:

$$f_{ya} := 675MPa$$

Determination of effective widths for a plane element without stiffeners (5.5.2 EN 1993-1-3)

The effective widths of the element will be equal to the calculated widths, since the class of the cross-section is Class 1.

Axial compression resistance (6.1.3 EN 1993-1-3)

$$A_{full} := 4A_g = 21.02 \cdot cm^2$$

$k_{\sigma 1} := 0.43$ $k_{\sigma 3} := 4$ - buckling factors (Table 4.1 and 4.2, EN 1993-1-5)

$$\lambda_{p1} := \frac{\frac{b_{p1}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 1}}} = 0.465$$

- plate slendernesses (4.4 EN 1993-1-5)

$$\lambda_{p3} := \frac{\frac{b_{p3}}{t}}{28.4 \cdot \varepsilon \cdot \sqrt{k_{\sigma 3}}} = 0.375$$

$\lambda := \max(\lambda_{p1}, \lambda_{p3}) = 0.465$ - element slenderness

$$\lambda_{e0} := 0.673$$

$$\gamma_{M0} := 1$$

$$N_{c.Rd} := \frac{A_{full} \cdot \left[f_{yb} + (f_{ya} - f_{yb}) \cdot 4 \cdot \left(1 - \frac{\lambda}{\lambda_{e0}}\right)\right]}{\gamma_{M0}} = 1.431 \times 10^3 \cdot kN$$

but

$$N_{c.Rd} > \frac{A_{full} \cdot f_{ya}}{\gamma_{M0}} = 1418.85 \cdot kN$$

then:

$N_{c.Rd} := 1418.85kN$

Local buckling

Critical stress

$k_{\sigma 1} = 0.43$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.51} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_1^{\,2}} = 4030.323 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.31} := k_{\sigma 1} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_{p1}^{\,2}} = 3010.037 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.51} := \sigma_{crit.1.51} \cdot A_{full} = 8471.738 \cdot kN$$

Critical load according to EN 1993-1-3:

$$N_{crit.1.31} := \sigma_{crit.1.31} \cdot A_{full} = 6327.099 \cdot kN$$

Critical stress

$k_{\sigma 3} = 4$

Critical stress according to theory plate EN 1993-1-5:

$$\sigma_{crit.1.53} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_3^{\,2}} = 6892.734 \cdot MPa$$ (A.1 EN 1993-1-5)

Critical stress according to EN 1993-1-3:

$$\sigma_{crit.1.33} := k_{\sigma 3} \cdot \frac{\pi^2 \cdot E \cdot t^2}{12\left(1-\nu^2\right) \cdot b_{p3}^{\,2}} = 4640.241 \cdot MPa$$

Critical load according to EN 1993-1-5:

$$N_{crit.1.53} := \sigma_{crit.1.53} \cdot A_{full} = 14488.526 \cdot kN$$

Critical load according to EN 1993-1-3:

$N_{crit.1.33} := \sigma_{crit.1.33} \cdot A_{full} = 9753.787 \cdot kN$

Global buckling

$\gamma_{M1} := 1$

$\lambda_1 := 93.9 \cdot \varepsilon = 56.46$

$A_{full} = 21.02 \cdot cm^2$

$I := 4858804.6673 mm^4$

$i := \sqrt{\frac{I}{A_{full}}} = 48.078 \cdot mm$ -radius of gyration

$L_{cr} := 0.75 \cdot L_{plate} = 1.711 m$

$\lambda := \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1} = 0.63$ (6.50 EN 1993-1-1)

The buckling curve used will be curve "c".
Therefore, we adopt the following imperfection facto $\alpha := 0.49$

$\phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.804$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = 0.767$ $\chi < 1$ - reduction factor

$N_{b.Rdglob} := \frac{\chi \cdot N_{c.Rd} \cdot \gamma_{M0}}{\gamma_{M1}} = 1088.724 \cdot kN$

Ultimate load resistance

$N_{b.Rd} := \min\left(N_{c.Rd}, N_{b.Rdglob}\right) = 1088.724 \cdot kN$

Axial load in the top cmpressed chord:

$N_{Ed} := 1086.44kN$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.998$ < 1 Diagonal is resistant to compression force by applied load.

Design of top braces:

Top braces are made from CHS, the design is given in previous calculations of CHS, as they are identical.

Desing of U-shaped bottom chord in tension:

Bottom chord in the truss must sustain the tensile force:

$$\frac{N_{Ed}}{N_{t.Rd}} < 1 \qquad (6.5)\ \text{EN 1993-1-1}$$

From Autodesk Robot 2013 we obtained the following value of axial force in the lower chord:

$$N_{Ed.t} := 3258.90kN$$

$$N_{t.Rd} := 3259kN$$

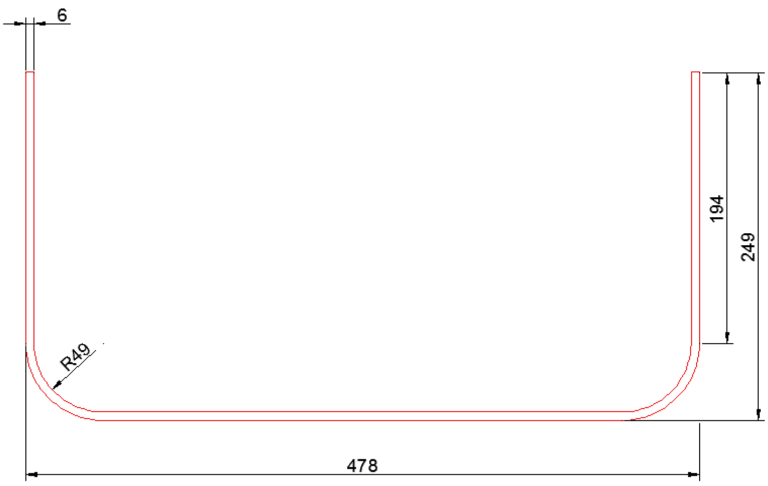
$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad \text{EN 1993-1-1}$$

$$\gamma_{M0} := 1.0$$

The type of steel used is S 650MC, therefore $f_y := 650 \frac{N}{mm^2}$ (Table 2, EN 1993-1-12)

$$A_{gross} := \frac{N_{t.Rd} \cdot \gamma_{M0}}{f_y} = 50.138 \cdot cm^2$$

We adopt the following u-shaped cross section, giving a gross area of 55.: cm^2



ANNEX C

Checking of uniform built-up compression members (for CHS using S650)

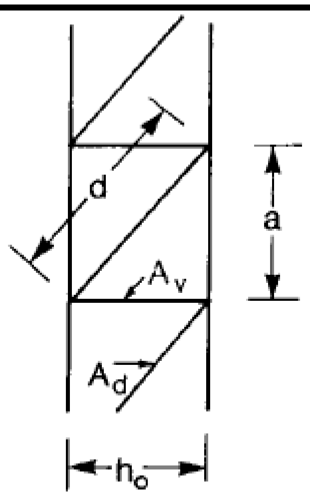
-according to EN 1993-1-1, (6.4)

The chords and diagonal bracings should be designed for buckling, in order to verify the following:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \leq 1$$

$$N_{ch.Ed} = 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} \qquad (6.69)$$

$$M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

System	
S_v	$\dfrac{nEA_d a h_0^2}{d^3\left[1 + \dfrac{A_d h_0^3}{A_V d^3}\right]}$

(Figure 6.9, EN 1993-1-1)

$h_0 := 1.8m \qquad A_{ch} := 57.71cm^2 \qquad L := 6m$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} = 934902 \cdot cm^4 \qquad e_0 := \frac{12m}{500} = 24 \cdot mm$

$A_d := 8.69cm^2 \qquad A_v := A_d = 8.69 \cdot cm^2$

$d := 4.39m \qquad n := 1$ (number of planes of lacings)

$E := 210GPa \qquad a := 4m$

$$S_v := \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^2}{d^3 \cdot \left(1 + \frac{A_d \cdot h_0^3}{A_v \cdot d^3}\right)} = 26151.719 \cdot kN$$

$N_{Ed} := 2 \cdot 1036.96kN$ - design value of the compression force to the built-up member

$M_{I.Ed} := 0kN \cdot m$ - design value of the maximum moment in the middle of the built-up mem

We will determine an in span axial distributed load, according to ...Pilkey....Table 11-7.

$q_x := 345.48 \frac{kN}{m}$ - distributed load needed to obtain the resulted axial force diagram from Robot

$$p_x \cdot L = \eta \cdot \frac{E \cdot I_{eff}}{L^2}$$

$\eta := 5.38 + 2.47 = 7.85$ (free - fixed condition)

$$N_{cr} = p_x \cdot L$$

$$N_{cr} := \eta \cdot \frac{E \cdot I_{eff}}{L^2} = 428107.208 \cdot kN$$

$$M_{Ed} := \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = 54.347 \cdot kN \cdot m$$

$$N_{ch.Ed} := 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}}$$

$N_{ch.Ed} = 1067.153 \cdot kN$

- **Buckling resistance:**

$\frac{N_{ch.Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 57.71cm^2$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$\gamma_{M1} := 1$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$$

$I_y := 2441.59cm^4$

$L_{cr} := 4m$ $f_y := 650MPa$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3162.8 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 1.089$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$\alpha := 0.49$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.311$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.49 \qquad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1838.457 \cdot kN$$

$$\frac{N_{ch.Ed}}{N_{b.Rd}} = 0.58 \qquad < 1$$

Checking of uniform built-up compression members (for CHS using S500)

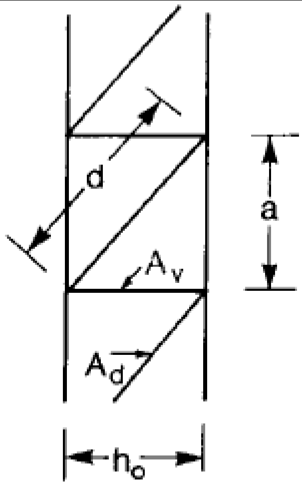
-according to EN 1993-1-1, (6.4)

The chords and diagonal bracings should be designed for buckling, in order to verify the following:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1$$

$$N_{ch.Ed} = 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} \qquad (6.69)$$

$$M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

System	
S_v	$\dfrac{nEA_d a h_0^2}{d^3\left[1+\dfrac{A_d h_0^3}{A_V d^3}\right]}$

(Figure 6.9, EN 1993-1-1)

$h_0 := 1.8m \qquad A_{ch} := 57.71cm^2 \qquad L := 6m$

$I_{eff} := 0.5 \cdot h_0^{\,2} \cdot A_{ch} = 934902 \cdot cm^4 \qquad e_0 := \dfrac{12m}{500} = 24 \cdot mm$

$A_d := 10.67cm^2 \qquad A_v := A_d = 10.67 \cdot cm^2$

$d := 4.39m \qquad n := 1$ (number of planes of lacings)

$E := 210GPa \qquad a := 4m$

$$S_v := \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^{\,2}}{d^3 \cdot \left(1 + \dfrac{A_d \cdot h_0^{\,3}}{A_v \cdot d^3}\right)} = 32110.338 \cdot kN$$

$N_{Ed} := 2 \cdot 1036.96kN$ - design value of the compression force to the built-up member

$M_{I.Ed} := 0kN \cdot m$ - design value of the maximum moment in the middle of the built-up mem

We will determine an in span axial distributed load, according to ...Pilkey....Table 11-7.

