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Masterproef
Weight optimization of steel trusses for industrial buildings

Promotor :
Prof. dr. Jose GOUVEIA HENRIQUES

Promotor :
ing. PHIL MELARD

Copromotor :
Dhr. GWEN WILLEMS

Koen Peeters

Scriptie ingediend tot het behalen van de graad van master in de industriële wetenschappen: bouwkunde

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Preface

With this thesis I conclude my education Industrial Engineering at the University of Hasselt. It has been an interesting four-year period that has had a remarkable influence on me. I would like to thank my parents for giving me the opportunity to take part in this adventure, and for their constant support during these four years. Researching and writing this thesis has helped me to grow as an engineer as well as a person. It wasn't always easy, but in the end I am satisfied with the result. I would like to thank professor José Gouveia Henriques for his support and help writing this thesis, even when things did not look good for me. I would also like to thank Phil Melard for his never-ending positivity and advice, which has been a great help to persist in this thesis.

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Glossary

Chapter 2

α	angle of pitch roof
γ	self-weight of steel
μ_i	snow load shape coefficient
C_e	exposure coefficient
c_{pe}	pressure coefficient for the external pressure
c_{pi}	pressure coefficient for the internal pressure
C_t	thermal coefficient
s_k	characteristic value of snow load on the ground
w_e	external wind pressure
w_i	internal wind pressure
z_e	reference height for the external pressure

Chapter 3

α	angle between the horizontal beam and the diagonal bars
B	width of one field of the truss
H	height of the columns
H_0	minimum height of the truss
H_{max}	maximum height of the truss
L	length of the truss

Chapter 4

$\gamma_{G,Q,P}$	safety factors
γ_{M0}	partial factor for resistance of cross-sections
γ_{M1}	partial factor for resistance members to instability assessed by member checks
$\psi_{0,1,2}$	combination factors
ρ	reduction factor to determine reduces design values of the resistance to bending moments making allowance for the presence of shear forces
A_i	cross-sectional area of an element
A_w	area of web
$A_{i,min}$	the minimum cross-sectional area of an element
d	lateral displacement
G_k	characteristic value of self-weight

k_{yy} interaction factor

k_{yz} interaction factor

k_{zy} interaction factor

k_{zz} interaction factor

L_i length of an element

m deflection

ΔM_y moments due to shift of the centroidal y-y axis

ΔM_z moments due to shift of the centroidal z-z axis

$M_{y,Rk}$ characteristic value of resistance to bending moments about y-y axis

$M_{z,Rk}$ characteristic value of resistance to bending moments about z-z axis

$M_{y,Ed}$ design bending moment, y-y axis

$M_{y,Rd}$ design values of the resistance to bending moments, y-y axis

n total number of elements in the truss

N_{Ed} design normal force

$N_{t,Rd}$ design values of the resistance to tension forces

$N_{c,Rd}$ design resistance to normal forces of the of the cross-sections for uniform compression

N_{Rk} characteristic value of resistance to compression

P_k characteristic value of permanent load

Q_k characteristic value of variable load

q_p peak velocity pressure

s snow load

S length of the diagonal bars

t_w web thickness

V_{Ed} design shear force

V_{Rd} design shear resistance

W weight of the truss

$W_{pl,y}$ plastic section modulus

Chapter 5

α_{av} average angle between diagonal web members and bottom chord

α_1 angle between outer diagonal web member and bottom chord

α_2 angle between middle diagonal web member and bottom chord

Abstract
AB Associates is een studiebureau dat stabiliteitsstudies uitvoert en stalen constructies ontwerpt. Vakwerken zijn vaak een economische oplossing om daken in stalen industriehallen te ondersteunen. Een van de belangrijkste kosten bij het bouwen van een vakwerk is de materiaalkost. Aangezien deze kost voor een groot deel afhangt van het gewicht zal dit onderzoek geometrische optimalisatie combineren met een optimalisatie van de doorsnedes, om zo de meest economische oplossing trachten te vinden.

Deze thesis zal vakwerkmodellen analyseren, onderworpen aan wind- en sneeuwlasten, daklast en eigengewicht, door middel van de software Diamonds. De optimalisaties zullen uitgevoerd worden op portieken met lengtes gaande van 25m tot 35m, en voor elke lengte zal de hoogte van het vakwerk tussen 1.25m en 2m variëren. Voor elke situatie zal de kleinst mogelijke sectie gezocht worden voor elk element.

Uit de optimalisatie blijkt dat wanneer enkel het vakwerk geoptimaliseerd wordt, er een omgekeerd evenredig verband is tussen de hoogte en het gewicht van het vakwerk. Wanneer er echter ook rekening gehouden wordt met de secties van de kolommen van de structuur zal de optimale hoogte niet meer altijd de hoogste zijn. Een analyse van de faalmodi leert dat de kritische elementen in UGT (knik of trek) zullen falen voor de randvoorwaarden in GGT (doorbuiging en verplaatsing) bereikt worden.

Abstract in English

AB Associates is a study bureau that performs stability studies and designs steel structures. Trusses are often an economical solution to support the roof of a steel industrial building. One of the main costs in the construction of a truss structure is the material cost of the steel. Since this cost depends largely on the weight of the elements, this thesis will combine both shape and size optimization to minimize the total weight, thus attempting to obtain the most economical solution.

This thesis will analyze truss models subject to wind load, snow load, roof load and self-weight, by means of computer aided structural software. The optimizations will be performed for spans with lengths varying from 25m up to 35m. For each length, the height of the truss will vary between 1.25m and 2m. for each situation, the smallest possible cross-section will be selected for each element.

The optimization results show that when only the truss itself is optimized, the weight is inversely proportional to the height. However, when the columns of the structure are optimized as well, the optimal height of the truss will not always be the largest anymore. An analysis of the failure modes shows that the critical elements will fail in ULS (buckling or tension), before the SLS constraints (deflection and displacement) are reached.

1 Introduction

"The perfect stability study saves money and unnecessary costs for both the client and the contractor". This is the key philosophy from which the engineering bureau AB Associates handles its assignments. Unnecessary costs can be avoided by the optimal usage of time and construction materials. This thesis will focus on reducing the material costs.

Industrial buildings often require spans of several tens of meters without columns inside. The use of steel truss structures is a wide-spread solution to this problem. In the construction of a steel building, the material cost saving is based in the optimization of the profiles and geometry of the structure. In an attempt to find the most economical solution for a steel truss structure, an optimization of the profiles in the truss seems like the most effective method. Since an optimization based on experimental testing is not realistic this investigation is based on numerical simulations.

The objective of this thesis is the weight optimization of steel trusses, used in industrial portal steel frames, by means of computer aided structural design software.

In chapter two there will be an introduction to trusses in general and the type of truss used in this paper, an overview of the Eurocodes that are applied and an introduction to optimization problems in general. The third chapter contains the case study, where the geometrical properties and the loads will be specified. Chapter four will explain the optimization problem, the constraints that have to be considered and the procedure of an optimization illustrated by an example. Chapter five explains the path that was followed to achieve the final results, and the different reductions made in order to minimize repetitive work. In the first part of the sixth chapter, the results will be presented for the truss optimization, both graphically and in the form of a section table. The second part of this chapter includes an optimization of the columns, and an analysis of the influence of this column optimization.. Finally, in chapter seven, an overview of the results and a conclusion is presented.

2 Literature

2.1 Truss structures

In structural frameworks, loads from roofs and upper floors are transmitted to the ground through columns. Structural members, therefore, must be provided to carry loads from the roofs and floors to the columns. When the column spacing is large, trusses often are an economical choice for those structural members. For economy, however, trusses usually have to be deep and consequently they can be used only if there will be sufficient space for them and adequate headroom under them. Because of the space requirements, the principal application of trusses in buildings is for supporting roof [1].

A truss is a stable configuration of interconnected tension and compression members. The connections between the members are assumed in truss design to be pinned, free to rotate, although actually the types of connections used may apply some restraint against rotation of truss joints [1].

The deeper a truss is made for a given span and loading the smaller will be the chord members, but that with deeper trusses the lengths of the web members increase. This fact means that the slenderness ratios of the web members may become a factor and require the use of heavier members [2].

Trusses can be flat or peaked. On the past the peaked roof trusses have probably been used more for short-span buildings and the flatter trusses for the longer spans. The trend today for spans long or short, however, seems to be away from the peaked trusses and towards the flatter ones, the change being due to the appearance desired and perhaps more economical construction of roof decks.

There are many different types of truss structures. The Pratt truss (Figure 2-1) has probably been used more for the flatter roofs (slopes of from $\frac{3}{4}$ to 1.5 in./ft or 6% to 10%) where built-up roofing can be satisfactory applied, than have the other types of trusses. These trusses can be economically used for flat roofs for spans of roughly 40 to 125 ft (12 to 38 m), although they have been used for spans as great as 200 ft (45 m). The roofs may be completely flat for spans not exceeding 30 or 40 ft (9 or 12 m), but for longer spans the slopes mentioned are used for drainage purposes [2].

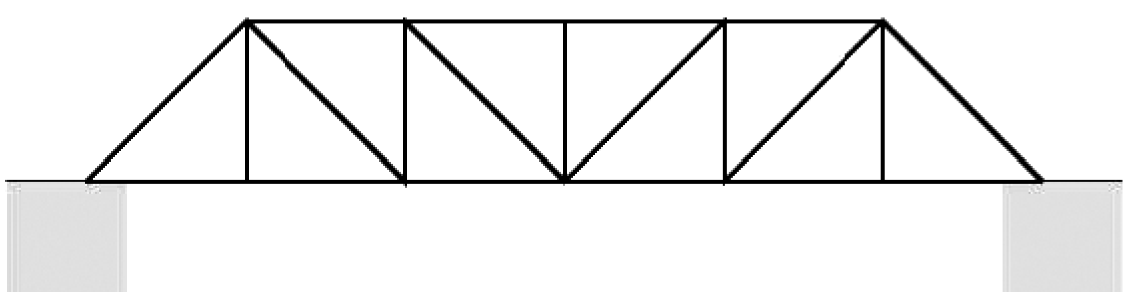


Figure 2-1: Pratt Truss [3]

2.2 Eurocodes

The design rules for calculating a steel structure according to the Eurocodes are given in EN 1990, EN 1991 and EN 1993. EN 1990 establishes principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability ([4]). EN 1991 (Eurocode 1) provides comprehensive information on all actions that should normally be considered in the design of buildings and other civil engineering works, including some geotechnical aspects.

It is in four main parts, the first part being divided into sub-parts that cover densities, self-weight and imposed loads; actions due to fire; snow; wind; thermal actions; loads during execution and accidental actions [5]. EN 1993 Eurocode 3 applies to the design of buildings and other civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural design. EN Eurocode 3 is concerned with requirements for resistance, serviceability, durability and fire resistance of steel structures [6].

Although all parts are relevant for the present research, special care is given for the quantification of the snow and wind loads. Therefore, in the following sections this two actions are further discussed.

2.2.1 Snow actions

The snow load on a roof is determined as follows:

$s = \mu_i \, C_e \, C_t \, s_k$ [7]

Where:

μ_i is the snow load shape coefficient

C_e is the exposure coefficient

C_t is the thermal coefficient

s_k is the characteristic value of snow load on the ground

The snow load coefficient μ_i is determined as in Table 2-1 [7]. The characteristic value s_k in Belgium is determined as $0.5 \frac{kN}{m^2}$ for buildings at a topographical height under 100m. The coefficients C_e and C_t both have a value of 1 in Belgium [8].

Table 2-1: Angle of pitch of roofs [7]

Angle of pitch of roof α	0° < α < 30°	30° < α < 60°	α > 60°
μ_1	0,8	$0{,}8(60-\alpha)/30$	0
μ_2	$0{,}8 + 0{,}8\,\alpha/30$	1,6	/

2.2.2 Wind actions

The wind pressure acting on the external surfaces, w_e, should be obtained as follows:

$w_e = q_p(z_e) \cdot c_{pe}$ [9]

Where:

$q_p(z_e)$	is the peak velocity pressure
z_e	is the reference height for the external pressure
c_{pe}	is the pressure coefficient for the external pressure

The wind pressure acting on the internal surfaces, w_i, should be determined in an analog way.

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their sings. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative. Examples are given in Figure 2-2 [9].

