Master's Thesis

The economical advantages of the use of S460 steel

- Based on a comparison with lower strength steel-

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27-05-2013

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For the past 7 months I have been working on this master’s thesis at Hollandia b.v. My colleagues at this company have been very helpful and provided me with the necessary information to complete this master’s thesis. I would like to thank everyone at Hollandia b.v. a few people in particular:

G. Kannekens for providing me with the opportunity to conduct my research at Hollandia b.v. ,for introducing me to everyone who could provide me with the information required for this master’s thesis and for providing me with helpful feedback.

T. van Lammeren for being part of the assessment committee and also for providing me with feedback prior to the meetings with the assessment committee.

M. van Outheusden and M. van Alphen for providing me with technical information and helping me with design issues.

H. Verleg for providing me with the basic information required for the determination of fabrication, assembly and transportation costs.

K. Ardon and J. Noordijk for providing me with in depth information about the fabrication costs.

I would like to thank all of the members of the assessment committee for their interest in this research and for all of the help and feedback they have given me.

I would also like to thank M. May (ArcelorMittal) for providing me with technical information about quenched and tempered steel grades.

Maikel Croes

27-05-2013

1 Abstract

This master's thesis is about the use of higher strength steel in buildings and the economical advantages of the use of this steel. The economical advantages are examined based on a comparison of similar designs with the use of the steel grades S355 and S460. The latter could in some cases result in large weight and cost savings. The costs are examined for trusses and columns. The trusses are applied to transfer loads from 5 different floors to columns up to 40 m apart. The columns are examined for a high-rise office with 24 stories. For all the components examined the costs have been calculated. These costs are divided into 4 different groups: Material, fabrication, transportation and assembly costs.

When designing constructions with the use of S460 large material cost savings can be expected. Due to the excellent properties of this steel grade the use of S460 is beneficial for both light weight and heavy sections. For heavy sections exceptionally large material cost savings can be expected. The use of this steel grade should however be limited to axially loaded members only.

Fabrication costs are more difficult to describe than the material costs. Stronger steel grades have a larger hardness which makes sawing and drilling for example more difficult. The reduced thicknesses that can be achieved by using S460 can however be assumed to compensate for this increased difficulty. Fabrication costs will ultimately not differ much from the costs required for a design with the use of S355.

Assembly costs can be reduced due to the lower weight of the components. If trusses are used the total weight can be reduced significantly and the use of smaller cranes is therefore possible. The time required for construction will not change much but the costs for the required equipment could decrease significantly due to the decreased weight.

The transportation costs can also be reduced because of the reduced weight. Less trucks are required to transport sections and plates to the construction site which results in a cost reduction proportional to the weight saving.

The total costs for a steel structure will be lower when S460 is used. The costs for beams cannot be reduced because the use of S460 is very inefficient in these cases. A steel structure can therefore be designed with the use of S460 for the columns and S355 for beams to achieve an optimal economical design.

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3 Introduction

3.1 Research

Through the years the use of S235 steel has declined, this is because the price of S355 steel is about the same as S235 steel. This steel grade (S355) has become the new standard in steel construction for parts where the strength of the steel is governing. Because the strength of S355 is about 1,5 times higher than S235 savings in material can be substantial. This provides enough economical reasons to switch to the use of S355. When not the strength but the stiffness of a section is governing (this is usually the case for girders) the use of higher strength steel would result in an increase in costs. Although higher strength steels already are available they are primarily used in cranes and bridges. This is because sections in high strength steels are not available. The last Dutch codes have been replaced in the Eurocode and the use of S460 steel is implemented. There are sections available in this strength class. These sections are manufactured by the company ArcelorMittal and the material is referred to as HISTAR460. It has the same strength properties as S460 but the yield strength does not have to be reduced that much if great thicknesses (t > 40 mm) are used (reduction not required for thicknesses up to 82 mm!). Sections in HISTAR460 have been used in numerous projects because of the additional weight saving and thus cost saving it enables. This higher strength steel is a bit more expensive than the more conventional steels. Per kilo it costs 5 cents more than S355, but the many kilos that can be saved will easily compensate for the increase in price. Still today's engineer will tend toward the use of S355 because there is not much experience in the use of S460, that is why it is interesting to investigate if the use of S460 or HISTAR460 will result in great savings compared to S355. In order to make accurate comparisons between the use of S355 and S460 steel the comparisons are based on real projects. In this master's thesis truss systems and columns are reviewed. For the truss systems a project in Delft is used. A new train station is under construction and on top of that station a new office building will be constructed. These offices are supported by trusses that span 40 m in lateral direction but can only be supported with 2 columns, hence the use of trusses. This kind of system has been used more than once in the Netherlands alone. Columns are reviewed by the use of a project in England. In London a new high-rise building is constructed which is called BP4. The tower has 24 floors and is supported by steel columns and a concrete core.

Next to savings in material other advantages can be identified: The reduction of material results in less space occupied by the buildings structural components resulting in a greater area available to use **[1]**. If composite construction with S460 allows for greater height reduction the total height of a multilevel building could be reduced **[2]**. Thanks to the low weight of the components the necessary building time could also be reduced.

The biggest advantage of high strength steel is when they are used as axially loaded elements. This enables the material to reach its full potential. The use of S460 in truss systems and columns will therefore probably lead to great weight and cost saving. The use of S460 in girders will probably be uneconomical because girders require a minimal stiffness and strength is usually not the dominating factor. When bolted connections are used the holes will have a significant effect on the strength of the sections. This is especially the case for higher strength steel because the cross-section will be smaller and the tensile strength is (relative to the yield strength) also smaller. In these cases the use of S460 would lead to an increase in the use of stiffeners which in turn reduces the cost saving.

3.2 Material properties HISTAR460

By adding an extra step to the production line of hot rolled sections an increase in strength can be achieved. The extra step required is the quenching and self-tempering **[3]** of the produced section. When the beam is quenched the temperature will drop fast. This is achieved by water cooling the entire surface of the hot rolled sections until the chore starts to cool down. The section is then self tempered, the inside of the section (chore) cools down and the surface warms up again until the temperature is uniformly distributed. The temperature of the surface area will remain below 600˚C. By doing this a fine grain and high strength is achieved. This results in the S460 (HISTAR460) steel grade which is delivered in a thermomechanically hot rolled condition. One of the main advantages of the HISTAR460 grade is the improved mechanical characteristics in thick products. According to EN 10025-4 **[4]** the following properties for S460M are valid (Table 3-1).

EN 10025-4	t≤40mm		40mm<t≤80mm	
	fy [N/mm²]	fu [N/mm²]	fy [N/mm²]	fu [N/mm²]
S460M/ML	460	540	430	530

Table 3-1: Material properties of S460M

The use of HISTAR460 allows for more convenient strength properties to be used in thick products (Table 3-2) **[5]**.

	t≤82mm		82mm<t≤140mm	
	fy [N/mm²]	fu [N/mm²]	fy [N/mm²]	fu [N/mm²]
HISTAR460	460	540	450	540

Table 3-2: Material properties of HISTAR460

These properties are important when designing highly loaded columns. When a large axial load bearing capacity is required HD sections are usually recommended. These sections can have thicknesses up to 140 mm. If conventional steel (S355) would be used the maximum allowable yield strength would have to be reduced to 295 N/mm² according to EN 10025-2 **[6]**. In that case the use of HISTAR460 becomes more attractive because the strength difference is even greater. When designing columns stability is always an important factor to consider. In this case HISTAR460 also has a slight advantage because more favorable buckling curves may be used. This is because the production process leads to less residual stresses in the sections which increases the resistance against buckling. For thicknesses up to 100 mm buckling curve b for buckling about the strong axis and curve c for buckling about the weak axis should be used for S355 **[7]**. For S460 buckling curve a which is more favorable should be used for buckling about both axis. For thicknesses greater than 100 mm buckling curve d for both axis (S355) and curve c (S460) should be used. These curves are valid when the height is not more than 1,2 times the width of the section which is valid for almost all HD sections.

HISTAR355 is also available. The use of this grade allows the use of a yield strength of 355 N/mm² regardless of the section used so when large sections are required the use of HISTAR355 could lead to a slightly smaller section than when S355 is used. The buckling curves are the same as the ones used for S355.

When the strength of steel increases usually the carbon equivalent also increases which means that it is more difficult to weld high strength steel. In this case the carbon equivalent (CEV) for HISTAR460 is not more than 0,43. If the combined thickness is not more than 50 mm preheating is not required. If the combined thickness is larger than 50 mm preheating is also not required provided that the diffusible hydrogen content in the weld material is kept low. These values are valid for a heat input of 1,0 kj/mm. When 1,5k j/mm is used preheating is not required when the diffusible hydrogen content is less than 10 ml/100 g which corresponds to class C. Weld consumables can be classified in 5 classes depending on their hydrogen content. The classes and maximum hydrogen content is shown in Table 3-3. With the use of EN 1011-2 **[8]** the maximum combined thickness of the materials for which no preheating is required is determined. This thickness is dependent on the carbon equivalent of the material. For S355M the CEV is 0,4 for thicknesses up to 63 mm. If the thickness is larger than 63 mm the CEV is 0,45. For S460M the CEV is 0,47 for thicknesses up to 63 mm and 0,48 for larger thicknesses. For both steel grades no preheating is required when a weld consumable in class D and a heat input of 1,5 kj/mm (S355M) or 2,0 kj/mm (S460M) is used. If the combined thickness is less than 63 mm and S355 is used a weld consumable in class C can be used with a heat input of 1,5 kj/mm.