$q_x := 345.48\frac{kN}{m}$ - distributed load needed to obtain the resulted axial force diagram from Robot

$$p_x \cdot L = \eta \cdot \frac{E \cdot I_{eff}}{L^2}$$

$\eta := 5.38 + 2.47 = 7.85$ (free - fixed condition)

$$N_{cr} = p_x \cdot L$$

$$N_{cr} := \eta \cdot \frac{E \cdot I_{eff}}{L^2} = 428107.208 \cdot kN$$

$$M_{Ed} := \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = 53.488 \cdot kN \cdot m$$

$$N_{ch.Ed} := 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}}$$

$$N_{ch.Ed} = 1066.675 \cdot kN$$

- **Buckling resistance:**

$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1$ (6.46) EN 1993-1-1

$$A := 57.71cm^2$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$\gamma_{M1} := 1$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$$

$I_y := 2441.59cm^4$

$L_{cr} := 4m$ $\quad$ $f_y := 500MPa$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3162.8 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.955$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$\alpha := 0.49$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.141$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.566 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1634.25 \cdot kN$$

$$\frac{N_{ch.Ed}}{N_{b.Rd}} = 0.653 \quad < 1$$

Checking of uniform built-up compression members (for CHS using S355)

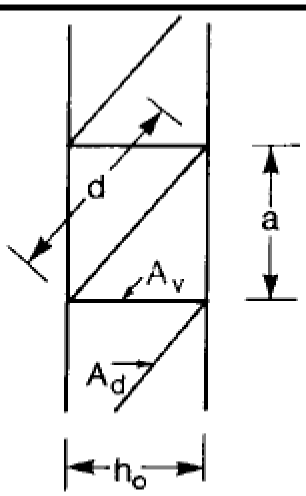
-according to EN 1993-1-1, (6.4)

The chords and diagonal bracings should be designed for buckling, in order to verify the following:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1$$

$$N_{ch.Ed} = 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} \qquad (6.69)$$

$$M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

System	
S_v	$\dfrac{nEA_d a h_0^2}{d^3\left[1 + \dfrac{A_d h_0^3}{A_V d^3}\right]}$

(Figure 6.9, EN 1993-1-1)

$h_0 := 1.8m \qquad A_{ch} := 81.13cm^2 \qquad L := 6m$

$I_{eff} := 0.5 \cdot h_0^{\,2} \cdot A_{ch} = 1314306 \cdot cm^4 \qquad e_0 := \frac{12m}{500} = 24 \cdot mm$

$A_d := 10.67cm^2 \qquad A_v := A_d = 10.67 \cdot cm^2$

$d := 4.39m \qquad n := 1$ (number of planes of lacings)

$E := 210GPa \qquad a := 4m$

$$S_v := \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^{\,2}}{d^3 \cdot \left(1 + \frac{A_d \cdot h_0^{\,3}}{A_v \cdot d^3}\right)} = 32110.338 \cdot kN$$

$N_{Ed} := 2 \cdot 1036.96kN$ - design value of the compression force to the built-up member

$M_{I.Ed} := 0kN \cdot m$ - design value of the maximum moment in the middle of the built-up mem

We will determine an in span axial distributed load, according to ...Pilkey....Table 11-7.

$q_x := 345.48 \frac{kN}{m}$ - distributed load needed to obtain the resulted axial force diagram from Robot

$$p_x \cdot L = \eta \cdot \frac{E \cdot I_{eff}}{L^2}$$

$\eta := 5.38 + 2.47 = 7.85$ (free - fixed condition)

$N_{cr} = p_x \cdot L$

$$N_{cr} := \eta \cdot \frac{E \cdot I_{eff}}{L^2} = 601842.623 \cdot kN$$

$$M_{Ed} := \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = 53.408 \cdot kN \cdot m$$

$$N_{ch.Ed} := 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}}$$

$$N_{ch.Ed} = 1066.631 \cdot kN$$

- **Buckling resistance:**

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1 \quad \text{(6.46) EN 1993-1-1}$$

$$A := 81.13cm^2$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^{\ 2}}$$

$$I_y := 4344.58cm^4$$

$$L_{cr} := 4m \qquad f_y := 355MPa$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^{\ 2}} = 5627.906 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.715$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5\cdot\left[1 + \alpha\cdot(\lambda - 0.2) + \lambda^2\right] = 0.882$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.715 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi\cdot A\cdot f_y}{\gamma_{M1}} = 2059.719\cdot kN$$

$$\frac{N_{ch.Ed}}{N_{b.Rd}} = 0.518 \quad < 1$$

Checking of uniform built-up compression members (for built of section using S650)

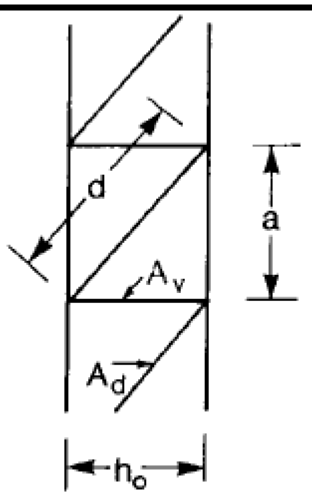
-according to EN 1993-1-1, (6.4)

The chords and diagonal bracings should be designed for buckling, in order to verify the following:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1$$

$$N_{ch.Ed} = 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} \qquad (6.69)$$

$$M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

System	
S_v	$\frac{nEA_d a h_0^2}{d^3\left[1+\frac{A_d h_0^3}{A_v d^3}\right]}$

(Figure 6.9, EN 1993-1-1)

$h_0 := 1.8m \qquad A_{ch} := 45.39cm^2 \qquad L := 6m$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} = 735318 \cdot cm^4 \qquad e_0 := \frac{12m}{500} = 24 \cdot mm$

$A_d := 8.24cm^2$ $A_v := A_d = 8.24 \cdot cm^2$

$d := 4.39m$ $n := 1$ (number of planes of lacings)

$E := 210GPa$ $a := 4m$

$$S_v := \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^2}{d^3 \cdot \left(1 + \frac{A_d \cdot h_0^3}{A_v \cdot d^3}\right)} = 24797.487 \cdot kN$$

$N_{Ed} := 2 \cdot 1036.96kN$ - design value of the compression force to the built-up member

$M_{I.Ed} := 0kN \cdot m$ - design value of the maximum moment in the middle of the built-up mer

We will determine an in span axial distributed load, according to ...Pilkey....Table 11-7.

$q_x := 345.48 \frac{kN}{m}$ - distributed load needed to obtain the resulted axial force diagram from Robo

$$p_x \cdot L = \eta \cdot \frac{E \cdot I_{eff}}{L^2}$$

$\eta := 5.38 + 2.47 = 7.85$ (free - fixed condition)

$$N_{cr} = p_x \cdot L$$

$$N_{cr} := \eta \cdot \frac{E \cdot I_{eff}}{L^2} = 336714.368 \cdot kN$$

$$M_{Ed} := \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = 54.684 \cdot kN \cdot m$$

$$N_{ch.Ed} := 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}}$$

$N_{ch.Ed} = 1067.34 \cdot kN$

- **Buckling resistance:**

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1 \quad \text{(6.46) EN 1993-1-1}$$

$$A := 45.39cm^2$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$$

$$I_y := 2347.605\ cm^4$$

$$L_{cr} := 4m \qquad f_y := 650MPa$$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3041.054 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.985$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.177$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.549 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1618.873 \cdot kN$$

$$\frac{N_{ch.Ed}}{N_{b.Rd}} = 0.659 \quad < 1$$

Checking of uniform built-up compression members (for built of section using S500)

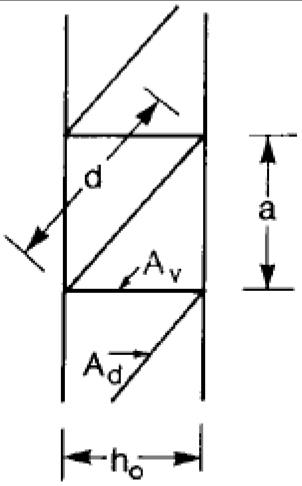
-according to EN 1993-1-1, (6.4)

The chords and diagonal bracings should be designed for buckling, in order to verify the following:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \leq 1$$

$$N_{ch.Ed} = 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} \qquad (6.69)$$

$$M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

System	
S_v	$\dfrac{nEA_d a h_0^2}{d^3\left[1+\dfrac{A_d h_0^3}{A_V d^3}\right]}$

(Figure 6.9, EN 1993-1-1)

$h_0 := 1.8m \qquad A_{ch} := 48.535cm^2 \qquad L := 6m$

$I_{eff} := 0.5 \cdot h_0^{\,2} \cdot A_{ch} = 786267 \cdot cm^4 \qquad e_0 := \dfrac{12m}{500} = 24 \cdot mm$

$A_d := 9.56cm^2 \qquad A_v := A_d = 9.56 \cdot cm^2$

$d := 4.39m \qquad n := 1$ (number of planes of lacings)

$E := 210GPa \qquad a := 4m$

$$S_v := \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^{\,2}}{d^3 \cdot \left(1 + \dfrac{A_d \cdot h_0^{\,3}}{A_v \cdot d^3}\right)} = 28769.9 \cdot kN$$

$N_{Ed} := 2 \cdot 1036.96kN$ - design value of the compression force to the built-up member

$M_{I.Ed} := 0kN \cdot m$ - design value of the maximum moment in the middle of the built-up mem

We will determine an in span axial distributed load, according to ...Pilkey....Table 11-7.