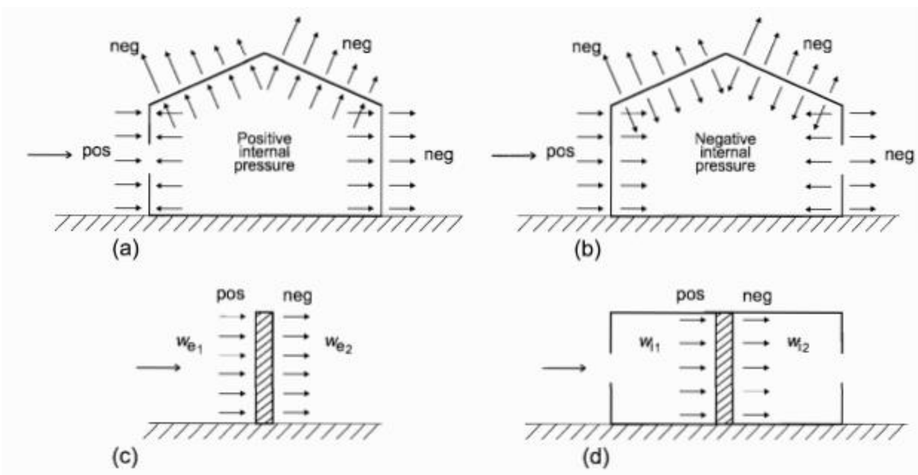


Figure 2-2: External and internal pressures in buildings [9]

The zones for the external pressure coefficients $c_{pe,10}$ and $c_{pe,1}$ for walls and roofs are defined in Figure 2-3. The coefficients can be derived from the tables in *NBN EN 1991-1-4,* the internal pressure coefficients can be obtained in the same way [9].

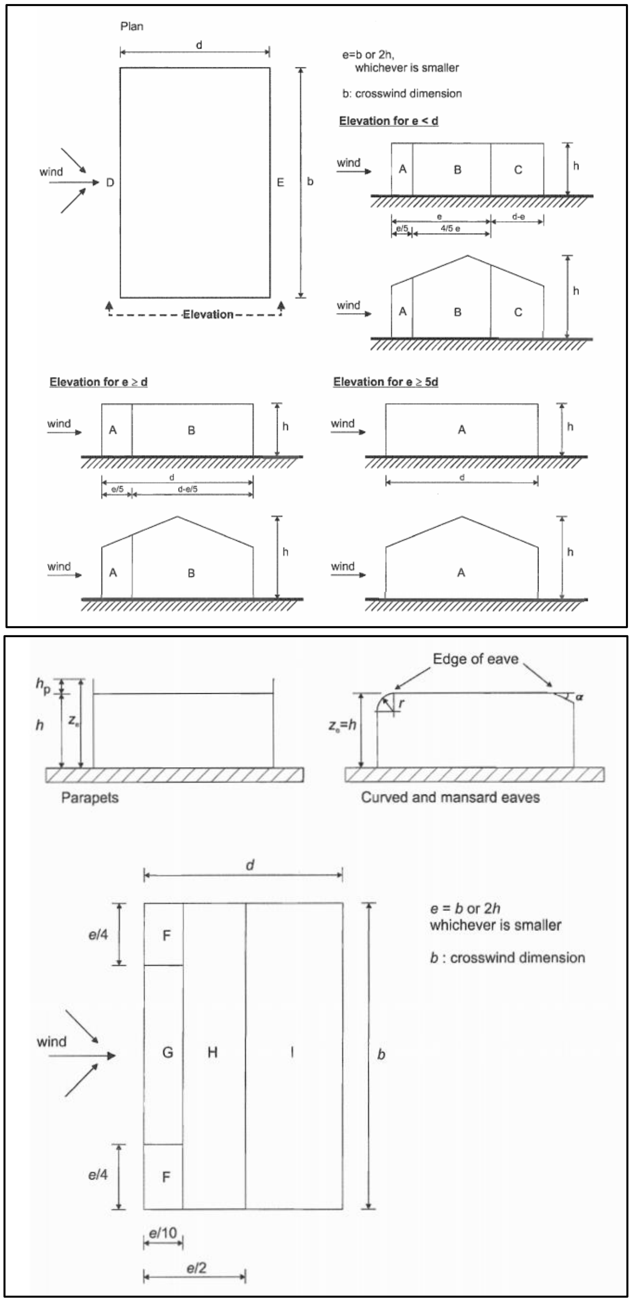


Figure 2-3: Pressure coefficient zones for walls and roofs [9]

The peak velocity pressure can be determined based on the wind velocity. In Belgium, there are four main zones with different wind velocities as shown in Figure 2-4. Once this value is obtained, the peak velocity pressure can be derived from the tables in *NBN EN 1991-1-4 ANB* [10].

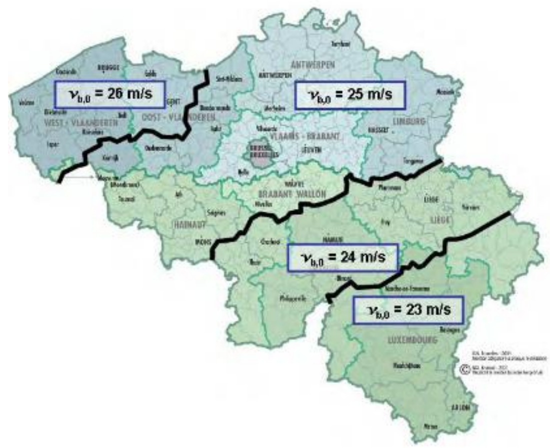


Figure 2-4: Wind velocity zones in Belgium [11]

2.3 Optimization

The aim of any structural optimization problem can be defined in the following way: To determine the best design for a given problem subject to certain restrictions. The design variables can be defined starting from the geometry, the topology or material properties of the structures. A set of derived parameters related to mechanical behavior can also be obtained: strains, stresses, deflections, natural frequencies and loads... The cost (or objective) function is given by the proper choice of the design criterion. This function can be either minimized or maximized [12].

The mathematical representation of optimization problems is the minimization of maximization of a scalar-valued objective function with respect to a vector of design parameters [12] Generally there are three main categories in truss optimization; namely size, shape and topology optimization [13].

The first category is the size optimization. The aim is usually to minimize the weight of the structure subject to certain behavioral constraints on stress and displacements. None modifications of the geometric model are allowed. Design variables influence neither the topology nor the geometry of the structure which are considered to be fixed. It concerns a modification of cross-sections of thickness of the structural elements [12].

The second category is the shape optimization which investigates the geometry of the truss. It concerns the changes of the geometrical dimensions of the initial design to satisfy a large variety of objective functions (stress, weight, displacements …) [12]. In this category of truss optimization, the nodal coordinates of a given truss are taken as design variables and the cost or weight of truss is the goal function [13].

Topology optimization allows the control of not only geometry but also of topology of a structure without any restriction on the number or the nature of the structural members. It involves the determination of the type of structural members (location, number, shape of holes) and their connectivity within the design space available when the loads and the boundary conditions are supposed known [12].

3 Case study

3.1 Portal frame configuration

This paper will study portals with a structure as is illustrated in Figure 3-1. The parameters that define the structure are:

H the height of the columns

H_0 the minimum height of the truss

H_{max} the maximum height of the truss

L the length of the truss

B the width of one field of the truss

S the length of the diagonal bars

α the angle between the horizontal beam and the diagonal bars

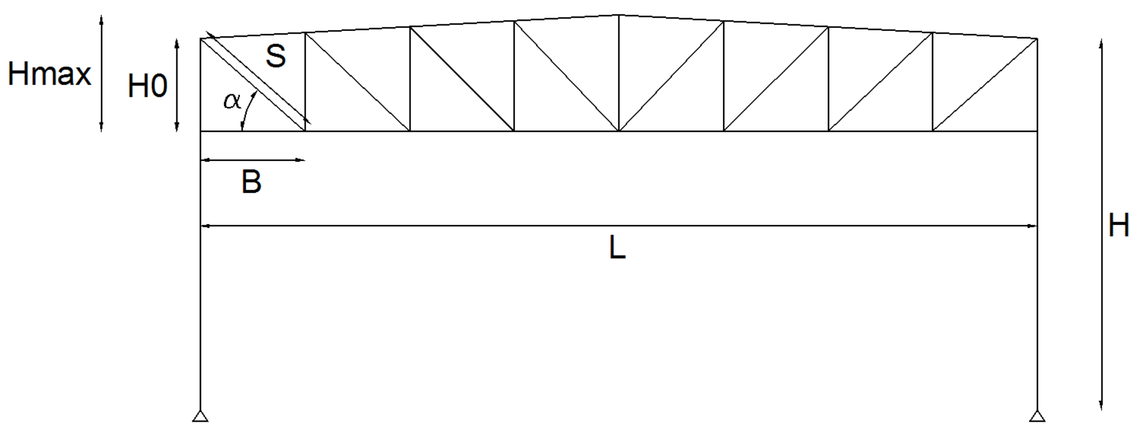
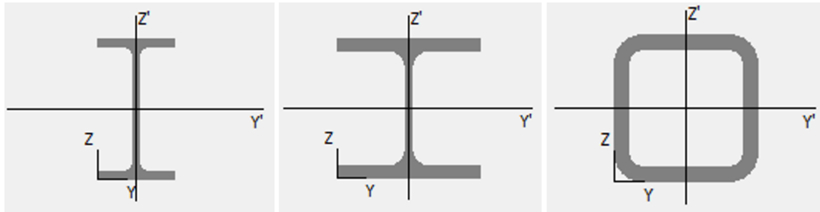


Figure 3-1: Portal configuration

Both the truss and the columns will consist of steel members. The columns will be steel profiles of the type IPE, the top and lower chords of the truss will be HEA profiles and the web members will be square tubes of the type SHS. The sections of these three types are depicted in Figure 3-2. The spacing in between two portals is assumed to be 6m.



(a) (b) (c)

Figure 3-2: Cross-sections of elements with: (a) IPE, (b) HEA and (c) SHS

3.2 Loads

3.2.1 Self-weight

The self-weight of steel is $7850\,\frac{kg}{m^3}$. The weight of an element is calculated by multiplying the area of the section with the length of the element.

3.2.2 Roof load

The roof load consists of four different elements:

- PVC roofing: $5\,\frac{kg}{m^2}$
- PIR insulation: : $5\,\frac{kg}{m^2}$
- Steel deck: $12\,\frac{kg}{m^2}$
- Techniques: $3\,\frac{kg}{m^2}$

The total weight of the roof load is therefore $25\,\frac{kg}{m^2}$, or $0.25\,\frac{kN}{m^2}$. This is taken into account as a permanent load. The spacing between the spans is $6\ m$, therefore the roof load carried by one span is $1.5\,\frac{kN}{m}$.

The service load of a roof as given in the Eurocode is neglected, since the roof has no access points.

3.2.3 Snow load

In order to calculate the snow load, the height of the location of the structure has to be determined. Since the majority of the projects of *AB Associates* is located in Flanders, the height will be assumed to be under 100 meters. The snow load will be calculated as prescribed in *NBN EN 1991-1-3*, therefore it will be $0.4\,\frac{kN}{m^2}$.

3.2.4 Wind load

To calculate the wind load, three parameters have to be determined: the wind velocity, the terrain category and the height of the building. Since the majority of the projects of *AB Associates* is located in the area with the velocity set at $25\,\frac{m}{s}$ according to *NBN EN 1991-1-4 ANB* [11], this value will be used in the calculations. Since the considered buildings are industrial buildings, they will be mostly located on industrial zones. This means that the terrain category can be assumed as III. When these parameters are combined with the height of the structure, the wind pressure can be derived from table 4.8 in *NBN EN 1991-1-4 ANB* [11].

3.2.5 Summary

A summary of the loads considered in this study is given in Table 3-1 and depicted in Figure 3-3.