Class	Hydrogen content [ml/100g]
A	>15
B	10≤15
C	5≤10
D	3≤5
E	≤3

Table 3-3: Hydrogen content classes

3.3 Expectations

When using higher strength steel it is expected that the decrease in material results in cost saving. In general for axially loaded structural elements the weight reduction can be related to the yield strengths of the compared steel grades. In this case the steel grade S460 is compared to S355. The actual weight reduction when the cross-sectional dimensions are known is shown in the following formula:

$$\frac{A_{355} - A_{460}}{A_{355}} * 100\%$$

In the formula the subscript refers to the used steel grade. When the cross-sectional dimensions are not known they can be related to the yield strengths of the materials. If a tensile member of a truss is loaded with a force F the required cross-sectional area is $\frac{F}{f_y}$. By replacing the areas in the previous formula the following formula is obtained:

$$\frac{\frac{F}{355} - \frac{F}{460}}{\frac{F}{355}} * 100\%$$

The force F can be replaced by 1 in the formula. Multiplying by $\frac{355}{355}$ results in:

$$\frac{1 - \frac{355}{460}}{1} * 100\%$$

Finally multiplying by $\frac{460}{460}$ results in:

$$\frac{460-355}{460} * 100\% \approx 23\%$$

The expected weight reduction by the use of S460 in comparison with S355 is about 23%. Often the expected weight reduction is related to the increase in yield strength. The yield strength of S460 is about 30% higher than S355. Based on this fact the general assumption is that the weight reduction is also 30%. As shown the expected weight reduction is smaller. This is because of the inverse proportionality of required cross-sectional area and yield strength. Because of this the increase in yield strength should also have been inversed to obtain the weight reduction of 23%. This estimate is only valid for axially loaded components. Girders loaded in bending will not result in the same weight reduction because an easy linear relation between the required cross-sectional area and a bending moment cannot be made. A relation between the bending moment and the required section modulus can be made by using the yield strength. It is however not possible to derive the required cross-sectional area and thus weight by using the section modulus only, there is no linear relation between these values. Because the section modulus is dependent on the height squared an increase in cross-sectional area will have a larger effect. Because of this the weight reduction will be smaller than the expected 23% for axially loaded components. Girders also require a minimum stiffness to limit the deflections. This could result in stiffness requirements that exceed the strength requirements. In that case it would not be beneficial to use S460.

When tensile members are connected with bolts the section will be weakened by the holes required for the bolts. The design of the member is in this case governed by the tensile strength of the material due to the requirements stated in the Eurocode. For every section with holes for bolts the following formulas are valid:

$$N_{pl,Rd} = \frac{A * f_y}{\gamma_{M0}}$$

$$N_{u,Rd} = \frac{0{,}9 * A_{net} * f_u}{\gamma_{M2}}$$

In these formulas γ_{M0} = 1,0 and γ_{M2} = 1,25. For S355 these formulas can be transformed to:

$$N_{pl,Rd} = 355 * A$$

$$N_{u,Rd} = 352{,}8 * A_{net}$$

When S460 or HISTAR460 is used these formulas become:

$$N_{pl,Rd} = 460 * A$$

$$N_{u,Rd} = 388{,}8 * A_{net}$$

The formulas show that no matter the size of the net section (which cannot be larger than the gross section) the tensile strength will always govern the design. This implies that high strength tensile members cannot be used in areas with seismic activity without strengthening of the members at the connections because deformation capacity is required in these areas. If deformation capacity is required the value $N_{u,Rd}$ must always be larger than $N_{pl,Rd}$ which is never the case if the sections are not thickened at the location of the bolts. It is also visible that S355 also does not comply with the requirements for the deformation capacity. This is because in the Eurocode the tensile strength has been reduced from 510 N/mm² to 490 N/mm². If these steel grades are used in tensile members that require deformation capacity the connections must either be welded or the sections have to be thickened at the location of the bolts. Thickening can be done by welding cover plates to the flange and/or web. When the tensile strength is the governing factor the expected weight reduction will be smaller due to the smaller difference in tensile strengths of S355 and S460:

$$\frac{540-490}{540} * 100\% \approx 9{,}3\%$$

This weight reduction is valid when a section is based on the tensile strength without the use of plates to strengthen the section at the connection. When local thickening of the section is used the savings could be higher but not more than the expected 23%.

The costs of the required material can directly be related to the weight saving. Because of the extra costs of S460 the cost saving on sections are expected to be a bit lower than 23%. Savings could also be higher in certain cases. In trusses the top and lower chords are usually continuous beams. Next to the normal forces in these chords there could also be bending moments present dependent on the stiffness of the chords. If higher strength steel is used the cross-section becomes smaller and so does the bending stiffness resulting in smaller bending moments. This could enable the use of an even smaller cross-section and thus increasing the savings in weight and costs.

Next to material costs it is also interesting to consider the costs of manufacturing the connections, transporting the structural components and the costs to assemble the pieces on the construction site. Fabrication costs involve the cost to drill, saw, weld etc. It basically describes the costs that are necessary to get the structural components ready for assembly. This involves sawing sections, flame cutting plates, welding plates together, drilling holes for bolted connections and also transport inside the factory. The stronger a material is the tougher it gets. It is therefore expected that a section in S460 steel will be more difficult to saw than the same section in S355 steel which could result in an increase in costs. Transportation of higher strength steel could be very economical. Because of the lower weight that needs to be transported it could be possible that less trucks driving to the construction site are needed which would lead to lower transportation costs. If not the weight but the size (complete truss for example) is governing the amount of trucks necessary additional savings cannot be expected. The assembly at the construction site could also be an important factor in the final cost saving. When bolted connections are used the amount of bolts required is independent of the strength class of the connected member but dependent on the strength of the bolts. This means that when bolted connections are used savings cannot be expected when assembling the construction. The downside of high strength steel is the relatively low tensile strength. The ratio fu/fy decreases with stronger steel grades. This means that the net section at a connection in a tensile member is often governing the design and special considerations need to be given in these situations. An alternative would be to weld the connections on site which is expensive because of the necessary precautions that need to be taken. On the other hand the lighter weight of the structural components could prove to be very beneficial on site. The required crane is determined by the heaviest pieces it needs to lift. If the weight of these pieces could be reduced by the use of stronger steel there could be some cost saving by using a cheaper crane that has a lesser capacity.

3.4 Determination of costs

The total costs can be divided into several parts. Material costs cover the costs of the necessary sections. Costs are given per ton steel so the unit weight and the length of the sections are important. Fabrication costs cover the costs that are necessary to make all the adjustments to the sections like sawing and welding. These costs also cover the material costs of the connections so the costs of plates and stiffeners etc are in this category. Transportation costs involve the costs of transportation but also additional costs if roads need to be locked down when big pieces are transported. The assembly costs cover everything at the construction site. This involves the costs of cranes and welding on site etc.

3.4.1 Material Costs

The costs of the sections are based on a price list provided by ArcelorMittal **[9]**. The price list consists of a range of prices that need to be summed. There is a basic price which is valid for certain groups of sections, a size extra which is dependent on the section and a grade extra which is dependent on the steel grade used. There are also length, certificate, quantity extra's etc. but these are not included in the calculation because they depend on the amount of steel used in a certain project. An overview of the used prices (for the Delft/trusses part) is shown in Table 3-4. For plates a price of 950 €/t is used for S355 and a price of 1000 €/t is used for S460.

Section	Basic	Size	Grade	Total
S355J2	[€/t]	[€/t]	[€/t]	[€/t]
HEB260	565	305	50	920
HEB280	565	305	50	920
HEB360	585	315	50	950
HEM260	565	325	50	940
HD400x216	595	330	50	975
HD400x287	595	330	50	975
HD400x382	605	320	50	975
HD400x463	715	320	50	1085
HD400x509	730	350	50	1130
HD400x677	730	350	50	1130
HISTAR460				
HEB220	560	305	100	965
HEB260	565	305	100	970
HEB280	565	305	100	970
HD360x147	595	335	100	1030
HD400x187	595	330	100	1025
HD400x216	595	330	100	1025
HD400x287	595	330	100	1025
HD400x382	605	320	100	1025
HD400x421	715	320	100	1135

Table 3-4: Material costs

It is visible that with increasing size of the sections the basic price increases. The size extra does not vary that much. The grade extra is based on the price of S235JR/J0. It is visible that HISTAR460 is indeed 5 cent per kilo more expensive than S355J2. S355J2 is indicated because it is used often in the steel structures considered in the projects and because it has similar fracture toughness properties as HISTAR460. From the table it is visible that a higher strength section could even be cheaper per ton steel. This could result in an increase in cost saving because of the reduction in basic price and a slight reduction in size extra. An HD400x463 column in S355 could easily be replaced by an HD400x382 column in S460. Not only would the weight of the column decrease but also the price per ton which indicates that in this case the use of stronger steel could be very economical.

3.4.2 Fabrication costs

The fabrication costs are summed up in the corresponding chapters. The costs are based on the amount of hours necessary to complete a certain activity and the hourly fees corresponding to those activities. Implemented in the amount of hours required is the machinability of the materials. Higher strength steel is harder so it is more difficult to perform operations such as drilling. This is factored in by increasing the amount of hours required by 5% compared to S355 for sawing, drilling and flattening of column ends. 5% seems like an overestimate but it also includes the wear of the machinery. Due to the increased hardness of the material the equipment used for sawing, drilling etc. results in an increased wear which results in increased fabrication costs. Costs for positioning, preparing and welding plates, flame cutting, shot blasting, and internal transport are also reviewed however a 5% increase for the required amount of hours is not applied. The amount of hours required is based on the thickness of plates, size of sections, volume of the welds and the length of individual elements. For sawing not only the surface area of the cross-section is relevant but also the thickness of the material. By evaluating both properties the required time for sawing a certain section is determined. The time required for drilling depends on the thickness of the material and the amount of bolts required. The time required is not assumed to have a linear relation between the thickness and/or the amount of bolts required. This is because setting up the machine and positioning of the material has to be factored in. When a few holes are required in thin material these factors become more significant than when a lot of holes are required in thick material, therefore drilling 4 holes in 10 mm thick material is not 6 times as fast as drilling 12 holes in 20 mm thick material. This is because relatively more time is required for setting up the drilling process. When column ends are flattened for contact pressure the positioning of the sections will take the most time. This is because it has to be done very precisely which takes a lot of time. The required time for flattening of column ends is therefore assumed to be constant regardless of the section used. Weld preparation is done with the use of flame cutting. The plates are cut in an angle of 45 degrees on both sides. The positioning and set-up of the machine takes more time than regular flame cutting so more time is required compared to flame cutting. The speed of flame cutting and weld preparation is dependent on the thickness of the material. Welding is done with the use of layers. For small fillet welds (a = 5 mm) a single layer weld will suffice. For large beveled welds much more layers are required which significantly increase the time required for welding. The welds are all categorized to their size and length. For large welds the time required for setting up the plates and cleaning of the welds become less significant. Therefore for large welds a higher welding speed is used to calculate the required time. For small welds this speed is lower because the set-up and cleaning takes relatively more time. The time required for shot blasting also independent of the section used because a constant speed is used for shot blasting. The time required for internal transport is based on the total weight of all the components. By using hourly fees for the different processes the total fabrication costs are calculated. The costs will probably differ from the actual fabrication costs because a generalization has been made. If the fabrication costs were examined for each component separately the calculated costs would be closer to reality. The difference in costs between the various components is assumed to be relatively low. This and the fact that calculating the costs for each individual component and process is impractical has led to the generalization of the costs calculation.