$q_x := 345.48 \frac{kN}{m}$ - distributed load needed to obtain the resulted axial force diagram from Robot

$$p_x \cdot L = \eta \cdot \frac{E \cdot I_{eff}}{L^2}$$

$\eta := 5.38 + 2.47 = 7.85$ (free - fixed condition)

$$N_{cr} = p_x \cdot L$$

$$N_{cr} := \eta \cdot \frac{E \cdot I_{eff}}{L^2} = 360044.764 \cdot kN$$

$$M_{Ed} := \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = 53.976 \cdot kN \cdot m$$

$$N_{ch.Ed} := 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}}$$

$$N_{ch.Ed} = 1066.947 \cdot kN$$

- **Buckling resistance:**

$\frac{N_{ch.Ed}}{N_{b.Rd}} \leq 1$ (6.46) EN 1993-1-1

$A := 48.535 cm^2$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$\gamma_{M1} := 1$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$$

$$I_y := 2751.924\ cm^4$$

$L_{cr} := 4m$ $f_y := 500MPa$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 3564.803 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.825$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.994$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.646 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 1568.69 \cdot kN$$

$$\frac{N_{ch.Ed}}{N_{b.Rd}} = 0.68 \quad < 1$$

Checking of uniform built-up compression members (for built of section using S355)

-according to EN 1993-1-1, (6.4)

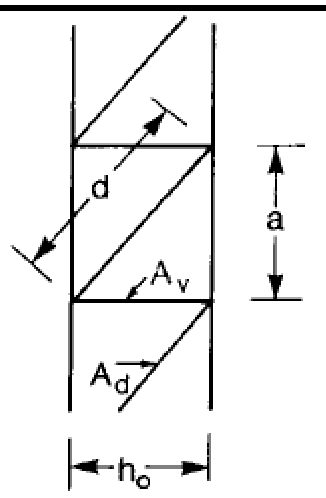
The chords and diagonal bracings should be designed for buckling, in order to verify the following:

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \le 1$$

$$N_{ch.Ed} = 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}} \qquad (6.69)$$

$$M_{Ed} = \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}}$$

System



(Figure 6.9, EN 1993-1-1)

S_v

$$\frac{nEA_d a h_0^2}{d^3\left[1+\frac{A_d h_0^3}{A_V d^3}\right]}$$

$h_0 := 1.8m \qquad A_{ch} := 51.65cm^2 \qquad L := 6m$

$I_{eff} := 0.5 \cdot h_0^2 \cdot A_{ch} = 836730 \cdot cm^4 \qquad e_0 := \frac{12m}{500} = 24 \cdot mm$

$A_d := 10.43cm^2$ $A_v := A_d = 10.43 \cdot cm^2$

$d := 4.39m$ $n := 1$ (number of planes of lacings)

$E := 210GPa$ $a := 4m$

$$S_v := \frac{n \cdot E \cdot A_d \cdot a \cdot h_0^2}{d^3 \cdot \left(1 + \frac{A_d \cdot h_0^3}{A_v \cdot d^3}\right)} = 31388.081 \cdot kN$$

$N_{Ed} := 2 \cdot 1036.96kN$ - design value of the compression force to the built-up member

$M_{I.Ed} := 0kN \cdot m$ - design value of the maximum moment in the middle of the built-up mer

We will determine an in span axial distributed load, according to ...Pilkey....Table 11-7.

$q_x := 345.48 \frac{kN}{m}$ - distributed load needed to obtain the resulted axial force diagram from Robo

$$p_x \cdot L = \eta \cdot \frac{E \cdot I_{eff}}{L^2}$$

$\eta := 5.38 + 2.47 = 7.85$ (free - fixed condition)

$$N_{cr} = p_x \cdot L$$

$$N_{cr} := \eta \cdot \frac{E \cdot I_{eff}}{L^2} = 383152.613 \cdot kN$$

$$M_{Ed} := \frac{N_{Ed} \cdot e_0 + M_{I.Ed}}{1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_v}} = 53.606 \cdot kN \cdot m$$

$$N_{ch.Ed} := 0.5 \cdot N_{Ed} + \frac{M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{eff}}$$

$N_{ch.Ed} = 1066.741 \cdot kN$

- **Buckling resistance:**

$$\frac{N_{ch.Ed}}{N_{b.Rd}} \leq 1 \quad (6.46)\ \text{EN 1993-1-1}$$

$$A := 51.65cm^2$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$$

$$\gamma_{M1} := 1$$

$$N_{cr} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2}$$

$$I_y := 3472.119\ cm^4$$

$L_{cr} := 4m$ $\qquad f_y := 650MPa$

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_y}{L_{cr}^2} = 4497.733 \cdot kN$$

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.864$$

According to Table 6.2, EN 1993-1-1, buckling curve "c" must be used.

$$\alpha := 0.49$$

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 1.036$$

$$\chi := \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.622 \quad < 1 \quad (6.49)$$

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 2088.619 \cdot kN$$

$$\frac{N_{ch.Ed}}{N_{b.Rd}} = 0.511 \quad < 1$$

ANNEX D

Determination of the total cost of S650 circular truss

Total cost is determined by:

$$C_T = C_{SM} + C_B + C_s + C_P + C_t + C_E$$

where:

C_T -total cost

C_{SM} - material cost

C_B - blasting cost

C_s - sawing cost

C_P - painting cost

C_t - tranporting cost

C_E - erecting cost

All prices are expressed in euro.

Material cost

The material cost will be calculated according to Haapio (2012) with the following formula:

$$C_{SM} = W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right)$$

W_{smpl} -weight of the plate [kg]

C_{smbp} - is basic cost

C_{smg} - is steel grade add-on

C_{smt} -is thickness add-on

C_{smq} - is quantity add-on

Top chords

$W_{smpl} := 1087.2kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{12}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 2246.155$ €

Diagonals

$W_{smpl} := 492.608kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{82}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.db} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1052.211$ €

Top braces

$W_{smpl} := 172.83kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tb} := W_{smpl} \cdot (C_{smbp} + C_{smg} + C_{smt} + C_{smq}) = 379.189$ €

Bottom chord

$W_{smpl} := 448.92kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{23}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.bc} := W_{smpl} \cdot (C_{smbp} + C_{smg} + C_{smt} + C_{smq}) = 932.407$ €

Total material cost:

$C_{SM} := C_{SM.tc} + C_{SM.db} + C_{SM.tb} + C_{SM.bc} = 4609.962$ €

Blasting cost

The blasting cost depends on the length of the beam.

$C_B = \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)$ €/min

$L := 81.22m$ -total length of chords and braces

$\upsilon_c := 3000\frac{mm}{min}$ - conveyor speed (Gietart)

$C_B := \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07) \cdot \frac{1}{min} = 29.51$ €

Sawing cost

$C_s = 1.2013(T_{NS} + T_{PS}) + T_{PS} \cdot (c_{cs} + c_{ens})$

T_{NS} - non-productive time

T_{PS} - productive time function

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

Top chord

$L := 24m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.7 \ min$ $\quad T_{NS} := 5.7min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 5771mm^2$

$Q := 4000\frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 1.443 \cdot min$

$c_{cs} := 1.2625$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tc} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 8.608$ €

Diagonals

$L := 27.52m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.876 \text{ min} \quad T_{NS} := 5.876min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 2281mm^2$

$Q := 4000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.57 \cdot min$

$c_{cs} := 1.2625$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.db} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.75$ €

Top braces

$L := 21m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.55 \text{ min} \quad T_{NS} := 5.55min$

$$T_{PS} = \frac{h}{S \cdot S_m}$$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 1049mm^2$

$Q := 4000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.262 \cdot min$

$c_{cs} := 1.2625$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tb} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 6.981$ €

Bottom chord

$L := 8.70m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 4.935$ min $T_{NS} := 4.935min$

$$T_{PS} = \frac{h}{S \cdot S_m}$$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 6569mm^2$

$$Q := 4000\,\frac{mm^2}{min}$$

$$T_{PS} := \frac{A_h}{Q} = 1.642 \cdot min$$

$$c_{cs} := 1.2625$$

$$c_{ens} := 0.02 \cdot \frac{1}{min} \quad €$$

$$C_{s.bc} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.934 \quad €$$

Total sawing cost:

$$C_s := C_{s.tc} + C_{s.db} + C_{s.tb} + C_{s.bc} = 31.273 \quad €$$

Painting cost (including drying)

$$C_p = 4.17 \cdot 10^{-6} \cdot L \cdot A_u + 0.36L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3}$$

A - painted area per unit length

W_{Amin} - smallest dimension of beam

Top chord

$$L := 24m$$

$$r := 96.85mm$$

$$A_u := 2 \cdot \pi \cdot r = 0.609\,m$$

$$W_{Amin} := 193.7mm$$

$$C_{p.tc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 62.575 \; €$$

Diagonals

$L := 27.52m$

$r := 63.5mm$

$A_u := 2 \cdot \pi \cdot r = 0.399\,m$

$W_{Amin} := 127mm$

$$C_{p.db} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 47.045 \ €$$

Top bracings

$L := 19.6m$

$r := 57.15mm$

$A_u := 2 \cdot \pi \cdot r = 0.359\,m$

$W_{Amin} := 114.3mm$

$$C_{p.tb} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 30.155 \ €$$

Bottom chord

$L := 8.70m$

$r := 109.55mm$

$A_u := 2 \cdot \pi \cdot r = 0.688\,m$

$W_{Amin} := 219.1mm$

$$C_{p.bc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 25.658 \ €$$

Total painting cost:

$C_p := C_{p.tc} + C_{p.db} + C_{p.tb} + C_{p.bc} = 165.433$ €

Transportation cost

$$C_t = \begin{cases} \left[V\cdot\left(0.0106 d_{ws} + 1.2729\right)\right] & \text{if } \frac{W}{V} \le 264\cdot\frac{kg}{m^3} \\ \left[W\cdot\left(4\cdot 10^{-5}\cdot d_{ws} + 4.8\cdot 10^{-3}\right)\right] & \text{otherwise} \end{cases}$$

V - the volume occupied by the beam

W - weight of the beam

d_{ws} -distance between workshop and site [km]

We assume that the distance between the workshop and the site i $d_{ws} := 100km$

Top chord

$L := 24m$

$d := 193.7mm$

$$A_u := \pi\cdot\frac{d^2}{4}$$

$V := L\cdot A_u = 0.707\cdot m^3$

$W := 1087.2kg$

$$\frac{W}{V} = 1537.266\frac{kg}{m^3}$$

$$C_{t.tc} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 9.567 \text{ €}$$

Diagonals

$L := 27.52m$

$d := 127mm$

$A_u := \pi \cdot \frac{d^2}{4}$

$V := L \cdot A_u = 0.349 \cdot m^3$

$W := 492.608kg$

$\frac{W}{V} = 1413.044 \frac{kg}{m^3}$

$C_{t.db} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 4.335$ €

Top bracings

$L := 21m$

$d := 114.3mm$

$A_u := \pi \cdot \frac{d^2}{4}$

$V := L \cdot A_u = 0.215 \cdot m^3$

$W := 172.83kg$

$\frac{W}{V} = 802.08 \frac{kg}{m^3}$

$C_{t.tb} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 1.521$ €

Bottom chord

$L := 8.70m$

$d := 219.1mm$

$A_u := \pi \cdot \frac{d^2}{4}$

$V := L \cdot A_u = 0.328 \cdot m^3$

$W := 448.92kg$

$\frac{W}{V} = 1368.595 \frac{kg}{m^3}$

$C_{t.bc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 3.95 \quad €$

<u>Total transportation cost:</u>

$C_t := C_{t.tc} + C_{t.db} + C_{t.tb} + C_{t.bc} = 19.374 \quad €$

<u>Erecting cost</u>

$C_E = T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E}$

$C_{LE} := 3.1 \cdot \frac{1}{min} \; €$ $\qquad C_{EqE} := 1.3460 \cdot \frac{1}{min} \; €$ $\qquad u_E := 0.36$

$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$

$L_s := 15m$ - distance from lifting area to final position

$L := 12m$

$T_E := \frac{L}{30000 \frac{mm}{min}} + \frac{L_s}{27 \frac{m}{min}} + \frac{L_s}{36 \frac{m}{min}} = 1.372 \cdot min$

$C_E := T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E} = 16.947 \; €$

The **total cost** for the S650 CHS is:

$C_{T.650} := C_{SM} + C_B + C_s + C_p + C_t + C_E = 4872.498$ €

Determination of the total cost of S500 circular truss

Total cost is determined by:

$$C_T = C_{SM} + C_B + C_s + C_P + C_t + C_E$$

where:

C_T -total cost

C_{SM} - material cost

C_B - blasting cost

C_s - sawing cost

C_P - painting cost

C_t - tranporting cost

C_E - erecting cost

All prices are expressed in euro.