Table 3-1: Load Summary

Self-weight	7850 kg/m²
Roof load	0,25 kN/m²
Wind load	25 m/s
Snow load	0,4 kN/m²

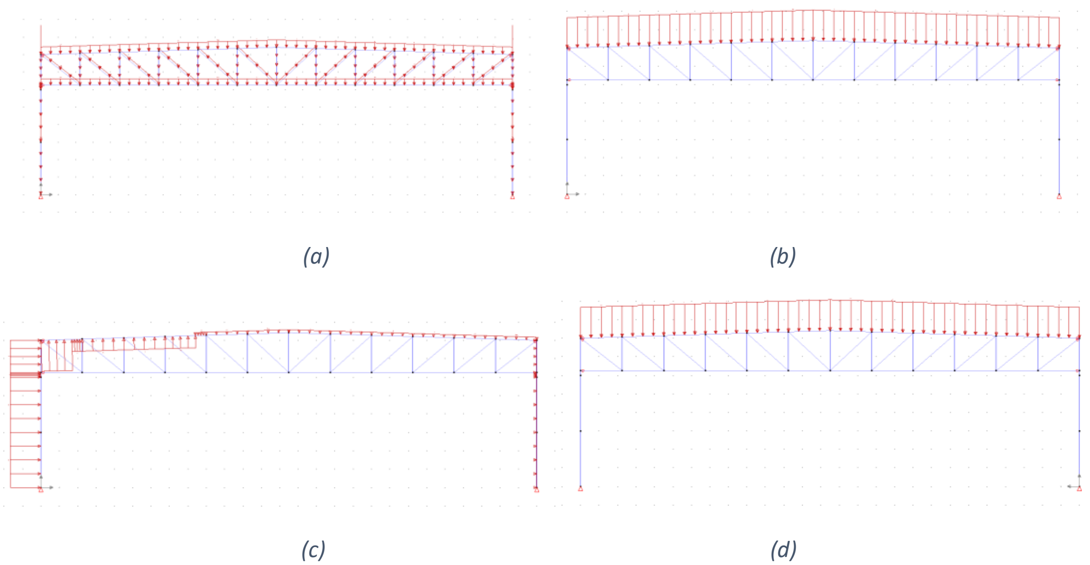


Figure 3-3: Load summary with: (a) Self-weight, (b) Roof load, (c) Wind load, (d) Snow load

4 Optimization problem

4.1 Objective function

One of the decisive factors in the calculation of the total cost of a steel structure is the total weight W of the elements. The weight of an element is proportional to the area A of its section. The optimization problem can be expressed as:

Minimize $W(A_i) =$ Minimize $\sum_{i=1}^{n} A_i L_i \gamma$

By reducing $A_i = A_{i,min}$

Where:

W the weight of the truss

n the total number of elements in the truss

A_i the cross-sectional area of an element

$A_{i,min}$ the minimum cross-sectional area of an element

L_i the length of an element

γ the self-weight of steel

Since this paper focuses the optimization of trusses, only the weight of the truss elements will be analyzed, the column weight will not be considered. The cross-sectional areas of the columns will be assumed invariable for each case, even though the section of the columns will have an influence on the behavior of the truss. Therefore, an analysis of the influence of different column cross-sections will be made in Chapter 0.

4.2 Constraints

4.2.1 Load combinations

To check the resistance and stability of the structure, the load combinations will be calculated in ULS-F (fundamental), SLS-R (rare) and SLS-Q (quasi-permanent). The safety factors for both limit states are given in Table 4-1 and the combination factors ($\psi_{0,i}$) for the variable loads are given in Table 4-2. The load combinations for the two limit states are:

$\sum \gamma_G G_k + \gamma_p P_k + \gamma_Q Q_k + \sum \gamma_Q \psi_0 Q_k$ ULS-F [14]

$\sum G_k + P_k + \psi_1 Q_k + \sum \psi_1 Q_k$ SLS-R [14]

$\sum G_k + P_k + \sum \psi_2 Q_k$ SLS-Q [14]

Table 4-1: Partial safety factors [15]

	ULS-F	SLS
γ_G	1,35	1
γ_Q	1,5	1
γ_p	1	1

Table 4-2: ψ factors [15]

	Self-weight	Roof load	Wind	Snow
ψ_0	1	1	0,6	0,5
ψ_1	1	1	0,2	0
ψ_2	1	1	0	0

4.2.2 Ultimate Limit State

In order to check the resistance of the cross-sections of both the truss and the columns in ULS, the following constraints are adopted from *EN 1993-1-1* [16]*:*

Tension

$$N_{Ed} \leq N_{t,Rd}$$

Compression

$$N_{Ed} \leq N_{c,Rd}$$

Bending

$$M_{y',Ed} \leq M_{y',Rd}$$

Shear

$$V_{z',Ed} \leq V_{z',Rd}$$

Interaction bending + shear

$$M_{y,V,Rd} \leq \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w}\right] f_y}{\gamma_{M0}}$$

Interaction biaxial bending + axial force

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1 \qquad \text{(class 1 \& 2 section)}$$

Interaction biaxial bending + shear + axial force

If $V_{Ed} \leq 0.5\, V_{pl,Rd}$ no interaction check is needed.

In order to check the stability of the columns and the truss elements in ULS, the following constraints are adopted from *EN 1993-1-1:*

Buckling

$$N_{y,Ed} \leq N_{y',Rd}$$

$$N_{z,Ed} \leq N_{z',Rd}$$

Buckling (M + N)

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

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y;Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z;Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

4.2.3 Serviceability Limit State

To avoid damage on non-structural elements and for comfort of users, the lateral displacement and the deflection of the truss have to be limited. The limit values are checked as prescribed in *NBN B 03-003* [17]*:*

Lateral displacement

Maximum displacement, taking in account the appearance (SLS-F).

$d \leq \frac{H}{250}$

A further restriction is imposed by the study bureau AB Associates. A rule of thumb that is used is a maximum displacement of 3 cm, for aesthetic reasons.

Deflection

Resistance of stiff roofing (SLS-Q).

$m \leq \frac{L}{250}$

4.2.4 Sections

The sections of the tubes in the truss are from the type SHS. There is a great variety within this type of truss, as different widths and thicknesses are possible. The catalog implemented in the *Diamonds* software can model every existing section in the EU from a width of $10\ mm$ and a thickness of $1\ mm$ up to a width of $250\ mm$ and a thickness of $10\ mm$. However, some sections are more commonly produced in Belgium, which means that these are more economical to purchase . The three types of SHS sections that are most common, and therefore used in this research, are given in Table 4-3 and a cross-section is depicted in Figure 4-1.

Table 4-3: Common SHS cross-sections

	60/4	**80/4**	**100/5**
B, H	60 mm	80 mm	100 mm
t1, t2	4 mm	4 mm	5 mm
r1	4 mm	4 mm	5 mm
r2	8 mm	8 mm	10 mm

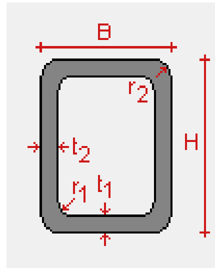


Figure 4-1: SHS cross-section

Where:

B, H are the width and height of the profile

$t1, t2$ are the thicknesses of the tube

$r1, r2$ are the inner and outer radius of the tube

4.2.5 Variables

In order to optimize a truss, adjustments can be made in both the cross-sections and the geometry of the structure. To optimize the geometry, certain parameters have to be assumed as variable (see Figure 3-1).

The goal of this paper is to optimize the weight for a number of lengths of the span, this means that L is a variable. This paper will focus on a length L between $25\ m$ and $35\ m$, considering an increment of $1\ m$. The next parameter that will be considered a variable is the minimum height of the truss H_0. This height will vary between $1\ m$ and $2\ m$ with increments of $0.25\ m$. To reduce the amount of calculations, the height of the columns H is fixed at $8\ m$. The roof has a slope of 3% to ensure that there can be no water stagnation, which would lead to an excessive loading on the roof. The other parameter (B, S, α and H_{max}) will vary in function of the variables determined above. For the parameter α there is a restriction: the angle between the diagonal bars and the horizontal beams cannot be less than $30°$ or more than $60°$. This restriction is purely practical, as it is difficult to weld the connections if the angle falls outside this limits.

One other variable is the number of fields in the truss. This variable depends on the angle of the diagonals. A number of fields can be realized by changing α, but because the fields have to be equally wide (B) and symmetric, there can only be an even number of fields. To reduce the amount of calculations, the number of fields is limited to 5 different cases: $12, 14, 16, 18$ and 20. This also means that the number of angles is limited for each L and H_0.

Table 4-4 shows an overview of all the different cases. For each length L and height H_0, the number of possible angles (and fields) is given, with the restrictions for the angles in mind.

Table 4-4: Number of possible angles for each length L and height H_0

Number of angles for each case		Length L (m)										
		25	26	27	28	29	30	31	32	33	34	35
Height H_0 (m)	1,00	2	3	3	3	2	2	2	1	1	1	1
	1,25	5	5	4	4	4	4	3	3	3	3	3
	1,50	5	5	5	5	5	5	5	4	4	4	4
	1,75	5	5	5	5	5	5	5	5	5	5	5
	2,00	4	4	5	5	5	5	5	5	5	5	5

When all the numbers of angles in Table 4-4 are added, there are 221 different cases to be calculated.

To design a structure, there are several other constraints that have not yet been mentioned but that have to be taken into account as well. The first one is the support conditions of the columns. In this investigation the supports of the columns will prevent movement in all directions, but they will allow rotation. This support condition is possible because the lateral stability of the structure is ensured by the truss.

As stated in *Structural steel design* by Jack McCormac [2], the connections in a truss are pinned and free to rotate, although some restraint against rotation can be applied by the type of connection. Since the connections in the structures that are investigated in this paper are welded, this kind of restraint against rotation is present in the connections. Therefore, the connections in the truss will be assumed to be pinned and to restrict rotation. The connection between the truss and the column, however, will be assumed to be free to rotate to prevent the transfer of bending moment from the truss members to the columns.

The final constraint is the buckling length of the members. Buckling can occur in two directions of the member. The bucking length of the chords of the truss around the y-axis is limited by the web members that act as buckling stiffeners. In the z-direction, however, these buckling stabilizers are not provided by the truss and have to be added in order to reduce the buckling length of the element. These out-of-plane supports will be simulated in the software and will be provided every 5 m. Similar out-of-plane supports are added in the columns every 3 m.

4.3 Optimization procedure

4.3.1 Model

To illustrate the procedure of the optimization, one of the 221 cases will be explained in detail. This example case will be the structure with a span of 25 m, a truss height H_0 of 2 m and 12 fields (which results in an average angle α of $46°$.

First, an assumption of the sections is made to design the structure. For example: an HEA 160 for the chords and a SHS 80/4 for the web members. As already mentioned in 4.1, the columns will initially be considered invariable for each case. To ensure the stability of the heaviest case, an IPE 450 will be used for he optimization. This configuration will then be drawn in the Diamonds software, and the constraints in the previous paragraph are applied.

4.3.2 Loads

When the structure is completed, the loads summed up in 4.3 will be applied. First, the different load groups have to be defined. Self-weight, wind and snow loads are automatically generated, the permanent load of the roof has to be manually converted to a distributed load on the span. Then, the load combinations can be automatically generated. The combinations are made for ULS-F, SLS-Q and SLS-F.

4.3.3 Displacement and deflection

In order to find the deflection and the lateral displacement of the structure, a first order elastic analysis is generated by the software. The results then are generated as is depicted in Figure 4-2 and . Figure 4-2 represents the vertical deflection in SLS-Q for a structure with a 25m span and a height H_0 of 1m, and Figure 4-3 represents the lateral displacement of this case.

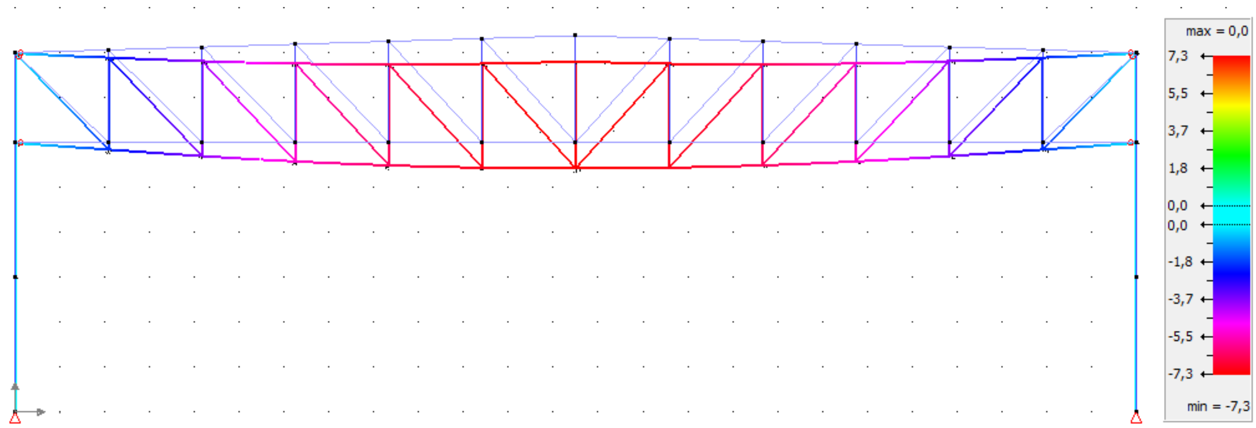


Figure 4-2: Vertical deflection in SLS-Q

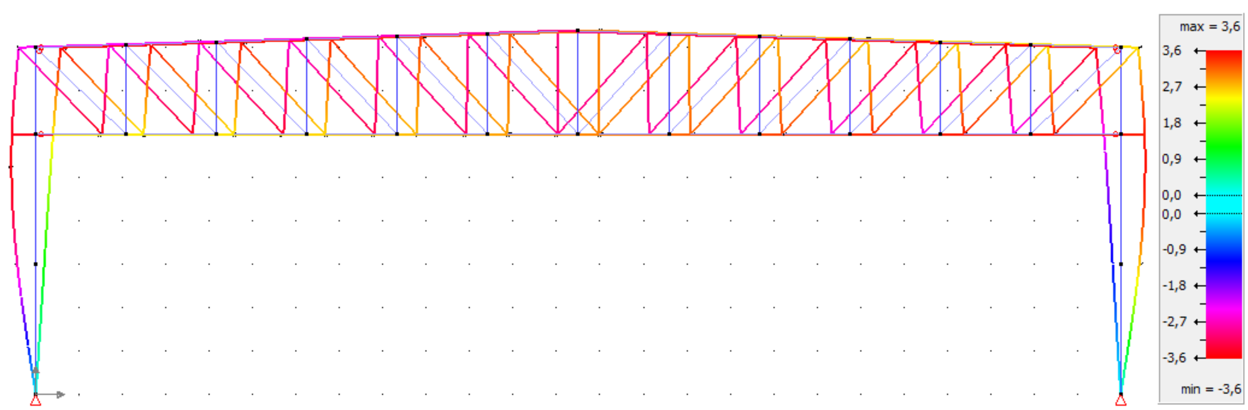


Figure 4-3: Lateral displacement in SLS-F

For this particular case, the conditions for displacement and deflection (see 4.2.3) are as follows:

Deflection (SLS-Q)

$m \leq \frac{L}{250}$ [17]

$m \leq 100\ mm$

$m = 7.3\ mm \leq 100 mm$ => OK

Displacement (SLS-F)

$d \leq \frac{H}{250}$ [17] and $d \leq 30\ mm$

$d \leq 32\ mm$ and $d \leq 30\ mm$

$d = 3.6\ mm \leq 30\ mm$ => OK

4.3.4 Cross-sectional resistance

When both conditions are met, a normative check is executed on the structure. In this check, the results of the elastic analysis are compared with the results of a design calculation according to *EN 1993-1-1.* After the check, the acting internal forces are given as a percentage of the designed forces. In the structure, four elements have to be checked: the upper and lower chords and the vertical and diagonal web members. The results are divided into two parts: resistance of the cross-section and buckling resistance.