3.4.3 Transportation costs

The transportation costs are dependent on the total weight of the structural components, their size and the distance. When size is not governing the trucks can be loaded to their capacity. Trucks will generally have a capacity of 25 t. Because the full capacity will never be utilized it is assumed that trucks are all loaded up to 20 t. In some cases only parts of the structures are reviewed. When a part weights 50 t this would mean that 3 trucks have to be used. If a structure would have 10 of these parts the total weight would be 500 t and 25 trucks would be required to transport the structural components. The review of a single part in this case would lead to higher transportation costs because the amount of trucks required would have to be an integer. When only parts of a structure are reviewed the amount of trucks required are therefore not rounded because in reality the trucks could also carry the other parts as well.

When the size is irrelevant the transportation costs are determined by the distance traveled and the amount of trucks required. When the distance is large the costs are also large but the costs per km will become smaller as shown in Table 3-5.

Distance	Costs per truck		Distance	Costs per truck	
[km]	[€]	[€/km]	[km]	[€]	[€/km]
20	206	8,60	200	524	2,19
40	253	5,28	240	559	1,94
60	295	4,10	280	629	1,87
80	337	3,51	320	702	1,83
100	365	3,04	360	778	1,80
125	409	2,73	400	826	1,72
150	432	2,40	450	892	1,65
175	486	2,31			

Table 3-5: Transportation costs per truck [source: Rijnart transport 2012]

It is visible that weight reduction becomes more significant when the distance is large. These costs are only valid when the distance is relatively small. In some cases the structural components have to be transported overseas. In these cases transportation costs become relatively larger.

3.4.4 Assembly costs

The assembly costs of structures are dependent on the weight of the individual components, the height of the structure, the use of welds on site, the size of connections, the available space and of course the weather. The weight is important because it determines the size of the required crane. A weight reduction could lead to the use of smaller cranes which is favorable because that would lead to a reduction in costs. The height of the structure is important because it increases the time required for a certain component to be lifted in place. For instance it takes more time to lift a column to the 20th floor than to lift it to the 2nd floor. The available space is of importance because it could lead to unfavorable crane positions which would increase the costs because heavier cranes would be required. When the structure needs to be welded on site special precautions have to be taken. Sometimes a welding tent could be required to guarantee a good quality weld. Because of the required precautions welding on site is very expensive hence bolted connections are usually preferred. When large connections are present it takes more time to finish them. A relatively light weight connection with 10 bolts is made much faster and with much more ease than a heavy connection with 50 bolts. All of these aspects result in a certain time required to complete a certain part of the structure. When the amount of time required to assemble (a part of) the construction is known the costs can be calculated by using hourly fees. The amount of hours required are dependent on the amount of construction workers assigned to that certain part. The required crane is dependent on the weight of the heaviest part it needs to lift and on the distance between the crane and the location of where that part needs to be. Lifting capacities of different cranes at different distances are shown in Table 3-6. These capacities are valid for mobile hydraulic cranes **[10]**.

	Distance [m]					
Crane	18	20	22	24	26	28
50t	5,1	4,3	3,7	3,1	2,6	2,2
60t	6,8	5,8	4,9	4,2	3,5	3,1
70t	7,2	6,1	5,2	4,6	4,0	3,4
80t	9,2	8,0	6,8	5,9	5,1	4,3
100t	10,0	8,4	7,2	6,2	5,4	4,8
120t	13,0	11,1	9,7	8,5	7,7	6,8
140t	14,7	12,7	11,1	9,7	8,5	7,7
160t	18,8	16,1	14,1	12,2	10,8	9,5
400t	70,0	63,0	56,0	50,0	45,5	41,5
500t	87,3	77,4	70,2	64,0	58,1	53,1
700t	109,0	98,0	86,0	79,0	72,0	65,5

Table 3-6: Crane capacities [t]

The costs of the cranes are dependent on the hourly fees of the cranes used. These hourly fees are transformed to daily fees because cranes used for construction are generally used for several weeks. The costs for the different cranes are shown in Table 3-7.

	Costs	
Crane	[€/hr]	[€/d]
50t	95,00	760,00
60t	105,00	840,00
70t	135,00	1080,00
80t	145,00	1160,00
100t	155,00	1240,00
120t	195,00	1560,00
140t	220,00	1760,00
160t	250,00	2000,00
400t	430,00	3440,00
500t	575,00	4600,00
700t	700,00	5600,00

Table 3-7: Crane costs

Other cranes are not relevant for the projects examined in this paper. This is mainly due to the capacity required and the time needed for all the relevant parts. Crawler cranes for example need to be mobilized and demobilized which increases the costs significantly, also they have to be rented for at least 4-8 weeks which is too long for the relevant cases studied. The mobilization costs for hydraulic cranes is only relevant when the cranes are rented for a few hours. As mentioned earlier in construction these cranes are rented for several weeks which makes the mobilization costs for these cranes irrelevant.

At the Delft project boom lifts are also required. The required boom lifts have a working height of 22 m. The costs are 430 € per day or 1250 € per week **[11]**.

4 Trusses: Delft railway station and offices

4.1 Introduction

In Delft a new railway station is under construction. Currently there is an old station with railway tracks above ground. In the future these tracks will be located below ground level in a tunnel. Above this tunnel on the ground floor the new station will be built. The building will however have 4 more floors to accommodate offices. The first to third floor will be used as office space and the fourth floor will be used for installations. These floors are supported by large trusses because of the tunnel underneath. The free space required for the station and the location of the tunnel walls prevent the use of multiple columns at the ground floor. For spans greater than 40 m only 2 columns can be used with a fixed position. This is why trusses are used. 14 trusses are used to support the part of the building above the tunnel. The spacing between these trusses is 8,1 m. About half of the trusses have end supports. The others have supports above the middle wall of the tunnel. These supports are moved inward resulting in a cantilever. Because of the required free space and the unfavorable positions of the inward supports extra columns are required in the trusses resulting in an uneconomical but necessary design. The concepts of both trusses are shown in Figure 4-1.

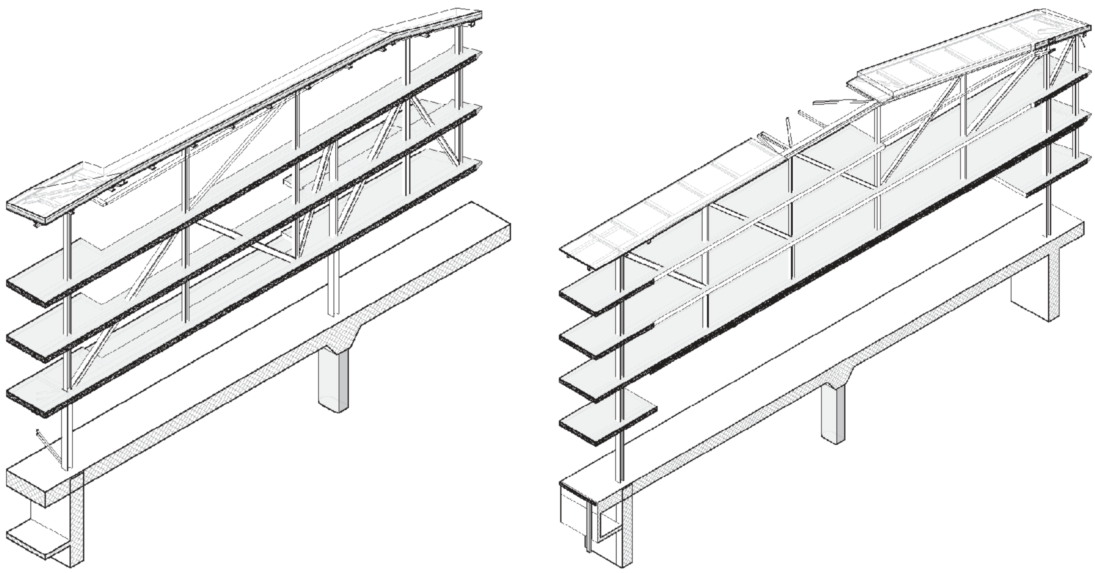


Figure 4-1: Concepts of cantilever and simply supported truss [source: ABT]

The trusses have a height of 2 storey’s (7 m) so between the top and bottom chords other horizontal beams are present as well. These beams (IFBA) are simply supported between the verticals and diagonals. On each floor hollow core slabs are used that span between the trusses. The slabs are also used to stabilize the chords against lateral torsional buckling. The buckling length about the weak axis however is not decreased by the slabs.

In order to make a proper comparison 2 different trusses are examined. Both trusses have different support conditions (one with both end supports and one with an inward support). The trusses considered are the heaviest loaded trusses and they happen to be adjacent. The truss with the inward support is the truss with the largest cantilever. Because all the trusses are alike it is not expected that individual trusses with the same support conditions give different cost savings, hence only 2 trusses are examined. The locations of the trusses and their supports are shown in Figure 4-2.