Material cost

$$C_{SM} = W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right)$$

W_{smpl} -weight of the plate [kg]

C_{smbp} - is basic cost

C_{smg} - is steel grade add-on

C_{smt} -is thickness add-on

C_{smq} - is quantity add-on

Top chords

$W_{smpl} := 1087.2kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{80}{tonne}$ €

$C_{smt} := \frac{12}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 2213.539$ €

Diagonals

$W_{smpl} := 652.8kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{80}{tonne}$ €

$C_{smt} := \frac{82}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.db} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1374.797$ €

Top braces

$W_{smpl} := 172.83kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{80}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$$C_{smq} := \frac{64}{tonne} \text{ €}$$

$$C_{SM.tb} := W_{smpl} \cdot (C_{smbp} + C_{smg} + C_{smt} + C_{smq}) = 374.004 \text{ €}$$

Bottom chord

$$W_{smpl} := 571.12kg$$

$$C_{smbp} := \frac{1.88}{kg} \text{ €}$$

$$C_{smg} := \frac{80}{tonne} \text{ €}$$

$$C_{smt} := \frac{12}{tonne} \text{ €}$$

$$C_{smq} := \frac{64}{tonne} \text{ €}$$

$$C_{SM.bc} := W_{smpl} \cdot (C_{smbp} + C_{smg} + C_{smt} + C_{smq}) = 1162.8 \text{ €}$$

Total material cost:

$$C_{SM} := C_{SM.tc} + C_{SM.db} + C_{SM.tb} + C_{SM.bc} = 5125.14 \text{ €}$$

Blasting cost

The blasting cost deppends on the length of the beam.

$$C_B = \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07) \text{ €/min}$$

$L := 81m$ -total length of chords and braces

$\upsilon_c := 3000 \frac{mm}{min}$ - conveyor speed (Gietart)

$$C_B := \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07) \cdot \frac{1}{min} = 29.43 \text{ €}$$

Sawing cost

$$C_s = 1.2013(T_{NS} + T_{PS}) + T_{PS} \cdot (c_{cs} + c_{ens})$$

T_{NS} - non-productive time

T_{PS} - productive time function

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

Top chord

$L := 24m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.7 \quad min \quad T_{NS} := 5.7min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 5771mm^2$

$Q := 5000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 1.154 \cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tc} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 8.252$ €

Diagonals

$L := 27.20m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.86 \quad min \quad T_{NS} := 5.86min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 3059mm^2$

$Q := 5000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.612 \cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.db} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.78$ €

Top braces

$L := 21m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.55 \quad min \quad T_{NS} := 5.55min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 1049mm^2$

$Q := 5000\frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.21 \cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tb} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 6.917$ €

Bottom chord

$L := 8.80m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 4.94$ min $T_{NS} := 4.94min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 8262mm^2$

$Q := 5000\frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 1.652\cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02\cdot\frac{1}{min}$ €

$C_{s.bc} := 1.2\cdot\frac{1}{min}\cdot(T_{NS} + T_{PS}) + T_{PS}\cdot c_{cs}\cdot c_{ens} = 7.949$ €

Total sawing cost:

$C_s := C_{s.tc} + C_{s.db} + C_{s.tb} + C_{s.bc} = 30.897$ €

Painting cost (including drying)

$C_p = 4.17\cdot10^{-6}\cdot L\cdot A_u + 0.36L\cdot10^{-3}\cdot W_{Amin}\cdot10^{-3}$

A - painted area per unit length

W_{Amin} - smallest dimension of beam

Top chord

$L := 24m$

$r := 96.85mm$

$A_u := 2\cdot\pi\cdot r = 0.609\,m$

$W_{Amin} := 193.7mm$

$$C_{p.tc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 62.575 \text{ €}$$

Diagonals

$L := 27.20m$

$r := 84.15mm$

$A_u := 2 \cdot \pi \cdot r = 0.529\,m$

$W_{Amin} := 168.3mm$

$$C_{p.db} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 61.619 \text{ €}$$

Top bracings

$L := 21m$

$r := 57.15mm$

$A_u := 2 \cdot \pi \cdot r = 0.359\,m$

$W_{Amin} := 114.3mm$

$$C_{p.tb} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 32.309 \text{ €}$$

Bottom chord

$L := 8.80m$

$r := 136.5mm$

$A_u := 2 \cdot \pi \cdot r = 0.858\,m$

$W_{Amin} := 273mm$

$$C_{p.bc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 32.337 \text{ €}$$

Total painting cost:

$$C_p := C_{p.tc} + C_{p.db} + C_{p.tb} + C_{p.bc} = 188.84 \quad €$$

Transportation cost

$$C_t = \begin{cases} \left[V\cdot(0.0106 d_{ws} + 1.2729)\right] & \text{if } \frac{W}{V} \le 264\cdot\frac{kg}{m^3} \\ \left[W\cdot\left(4\cdot10^{-5}\cdot d_{ws} + 4.8\cdot10^{-3}\right)\right] & \text{otherwise} \end{cases}$$

V - the volume occupied by the beam

W - weight of the beam

d_{ws} -distance between workshop and site [km]

We assume that the distance between the workshop and the site i $d_{ws} := 100km$

Top chord

$L := 24m$

$d := 193.7mm$

$$A_u := \pi\cdot\frac{d^2}{4}$$

$$V := L\cdot A_u = 0.707\cdot m^3$$

$$W := 1087.2kg$$

$$\frac{W}{V} = 1537.266\frac{kg}{m^3}$$

$$C_{t.tc} := W\cdot\left(\frac{4\cdot10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot10^{-3}}{kg}\right) = 9.567 \quad €$$

Diagonals

$L := 27.2m$

$d := 168.3mm$

$$A_u := \pi \cdot \frac{d^2}{4}$$

$$V := L \cdot A_u = 0.605 \cdot m^3$$

$$W := 652.8kg$$

$$\frac{W}{V} = 1078.83 \frac{kg}{m^3}$$

$$C_{t.db} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 5.745 \ €$$

Top bracings

$$L := 21m$$

$$d := 114.3mm$$

$$A_u := \pi \cdot \frac{d^2}{4}$$

$$V := L \cdot A_u = 0.215 \cdot m^3$$

$$W := 172.83kg$$

$$\frac{W}{V} = 802.08 \frac{kg}{m^3}$$

$$C_{t.tb} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 1.521 \ €$$

Bottom chord

$$L := 8.80m$$

$$d := 273mm$$

$$A_u := \pi \cdot \frac{d^2}{4}$$

$$V := L \cdot A_u = 0.515 \cdot m^3$$

$$W := 571.12kg$$

$$\frac{W}{V} = 1108.74\frac{kg}{m^3}$$

$$C_{t.bc} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 5.026\ €$$

Total transportation cost:

$$C_t := C_{t.tc} + C_{t.db} + C_{t.tb} + C_{t.bc} = 21.859 \quad €$$

Erecting cost

$$C_E = T_E\cdot\frac{C_{LE} + C_{EqE}}{u_E}$$

$$C_{LE} := 3.1\cdot\frac{1}{min}\ € \qquad C_{EqE} := 1.3460\cdot\frac{1}{min}\ € \qquad u_E := 0.36$$

$$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$$

$L_s := 15m$ - distance from lifting area to final position

$L := 12m$

$$T_E := \frac{L}{30000\frac{mm}{min}} + \frac{L_s}{27\frac{m}{min}} + \frac{L_s}{36\frac{m}{min}} = 1.372\cdot min$$

$$C_E := T_E\cdot\frac{C_{LE} + C_{EqE}}{u_E} = 16.947\ €$$

The **total cost** for the S500 CHS is:

$$C_{T.500} := C_{SM} + C_B + C_s + C_p + C_t + C_E = 5413.113 \quad €$$

Determination of the total cost of S355 circular truss

Total cost is determined by:

$$C_T = C_{SM} + C_B + C_s + C_P + C_t + C_E$$

where:

C_T -total cost

C_{SM} - material cost

C_B - blasting cost

C_s - sawing cost

C_P - painting cost

C_t - tranporting cost

C_E - erecting cost

All prices are expressed in euro.

Material cost

$$C_{SM} = W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right)$$

W_{smpl} -weight of the plate [kg]

C_{smbp} - is basic cost

C_{smg} - is steel grade add-on

C_{smt} -is thickness add-on

C_{smq} - is quantity add-on

Top chords

$W_{smpl} := 1528.8kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{35}{tonne}$ €

$C_{smt} := \frac{0}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 3025.495$ €

Diagonals

$W_{smpl} := 673.848kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{35}{tonne}$ €

$C_{smt} := \frac{82}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.db} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1388.801$ €

Top braces

$W_{smpl} := 216.3kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{35}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tb} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 458.34$ €

Bottom chord

$W_{smpl} := 864kg$

$C_{smbp} := \frac{1.88}{kg}$ €

$C_{smg} := \frac{35}{tonne}$ €

$C_{smt} := \frac{0}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.bc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1709.856$ €

Total material cost:

$C_{SM} := C_{SM.tc} + C_{SM.db} + C_{SM.tb} + C_{SM.bc} = 6582.492$ €

Blasting cost

The blasting cost deppends on the length of the beam.