Each member has its critical area for both the cross-sectional resistance as the buckling resistance. The critical area for the cross-sectional resistance is in the middle for both the upper and the lower chord, and for the web members the critical members are located on both the ends of the truss. For the buckling resistance the critical areas are similar as the ones mentioned for the cross-section, however, the critical area for the lower chord is now on the end of the truss.

Figure 4-4 shows the results for the cross-sectional resistance graphically, Figure 4-5 shows the results of the calculation of one of the critical members of the upper chord, circled in Figure 4-4. The details of the calculation are presented in Annex B.

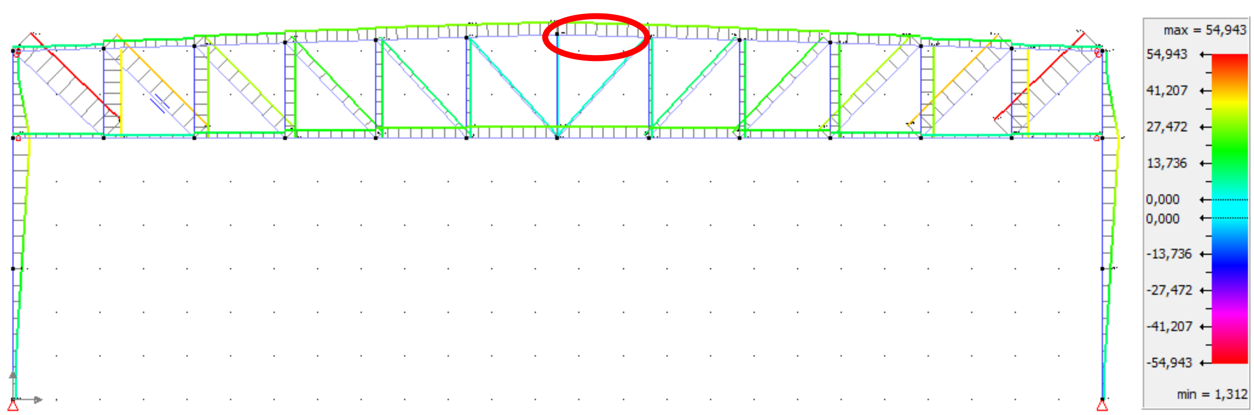


Figure 4-4: Cross-sectional resistance

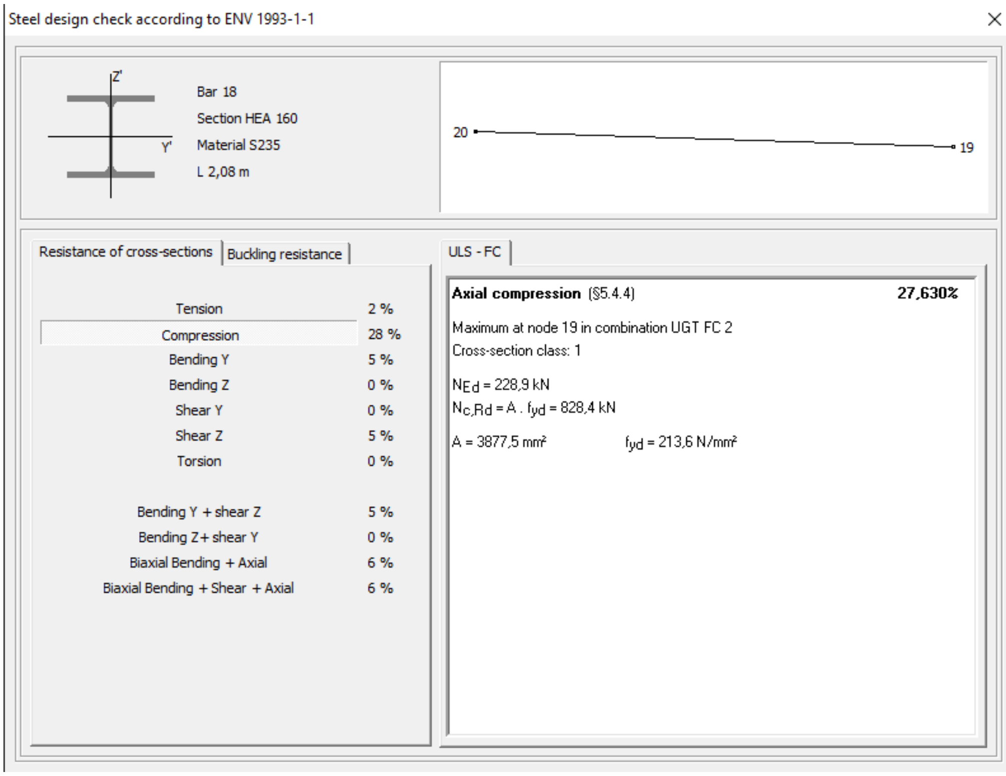


Figure 4-5: Steel design check, resistance of cross-section

4.3.5 Buckling resistance

In Figure 4-6 the results for the buckling resistance are depicted graphically and in Figure 4-7 the result of the calculation for the outer vertical web member circled in Figure 4-6 is depicted.

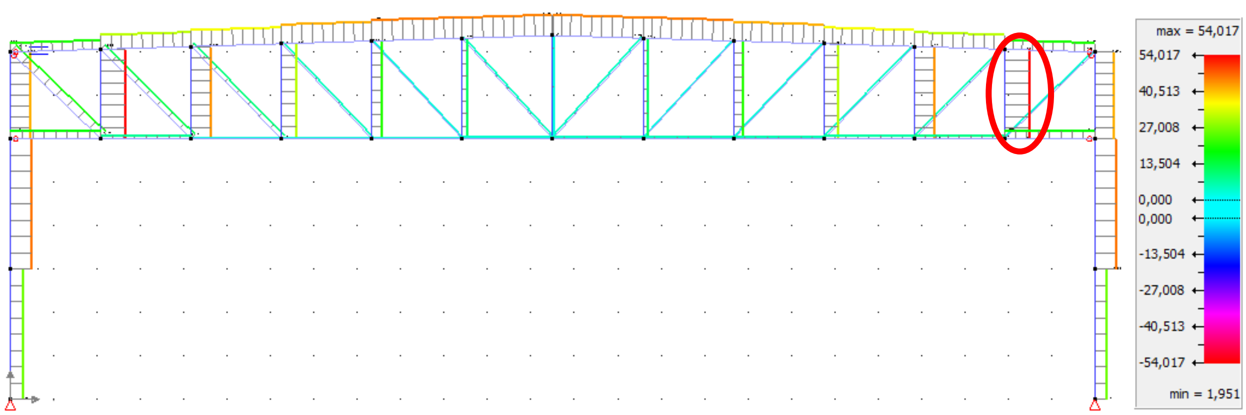


Figure 4-6: Buckling resistance

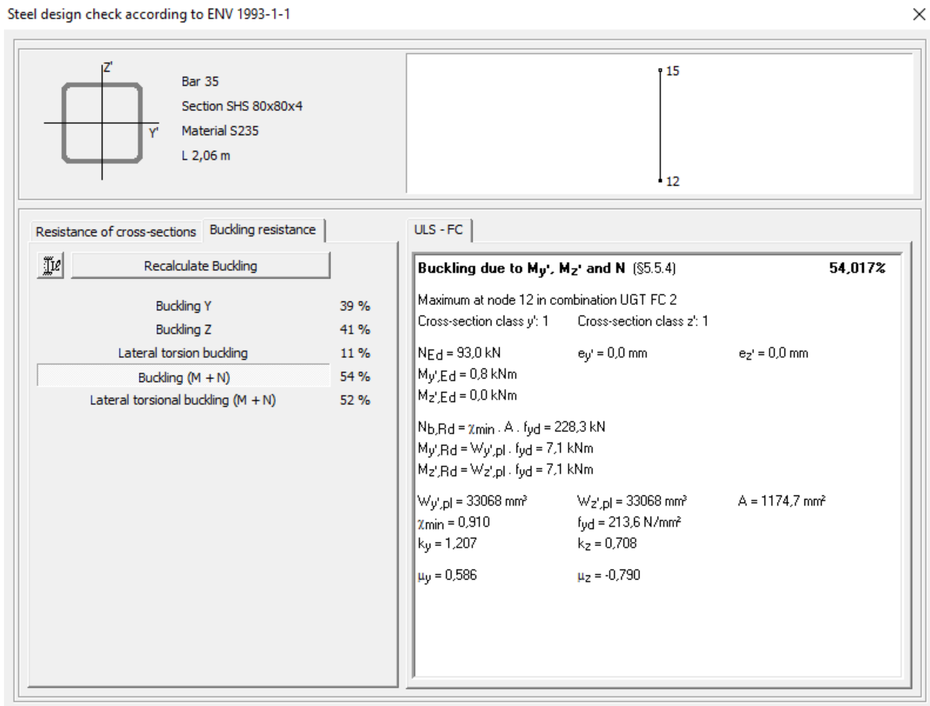


Figure 4-7: Steel design check, buckling resistance

4.3.6 Optimization process

The optimization process performed consisted in an iterative procedure which is here described:

- If the resistance of any member is exceeded, the cross-section has to be increased. However, if the acting forces do not exceed the resistance the cross-section reduction is attempted.
- When these adaptations have been completed, the calculations have to be restarted, and the procedure has to start over from the calculation part.
- When the resistance of each element reaches its limit (higher than the acting forces, but as low as possible), the structure is at its lightest point. The truss is then optimized.

In the example case, no member of the truss has exceeded the resistance values. This means that the cross-section of each element can be reduced, and the process can restart. After several iterations, the new sections are: HEA 120 for the upper chord, HEA 100 for the bottom chord and SLS 60/4 for the web members. The deflection and displacement are now respectively 9.5 mm and 3.9 mm, which is still within the permitted boundaries. The cross-sectional resistance and buckling resistance are still not exceeded in any member. However, the acting forces begin to approach the resistance values, as is clear in Figure 4-8. The axial force in the upper chord is at 80.5% of its buckling resistance. When the section is reduced this becomes 119%, which means it will fail. The bottom chord and the web members cannot be reduced further, as they have reached the minimum possible cross-sections.

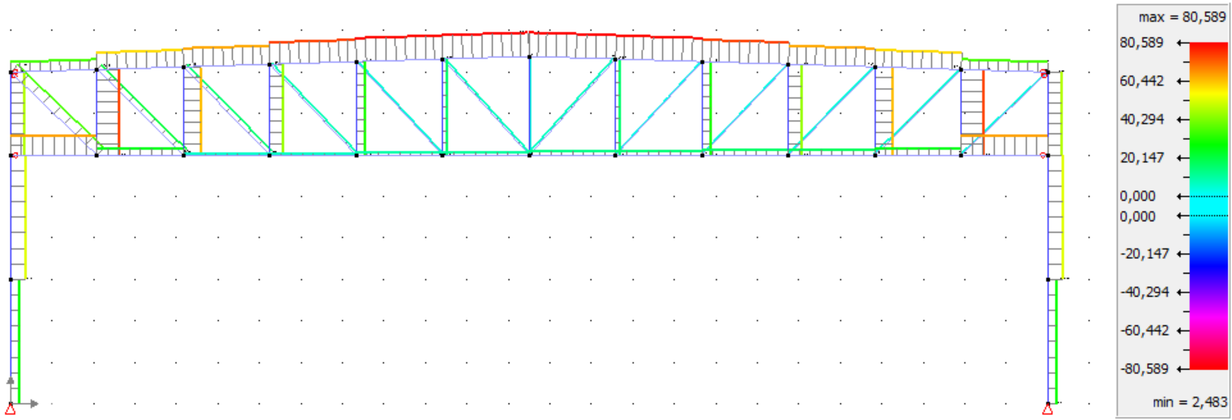


Figure 4-8: Buckling resistance after optimization

5 Methodology

Based on the geometry presented in 4.2.5 , 221 different cases were analyzed. After consideration, the decision was made to reduce the number of cases, as this would prevent a lot of unnecessary and repetitive work. In an attempt to find a relationship between the geometric values of the different cases, a couple of reductions were made in order to find a pattern in the variables of the structures.