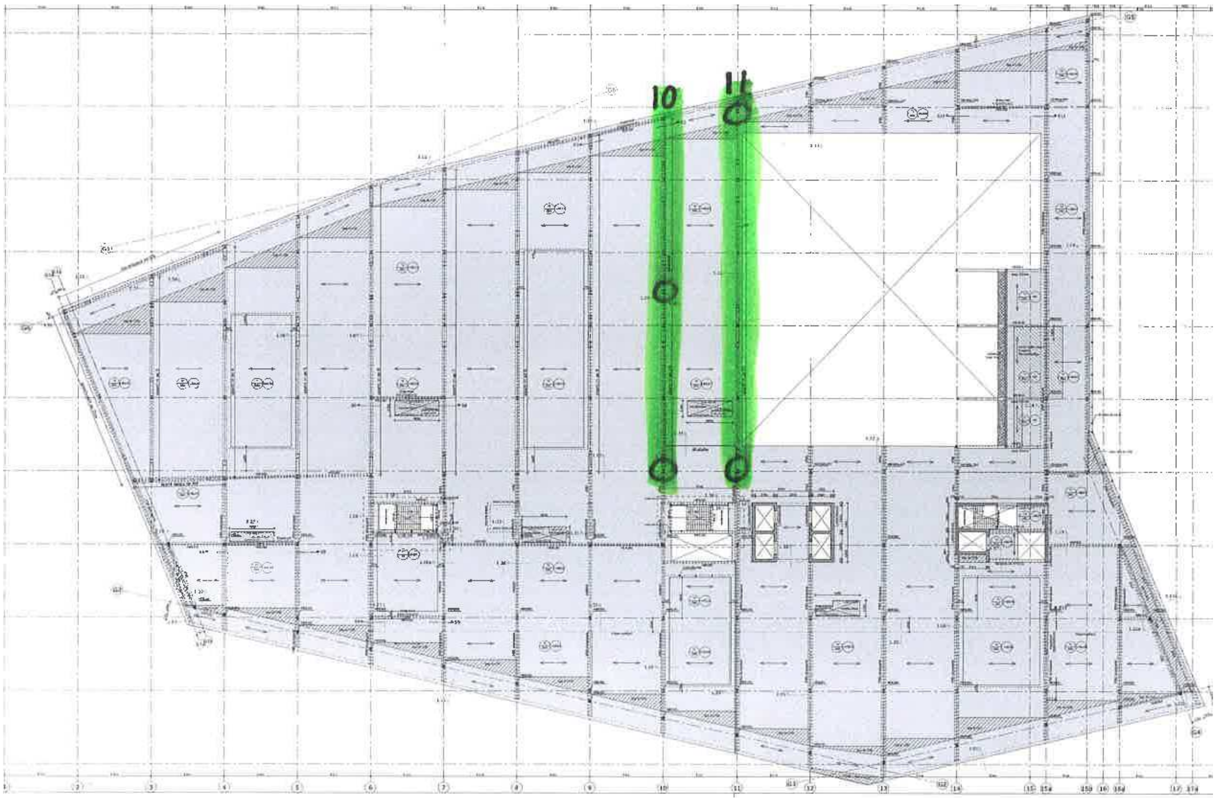


Figure 4-2: Truss and support locations [source: ABT]

The length of these trusses is about the same so the influence of the support conditions can also be considered when comparing the final costs. The used trusses are visible in Figure 4-3 and Figure 4-4. The names correspond to the location on the grid.

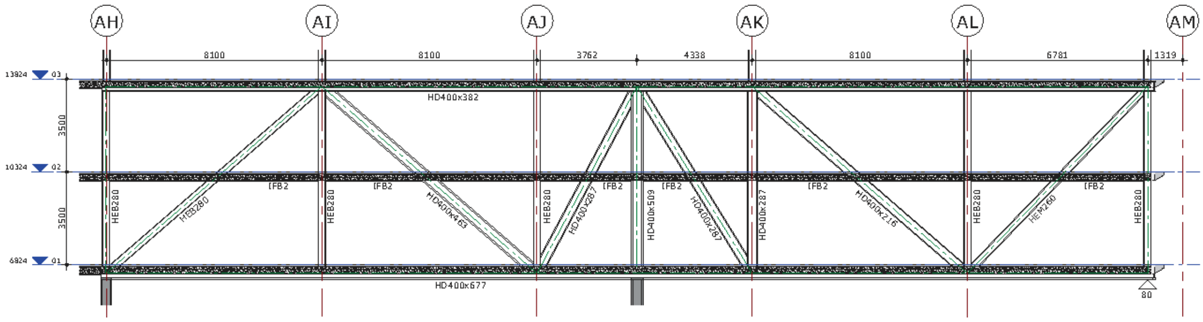


Figure 4-3: Truss 10 [source: ABT]

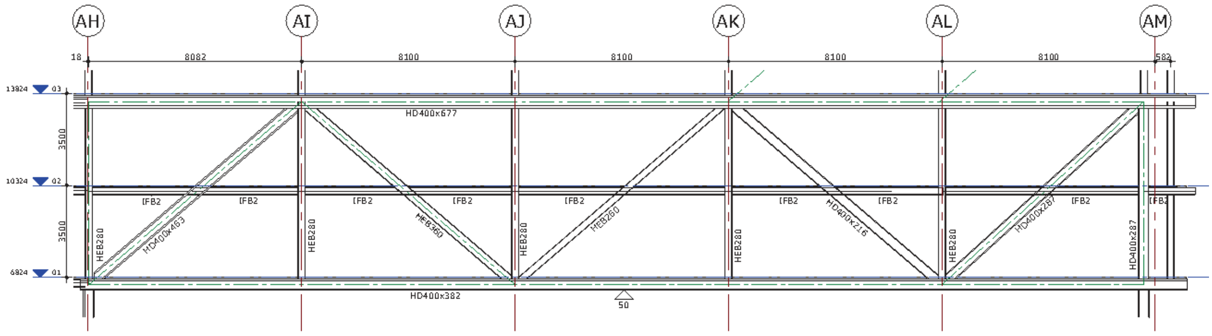


Figure 4-4: Truss 11 [source: ABT]

Nearly all loads are acting as point loads in the nodes of the trusses. The top and bottom chords are also loaded by a distributed force so these cords need to be designed by taking account of both an axial force and a bending moment. The total load consists primarily of permanent loads. The hollow core slabs have a height of 260 mm resulting in a relatively high dead load. Used loads are:

Permanent loads:

- Hollow core slab d=260mm	3,80 kN/m^2
- Screed d=70mm	1,70 kN/m^2
- Floor covering d=70mm	1,40 kN/m^2
- Ceiling, pipes	0,50 kN/m^2

Variable loads:

- Office floor	4,00 kN/m^2
- Installations	5,00 kN/m^2

The calculations of the acting forces in the structural components of the trusses are made by using SCIA Engineer. The relevant members of the 2 considered trusses are shown in Figure 4-5 and Figure 4-6.

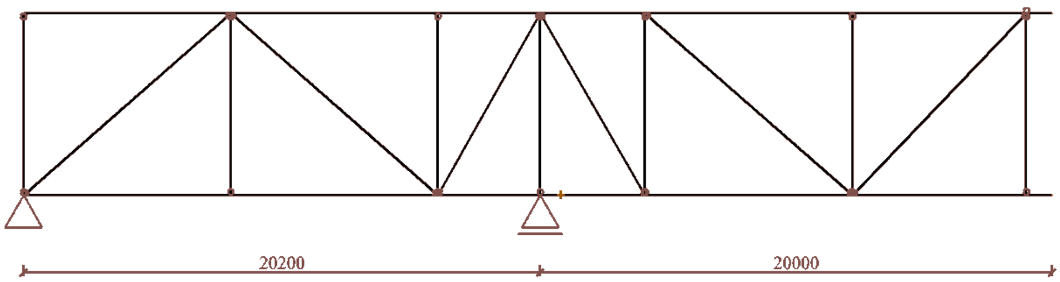


Figure 4-5: Truss 10

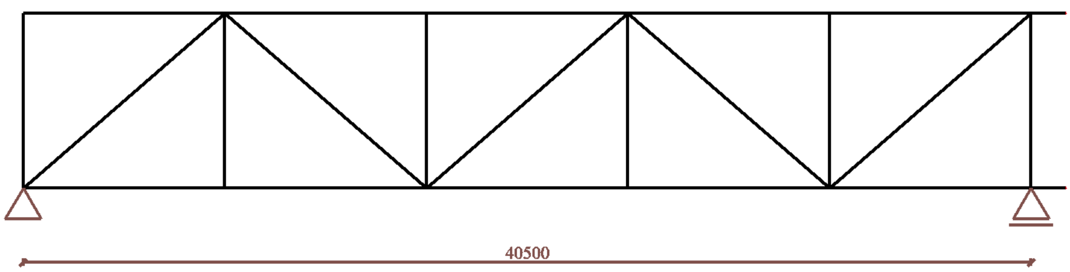


Figure 4-6: Truss 11

In the current design all components of the trusses are made with the S355J2 steel grade. The vertical and diagonal members are primarily loaded by an axial force. Because of the simply supported beams located between the top and bottom chords it is possible for bending moments to occur in the diagonal members. These bending moment are not large enough to cause a significant impact on the design of the members. Because of this all vertical and diagonal members are considered to be loaded by an axial force only. Because of the distributed load on the top and bottom chords not only an axial force is present but also a bending moment and a shear force. The bending moment has a significant effect on the required section especially in compressed members (depending on the location of the supports this could be the top or bottom chord).

A first impression of the cost saving can be made by comparing the required sections and their costs. In order to do that the required sections when S460 is used have to be determined. The next step is to design the connections. The connection type is dependent on the transportation. If the truss can be transported in large pieces welded connections could be an interesting connection method because the sections would not be weakened by holes for bolts. In this case transportation of large parts is not possible so the truss has to be bolted or welded on site. Bolted connections are preferable in this case because of the ease of assembly. Welded connections could be very expensive because of the precautions that are necessary. Because large parts cannot be transported the top and bottom chords have to be split in parts as well. It is necessary to divide these chords in 2 pieces. The fire resistance of the structure is ensured by using timber components surrounding the steel.

4.2 Truss design

For each truss the required sections are determined by using the guidelines provided in the Eurocode. The used forces are exactly the same as the ones used in the current design (S355). These forces are based on the last Dutch code (TGB) **[12]**, to make an accurate comparison the same forces have to be used when redesigning the structure with the use of S460. When all components are checked a first impression of the savings can be made. The savings are based on the weight difference between the 2 separate designs. As mentioned earlier the expected savings will amount to about 23%.