$C_B = \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)$ €/min

$L := 80.74m$ -total length of chords and braces

$\upsilon_c := 3000 \frac{mm}{min}$ - conveyor speed (Gietart)

$$C_B := \frac{L}{\upsilon_c}\cdot(0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)\cdot\frac{1}{min} = 29.336 \quad €$$

Sawing cost

$$C_s = 1.2013\left(T_{NS} + T_{PS}\right) + T_{PS}\cdot\left(c_{cs} + c_{ens}\right)$$

T_{NS} - non-productive time

T_{PS} - productive time function

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

Top chord

$L := 24m$

$$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.7 \quad min \quad T_{NS} := 5.7min$$

$$T_{PS} = \frac{h}{S\cdot S_m}$$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 8113mm^2$

$$Q := 8800\frac{mm^2}{min}$$

$$T_{PS} := \frac{A_h}{Q} = 0.922\cdot min$$

$c_{cs} := 1$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tc} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.965$ €

Diagonals

$L := 26.74m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.837$ min $T_{NS} := 5.837min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 3206mm^2$

$Q := 8800 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.364 \cdot min$

$c_{cs} := 1$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.db} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.449$ €

Top braces

$L := 21m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.55 \quad min \quad T_{NS} := 5.55min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 1307mm^2$

$Q := 8800 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.149 \cdot min$

$c_{cs} := 1$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tb} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 6.841$ €

Bottom chord

$L := 9m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 4.95 \quad min \quad T_{NS} := 4.95min$

$T_{PS} = \frac{h}{S \cdot S_m}$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$$A_h := 9600mm^2$$

$$Q := 8800\frac{mm^2}{min}$$

$$T_{PS} := \frac{A_h}{Q} = 1.091 \cdot min$$

$$c_{cs} := 1$$

$$c_{ens} := 0.02 \cdot \frac{1}{min} \quad €$$

$$C_{s.bc} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.271 \quad €$$

Total sawing cost:

$$C_s := C_{s.tc} + C_{s.db} + C_{s.tb} + C_{s.bc} = 29.526 \quad €$$

Painting cost (including drying)

$$C_p = 4.17 \cdot 10^{-6} \cdot L \cdot A_u + 0.36L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3}$$

A - painted area per unit length

W_{Amin} - smallest dimension of beam

Top chord

$$L := 24m$$

$r := 109.55mm$

$A_u := 2 \cdot \pi \cdot r = 0.688\,m$

$W_{Amin} := 219.1mm$

$$C_{p.tc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 70.78 \quad €$$

Diagonals

$L := 26.74m$

$r := 84.15mm$

$A_u := 2 \cdot \pi \cdot r = 0.529\,m$

$W_{Amin} := 168.3mm$

$$C_{p.db} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 60.577 \quad €$$

Top bracings

$L := 21m$

$r := 54mm$

$A_u := 2 \cdot \pi \cdot r = 0.339\,m$

$W_{Amin} := 108mm$

$$C_{p.tb} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 30.528 \quad €$$

Bottom chord

$L := 9m$

$r := 161.95mm$

$A_u := 2\cdot\pi\cdot r = 1.018\,m$

$W_{Amin} := 323.9mm$

$$C_{p.bc} := \frac{4.17}{mm^2}\cdot 10^{-6}\cdot L\cdot A_u + \frac{0.36}{mm^2}L\cdot 10^{-3}\cdot W_{Amin}\cdot 10^{-3} = 39.239 \quad €$$

Total painting cost:

$$C_p := C_{p.tc} + C_{p.db} + C_{p.tb} + C_{p.bc} = 201.124 \quad €$$

Transportation cost

$$C_t = \begin{cases} \left[V\cdot\left(0.0106d_{ws} + 1.2729\right)\right] & \text{if } \frac{W}{V} \le 264\cdot\frac{kg}{m^3} \\ \left[W\cdot\left(4\cdot 10^{-5}\cdot d_{ws} + 4.8\cdot 10^{-3}\right)\right] & \text{otherwise} \end{cases}$$

V - the volume occupied by the beam

W - weight of the beam

d_{ws} -distance between workshop and site [km]

We assume that the distance between the workshop and the site i $d_{ws} := 100km$

Top chord

$L := 24m$

$d := 219.1mm$

$$A_u := \pi\cdot\frac{d^2}{4} = 37702.89\cdot mm^2$$

$V := L\cdot A_u = 0.905\cdot m^3$

$W := 1528.8kg$

$$\frac{W}{V} = 1689.526\frac{kg}{m^3}$$

$$C_{t.tc} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 13.453 \quad €$$

Diagonals

$$L := 26.74m$$

$$d := 168.3mm$$

$$A_u := \pi\cdot\frac{d^2}{4}$$

$$V := L\cdot A_u = 0.595\cdot m^3$$

$$W := 673.848kg$$

$$\frac{W}{V} = 1132.772\frac{kg}{m^3}$$

$$C_{t.db} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 5.93 \quad €$$

Top bracings

$$L := 21m$$

$$d := 108mm$$

$$A_u := \pi\cdot\frac{d^2}{4} = 9160.884\cdot mm^2$$

$$V := L\cdot A_u = 0.192\cdot m^3$$

$$W := 216.3kg$$

$$\frac{W}{V} = 1124.346\frac{kg}{m^3}$$

$$C_{t.tb} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 1.903 \quad €$$

Bottom chord

$L := 9m$

$d := 323.9mm$

$A_u := \pi \cdot \frac{d^2}{4}$

$V := L \cdot A_u = 0.742 \cdot m^3$

$W := 864kg$

$\frac{W}{V} = 1165.09 \frac{kg}{m^3}$

$C_{t.bc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 7.603$ €

Total transportation cost:

$C_t := C_{t.tc} + C_{t.db} + C_{t.tb} + C_{t.bc} = 28.89$ €

Erecting cost

$C_E = T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E}$

$C_{LE} := 3.1 \cdot \frac{1}{min}$ € $C_{EqE} := 1.3460 \cdot \frac{1}{min}$ € $u_E := 0.36$

$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$

$L_s := 15m$ - distance from lifting area to final position

$L := 12m$

$T_E := \frac{L}{30000 \frac{mm}{min}} + \frac{L_s}{27 \frac{m}{min}} + \frac{L_s}{36 \frac{m}{min}} = 1.372 \cdot min$

$$C_E := T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E} = 16.947 \text{ €}$$

The **total cost** for the S355 CHS is:

$$C_{T.355} := C_{SM} + C_B + C_s + C_p + C_t + C_E = 6888.314 \quad \text{€}$$

Determination of the total cost of S650 polygonal truss

Total cost is determined by:

$$C_T = C_{SM} + C_B + C_s + C_P + C_t + C_E$$

where:

C_T -total cost

C_{SM} - material cost

C_B - blasting cost

C_s - sawing cost

C_P - painting cost

C_t - tranporting cost

C_E - erecting cost

All prices are expressed in euro.

Material cost

The material cost will be calculated according to Haapio (2012) with the following formula:

$$C_{SM} = W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right)$$

W_{smpl} -weight of the plate [kg]

C_{smbp} - is basic cost

C_{smg} - is steel grade add-on

C_{smt} -is thickness add-on

C_{smq} - is quantity add-on

Top chords

$W_{smpl} := 855.299kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{82}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1218.801$ €

Diagonals

$W_{smpl} := 448.82kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.db} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 665.6$ €

Top braces

$W_{smpl} := 172.83kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tb} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 256.307$ €

Bottom chord

$W_{smpl} := 376.783kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{110}{tonne}$ €

$C_{smt} := \frac{82}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.bc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 536.916$ €

Total material cost:

$C_{SM} := C_{SM.tc} + C_{SM.db} + C_{SM.tb} + C_{SM.bc} = 2677.624$ €

Blasting cost

The blasting cost depends on the length of the beam.

$C_B = \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)$ €/min

$L := 80.9m$ -total length of chords and braces

$\upsilon_c := 3000 \frac{mm}{min}$ - conveyor speed (Gietart)

$$C_B := \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07) \cdot \frac{1}{min} = 29.394 \quad €$$

Sawing cost

$$C_s = 1.2013\left(T_{NS} + T_{PS}\right) + T_{PS} \cdot \left(c_{cs} + c_{ens}\right)$$

T_{NS} - non-productive time

T_{PS} - productive time function

A_h - area of parts of the profile

Q - sawing efficiency of the blade for solid material

Top chord

$$L := 24m$$

$$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.7 \; min \qquad T_{NS} := 5.7min$$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$$A_h := 4539.8mm^2$$

$$Q := 4000 \frac{mm^2}{min}$$

$$T_{PS} := \frac{A_h}{Q} = 1.135 \cdot min$$

$$c_{cs} := 1.2625$$

$$c_{ens} := 0.02 \cdot \frac{1}{min} \quad €$$

$$C_{s.tc} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 8.231 \; €$$

Diagonals

$L := 27.2m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.86 \quad min \quad T_{NS} := 5.86min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 2102mm^2$

$Q := 4000\frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.525 \cdot min$

$c_{cs} := 1.2625$

$c_{ens} := 0.02 \cdot \frac{1}{min} \quad €$

$C_{s.db} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.676€$

Top braces

$L := 21m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.55 \quad min \quad T_{NS} := 5.55min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 1049 mm^2$

$Q := 4000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.262 \cdot min$

$c_{cs} := 1.2625$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tb} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 6.981$ €

Bottom chord

$L := 8.7m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 4.935$ min $\quad T_{NS} := 4.935 min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 5516.73 mm^2$

$Q := 4000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 1.379 \cdot min$

$c_{cs} := 1.2625$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.bc} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.612$ €

Total sawing cost:

$C_s := C_{s.tc} + C_{s.db} + C_{s.tb} + C_{s.bc} = 30.5$ €

Painting cost (including drying)

$C_p = 4.17 \cdot 10^{-6} \cdot L \cdot A_u + 0.36 L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3}$

A - painted area per unit length

W_{Amin} - smallest dimension of beam

Top chord

$L := 24m$

$A_u := 618mm$

$W_{Amin} := 200mm$

$C_{p.tc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 63.577$ €

Diagonals

$L := 27.52m$

$A_u := 432.62mm$

$W_{Amin} := 140mm$

$C_{p.db} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 51.034$ €

Top bracings

$L := 19.6m$

$r := 57.15mm$

$A_u := 2\pi \cdot r = 359.084 \cdot mm$

$W_{Amin} := 114.3mm$

$$C_{p.tb} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 30.155 \text{ €}$$

Bottom chord

$L := 8.70m$

$A_u := 1854.72mm$

$W_{Amin} := 249mm$

$$C_{p.bc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 68.067 \text{ €}$$

Total painting cost:

$$C_p := C_{p.tc} + C_{p.db} + C_{p.tb} + C_{p.bc} = 212.834 \quad \text{€}$$

Transportation cost

$$C_t = \begin{cases} \left[V \cdot \left(0.0106 d_{ws} + 1.2729\right)\right] & \text{if } \frac{W}{V} \le 264 \cdot \frac{kg}{m^3} \\ \left[W \cdot \left(4 \cdot 10^{-5} \cdot d_{ws} + 4.8 \cdot 10^{-3}\right)\right] & \text{otherwise} \end{cases}$$