The first reduction has already been mentioned in paragraph 4.2.5: a limitation in the number of fields. The decisive factor in the maximum number of fields was the maximum that the Diamonds software could automatically generate, namely 20. The minimum was set at 12, because the values of the angle α dropped under the minimum value of 30°.

The second measure was to check three different lengths of the truss, to get a global view of the behavior of the trusses. These three values were the two limit values 25m and 35m, and the middle value of 30m (Table 5-1). These cases were selected in order to achieve an general view, since the other cases were all situated in between these values, without having to calculate each case seperately. These other cases will also be calculated after some sort of pattern has been found. However, this reduction still left as many as 60 cases to be investigated, which was still a large amount considering this had to be repeated for the other 8 lengths.

Table 5-1: Reduction of cases

Number of angles for each case		Length L (m)										
		25	26	27	28	29	30	31	32	33	34	35
Height H_0 (m)	1,00	2	3	3	3	2	2	2	1	1	1	1
	1,25	5	5	4	4	4	4	3	3	3	3	3
	1,50	5	5	5	5	5	5	5	4	4	4	4
	1,75	5	5	5	5	5	5	5	5	5	5	5
	2,00	4	4	5	5	5	5	5	5	5	5	5

The third manner to reduce the number of optimizations was to isolate one of the heights. The maximum height of 2 meters was chosen to conduct this preliminary research on. For the three lengths (25m, 30m and 34m), 5 trusses were modeled with a height H_0 of 2m and fields varying between 12 and 20, with steps of two. These models were optimized following the procedure in 4.3, and the results of these first optimizations are given in Table 5-2.

Table 5-2: Weight for each length and number of fields

Weight (kg)		Length L (m)		
		25	30	35
Fields	12	1326	1682	2590
	14	1374	1587	2478
	16	1429	1887	2436
	18	1483	1934	2802
	20	/	2095	2695

An interesting approach to obtain a relationship between geometric properties is to analyze the angle α between the diagonal web members and the chords. Since the height of the truss increases gradually towards the middle, α will increase as well. That is why the concept of the average angle α_{av} is introduced as the average of the angle α_1 at the beginning of the truss, and the angle α_2 in the middle of the truss (Figure 5-1).

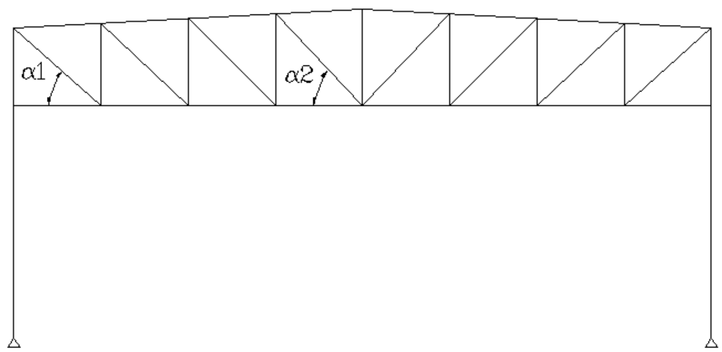


Figure 5-1: α_1 and α_2

Table 5-3 shows the average angle for the three lengths and each number of fields.

Table 5-3: Angle α_{av}

α_{av} (°)		Length L (m)		
		25	30	35
Fields	12	46	41	37
	14	50	46	42
	16	54	49	45
	18	57	53	49
	20	/	56	52

The results of Table 5-2 and Table 5-3 can now be combined and visualized in a graph. Figure 5-2 shows the result of the first optimization, where the weight of the truss is expressed as a function of the angle α_{av} for every one of the three lengths.

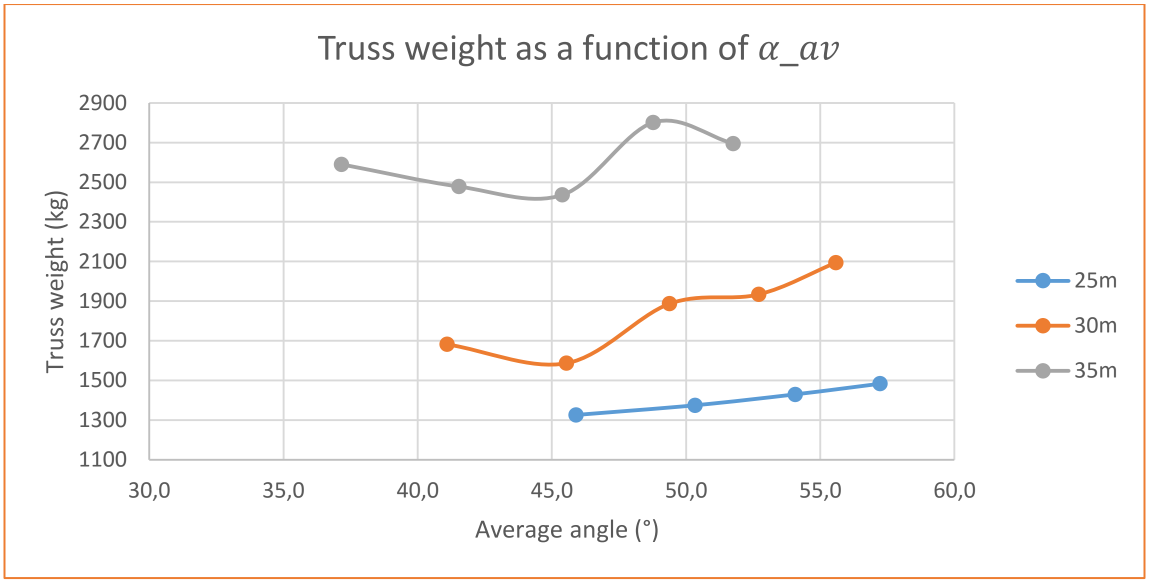


Figure 5-2: Truss weight as a function of α_{av}

The graph clearly shows that the optimal weight for each length coincides with an angle of approximately 45°, which is what one would expect. Therefore, the assumption can be made that this is the optimal angle for each length in between these three. This helps to reduce the total number of cases to be optimized, since only the configuration with an α_{av} closest to 45° will be taken into consideration. In Table 5-4 every average angle for the number of fields is presented, with the optimums marked. For these marked cases, the optimization will now be executed with H_0 as the only remaining geometric variable.

Table 5-4: Angle between diagonal web members and bottom chord

α		Length L										
		25	26	27	28	29	30	31	32	33	34	35
Fields	12	46	45	44	43	42	41	40	39	39	38	37
	14	50	49	48	47	46	46	45	44	43	42	42
	16	54	53	52	51	50	49	49	48	47	46	45
	18	57	56	55	54	54	53	52	51	50	50	49
	20	60	59	58	57	56	56	55	54	53	52	52

6 Results

6.1 Truss optimization

The first part of the optimization consists of truss optimization. In this part, the columns are dimensioned for the biggest span, which is expected to be the heaviest, and these columns are used for the other cases as well. This way, only the effect of the increasing length can be examined.

In Table 6-1 the results for the truss optimization of each case is presented. The table gives the optimal weight of the truss for each length and each height H_0. For the spans with H_0 of 1.00m, there are no results. The reason is that for this height, the angle α is not in between the interval of 30° and 60°, and therefore is not suited for a welded connection. The same situation occurs in the spans with a height of 27m, 31m and 32m. A choice was made to keep the number of fields constant for each given length, but as a consequence, the diagonals in these three spans have angles that do not lay within the boundary conditions. The results are graphically depicted in Figure 6-1, where the weights are expressed as a function of the height H_0, for each length of the truss between 25m and 35m.

Table 6-1: Truss optimization results

Weight (kg)		Length (m)										
		25	26	27	28	29	30	31	32	33	34	35
Height (m)	1,25	1307	1557	/	1698	1751	2028	/	/	2618	2646	2935
	1,50	1258	1301	1646	1651	1793	1846	1899	1952	2308	2468	2606
	1,75	1291	1334	1376	1462	1645	1692	1943	1996	2100	2235	2487
	2,00	1326	1368	1410	1502	1545	1587	1779	1929	2047	2081	2436

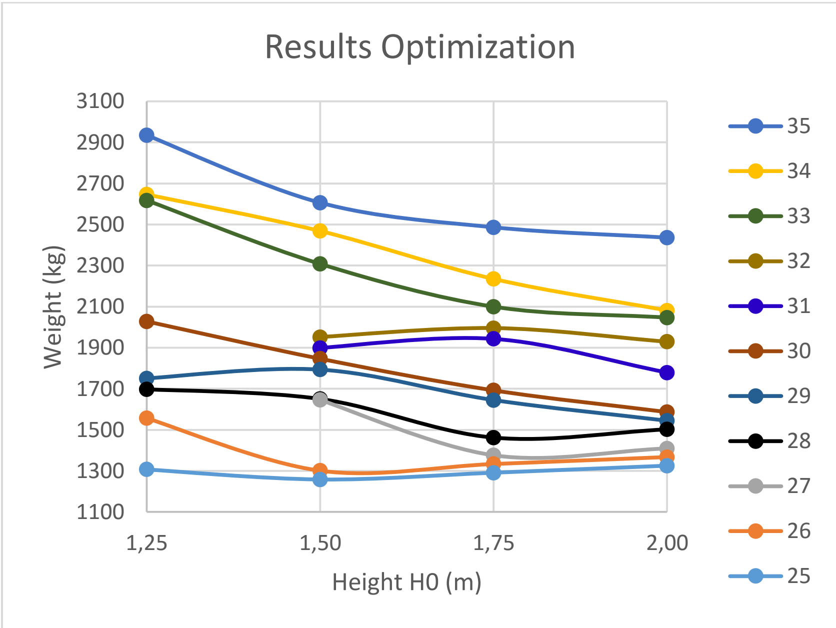


Figure 6-1: Graphical optimization results

The graph in Figure 6-1 shows that the spans with a length of 26m and 25m have an optimal weight which appears at a truss height of 1.50m. When the spans of 27m and 28m are examined, the minimum weight is situated at the height of 1.75m, and there is a slight increase in weight towards 2.00m. For the other lengths, from 29m up to 35m, it is clear that the best height is 2.00m, since all the graphs show that their minimal weight is situated there.

When the graph is examined closely, it becomes clear that in general the optimal height decreases as the length grows. However, in some cases, the line reaches a minimum and starts increasing again. To analyse this phenomenon, two cases will be taken as example. The 28m span reaches a minimum at 1.75m and then goes up again, while the 30m span has an almost constant decrease (see Figure 6-2). An explanation can be found when the cross-sections of the truss elements in Table 6-2 are examined. The 28m span with a height of 1.75m has the same HEA profiles for the chords as the length with a height of 2m. The reason for this is that for the 1.75m case, the axial force came close to the limit of buckling resistance for both the upper and the lower chord. When the height was increased, the axial forces in these chords were reduced, but not enough to allow a reduction in the cross-sections. A consequence of the increase in truss height is that the lengths of the web members increase as well. This extra weight is the reason of the augmentation of the graph towards the 2m height.

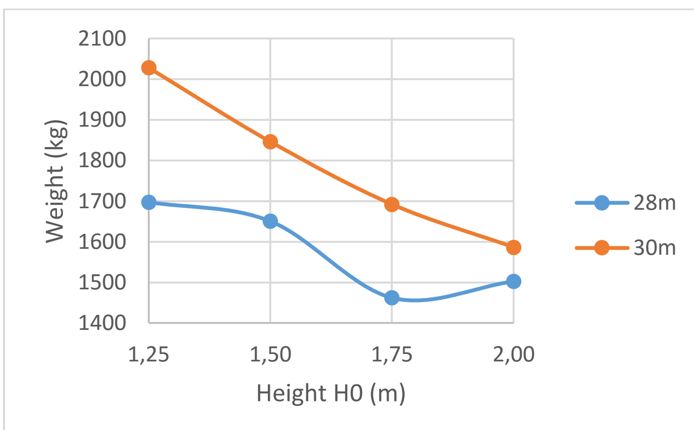


Figure 6-2: Results 28m and 30m spans

In the 29m case the same occurred for the upper chord and this cross-section remained unchanged. However, the lower chord had not reached its buckling resistance at 1.75m. When the height increased, the axial forces were reduced enough to allow the reduction of the lower chord section. The reduction of the weight due to this change was more than enough to compensate for the extra web member weight gained as a result of the increase of the height, as explained above. Therefore the graph has a constant decline towards the 2m height.