Because of the lower tensile strength/yield strength ratio holes will have a more significant effect on high strength steel sections. Thickening of tensile members at the joints is therefore required more often when using S460. In these cases it might be favorable to upsize the section to remove the need for thickening. This is only the case when the additional costs for the larger sections are smaller than the additional costs required to thicken the sections. If the sections are thickened the material costs will be lower but the fabrication costs will increase. If the sections are upsized the material costs will be larger but additional fabrication costs are not required. The additional weight is usually small compared to the loads present so extra costs to take account of this extra weight are probably not required. It could however have an effect on the assembly of the structure. Heavier sections require larger equipment which could lead to an increase in costs.

The calculation of the governing forces in the ultimate limit state (ULS) has been done by using 37 different load combinations each consisting of 12 load cases. The high amount of load combinations is due to the amount of floors and the different loads on these floors. The cantilever also increases the amount of load combinations because variable loads can be separated in 2 parts: an internal part (between the supports) and an external part (at the cantilever). The governing deflection is calculated by using 13 different load combinations. The decrease in load combinations is caused by the use of the serviceability limit state (SLS). When using the SLS the partial safety factors are all considered to be 1,0 causing the reduced amount of load combinations. A load combination including only permanent loads is also used to determine the permanent deflection. This permanent deflection is used to determine the required camber.

4.2.1 Truss 10

Truss 10 is the largest cantilever truss and it is located on the 10^{th} axis of the grid hence the name truss 10. It has 7 verticals and 6 diagonals. The length of the cantilever is 20 m (Figure 4-5). Element numbers and dimensions used in the model are displayed in Figure 4-7.

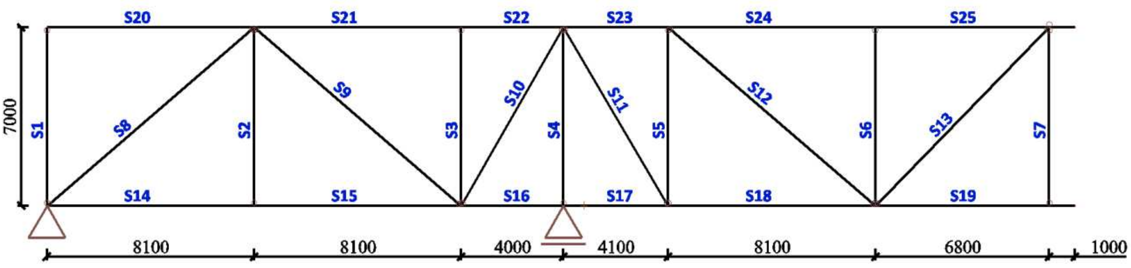


Figure 4-7: Element numbers and dimensions truss 10

In future references the terms top chord, bottom chord, vertical 1-7 and diagonal 1-6 are used to describe the different elements of the truss. The bottom chord consists of elements S14-S19, the top chord: S20-S25. Verticals and diagonals are described from left to right by their corresponding element numbers so vertical 1's element number is S1 and diagonal 1's element number is S8.

4.2.1.1 Sections

Because of the relatively large height of the truss the stiffness will be large as well. This implies that the required sections for the chords should be based on the acting forces. In the current design the largest deflection due to permanent loads is 80 mm. This deflection is located at the end of the cantilever. This deflection is counteracted by using a camber of 80 mm. The additional deflection due to the variable loads is equal to 30 mm. The limits for the deflection are governed by the following formulas **[13]**:

$$u_{end} = 0{,}004 * l_{rep}$$

$$u_{add} = 0{,}003 * l_{rep}$$

L_{rep} is equal to the length of a simply supported beam or twice the length of a cantilever. In this case the cantilever has the largest deflection which makes L_{rep} equal to 2x20 = 40 m. u_{end} is in this case 160 mm and u_{add} is equal to 120 mm. These values indicate the limits for the final deflection and the additional deflection. The final deflection (u_{end}) is the deflection caused by all loads including the used camber. The additional deflection (u_{add}) is the deflection caused by the variable loads only. Both the final and additional deflection are in this case 30 mm which is well below the limits. In the new design the local deflection between the verticals is also considered. If the global stiffness is adequate it does not necessarily mean that the local stiffness is also high enough. Due to the distributed loads on the top and bottom chords the chords can also deflect between the verticals. The largest distance between the verticals is 8100 mm. It is difficult to use a camber for every part between the verticals so it is assumed that the use of a camber to counteract local permanent deflections is not possible. Due to the relatively low variable loads the final deflection will be the governing factor for the minimum required stiffness. The maximum allowable deflection u_{end} is in this case equal to 32,4 mm.

Although it could be possible that the deflection is the governing factor for the choice of sections, strength is still considered first. The current design consists of HEB280 and larger sections. For some verticals smaller sections could have been used but this would alter the possibility of making certain connections. The HEB280 sections allow bolted connections in which the shear resistance of the bolts is governing. If smaller sections were chosen the bearing strength could have been governing resulting in the use of more bolts and thus longer connections. In order to keep the connections as small as possible the designer chose to use HEB280 sections instead. These sections are also used for most of the other columns in the building. The redesigned truss consists of HEB220 and larger sections. Although the bearing strength could be the governing factor this lighter section is chosen to investigate the influence on the costs. Because of the higher tensile strength the bearing strength will also be larger which makes it possible to reduce the size of sections. Smaller sections cannot be used due to the minimum edge distance required for bolted connections. Also the use of HEB220 sections leads to similar unity checks as the HEB280 sections in the current design. This increases the accuracy of the cost reduction.

In order to redesign the truss using S460 the relevant forces are required. The verticals and diagonals can be designed by using the axial force only. The top and bottom chords also require the relevant shear force and bending moment. To obtain the critical cross-section for the top and bottom chords the forces must be displayed for the entire truss (Figure 4-8 - Figure 4-10).

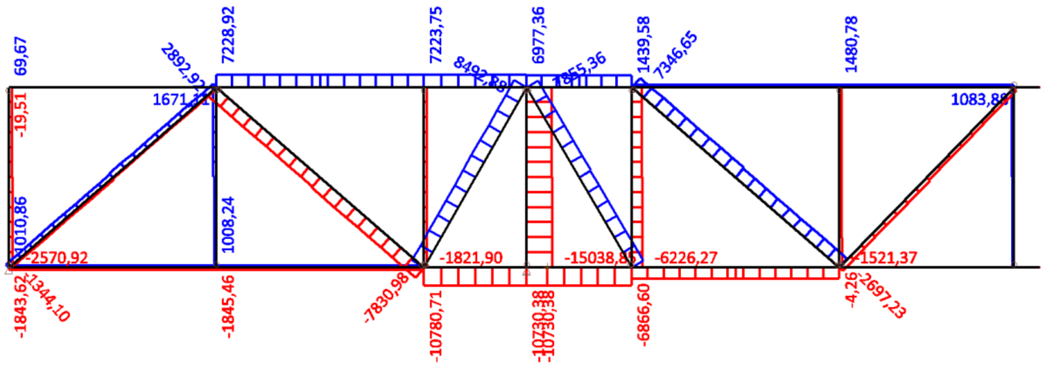


Figure 4-8: Axial force truss 10

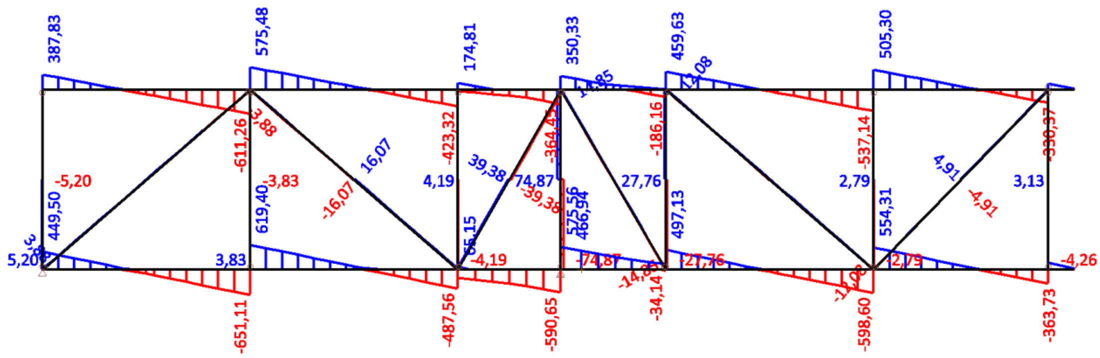


Figure 4-9: Shear force truss 10

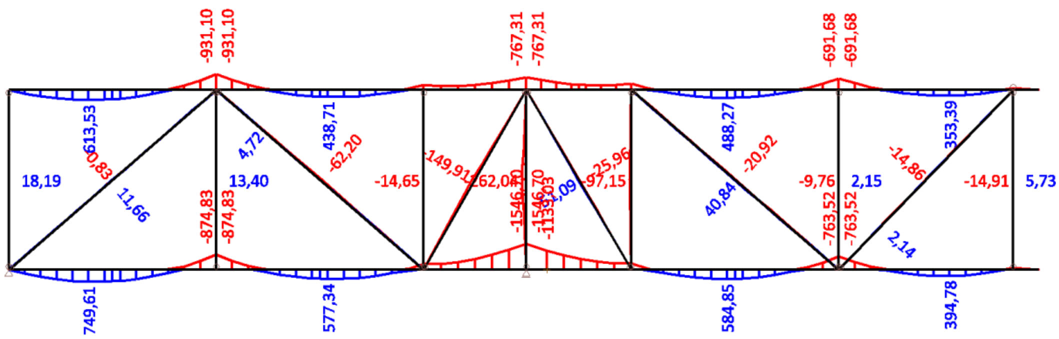


Figure 4-10: Bending moment truss 10

The governing cross-section of the top chord is just to the right of vertical 2. All forces are critical in this position. The buckling lengths are also critical in this position which confirms the critical cross-section. The governing cross-section of the bottom chord is just right of vertical 4. The axial force and bending moment are critical in this position. Although the buckling length of the chord is shorter in this part of the truss the forces in the other parts with a greater buckling length are too small to be governing. The forces used to calculate the required cross-sections are shown in Table 4-1.