V - the volume occupied by the beam

W - weight of the beam

d_{ws} -distance between workshop and site [km]

We assume that the distance between the workshop and the site i $d_{ws} := 100km$

Top chord

$L := 24m$

$A_u := 29389.26mm^2$

$V := L \cdot A_u = 0.705 \cdot m^3$

$W := 855.29kg$

$\frac{W}{V} = 1212.589 \frac{kg}{m^3}$

$C_{t.tc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 7.527$ €

Diagonals

$L := 27.2m$

$A_u := 14400.73mm^2$

$V := L \cdot A_u = 0.392 \cdot m^3$

$W := 448.82kg$

$\frac{W}{V} = 1145.826 \frac{kg}{m^3}$

$C_{t.db} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 3.95$ €

Top bracings

$L := 21m$

$r := 57.15mm$

$A_u := \pi \cdot r^2 = 1.026 \times 10^4 \cdot mm^2$

$V := L \cdot A_u = 0.215 \cdot m^3$

$W := 172.83kg$

$$\frac{W}{V} = 802.08 \frac{kg}{m^3}$$

$$C_{t.tb} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 1.521 \quad €$$

Bottom chord

$$L := 8.70m$$

$$A_u := 1.19 \times 10^5 mm^2$$

$$V := L \cdot A_u = 1.035 \cdot m^3$$

$$W := 376.783kg$$

$$\frac{W}{V} = 363.936 \frac{kg}{m^3}$$

$$C_{t.bc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 3.316 \quad €$$

Total transportation cost:

$$C_t := C_{t.tc} + C_{t.db} + C_{t.tb} + C_{t.bc} = 16.313 \quad €$$

Erecting cost

$$C_E = T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E}$$

$$C_{LE} := 3.1 \cdot \frac{1}{min} \; € \qquad C_{EqE} := 1.3460 \cdot \frac{1}{min} \; € \qquad u_E := 0.36$$

$$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$$

$L_s := 15m$ - distance from lifting area to final position

$L := 12m$

$$T_E := \frac{L}{30000\frac{mm}{min}} + \frac{L_s}{27\frac{m}{min}} + \frac{L_s}{36\frac{m}{min}} = 1.372 \cdot min$$

$$C_E := T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E} = 16.947 \ €$$

The **total cost** for the S650 built-up section is:

$$C_{T.650} := C_{SM} + C_B + C_s + C_p + C_t + C_E = 2983.61 \quad €$$

Determination of the total cost of S500 polygonal truss

Total cost is determined by:

$$C_T = C_{SM} + C_B + C_s + C_P + C_t + C_E$$

where:

C_T -total cost

C_{SM} - material cost

C_B - blasting cost

C_s - sawing cost

C_P - painting cost

C_t - tranporting cost

C_E - erecting cost

All prices are expressed in euro.

Material cost

The material cost will be calculated according to Haapio (2012) with the following formula:

$$C_{SM} = W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right)$$

W_{smpl} -weight of the plate [kg]

C_{smbp} - is basic cost

C_{smg} - is steel grade add-on

C_{smt} -is thickness add-on

C_{smq} - is quantity add-on

Top chords

$$W_{smpl} := 914.4kg$$

$$C_{smbp} := \frac{1.169}{kg} \text{ €}$$

$$C_{smg} := \frac{80}{tonne} \text{ €}$$

$$C_{smt} := \frac{82}{tonne} \text{ €}$$

$$C_{smq} := \frac{64}{tonne} \text{ €}$$

$$C_{SM.tc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1275.588 \text{ €}$$

Diagonals

$$W_{smpl} := 569.67kg$$

$$C_{smbp} := \frac{1.169}{kg} \text{ €}$$

$$C_{smg} := \frac{80}{tonne} \text{ €}$$

$$C_{smt} := \frac{140}{tonne} \text{ €}$$

$$C_{smq} := \frac{64}{tonne} \text{ €}$$

$$C_{SM.db} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 827.731 \text{ €}$$

Top braces

$W_{smpl} := 172.83kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{80}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tb} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 251.122$ €

Bottom chord

$W_{smpl} := 455.92kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{80}{tonne}$ €

$C_{smt} := \frac{82}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.bc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 636.008$ €

Total material cost:

$C_{SM} := C_{SM.tc} + C_{SM.db} + C_{SM.tb} + C_{SM.bc} = 2990.449$ €

Blasting cost

The blasting cost depends on the length of the beam.

$C_B = \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)$ €/min

$L := 81m$ -total length of chords and braces

$\upsilon_c := 3000 \frac{mm}{min}$ - conveyor speed (Gietart)

$C_B := \frac{L}{\upsilon_c} \cdot (0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07) \cdot \frac{1}{min} = 29.43$ €

<u>Sawing cost</u>

$C_s = 1.2013(T_{NS} + T_{PS}) + T_{PS} \cdot (c_{cs} + c_{ens})$

T_{NS} - non-productive time

T_{PS} - productive time function

A_h - area of parts of the profile

Q - sawing efficiency of the blade for solid material

<u>Top chord</u>

$L := 24m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.7\ min$ $T_{NS} := 5.7min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 4853.5mm^2$

$Q := 5000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.971 \cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tc} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 8.027$ €

Diagonals

$L := 27.2m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.86$ min $T_{NS} := 5.86min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 2668mm^2$

$Q := 5000 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.534 \cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.db} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.685$€

Top braces

$L := 21m$

$$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.55 \quad min \quad T_{NS} := 5.55min$$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$$A_h := 1049mm^2$$

$$Q := 5000 \frac{mm^2}{min}$$

$$T_{PS} := \frac{A_h}{Q} = 0.21 \cdot min$$

$$c_{cs} := 1.15$$

$$c_{ens} := 0.02 \cdot \frac{1}{min} \ €$$

$$C_{s.tb} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 6.917 \ €$$

Bottom chord

$$L := 8.8m$$

$$T_{NS} := 4.5 + \frac{L}{20000mm} = 4.94 \quad min \quad T_{NS} := 4.94min$$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 6600mm^2$

$Q := 5000\frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 1.32 \cdot min$

$c_{cs} := 1.15$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.bc} := 1.2 \cdot \frac{1}{min} \cdot (T_{NS} + T_{PS}) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.542$ €

Total sawing cost:

$C_s := C_{s.tc} + C_{s.db} + C_{s.tb} + C_{s.bc} = 30.171$ €

Painting cost (including drying)

$C_p = 4.17 \cdot 10^{-6} \cdot L \cdot A_u + 0.36L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3}$

A - painted area per unit length

W_{Amin} - smallest dimension of beam

Top chord

$L := 24m$

$A_u := 680mm$

$W_{Amin} := 220mm$

$C_{p.tc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 69.955$ €

Diagonals

$L := 27.2m$

$A_u := 494.42mm$

$W_{Amin} := 160mm$

$$C_{p.db} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 57.646 €$$

Top bracings

$L := 21m \qquad r := 57.15mm$

$A_u := 2 \cdot \pi \cdot r = 359.084 \cdot mm$

$W_{Amin} := 114.3mm$

$$C_{p.tb} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 32.309 \text{ €}$$

Bottom chord

$L := 8.80m$

$A_u := 2035.79mm$

$W_{Amin} := 273.9mm$

$$C_{p.bc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 75.573 €$$

Total painting cost:

$C_p := C_{p.tc} + C_{p.db} + C_{p.tb} + C_{p.bc} = 235.483 \quad €$

Transportation cost

$$C_t = \begin{cases} \left[V \cdot \left(0.0106 d_{ws} + 1.2729\right)\right] & \text{if } \frac{W}{V} \le 264 \cdot \frac{kg}{m^3} \\ \left[W \cdot \left(4 \cdot 10^{-5} \cdot d_{ws} + 4.8 \cdot 10^{-3}\right)\right] & \text{otherwise} \end{cases}$$

V - the volume occupied by the beam

W - weight of the beam

d_{ws} -distance between workshop and site [km]

We assume that the distance between the workshop and the site i $d_{ws} := 100km$

Top chord

$L := 24m$

$A_u := 35561mm^2$

$V := L \cdot A_u = 0.853 \cdot m^3$

$W := 914.4kg$

$\frac{W}{V} = 1071.398 \frac{kg}{m^3}$

$C_{t.tc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 8.047$ €

Diagonals

$L := 27.2m$

$A_u := 18809.12mm^2$

$V := L \cdot A_u = 0.512 \cdot m^3$

$W := 569.67kg$

$\frac{W}{V} = 1113.489 \frac{kg}{m^3}$

$C_{t.db} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 5.013$ €

Top bracings

$L := 21m$ $r := 57.15mm$

$A_u := \pi \cdot r^2 = 0.01 m^2$

$V := L \cdot A_u = 0.215 \cdot m^3$

$W := 172.83kg$

$\frac{W}{V} = 802.08 \frac{kg}{m^3}$

$C_{t.tb} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 1.521$ €

Bottom chord

$L := 8.80m$

$A_u := 145260.1mm^2$

$V := L \cdot A_u = 1.278 \cdot m^3$

$W := 455.93kg$

$\frac{W}{V} = 356.672 \frac{kg}{m^3}$

$C_{t.bc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 4.012$ €

Total transportation cost:

$C_t := C_{t.tc} + C_{t.db} + C_{t.tb} + C_{t.bc} = 18.593$ €

Erecting cost

$C_E = T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E}$

$C_{LE} := 3.1 \cdot \frac{1}{min}$ € $C_{EqE} := 1.3460 \cdot \frac{1}{min}$ € $u_E := 0.36$

$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$

$L_s := 15m$ - distance from lifting area to final position

$L := 12m$

$$T_E := \frac{L}{30000\frac{mm}{min}} + \frac{L_s}{27\frac{m}{min}} + \frac{L_s}{36\frac{m}{min}} = 1.372 \cdot min$$

$$C_E := T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E} = 16.947 \ €$$

The **total cost** for the S500 built-up section is:

$$C_{T.500} := C_{SM} + C_B + C_s + C_p + C_t + C_E = 3321.073 \quad €$$

Determination of the total cost of S355 polygonal truss

Total cost is determined by:

$$C_T = C_{SM} + C_B + C_s + C_P + C_t + C_E$$

where:

C_T -total cost

C_{SM} - material cost

C_B - blasting cost

C_s - sawing cost

C_P - painting cost

C_t - tranporting cost

C_E - erecting cost

All prices are expressed in euro.