A possible explanation for the fact that in the 28m case the decrease of axial force does not allow a reduction in cross-section in contrary to the 29m case can be found when the angles of the diagonal bars are investigated in detail. When the height of the 28m case increases from 1.75m to 2m, the average angle changes from 44° to 47°. The first angle is closer to the optimal angle of 45°, so it is possible that the distribution of forces is not optimal in the 2m height. In the 29m case, this angle changes from 42° to 44°, which means a more optimal angle and thus a better force distribution for the 2m height, and this could explain the reduction of the cross-section.

Table 6-2: Cross-sections 28m and 30m span

	L			
H0	28		30	
1,25	Column	IPE 360	Column	IPE 360
	Upper chord	HEA 140	Upper chord	HEA 160
	Bottom chord	HEA 100	Bottom chord	HEA 120
	Diagonal web	SHS 80/4	Diagonal web	SHS 80/4
	Vertical web	SHS 60/4	Vertical web	SHS 60/4
1,50	Column	IPE 360	Column	IPE 360
	Upper chord	HEA 140	Upper chord	HEA 140
	Bottom chord	HEA 100	Bottom chord	HEA 120
	Diagonal web	SHS 80/4	Diagonal web	SHS 80/4
	Vertical web	SHS 60/4	Vertical web	SHS 60/4
1,75	Column	IPE 360	Column	IPE 360
	Upper chord	HEA 120	Upper chord	HEA 140
	Bottom chord	HEA 100	Bottom chord	HEA 100
	Diagonal web	SHS 60/4	Diagonal web	SHS 60/4
	Vertical web	SHS 60/4	Vertical web	SHS 60/4
2,00	Column	IPE 360	Column	IPE 360
	Upper chord	HEA 120	Upper chord	HEA 120
	Bottom chord	HEA 100	Bottom chord	HEA 100
	Diagonal web	SHS 60/4	Diagonal web	SHS 60/4
	Vertical web	SHS 60/4	Vertical web	SHS 60/4

However, when other cases were checked for the same pattern, no correlation was found. The 27m span for example also reaches a minimum at a height of 1.75m, however, the angle in this minimum is 40° whereas the angle in the 2m height is 44°. A similar observation can be made in the 31m and 32m spans. This nullifies the hypothesis formulated in the previous paragraph.

6.2 Portal frame optimization

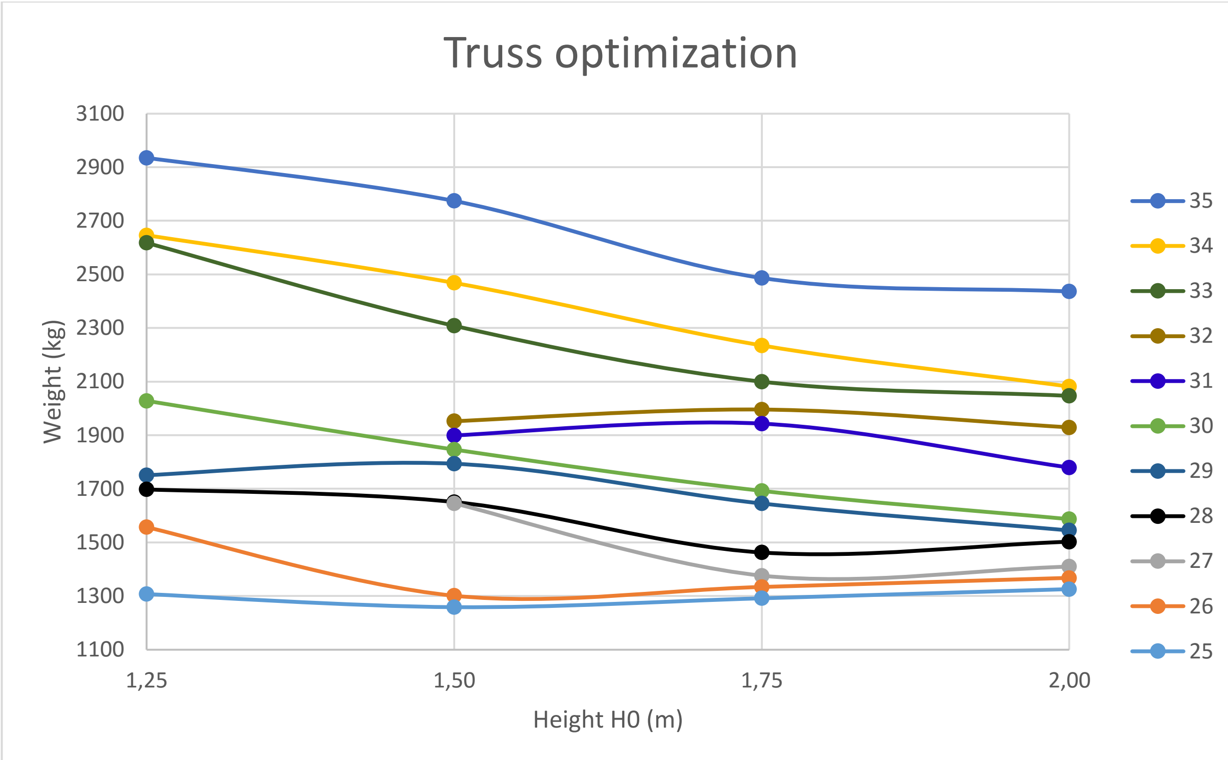
As mentioned in 6.1, the previous results are all derived from a model that presumes the sections of columns to be the same in each case. However, this section does have an influence on the behavior of the structure, as it adds to the total stiffness. In order to take this influence into account, the column sections on the optimized models were decreased until the constraints for either buckling or displacement were reached. After this modification, the optimization process was repeated until a new optimized model was created. The results of this second optimization are presented in Table 6-3.

Table 6-3: Portal frame optimization results

Weight (kg)		Length (m)										
		25	26	27	28	29	30	31	32	33	34	35
Height (m)	1,25	1428	1557	/	1607	1657	2028	/	/	2460	2548	2935
	1,50	1258	1425	1559	1651	1700	1754	1979	2035	2202	2305	2606
	1,75	1291	1334	1505	1462	1645	1692	1843	1996	1994	2126	2487
	2,00	1326	1368	1539	1502	1545	1733	1779	1929	2047	2081	2436

When this table is compared to Table 6-1, there are clearly some differences. At first sight these differences may seem random, since some weights are higher, some are lower and some are not different at all. The differences between the two optimizations can be directly related to the stiffness of the columns. To explain this 'randomness', an analysis of the acting forces in the truss has to be made.

When the column sections are reduced, a larger deformation of the structure will occur. This results in an increase in the compression in the upper chord of the truss, and an increase in the tension in the lower chord. Another result of this modification is a decrease in the compression at the ends of the lower chord, which is a decisive factor in its buckling strength. When the increase of axial force in the upper chord causes the overcome of the buckling resistance, a larger section is necessary. The same goes for the lower chord. However, as it turns out the tension resistance will not be exceeded. On contrary, due to the decrease of compression in these chords, the section can even be reduced when the decrease is enough. The web members seem to be hardly effected by the change of the columns, therefore, their sections remain unchanged. In Figure 6-3 a graphical comparison of the two optimizations is shown.



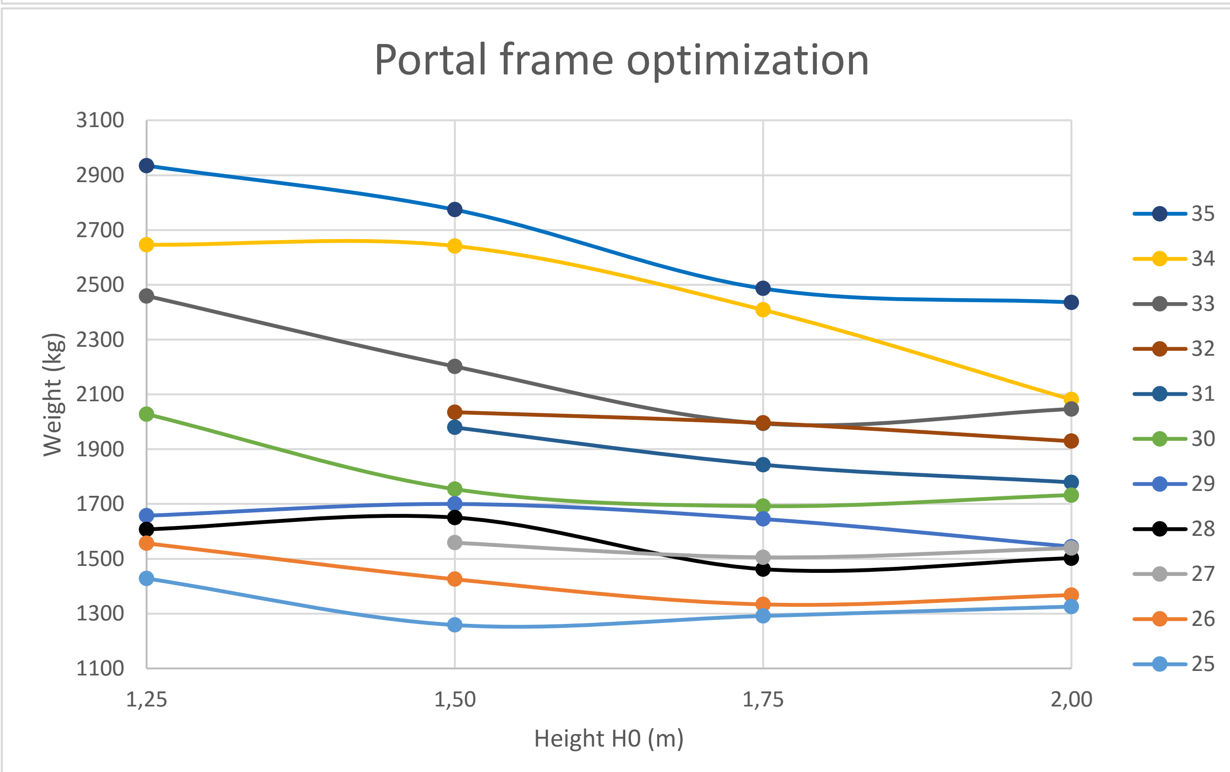


Figure 6-3: Graphical comparison of optimization results

In general the form of the graphs are similar, however, there are some small differences between the two. One noticeable difference is that the optimal weight of the 26m span has shifted from 1.50m to 1.75m (Figure 6-4). The opposite has happened for the spans of 30m and 33m. Here the optimal height has become 1.75m instead of 2.00m (Figure 6-5).