	Nmin [kN]	Nmax [kN]
Vertical 1	-2684	-1340
Vertical 2	732	1769
Vertical 3	-1883	-999
Vertical 4	-15253	-11111
Vertical 5	-6418	-3784
Vertical 6	-1490	-547
Vertical 7	39	1114

	Nmin [kN]	Nmax [kN]
Diagonal 1	-1476	2751
Diagonal 2	-8229	-4955
Diagonal 3	6520	8543
Diagonal 4	4882	7930
Diagonal 5	4058	7468
Diagonal 6	-2821	-1013

	N [kN]	Vz [kN]	My [kNm]
Top chord	6794	611	1020
Bottom chord	-10964	842	2247

Table 4-1: Forces in current design

After the new sections had been entered and the force flow of the truss was recalculated there appeared to be differences in the acting forces. This was especially noticeable in the top and bottom chords because the acting bending moment had been reduced significantly. Some of the verticals and diagonals appear to be loaded more heavily. Because of these differences the sections have been checked with the forces resulting from the recalculation. These forces are shown in Table 4-2.

	Nmin [kN]	Nmax [kN]
Vertical 1	-2571	-1237
Vertical 2	645	1671
Vertical 3	-1822	-958
Vertical 4	-15039	-10819
Vertical 5	-6226	-3613
Vertical 6	-1521	-566
Vertical 7	18	1084

	Nmin [kN]	Nmax [kN]
Diagonal 1	-1344	2893
Diagonal 2	-7831	-4666
Diagonal 3	6543	8493
Diagonal 4	4805	7855
Diagonal 5	3913	7347
Diagonal 6	-2697	-934

	N [kN]	Vz [kN]	My [kNm]
Top chord	6630	575	931
Bottom chord	-10714	576	1547

Table 4-2: Forces in new design

It is also important to notice the difference between the maximum and minimum acting force in a member. If both tension and compression can be present in a certain section the connection has to be modified accordingly. In this case only diagonal 1 can have both tension and compression.

The calculation of the unity checks has been split in 2 separate parts. The verticals and diagonals are considered to be loaded by an axial force only. In this case a simplified calculation can be made for these elements. For the top and bottom chords more extensive calculations are required because there is an interaction between an axial force and a bending moment. As mentioned earlier the hollow core slabs support the top and bottom chords against lateral torsional buckling. In the calculations made for the current design a lateral torsional buckling length of $0{,}1xL_{sys}$ is used. The effective lateral torsional buckling length is 810 mm and this length is also used in the calculation of the shorter parts at the inward support. The buckling length of the chords is equal to the length between the corresponding verticals. The buckling length for the verticals and diagonals is $0{,}5xL_{sys}$. This reduction of the buckling length is because the floor between the top and bottom chords supports all verticals and diagonals against buckling in both directions.

The calculations of the unity checks are shown in Appendix I. The chords are reviewed separately because more extensive calculations are required. The sections used for the verticals and diagonals are all checked once for the maximum occurring forces because there's no need to check similar sections with a smaller force. The section checks have been made according to NEN-EN 1993-1-1 sections:

- 6.2.3: Tension
- 6.2.4: Compression
- 6.2.5: Bending moment
- 6.2.6: Shear force
- 6.2.9: Combined axial force and bending moment
- 6.3.1: Compression (stability)
- 6.3.3: Combined compression and bending moment (stability)
- Appendix B: Interaction factors for combined compression and bending moment (stability)

The interaction factors used in the calculations of combined compression and bending moment are dependent on the compressive force present and on the type of system. For non-sway systems the factors are also dependent on the bending moment diagram. In this case the values of the bending moments at 3 different positions are required. The trusses considered are classified as sway systems in plane of the truss and non sway out of plane because all nodes are restrained in the horizontal direction but not in the vertical direction.

The described sections all fulfill the strength requirements written in the Eurocode. The deflection should also be smaller than the described limits. If the deflection is too high and camber is not an option it means that the top and bottom chords need to be stiffer. There is no need to choose stiffer verticals and diagonals since their effect on the stiffness of the truss is far less than the effect of the top and bottom chords. The deflection is calculated by using SCIA Engineer (Figure 4-11).

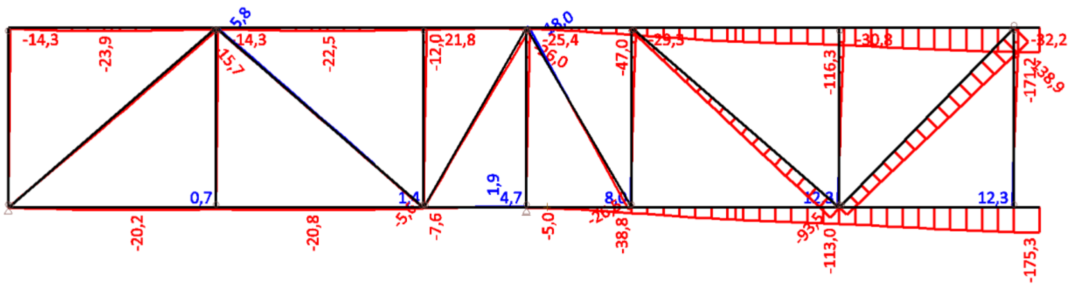


Figure 4-11: Deflection truss 10

The final deflection (u_{end}) is 175,3 mm. This value is larger than the allowed deflection (160 mm) but a large part of the deflection can be counteracted by using a camber. The deflection caused by the permanent loads is 125,4 mm. To compensate for this relatively large deflection a camber of 120 mm is used. Because of the camber the final deflection is no longer the governing factor for the global deflection check. The additional deflection is 49,9 mm, this is well below the limit of 120 mm. The local deflection is checked by using relative displacements. These are the displacements of the individual elements between the nodes (Figure 4-12).

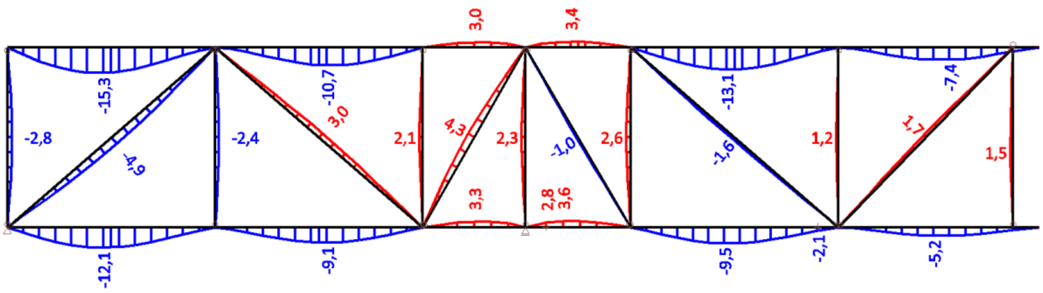


Figure 4-12: Relative displacement truss 10

It is visible that the maximum value of the relative displacement is 15,3 mm. The local stiffness requirement is met because the maximum relative displacement is smaller than the limit of 32,4 mm. The relative deformation has been calculated by using all the loads because a camber cannot be used to reduce the permanent deflection. The truss is stiff enough to limit the global deflection. This is mainly because of the large height of the truss and the placement of the supports.

Not only the deflection but also the maximum rotation should be limited. For simply supported beams the stiffness requirement limits the allowable deflection. Due to this limit the rotation at the supports is also limited. In this case the rotation depends on the local deflection and the global deflection which complicates the calculation of the rotations. A conservative estimation based on the deflections can however be made. The relative displacement in this case is about 50% of the allowed value. Because the maximum allowable rotation can be related to the maximum allowable deflection it is assumed that 50% of the allowable rotation occurs due to the local deflection. This implies that the rotation due to the global deflection should not be more than 50% either so the total rotation is less than the allowable rotation. This is illustrated in Figure 4-13. In this figure an assumed global and local deflection is shown. The values on the vertical axis show the unity checks for the global deflection. When the global deflection is reviewed one would conclude that the deflection is (in this case) 50% of the allowable deflection. The same would apply for the local deflection (also 50% of the allowable local deflection which is not visible in the figure). When looking at the rotations the local and global rotations are both 50% of their limit (because they are based on the deflections), but when these rotations are combined it can be shown that the total rotation at the left and right end are at 100% of the allowable value. By simply adding up the unity checks for the local and global deflection the right unity check for the rotation is obtained. The same can be applied when examining the trusses however it is not always the case that the maximum global rotation occurs at the same place as the maximum local rotation. This approach is therefore conservative. In this case the unity check would be: 49,9/120 + 15,3/32,4 = 0,89. This value is much larger than the ones obtained when reviewing the global and local deflections separately and it shows that the stiffness could be significant when designing trusses with the use of higher strength steel.

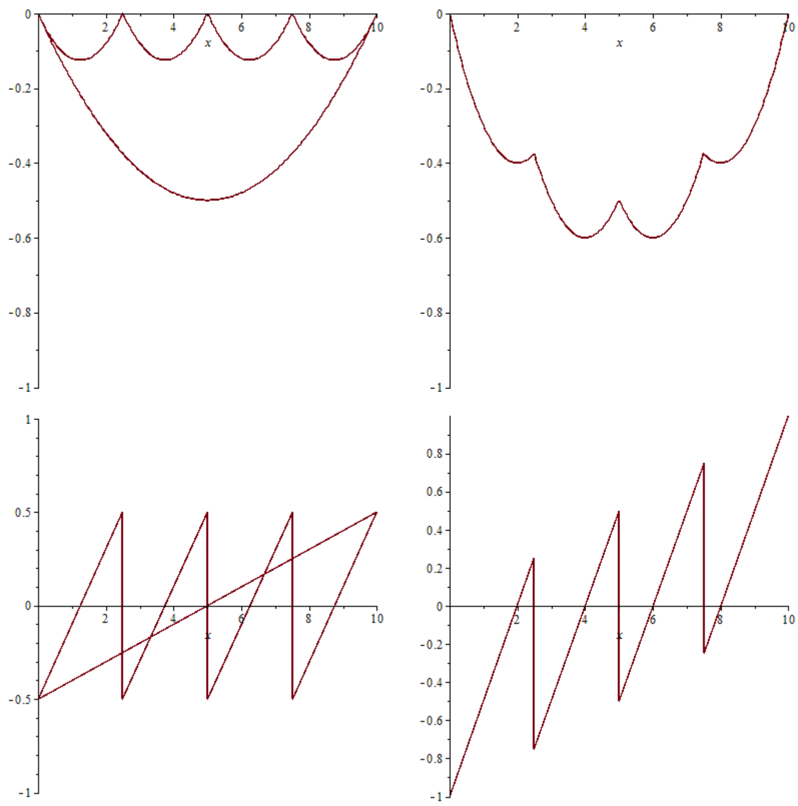


Figure 4-13: Global and local deflections (top) and rotations (bottom)

The used sections in the current design are displayed in Table 4-3. For each element the length, weight per meter, the unity check, the direct weight and the modified weight are displayed. The modified weight takes account of the difference in the unity checks between the current and the new design. This modification has been made because the difference between the unity checks can be substantial. If these differences are neglected the saving would be based on a faulty comparison and could therefore turn out to be more favorable than the actual possible saving. The sections in the new design result in generally higher unity checks which results in an increased weight saving. To take account of the different unity checks the direct weight is rescaled by multiplying it with the according unity check. This results in a modified weight which is always lower than the direct weight. By using the modified weights a more accurate comparison can be made. The unity checks used to calculate the modified weights are based on the TGB (last Dutch code) **[14][15]** because that is the code which was used to calculate the unity checks for the current design.