Material cost

The material cost will be calculated according to Haapio (2012) with the following formula:

$$C_{SM} = W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right)$$

W_{smpl} -weight of the plate [kg]

C_{smbp} - is basic cost

C_{smg} - is steel grade add-on

C_{smt} -is thickness add-on

C_{smq} - is quantity add-on

Top chords

$$W_{smpl} := 973.09kg$$

$$C_{smbp} := \frac{1.169}{kg} \ €$$

$$C_{smg} := \frac{35}{tonne} \ €$$

$$C_{smt} := \frac{82}{tonne} \ €$$

$$C_{smq} := \frac{64}{tonne} \ €$$

$$C_{SM.tc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 1313.672 \ €$$

Diagonals

$$W_{smpl} := 626.96kg$$

$$C_{smbp} := \frac{1.169}{kg} \ €$$

$$C_{smg} := \frac{35}{tonne} \ €$$

$$C_{smt} := \frac{140}{tonne} \ €$$

$$C_{smq} := \frac{64}{tonne} \ €$$

$$C_{SM.db} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 882.76 \ €$$

Top braces

$W_{smpl} := 216.3kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{35}{tonne}$ €

$C_{smt} := \frac{140}{tonne}$ €

$C_{smq} := \frac{64}{tonne}$ €

$C_{SM.tb} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 304.55$ €

Bottom chord

$W_{smpl} := 657kg$

$C_{smbp} := \frac{1.169}{kg}$ €

$C_{smg} := \frac{35}{tonne}$ €

$C_{smt} := \frac{23}{tonne}$ €

$C_{smq} := \frac{82}{tonne}$ €

$C_{SM.bc} := W_{smpl} \cdot \left(C_{smbp} + C_{smg} + C_{smt} + C_{smq}\right) = 860.013$ €

Total material cost:

$C_{SM} := C_{SM.tc} + C_{SM.db} + C_{SM.tb} + C_{SM.bc} = 3360.995$ €

Blasting cost

The blasting cost depends on the length of the beam.

$$C_B = \frac{L}{\upsilon_c}\cdot(0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07) \text{ €/min}$$

$L := 80.74m$ -total length of chords and braces

$\upsilon_c := 3000\frac{mm}{min}$ - conveyor speed (Gietart)

$$C_B := \frac{L}{\upsilon_c}\cdot(0.46 + 0.13 + 0.01 + 0.16 + 0.24 + 0.02 + 0.07)\cdot\frac{1}{min} = 29.336 \quad €$$

Sawing cost

$$C_s = 1.2013\left(T_{NS} + T_{PS}\right) + T_{PS}\cdot\left(c_{cs} + c_{ens}\right)$$

T_{NS} - non-productive time

T_{PS} - productive time function

A_h - area of parts of the profile

Q - sawing efficiency of the blade for solid material

Top chord

$L := 24m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.7 \text{ min}$ $\quad T_{NS} := 5.7min$

$$T_{PS} = \frac{A_h}{Q}$$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 5165mm^2$

$Q := 8800\frac{mm^2}{min}$

$$T_{PS} := \frac{A_h}{Q} = 0.587\cdot min$$

$c_{cs} := 1$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tc} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.556$ €

Diagonals

$L := 26.74m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.837$ min $\quad T_{NS} := 5.837min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 2986.86mm^2$

$Q := 8800 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.339 \cdot min$

$c_{cs} := 1$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.db} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.418$€

Top braces

$L := 21m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 5.55 \quad min \quad T_{NS} := 5.55min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 1307mm^2$

$Q := 8800 \frac{mm^2}{min}$

$T_{PS} := \frac{A_h}{Q} = 0.149 \cdot min$

$c_{cs} := 1$

$c_{ens} := 0.02 \cdot \frac{1}{min}$ €

$C_{s.tb} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 6.841$ €

Bottom chord

$L := 9m$

$T_{NS} := 4.5 + \frac{L}{20000mm} = 4.95 \quad min \quad T_{NS} := 4.95min$

$T_{PS} = \frac{A_h}{Q}$

c_{cs} - cost factor depending on the steel grade

c_{ens} - cost of energy

$A_h := 9300mm^2$

$$Q := 8800 \frac{mm^2}{min}$$

$$T_{PS} := \frac{A_h}{Q} = 1.057 \cdot min$$

$$c_{cs} := 1$$

$$c_{ens} := 0.02 \cdot \frac{1}{min} \ €$$

$$C_{s.bc} := 1.2 \cdot \frac{1}{min} \cdot \left(T_{NS} + T_{PS}\right) + T_{PS} \cdot c_{cs} \cdot c_{ens} = 7.229 \ €$$

Total sawing cost:

$$C_s := C_{s.tc} + C_{s.db} + C_{s.tb} + C_{s.bc} = 29.045 \ €$$

Painting cost (including drying)

$$C_p = 4.17 \cdot 10^{-6} \cdot L \cdot A_u + 0.36L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3}$$

A - painted area per unit length

W_{Amin} - smallest dimension of beam

Top chord

$$L := 24m$$

$$A_u := 741.64mm$$

$$W_{Amin} := 240mm$$

$$C_{p.tc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 76.297 \ €$$

Diagonals

$$L := 26.74m$$

$A_u := 587.13mm$

$W_{Amin} := 190mm$

$$C_{p.db} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 67.297 €$$

Top bracings

$L := 21m \quad r := 54mm$

$A_u := 2 \cdot \pi \cdot r = 339.292 \cdot mm$

$W_{Amin} := 108mm$

$$C_{p.tb} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 30.528\ €$$

Bottom chord

$L := 9m$

$A_u := 2414.34mm$

$W_{Amin} := 325mm$

$$C_{p.bc} := \frac{4.17}{mm^2} \cdot 10^{-6} \cdot L \cdot A_u + \frac{0.36}{mm^2} L \cdot 10^{-3} \cdot W_{Amin} \cdot 10^{-3} = 91.663 €$$

Total painting cost:

$C_p := C_{p.tc} + C_{p.db} + C_{p.tb} + C_{p.bc} = 265.786$ €

Transportation cost

$$C_t = \begin{cases} \left[V\cdot\left(0.0106 d_{ws} + 1.2729\right)\right] & \text{if } \frac{W}{V} \le 264\cdot\frac{kg}{m^3} \\ \left[W\cdot\left(4\cdot 10^{-5}\cdot d_{ws} + 4.8\cdot 10^{-3}\right)\right] & \text{otherwise} \end{cases}$$

V - the volume occupied by the beam

W - weight of the beam

d_{ws} -distance between workshop and site [km]

We assume that the distance between the workshop and the site i $d_{ws} := 100km$

Top chord

$L := 24m$

$A_u := 42320.54mm^2$

$V := L\cdot A_u = 1.016\cdot m^3$

$W := 973.08kg$

$\frac{W}{V} = 958.045\frac{kg}{m^3}$

$C_{t.tc} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 8.563$ €

Diagonals

$L := 26.74m$

$A_u := 26523.8mm^2$

$V := L\cdot A_u = 0.709\cdot m^3$

$W := 626.97kg$

$\frac{W}{V} = 883.995\frac{kg}{m^3}$

$C_{t.db} := W\cdot\left(\frac{4\cdot 10^{-5}}{km\cdot kg}\cdot d_{ws} + \frac{4.8\cdot 10^{-3}}{kg}\right) = 5.517$ €

Top bracings

$L := 21m \qquad r := 54mm$

$A_u := \pi \cdot r^2 = 0.009\,m^2$

$V := L \cdot A_u = 0.192 \cdot m^3$

$W := 159.57kg$

$\frac{W}{V} = 829.458 \frac{kg}{m^3}$

$C_{t.tb} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 1.404 \quad €$

Bottom chord

$L := 9m$

$A_u := 202554.2mm^2$

$V := L \cdot A_u = 1.823 \cdot m^3$

$W := 657.045kg$

$\frac{W}{V} = 360.422 \frac{kg}{m^3}$

$C_{t.bc} := W \cdot \left(\frac{4 \cdot 10^{-5}}{km \cdot kg} \cdot d_{ws} + \frac{4.8 \cdot 10^{-3}}{kg} \right) = 5.782 \quad €$

Total transportation cost:

$C_t := C_{t.tc} + C_{t.db} + C_{t.tb} + C_{t.bc} = 21.267 \quad €$

Erecting cost

$C_E = T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E}$

$C_{LE} := 3.1 \cdot \frac{1}{min} \; € \qquad C_{EqE} := 1.3460 \cdot \frac{1}{min} \; € \qquad u_E := 0.36$

$$T_E = \frac{L}{30000} + \frac{L_s}{27} + \frac{L_s}{36}$$

$L_s := 15m$ - distance from lifting area to final position

$L := 12m$

$$T_E := \frac{L}{30000\frac{mm}{min}} + \frac{L_s}{27\frac{m}{min}} + \frac{L_s}{36\frac{m}{min}} = 1.372 \cdot min$$

$$C_E := T_E \cdot \frac{C_{LE} + C_{EqE}}{u_E} = 16.947 \ \text{€}$$

The **total cost** for the S355 built-up section is:

$$C_{T.355} := C_{SM} + C_B + C_s + C_p + C_t + C_E = 3723.375 \quad \text{€}$$

ANNEX E

CO_2 emissions for the S650 CHS truss

Steel production

$m_{650} := 2201.55kg$ - total mass of the steel

The amount of CO_2 emissions is converted to a higher strength steel with the following formula, presented in "Jan-Olof steel eco-cycle":

$$co_{2.emiss.650} := \left[0.00018\cdot(650 - 355) + \frac{1070}{1000}\right]\cdot 1000 = 1123.1$$

$$co_{2.emiss.650} := \frac{1123.1gm}{kg}$$

$$co_{2.steel} := m_{650}\cdot co_{2.emiss.650} = 2472560.805\cdot gm\, CO_2$$

Painting

Acrylic paint will be used for the truss elements.

$m_{acryl} := 2.5\frac{kg}{L}$ -amount of CO_2 emission according to JouCO2&COSTi

$t_{paint} := 1.5mm$ - thickness of paint applied to the truss elements

$A_{paint} := 38.611\ m^2$ - painted area

$V_{paint} := A_{paint}\cdot t_{paint} = 57.916\,L$ -volume of paint needed

$$co_{2.acryl} := m_{acryl}\cdot V_{paint} = 144791.25\cdot gn\, CO_2$$

$$co_{2.paint} := co_{2.acryl} = 144791.25\cdot gm\ \ CO_2$$

Transportation

We will consider that a semi-track with the capacity of 25tonne is used for the transportation.