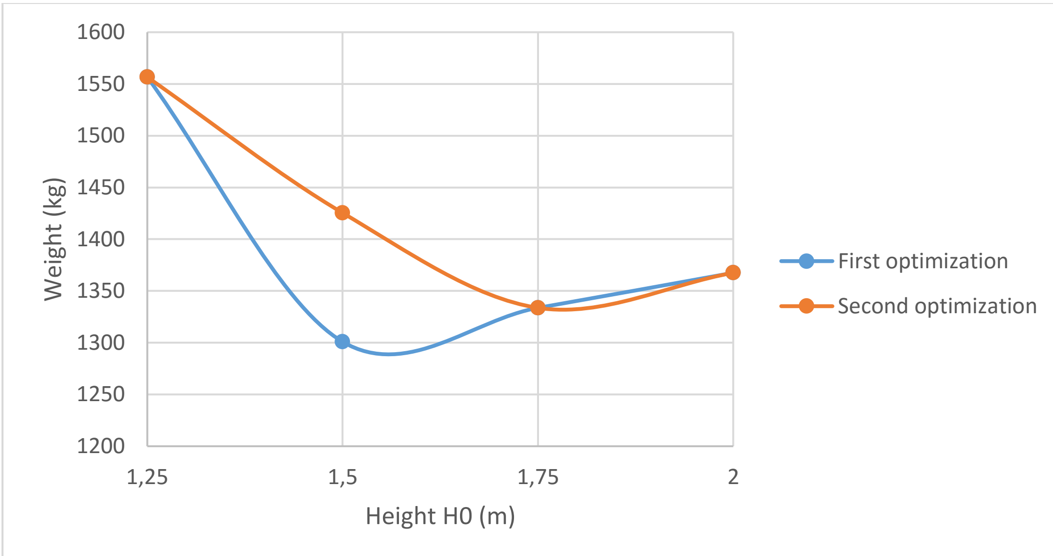


Figure 6-4: Shift of the optimal height of 26m span

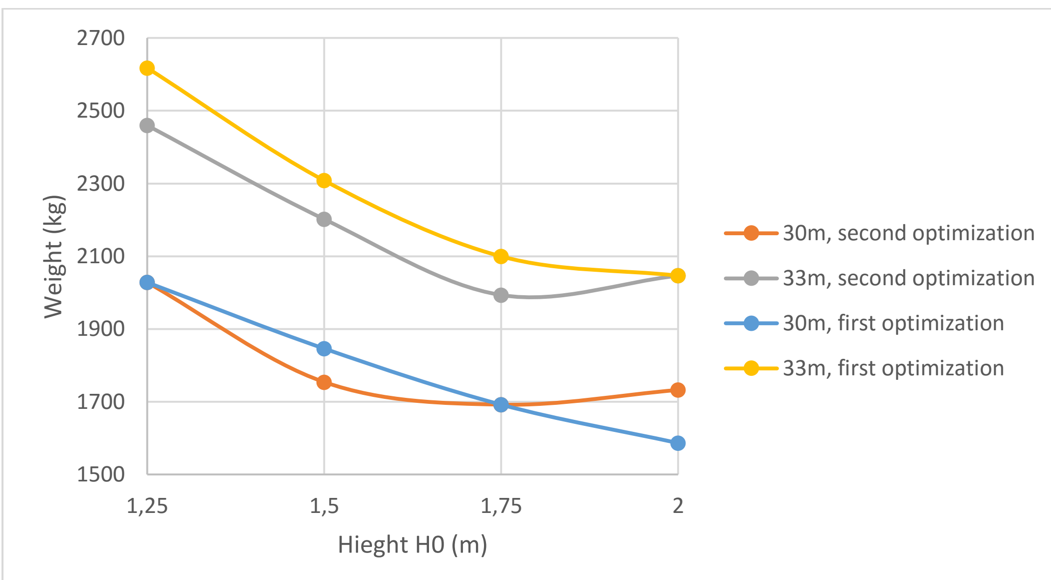


Figure 6-5: Shift of the optimal height of 30m and 33m spans

Another observation that immediately catches the eye is that the 27m span has become heavier than the 28m span when the column sections were changed. This seems odd at first sight, however, an explanation can be found after an analysis of the buckling lengths. As already mentioned in 4.2.5, the buckling length in the z-direction was limited to 5m by buckling stabilizers, for each case. However, a perfect length of 5m is not always possible in spans with a length of for example 27m. In one of these cases the length of the span is divided by 5m, and this number (which is the number of stabilizers) is rounded off. In the 27m span this number is: $\frac{27m}{5m} = 5.4$, which is rounded off to 5. In the 28m span it is: $\frac{28m}{5m} = 5.6$, which becomes 6. This difference in the numbers of stiffeners means that the buckling lengths in the 28m span are smaller than the ones in the 27m span, causing the buckling strength of the elements to be lower in the latter case. This results in larger sections for the chords in the 27m span, thus increasing its weight.

When the truss structures are calculated, a check is executed in the Serviceability Limit State (deflection and displacement) and in the Ultimate limit state (cross-sectional resistance and buckling). An examination of the main failure modes shows that the restrictions for displacement and deflection are never exceeded. The displacement is mostly related to the stiffness of the columns. The analysis shows that the column will always fail in buckling before the displacement becomes an issue. The same conclusion can be made for the deflection, which is related to the stiffness of the truss structure itself. Here, the chords will fail in buckling first before the deflection limit is reached. The vertical web members will also fail in buckling, in contrary to the diagonal members which will fail in tension. The conclusion that can be drawn is that the most important check of these structures is the Ultimate Limit State check.

6.3 Section table

In Table 6-4 the sections for each element are presented after the truss optimization, with the IPE 450 columns. In Table 6-5 the results of the portal frame optimization are presented.

Table 6-4: Optimization results IPE 450 columns

	L										
H0	25	26	27	28	29	30	31	32	33	34	35
1,25	C IPE 450	C IPE 450		C IPE 450	C IPE 450	C IPE 450			C IPE 450	C IPE 450	C IPE 450
	U HEA 120	U HEA 140		U HEA 140	U HEA 140	U HEA 160			U HEA 160	U HEA 180	U HEA 180
	B HEA 120	B HEA 100		B HEA 120	B HEA 120	B HEA 120			B HEA 140	B HEA 140	B HEA 140
	D SHS 60/4	D SHS 80/4		D SHS 80/4	D SHS 80/4	D SHS 80/4			D SHS 100/5	D SHS 80/4	D SHS 100/5
	V SHS 60/4	V SHS 60/4		V SHS 60/4	V SHS 60/4	V SHS 60/4			V SHS 80/4	V SHS 80/4	V SHS 80/4
1,50	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450
	U HEA 120	U HEA 120	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 160	U HEA 160	U HEA 180
	B HEA 100	B HEA 100	B HEA 120	B HEA 100	B HEA 120	B HEA 120	B HEA 120	B HEA 120	B HEA 120	B HEA 140	B HEA 120
	D SHS 60/4	D SHS 60/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4
	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 80/4	V SHS 60/4	V SHS 80/4
1,75	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450
	U HEA 120	U HEA 120	U HEA 120	U HEA 120	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 160	U HEA 160
	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 120	B HEA 120	B HEA 120	B HEA 120	B HEA 120
	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 60/4	D SHS 80/4
	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 80/4
2,00	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450	C IPE 450
	U HEA 120	U HEA 120	U HEA 120	U HEA 120	U HEA 120	U HEA 120	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 160
	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 120	B HEA 100	B HEA 120	B HEA 100
	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 80/4	D SHS 60/4	D SHS 80/4
	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 80/4

Wind load: $v_{b,0}$ = 25 m/s
Terrain category III
H = 8m

Snow load: 0,4 kN/m²

Roof load: 25 kg/m²

Steel: S235

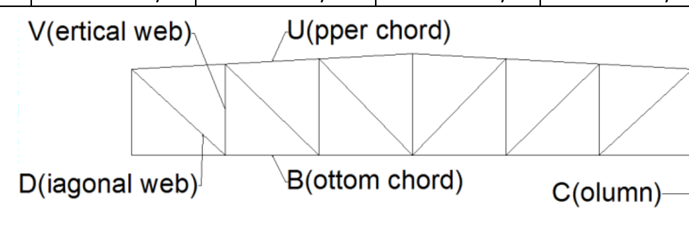
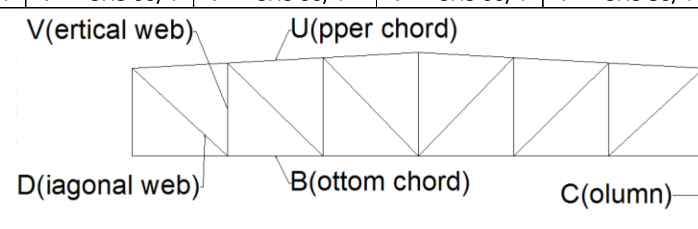


Table 6-5: Portal frame optimization results

	L										
H0	25	26	27	28	29	30	31	32	33	34	35
1,25	C IPE 330	C IPE 330		C IPE 360	C IPE 360	C IPE 360			C IPE 400	C IPE 400	C IPE 450
	U HEA 120	U HEA 140		U HEA 140	U HEA 140	U HEA 160			U HEA 160	U HEA 180	U HEA 180
	B HEA 120	B HEA 100		B HEA 100	B HEA 100	B HEA 120			B HEA 120	B HEA 140	B HEA 140
	D SHS 60/4	D SHS 80/4		D SHS 80/4	D SHS 80/4	D SHS 80/4			D SHS 100/5	D SHS 80/4	D SHS 100/5
	V SHS 60/4	V SHS 60/4		V SHS 60/4	V SHS 60/4	V SHS 60/4			V SHS 80/4	V SHS 80/4	V SHS 80/4
1,50	C IPE 330	C IPE 330	C IPE 330	C IPE 360	C IPE 360	C IPE 360	C IPE 360	C IPE 360	C IPE 400	C IPE 400	C IPE 450
	U HEA 120	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 160	U HEA 160	U HEA 160	U HEA 180	U HEA 180
	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 120	B HEA 100	B HEA 120	B HEA 140	B HEA 120
	D SHS 60/4	D SHS 60/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 80/4
	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 80/4	V SHS 60/4	V SHS 80/4
1,75	C IPE 330	C IPE 330	C IPE 330	C IPE 360	C IPE 360	C IPE 360	C IPE 360	C IPE 360	C IPE 400	C IPE 400	C IPE 450
	U HEA 120	U HEA 120	U HEA 140	U HEA 120	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 180	U HEA 160
	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 120	B HEA 100	B HEA 120	B HEA 120
	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 80/4	D SHS 80/4	D SHS 80/4	D SHS 60/4	D SHS 80/4
	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 80/4
2,00	C IPE 330	C IPE 330	C IPE 330	C IPE 360	C IPE 360	C IPE 360	C IPE 360	C IPE 360	C IPE 400	C IPE 400	C IPE 450
	U HEA 120	U HEA 120	U HEA 140	U HEA 120	U HEA 120	U HEA 140	U HEA 140	U HEA 140	U HEA 140	U HEA 160	U HEA 160
	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 100	B HEA 120	B HEA 100	B HEA 120	B HEA 100
	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 60/4	D SHS 80/4	D SHS 60/4	D SHS 80/4
	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 60/4	V SHS 80/4

Wind load: $v_{b,0}$ = 25 m/s
Terrain category III
H = 8m
Snow load: 0,4 kN/m²
Roof load: 25 kg/m²

Steel: S235



6.4 Additional wall coverage weight

In this paper, the main objective was to optimize the weight of the truss structures, in order to find the most economical solution. To reach this goal, only the truss weight was optimized. However, the cost of a structure obviously does not depend solely on the truss weight. Another factor for example is the additional weight gained due to the increased wall height that is needed when the height of the truss increases. Initial research on this subject indicates that the influence of this weight causes the optimal height to shift towards the lower truss heights.

To get an impression of this influence, an assumption of the additional weight is added to the weights of the optimized trusses. For an increase of 0.25 in the truss height, an additional weight of 180 kg is assumed. When this is added to the truss weights, a graph occurs as depicted in Figure 6-6.

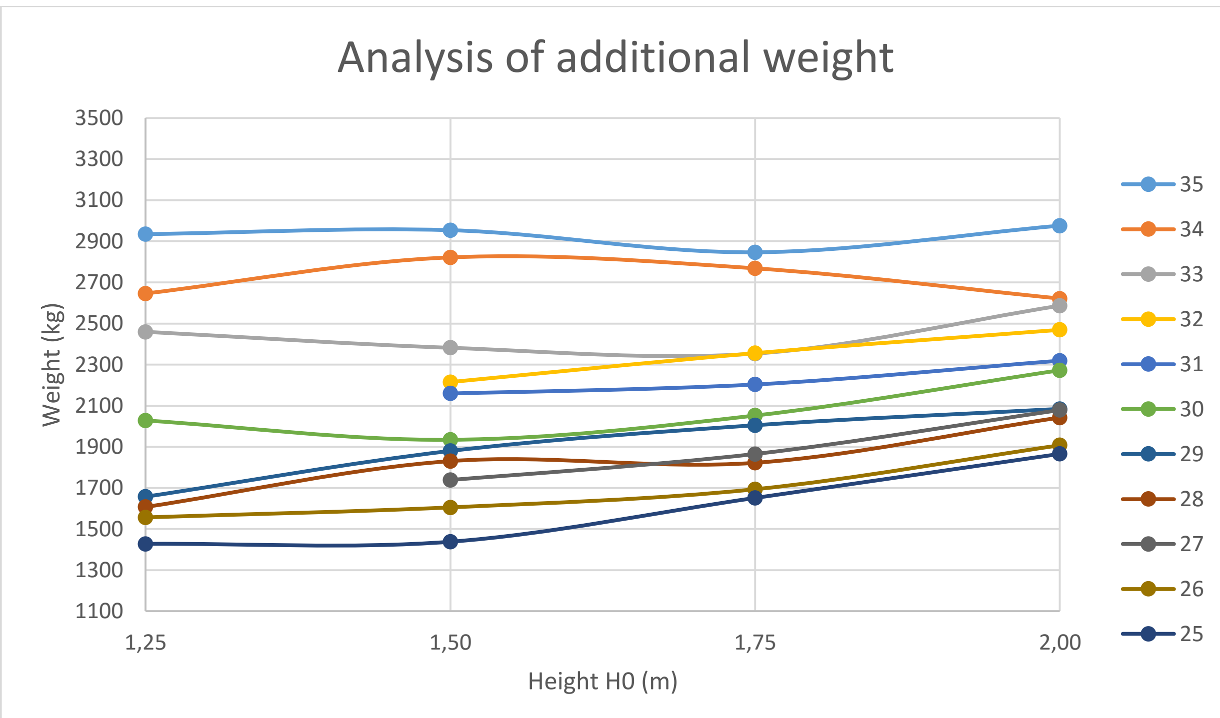


Figure 6-6: Analysis of additional weight

The graph suggests that the optimal weight for practically every truss length is now 1.25m. However, this analysis is just an initial impression. A more thorough investigation is recommended to calculate the exact influence on the economy of a truss structure.