Weight [kg] 66514,2 S355

	Section	Length [m]	Weight [kg/m]	Unity Check	Weight [kg] Direct	Weight [kg] Modified
Top chord	HD400x382	40,2	382,4	0,74	15372,5	11375,6
Bottom chord	HD400x677	40,2	677,8	0,83	27247,6	22615,5
Diagonal 1	HEB280	10,706	103,1	0,62	1103,8	684,3
Diagonal 2	HD400x463	10,706	462,8	0,54	4954,7	2675,6
Diagonal 3	HD400x287	8,062	287,5	0,66	2317,8	1529,8
Diagonal 4	HD400x287	8,112	287,5	0,6	2332,2	1399,3
Diagonal 5	HD400x216	10,706	216,3	0,76	2315,7	1759,9
Diagonal 6	HEM260	9,759	172,4	0,62	1682,5	1043,1
Vertical 1	HEB280	7	103,1	0,76	721,7	548,5
Vertical 2	HEB280	7	103,1	0,38	721,7	274,2
Vertical 3	HEB280	7	103,1	0,5	721,7	360,9
Vertical 4	HD400x509	7	509,5	0,82	3566,5	2924,5
Vertical 5	HD400x287	7	287,5	0,61	2012,5	1227,6
Vertical 6	HEB280	7	103,1	0,43	721,7	310,3
Vertical 7	HEB280	7	103,1	0,24	721,7	173,2

Modified weight [kg] 48902,4
Average unity 0,74

Table 4-3: Used sections current design (S355)

Weight [kg] 38961,4 S460

	Section	Length [m]	Weight [kg/m]	Unity Check	Weight [kg] Direct	Weight [kg] Modified
Top chord	HD400x216	40,2	216,3	0,94	8695,3	8173,5
Bottom chord	HD400x382	40,2	382,4	0,91	15372,5	13989,0
Diagonal 1	HEB220	10,706	71,5	0,79	765,5	604,7
Diagonal 2	HD400x187	10,706	186,5	0,92	1996,7	1836,9
Diagonal 3	HD400x187	8,062	186,5	0,82	1503,6	1232,9
Diagonal 4	HD400x187	8,112	186,5	0,72	1512,9	1089,3
Diagonal 5	HD400x187	10,706	186,5	0,67	1996,7	1337,8
Diagonal 6	HEB260	9,759	92,9	0,83	906,6	752,5
Vertical 1	HEB220	7	71,5	0,87	500,5	435,4
Vertical 2	HEB220	7	71,5	0,4	500,5	200,2
Vertical 3	HEB220	7	71,5	0,61	500,5	305,3
Vertical 4	HD400x382	7	382,4	0,77	2676,8	2061,1
Vertical 5	HD360x147	7	147,5	0,86	1032,5	888,0
Vertical 6	HEB220	7	71,5	0,51	500,5	255,3
Vertical 7	HEB220	7	71,5	0,26	500,5	130,1

Modified weight [kg] 33292,0
Average unity 0,85

Table 4-4: Used sections new design (S460)

The new design is made by using the steel grade S460. The used sections in this case are displayed in Table 4-4. The final savings have been calculated for each individual element to display individual differences if they are present. The savings are displayed in Table 4-5.

Direct saving [%]: 41,4

	Saving [%]	
	Direct	Modified
Top chord	43,4	28,1
Bottom chord	43,6	38,1
Diagonal 1	30,6	11,6
Diagonal 2	59,7	31,3
Diagonal 3	35,1	19,4
Diagonal 4	35,1	22,2
Diagonal 5	13,8	24,0
Diagonal 6	46,1	27,9
Vertical 1	30,6	20,6
Vertical 2	30,6	27,0
Vertical 3	30,6	15,4
Vertical 4	24,9	29,5
Vertical 5	48,7	27,7
Vertical 6	30,6	17,7
Vertical 7	30,6	24,9

Modified saving [%]: 31,9

Table 4-5: Weight saving

The modified saving is 31,9% in this case. This is about 1,5 times higher than the expected saving. This increase in saving is caused by the high savings in the top and bottom chords. Because of the use of heavy sections and the great length the chords have a large influence on the total saving. The savings of the chords are larger than expected because smaller bending moments are present in the critical cross-section. The cause of the lower bending moment is the decreased stiffness of the chords. Normally the bending stiffness does not have an influence on the bending moment but in this case there is an interaction between bending and axial elongation of the truss's elements. The bending stiffness has decreased more than the axial stiffness resulting in a decrease in shear force being transferred to the top and bottom chords at the nodes. This decrease in shear force results in a decrease in bending moment. Because of the decreased stiffness the hogging moment above the inward support is lower. This reduction in bending moment allows for more slender sections. It is visible that there is quite a large spread in the savings of the individual components. This spread is caused by the difference in resistance between tensile and compression members of the truss. Tensile members are designed by using the tensile strength reduced by the relevant factors required to calculate the resistance at the connections. This method is used to reduce or even prevent the use of cover plates at the connections. Structural components loaded by a compressive force require additional stability checks. To take account of stability issues a reduction factor is used in the calculations. This reduction factor is dependent on the used section and is therefore hard to predict.

Some elements turn out to have a low unity check. The scaling could cause the modified saving to be inaccurate in these cases. This is especially the case when a compression force is present. The scaling does not take account of stability and assumes a linear relation between force and cross-sectional area. If the unity check of a certain element is 0,5 the modified weight is based on a section that is twice as small. If the cross-section becomes smaller the radius of gyration ($\sqrt{I/A}$) will also decrease. This is because the moment of inertia (I) relatively decreases more than the cross-sectional area (A). If the radius of gyration decreases the relative slenderness will increase causing the reduction factor for buckling to become smaller which allows a lower stress n the section. In this case the modified weight is an underestimate of the actual required weight to support the compressive force. Because the unity checks of the current design (S355) are generally lower than the unity checks in the new design (S460) the underestimation of the required weight in the current design will be larger. This implies that the modified saving is an underestimate of the possible saving. The differences in the unity checks are considered small enough to neglect the influence on the final saving. This assumption is based on the fact that the radius of gyration does not decrease much when smaller sections are used. Because of this the underestimation of the modified saving will be small as well.

Another cause of the high saving is the change in acting forces. This change is displayed as a percentage in Table 4-6. The values indicate the difference between the governing forces used in the calculations for the current (S355) and the new design (S460). In some cases this difference is negative indicating that the use of higher strength steel resulted in larger forces in these elements.

	Difference	Saving
	[%]	[%]
Vertical 1	4,2	17,1
Vertical 2	5,5	22,7
Vertical 3	3,2	12,6
Vertical 4	1,4	28,5
Vertical 5	3,0	25,4
Vertical 6	-2,1	19,4
Vertical 7	2,7	22,8
Diagonal 1	-5,2	16,0
Diagonal 2	4,8	27,9
Diagonal 3	0,6	18,9
Diagonal 4	0,9	21,4
Diagonal 5	1,6	22,7
Diagonal 6	4,4	24,5

Table 4-6: Difference in force and recalculated modified saving

By using these differences in acting forces the modified weights are recalculated for the verticals and diagonals. Now the weight saving is shown if the forces would be exactly the same. These savings are shown in Table 4-6. It is visible that the saving in this case is as expected. The average saving for the verticals is 21,2% and the average saving for the diagonals is 21,9%. These averages are a bit below the expected value of 23% but this is caused by the stability checks in compression members.

Because the use of S460 results in generally lower acting forces the final saving is not modified for this reduction in force. The final cost saving is therefore based on the modified weight. The difference in acting forces is not included in the calculation of the cost saving because it is a direct result of the use of higher strength steel.

4.2.1.2 Connections

In order to make transportation of the truss elements possible a certain amount of splices are required. In order to keep the assembly costs as low as possible bolted connections are favorable. In the current design gusset plates are welded to the top and bottom chords and flange plates are welded tot the gusset plate. The sections are connected with the use of splice plates. In all cases only the flanges are connected because the thickness of the gusset plates and the webs of the sections could differ significantly. When the difference is large thick packing plates are required which reduce the effectiveness of the bolted connection. Because only the flanges are connected the effective cross-sectional area at the connection is reduced. Only the area of the flanges may be used to calculate the resistance at the connections. This is especially noticeable in tensile members, because the net cross-section will be governing which is reduced due to the bolt holes. In some cases the flanges need to be thickened to account for the loss of effective cross-sectional area. It is expected that this is required more often when using higher strength steel, especially when the unity check of the entire section is already high.

The calculations of the resistances of the connections are shown in Appendix II. The locations of the splices are shown in Figure 4-14.