According to Lipasto.vtt.fi (year 2011), the environmental impact of the fully loaded track is:

$$co_{2.track} := 41 \frac{gm}{tonne \cdot km} \quad CO_2$$

The emissions of the empty track are $co_{2.track.empty} := 757 \frac{gm}{km} \; CO_2$

Transportation towards site: $m_{650} \cdot 100km \cdot co_{2.track} = 9026.355 \cdot gm \; CO_2$

Return of track from the site $100km \cdot co_{2.track.empty} = 75700 \cdot gm \; CO_2$

$$co_{2.transp} := 9026.355\ gm + 75700\ gm = 84726.355 \cdot gm \quad CO_2$$

The total CO_2 emissions for the S650 CHS truss is:

$$co_{2.s650} := co_{2.steel} + co_{2.paint} + co_{2.transp} = 2702.078\, kg \; CO_2$$

CO_2 emissions for the S500 CHS truss

Steel production

$m_{500} := 2483.95kg$ - total mass of the steel

The amount of CO_2 emissions is converted to a higher strength steel with the following formula, presented in "Jan-Olof steel eco-cycle":

$$co_{2.emiss.500} := \left[0.00018\cdot(500 - 355) + \frac{1070}{1000}\right]\cdot 1000 = 1096.1$$

$$co_{2.emiss.500} := \frac{1096.1gm}{kg}$$

$$co_{2.steel} := m_{500}\cdot co_{2.emiss.500} = 2722657.595\cdot gm\,CO_2$$

Painting

Acryl paint will be used for the truss elements.

$m_{acryl} := 2.5\frac{kg}{L}$ -amount of CO_2 emission according to JouCO2&COSTi

$t_{paint} := 1.5mm$ - thickness of paint applied to the truss elements

$A_{paint} := 44.074\ m^2$ - painted area

$V_{paint} := A_{paint}\cdot t_{paint} = 66.111\,L$ -volume of paint needed

$co_{2.acryl} := m_{acryl} \cdot V_{paint} = 165277.5 \cdot gm$ CO_2

$co_{2.paint} := co_{2.acryl} = 165277.5 \cdot gm$ CO_2

Transportation

We will consider that a semi-track with the capacity of 25tonne is used for the transportation.

According to Lipasto.vtt.fi (year 2011), the environmental impact of the fully loaded track is:

$co_{2.track} := 41 \frac{gm}{tonne \cdot km}$ CO_2

The emissions of the empty track are $co_{2.track.empty} := 757 \frac{gm}{km}$ CO_2

Transportation towards site: $m_{500} \cdot 100km \cdot co_{2.track} = 10184.195 \cdot gm$ CO_2

Return of track from the site $100km \cdot co_{2.track.empty} = 75700 \cdot gm$ CO_2

$co_{2.transp} := 10184.195\ gm + 75700\ gm = 85884.195 \cdot gm$ CO_2

The total CO_2 emissions for the S500 CHS truss is:

$co_{2.s500} := co_{2.steel} + co_{2.paint} + co_{2.transp} = 2973.819\,kg$ CO_2

CO_2 emissions for the S355 CHS truss

Steel production

$m_{355} := 3282.95kg$ - total mass of the steel

$co_{2.emiss} := \frac{1070gm}{kg}$ - amount of CO_2 emission according to Ruukki EPD for tubular sections

$co_{2.steel} := m_{355} \cdot co_{2.emiss} = 3512756.5 \cdot gm \; CO_2$

Painting

Acryl paint will be used for the truss elements.

$m_{acryl} := 2.5 \frac{kg}{L}$ -amount of CO_2 emission according to JouCO2&COSTi

$t_{paint} := 1.5mm$ - thickness of paint applied to the truss elements

$A_{paint} := 46.941 \; m^2$ - painted area

$V_{paint} := A_{paint} \cdot t_{paint} = 70.412 L$ -volume of paint needed

$co_{2.acryl} := m_{acryl} \cdot V_{paint} = 176028.75 \cdot gn \, CO_2$

$co_{2.paint} := co_{2.acryl} = 176028.75 \cdot gm \; CO_2$

Transportation

We will consider that a semi-track with the capacity of 25tonne is used for the transportation.

According to Lipasto.vtt.fi (year 2011), the environmental impact of the fully loaded track is:

$$co_{2.track} := 41 \frac{gm}{tonne \cdot km} \quad CO_2$$

The emissions of the empty track are $co_{2.track.empty} := 757 \frac{gm}{km}$ CO_2

Transportation towards site: $m_{355} \cdot 100km \cdot co_{2.track} = 13460.095 \cdot gm$ CO_2

Return of track from the site $100km \cdot co_{2.track.empty} = 75700 \cdot gm$ CO_2

$$co_{2.transp} := 13460.095\ gm + 75700\ gm = 89160.095 \cdot gm \quad CO_2$$

The total CO_2 emissions for the S355 CHS truss is:

$$co_{2.s355} := co_{2.steel} + co_{2.paint} + co_{2.transp} = 3777.945\, kg \quad CO_2$$

CO_2 emissions for the S650 polygonal section truss

Steel production

$m_{650} := 1853.73kg$ - total mass of the steel

The amount of CO_2 emissions is converted to a higher strength steel with the following formula, presented in "Jan-Olof steel eco-cycle":

$$co_{2.emiss.650} := \left[0.00018\cdot(650 - 355) + \frac{710}{1000}\right]\cdot 1000 = 763.1$$

$$co_{2.emiss.650} := \frac{763.1gm}{kg}$$

$$co_{2.steel} := m_{650}\cdot co_{2.emiss.650} = 1414581.363\cdot gm\,CO_2$$

Painting

Acryl paint will be used for the truss elements.

$m_{acryl} := 2.5\frac{kg}{L}$ -amount of CO_2 emission according to JouCO2&COSTi (water)

$t_{paint} := 1.5mm$ - thickness of paint applied to the truss elements

$A_{paint} := 49.912\ m^2$ - painted area

$V_{paint} := A_{paint}\cdot t_{paint} = 74.868\,L$ -volume of paint needed

$$co_{2.acryl} := m_{acryl}\cdot V_{paint} = 187170\cdot gm\,CO_2$$

$$co_{2.paint} := co_{2.acryl} = 187170\cdot gm\,CO_2$$

Transportation

We will consider that a semi-track with the capacity of 25tonne is used for the transportation.

According to Lipasto.vtt.fi (year 2011), the environmental impact of the fully loaded track is:

$$co_{2.track} := 41 \frac{gm}{tonne \cdot km} \quad CO_2$$

The emissions of the empty track are $co_{2.track.empty} := 757 \frac{gm}{km} \; CO_2$

Transportation towards site: $m_{650} \cdot 100km \cdot co_{2.track} = 7600.293 \cdot gm \; CO_2$

Return of track from the site: $co_{2.track.empty} \cdot 100km = 75700 \cdot gm \; CO_2$

$$co_{2.transp} := 7600.293\ gm + 75700\ gm = 83300.293 \cdot gm \quad CO_2$$

The total CO_2 emissions for the S650 polygonal section truss is:

$$co_{2.s650} := co_{2.steel} + co_{2.paint} + co_{2.transp} = 1685.052\,kg \; CO_2$$

CO_2 emissions for the S500 polygonal section truss

Steel production

$m_{500} := 2112.82kg$ - total mass of the steel

The amount of CO_2 emissions is converted to a higher strength steel with the following formula, presented in "Jan-Olof steel eco-cycle":

$$co_{2.emiss.500} := \left[0.00018\cdot(500 - 355) + \frac{710}{1000}\right]\cdot 1000 = 736.1$$

$$co_{2.emiss.500} := \frac{736.1gm}{kg}$$

$$co_{2.steel} := m_{500}\cdot co_{2.emiss.500} = 1555246.802\cdot gm\,CO_2$$

Painting

Acryl paint will be used for the truss elements.

$m_{acryl} := 2.5\frac{kg}{L}$ -amount of CO_2 emission according to JouCO2&COSTi.

$t_{paint} := 1.5mm$ - thickness of paint applied to the truss elements

$A_{paint} := 55.224\ m^2$ - painted area

$V_{paint} := A_{paint}\cdot t_{paint} = 82.836L$ -volume of paint needed

$co_{2.acryl} := m_{acryl} \cdot V_{paint} = 207090 \cdot gm \quad CO_2$

$co_{2.paint} := co_{2.acryl} = 207090 \cdot gm \; CO_2$

Transportation

We will consider that a semi-track with the capacity of 25tonne is used for the transportation.

According to Lipasto.vtt.fi (year 2011), the environmental impact of the fully loaded track is:

$co_{2.track} := 41 \frac{gm}{tonne \cdot km} \quad CO_2$

The emissions of the empty track are $co_{2.track.empty} := 757 \frac{gm}{km} \; CO_2$

Transportation towards site: $m_{500} \cdot 100km \cdot co_{2.track} = 8662.562 \cdot gm \; CO_2$

Return of track from the site: $co_{2.track.empty} \cdot 100km = 75700 \cdot gm \; CO_2$

$co_{2.transp} := 8662.562 \; gm + 75700 \; gm = 84362.562 \cdot gm \; CO_2$

The total CO_2 emissions for the S500 polygonal section truss is:

$co_{2.s500} := co_{2.steel} + co_{2.paint} + co_{2.transp} = 1846.699 \, kg \; CO_2$

CO_2 emissions for the S355 polygonal truss

Steel production

$m_{355} := 2473.4kg$ - total mass of the steel

$co_{2.emiss} := \frac{710gm}{kg}$

$co_{2.steel} := m_{355} \cdot co_{2.emiss} = 1756114 \cdot gm$ CO_2

Painting

Acryl paint will be used for the truss elements.

$m_{acryl} := 2.5 \frac{kg}{L}$ -amount of CO_2 emission according to JouCO2&COSTi

$t_{paint} := 1.5mm$ - thickness of paint applied to the truss elements

$A_{paint} := 62.353\ m^2$ - painted area

$V_{paint} := A_{paint} \cdot t_{paint} = 93.529\,L$ -volume of paint needed

$co_{2.acryl} := m_{acryl} \cdot V_{paint} = 233823.75 \cdot gn\,CO_2$

$co_{2.paint} := co_{2.acryl} = 233823.75 \cdot gm$ CO_2

Transportation

We will consider that a semi-track with the capacity of 25tonne is used for the transportation.

According to Lipasto.vtt.fi (year 2011), the environmental impact of the fully loaded track is:

$$co_{2.track} := 41 \frac{gm}{tonne \cdot km} \quad CO_2$$

Transportation towards site: $m_{355} \cdot 100km \cdot co_{2.track} = 10140.94 \cdot gm \quad CO_2$

The emissions of the empty track are $co_{2.track.empty} := 757 \frac{gm}{km} \quad CO_2$

Return of track from the site $100km \cdot co_{2.track.empty} = 75700 \cdot gm \; CO_2$

$$co_{2.transp} := 10140.94\ gm + 75700\ gm = 85840.94 \cdot gm \qquad CO_2$$

The total CO_2 emissions for the S355 polygonal section truss is:

$$co_{2.s355} := co_{2.steel} + co_{2.paint} + co_{2.transp} = 2075.779\,kg \; CO_2$$