7 Conclusion

In this thesis the objective was to optimize the material weight of the structures, in order to reduce the cost. To achieve this goal, structural software was used in order to find the optimal truss height and sections for truss spans of different lengths. The lengths varied between 25m and 35m, and the height of the truss between 1.25m and 2m. The acting loads considered were wind load, snow load, roof load and self-weight.

First, an optimization of the truss structure was executed separately from the column section. For each length of the span, IPE 450 columns were used. The graphical results showed that the truss height was inversely proportional to the weight. When the columns were taken into account and optimized, the optimal height was not always the largest anymore. The column sections caused a reduction in compression force in the bottom chord, but an increase in the upper chord. Due to this change in compression force, the bottom section could be reduced in some cases, but the upper chord section had to be increased. This caused a shift in optimal height in several cases.

In all the cases, the analysis of the failure modes shows that the critical elements will fail in ULS (buckling or tension), before the SLS constraints (deflection and displacement) are reached.

The optimal economical cost is not exclusively related to the truss weight. A brief investigation of the influence of the coverage weight of the structure indicated that this additional weight caused the optimal truss height to shift towards the lower heights. Future research could include this parameter more extensively in its attempt to obtain a fully optimized structure.

8 References

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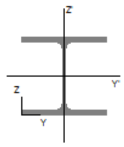
Annex A: Detailed calculation

Overview ENV 1993-1-1 : bar 18

Data



Cross-section: HEA 160
Material: S235
Bar length: 2,08 m
Buckling length in plane: 1,79 m
Buckling length out of plane: 2,84 m
Lateral torsional buckling length: 2,08 m

Checks

Axial tension	1,834%
Axial compression	**27,630%**
Bending around y'-axis	4,612%
Bending around z'-axis	0,000%
Shear force y'-axis	0,000%
Shear force z'-axis	5,073%
Torsion	0,000%
Bending y' + shear z'	4,612%
Bending z' + shear y'	0,000%
Biaxial bending + normal force	5,551%
Biaxial bending + shear + normal force	5,551%

Buckling round y' axis	28,560%
Buckling round z' axis	40,159%
Buckling due to $M_{y'}$, $M_{z'}$ and N	44,332%
Lateral torsional buckling due to $M_{y'}$, $M_{z'}$ and N	**44,967%**
Lateral torsional buckling	4,859%

Resistance checks according to ENV 1993-1-1 : bar 18 – FC

Axial tension (§5.4.3) **1,834%**

Maximum at node 20 in combination UGT FC 20

N_{Ed} = 15,2 kN
$N_{t,Rd} = A \cdot f_{yd}$ = 828,4 kN

A = 3877,5 mm² f_{yd} = 213,6 N/mm²

Axial compression (§5.4.4) **27,630%**

Maximum at node 19 in combination UGT FC 2
Cross-section class: 1

N_{Ed} = 228,9 kN
$N_{c,Rd} = A \cdot f_{yd}$ = 828,4 kN

A = 3877,5 mm² f_{yd} = 213,6 N/mm²

Bending around y'-axis (§5.4.5) **4,612%**

Maximum at 1,13 m of node 20 in combination UGT FC 2
Cross-section class: 1

$M_{y',Ed}$ = 2,4 kNm
$M_{y',Rd} = W_{y',pl} \cdot f_{yd}$ = 52,4 kNm

$W_{y',pl}$ = 245167 mm³ f_{yd} = 213,6 N/mm²

Bending around z'-axis (§5.4.5)

This bar is not subjected to bending around z'-axis

Shear force y'-axis (§5.4.6)

This bar is not subjected to shear force $V_{y'}$

Shear force z'-axis (§5.4.6) **5,073%**

Maximum at node 20 in combination UGT FC 2

$V_{z',Ed}$ = 8,3 kN
$V_{z',Rd} = A_{vz} \cdot f_{yd} / \sqrt{3}$ = 163,0 kN

Avz = 1321,5 mm² f_{yd} = 213,6 N/mm²

Torsion

This bar is not subjected to torsion

Bending y' + shear z' (§5.4.7) **4,612%**

Maximum at 1,13 m of node 20 in combination UGT FC 2
Cross-section class: 1

$M_{y',Ed}$ = 2,4 kNm
$V_{z',Ed}$ = 0,6 kN

$M_{Vy',Rd} = W_{y',pl} \cdot (1 - \rho) \cdot f_{yd}$ = 52,4 kNm
$V_{z',Rd} = A_{vz} \cdot f_{yd} / \sqrt{3}$ = 163,0 kN

ρ = 0,000	$W_{y',pl}$ = 245167 mm³
Avz = 1321,5 mm²	f_{yd} = 213,6 N/mm²

Bending z' + shear y' (§5.4.7)

This bar is not subjected to bending around z'-axis

Biaxial bending + normal force (§5.4.8) **5,551%**

Maximum at 1,13 m of node 20 in combination UGT FC 2
Cross-section class y': 1 Cross-section class z': 1

N_{Ed} = 228,7 kN	$e_{y'}$ = 0,0 mm	$e_{z'}$ = 0,0 mm
$M_{y',Ed}$ = 2,4 kNm		
$M_{z',Ed}$ = 0,0 kNm		

$M_{N,y',Rd} = M_{y',Rd} \cdot (1 - n)/(1 - 0.5a)$ = 43,5 kNm

$M_{N,z',Rd} = M_{z',Rd} \cdot [1 - (n - a)^2/(1 - a)^2]$ = 25,1 kNm

$N_{Rd} = A \cdot f_{yd}$ = 828,4 kN
$M_{y',Rd} = W_{y',pl} \cdot f_{yd}$ = 52,4 kNm
$M_{z',Rd} = W_{z',pl} \cdot f_{yd}$ = 25,1 kNm

A = 3877,5 mm²	$W_{y',pl}$ = 245167 mm³	$W_{z',pl}$ = 117635 mm³
f_{yd} = 213,6 N/mm²	n = 0,276	a = 0,257
α = 2,000	β = 1,380	

Biaxial bending + shear + normal force (§5.4.9) **5,551%**

Maximum at 1,13 m of node 20 in combination UGT FC 2
Cross-section class y': 1 Cross-section class z': 1

N_{Ed} = 228,7 kN	$e_{y'}$ = 0,0 mm	$e_{z'}$ = 0,0 mm
$M_{y',Ed}$ = 2,4 kNm	$V_{y,Ed}$ = 0,0 kN	
$M_{z',Ed}$ = 0,0 kNm	$V_{z,Ed}$ = 0,6 kN	

$M_{VN,y',Rd} = M_{V,y',Rd} \cdot (1 - n) / (1 - 0.5a)$ = 43,5 kNm
$M_{VN,z',Rd} = M_{V,z',Rd} \cdot [1 - ((n - a) / (1 - a))^2]$ = 25,1 kNm

$N_{Rd} = A \cdot f_{yd}$ = 828,4 kN
$M_{Vy',Rd} = W_{y',pl} \cdot (1 - \rho) \cdot f_{yd}$ = 52,4 kNm
$M_{Vz',Rd} = W_{z',pl} \cdot (1 - \rho) \cdot f_{yd}$ = 25,1 kNm
$V_{y,Rd} = A_{vy} \cdot f_{yd} / \sqrt{3}$ = 370,8 kN
$V_{z,Rd} = A_{vz} \cdot f_{yd} / \sqrt{3}$ = 163,0 kN

$\rho = 0{,}000$ | $W_{y',pl} = 245167\ mm^3$ | $W_{z',pl} = 117635\ mm^3$
$A = 3877{,}5\ mm^2$ | $A_{vy} = 3006{,}0\ mm^2$ | $A_{vz} = 1321{,}5\ mm^2$
$f_{yd} = 213{,}6\ N/mm^2$ | $n = 0{,}276$ | $a = 0{,}257$
$\alpha = 2{,}000$ | $\beta = 1{,}3$

Stability checks according to ENV 1993-1-1 : bar 18 -

Buckling round y' axis (§5.5.1) **28,560%**

Maximum at node 19 in combination UGT FC 2
Cross-section class: 1

$N_{Ed} = 228{,}9\ kN$
$N_{b,y',Rd} = \chi_{y'} . A . f_{yd} = 801{,}4\ kN$

$\chi_{y'} = 0{,}967$ | $A = 3877{,}5\ mm^2$ | $f_{yd} = 213{,}6\ N/mm^2$
$\vartheta_{y'} = 0{,}558$ | $\lambda_{y',rel} = 0{,}291$ | $\alpha_{y'} = 0{,}340$
$\lambda_{y'} = 27{,}320$

$L_{cr,y'} = 1{,}79\ m$

Buckling round z' axis (§5.5.1) **40,159%**

Maximum at node 19 in combination UGT FC 2
Cross-section class: 1

$N_{Ed} = 228{,}9\ kN$
$N_{b,z',Rd} = \chi_{z'} . A . f_{yd} = 569{,}9\ kN$

$\chi_{z'} = 0{,}688$ | $A = 3877{,}5\ mm^2$ | $f_{yd} = 213{,}6\ N/mm^2$
$\vartheta_{z'} = 0{,}925$ | $\lambda_{z',rel} = 0{,}759$ | $\alpha_{z'} = 0{,}490$
$\lambda_{z'} = 71{,}254$

$L_{cr,z'} = 2{,}84\ m$

Buckling due to $M_{y'}$, $M_{z'}$ and N (§5.5.4) **44,332%**

Maximum at 1,13 m of node 20 in combination UGT FC 2
Cross-section class y': 1 | Cross-section class z': 1

$N_{Ed} = 228{,}7\ kN$ | $e_{y'} = 0{,}0\ mm$ | $e_{z'} = 0{,}0\ mm$
$M_{y',Ed} = 2{,}4\ kNm$
$M_{z',Ed} = 0{,}0\ kNm$

$N_{b,Rd} = \chi_{min} . A . f_{yd} = 569{,}9\ kN$
$M_{y',Rd} = W_{y',pl} . f_{yd} = 52{,}4\ kNm$
$M_{z',Rd} = W_{z',pl} . f_{yd} = 25{,}1\ kNm$

$W_{y',pl} = 245167\ mm^3$ | $W_{z',pl} = 117635\ mm^3$ | $A = 3877{,}5\ mm^2$
$\chi_{min} = 0{,}688$ | $f_{yd} = 213{,}6\ N/mm^2$
$k_y = 0{,}911$ | $k_z = 0{,}543$

$\mu_y = -0{,}341$ | $\mu_z = -1{,}252$

Lateral torsional buckling due to $M_{y'}$, $M_{z'}$ and N (§5.5.4) **44,967%**

Maximum at 1,13 m of node 20 in combination UGT FC 2

Cross-section class y': 1 Cross-section class z': 1

N_{Ed} = 228,7 kN $e_{y'}$ = 0,0 mm $e_{z'}$ = 0,0 mm

$M_{y',Ed}$ = 2,4 kNm

$M_{z',Ed}$ = 0,0 kNm

$N_{b,z',Rd} = \chi_{z'} . A . f_{yd}$ = 569,9 kN

$M_{b,y',Rd} = \chi_{LT} . W_{y',pl} . f_{yd}$ = 49,7 kNm

$M_{z',Rd} = W_{z',pl} . f_{yd}$ = 25,1 kNm

$W_{y',pl}$ = 245167 mm³ $W_{z',pl}$ = 117635 mm³ A = 3877,5 mm²

$\chi_{z'}$ = 0,688 χ_{LT} = 0,949 f_{yd} = 213,6 N/mm²

k_z = 0,543 k_{LT} = 0,996

μ_z = -1,252 μ_{LT} = -0,011

Lateral torsional buckling (§5.5.2) **4,859%**

Maximum at 1,13 m of node 20 in combination UGT FC 2

Cross-section class: 1

$M_{y',Ed}$ = 2,4 kNm

$M_{b,Rd} = \chi_{LT} . W_{y',pl} . f_{yd}$ = 49,7 kNm

χ_{LT} = 0,949 $W_{y',pl}$ = 245167 mm³ f_{yd} = 213,6 N/mm²

ϑ_{LT} = 0,608 $\lambda_{LT,rel}$ = 0,414 α_{LT} = 0,210

M_{cr} = 336,1 kNm C_1 = 1,245

L_{crLT} = 2,08 m

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Richting: **master in de industriële wetenschappen: bouwkunde**
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