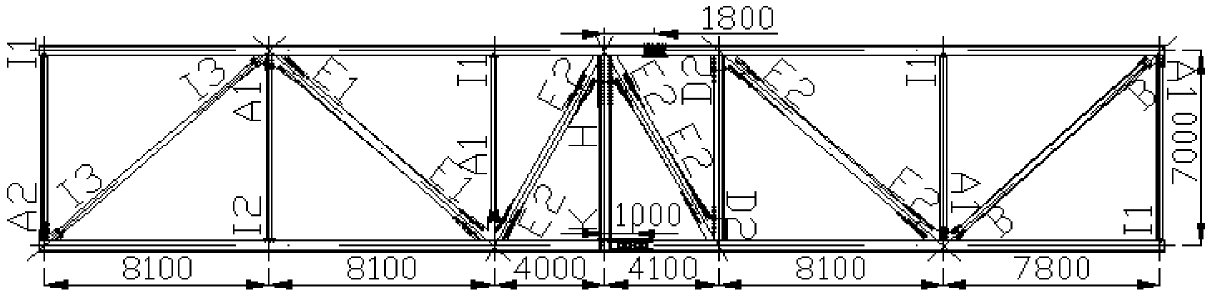


Figure 4-14: Splice locations and details

It is visible that in some cases 3 sections have to be connected to the top or bottom chord. All connections which involve diagonal members are made by using gusset plates with flange plates welded on it. A typical detail for this type of connection is displayed in Figure 4-15. When only a vertical member is connected there is no need for a gusset plate and the connection is simply made by using an end plate. The gusset plates all have a unique shape because of the demands of the designer. The edges of the gusset plates are parallel to the edges of the flanges of the connected sections. The edges are all at a certain distance perpendicular to these flanges. For aesthetic reasons this perpendicular distance (offset) is 80 mm. The length of the flange plates depends on the amount of bolts and the end distance and spacing applied. In some cases the flange plates are extended and welded to the flange of the corresponding chord. The flange plates are in these cases used to stabilize the gusset plates. Flange plates connecting tensile members are not used to stabilize the gusset plates.

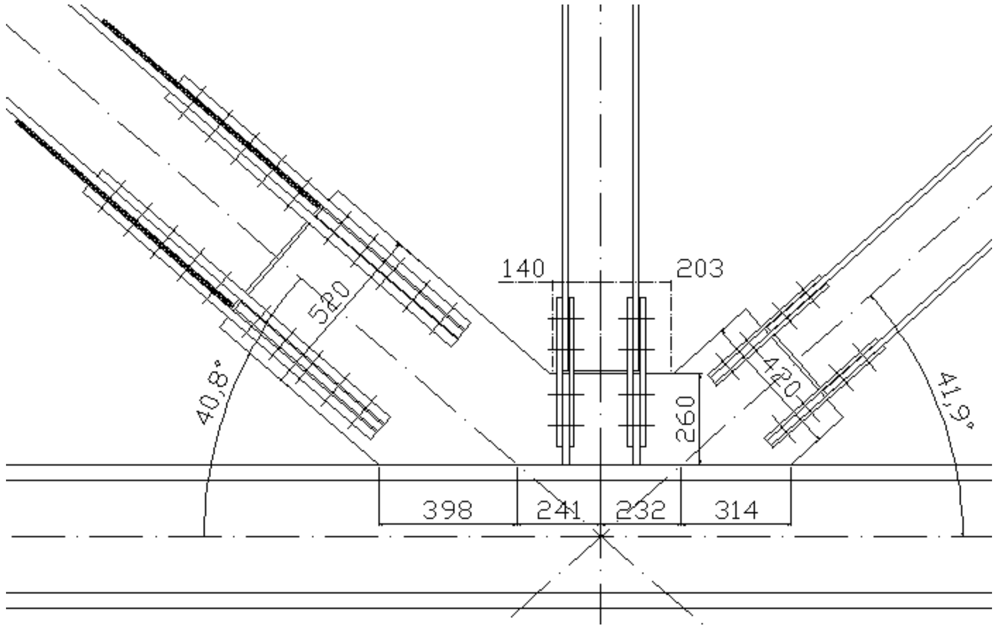
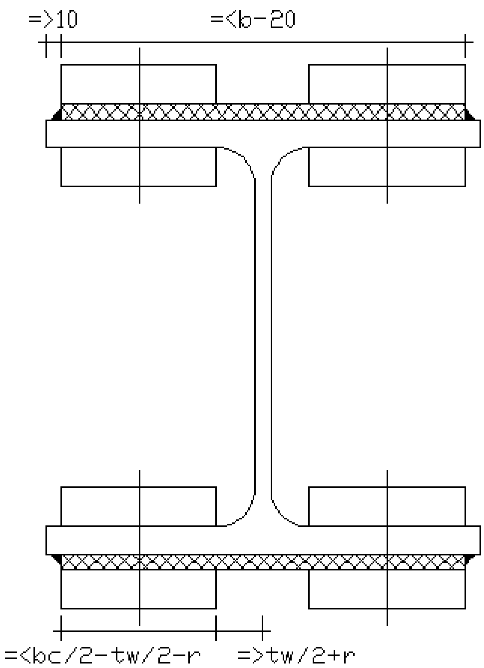


Figure 4-15: Typical detail of a connection with a gusset plate

M27 bolts are mainly used in the current design. Due to the minimum spacing and edge distance requirements it is not possible to use more than 2 bolts next to each other in the flanges. Smaller bolts would be required but their decreased shear resistance would not result in a great reduction in connection length. Also the assembly costs would be higher because more bolts would have to be used. In the heaviest sections M36 bolts are used to reduce the connection length. Figure 4-16 displays the location of the bolts. This drawing displays the cross-section of the left diagonal in Figure 4-15 at the location of the connection. In this drawing the maximum dimensions for the cover plates and splice plates are also shown.



The cover plates have to be welded to the flanges so on each side of the plate a minimum spacing of 10 mm is required to enable the use of fillet welds. The dimensions of the splice plates inside the section are bound by the edge of the flange or the edge of the cover plate (if present), and the radius and web width of the section. The dimension of the splice plates outside the section are bound by the edge of the cover plate (if present) and the thickness of the gusset plate. Usually the thickness of the gusset plate is not large enough to govern the width of the splice plates therefore the width of the splice plates both inside and outside the section are based on the boundaries inside the section.

Figure 4-16: Maximum dimensions splice- and cover plates

The length of all plates is dependent on the amount of bolts required and the spacing and end distances. The used spacing, end and edge distance is shown for 3 different bolts (Table 4-7).

	e1	e2	p1	p2
M27	60	45	90	90
M30	70	50	100	100
M36	80	60	120	120

Table 4-7: Minimum spacing, end and edge distance

The used values are larger than the required minimum values given in the Eurocode **[16]**. Because of this the shear capacity of the bolts is more often utilized resulting in a more economical design. The strength class of the bolts is 8.8. This means that the tensile strength is 800 N/mm² and the yield strength is 640 N/mm². The strength class 10.9 could also have been chosen but the shear resistance of the bolts is almost always governing. The shear resistance is calculated with the use of the formula:

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

The shear resistance is dependent on the area of the bolt at the thread and the strength class. For bolts with the same size the only variables that remain are α_v and the tensile strength f_{ub}. The factor α_v depends on the strength class of the bolts. For class 8.8 this factor is equal to 0,6 and for class 10.9 this factor equals 0,5. Because of the lower value for strength class 10.9 the advantage of using a higher strength class decreases. In fact if we multiply the tensile strength with the factor α_v this results in 480 N/mm² for class 8.8 and 500 N/mm² for class 10.9. The shear strengths differ slightly and because of this strength class 8.8 was chosen. The shear capacities of M27, M30 and M36 bolts are given in Table 4-8.

	Ab,s	d	d0	e1	e2	p1	p2	αb	k1	Fv,rd	Fb,rd
	[mm²]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]			[kN]	[kN/mm]
M27	459	27	30	60	40	90	90	0,667	2,033	176,3	15,81
M27	459	27	30	60	45	90	90	0,667	2,500	176,3	19,44
M30	561	30	33	70	50	100	100	0,707	2,500	215,4	22,91
M36	817	36	39	80	60	120	120	0,684	2,500	313,7	26,58

Table 4-8: Shear and bearing capacities of M27, M30 and M36 bolts

The bearing resistance is generally not lower than the shear resistance of the bolts. M27 is used for the smaller sections with relatively small forces. M30 is used to limit the connection length and for heavy loaded elements M36 is used also to reduce the length of the connection.

The bearing resistance is calculated for a unit thickness (1 mm). The following formula is used:

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t}{\gamma_{M2}}$$

The factor k_1 is the smallest value of $2{,}8\text{x}e_2/d_0 - 1{,}7$; $1{,}4\text{x}p_2/d_0\text{-}1{,}7$ and 2,5. If the minimum edge distance and spacing shown in Table 4-7 is used the value of k_1 becomes 2,5 for every bolt. This is the maximum value so the minimum edge distance and spacing provide maximum bearing capacity.

The factor α_b is the smallest value of $e_1/(3xd_0)$; $p_1/(3xd_0)$-0,25; f_{ub}/f_u and 1. For M27 bolts this value is equal to 2/3 for end bolts and 3/4 for internal bolts. An end distance of 90 mm and a spacing of 112,5 mm would be required for this factor to become 1 (and thus achieve maximum bearing capacity). This would however lead to an increase in connection length which could in turn decrease the shear capacity of the bolts. In most of the cases the thickness will be large enough for the bearing capacity to be larger than the shear capacity. For M27 bolts the bearing capacity is 19,44 kN (when the earlier specified spacing is used) per mm, so a component with a thickness of 10 mm has a bearing capacity of 194 kN. The bolts however are loaded in 2 shear planes which doubles the shear capacity for the bolts. This means that the flanges should have thicknesses of at least 19 mm in order to make the bearing capacity of the bolts larger than the shear capacity. The splice plates should have a thickness of at least 10 mm because each bolt divides its load to 2 splice plates. In the current design the bearing capacity is never smaller than the shear capacity. In the new design smaller thicknesses are used and in some cases the edge distance is smaller than the specified minimum edge distance of 45 mm. In these cases the bearing capacity has to be reduced. The minimum edge distance according to NEN-EN 1993-1-8 3.5 is 1,2xd_0. d_0 is the diameter of the holes. For bolts M27, M30 and M36 the diameter of the holes is the bolt shaft diameter + 3 mm **[17]**. For M27 d_0 is 30 mm and the minimum edge distance therefore becomes 36 mm. The smallest width used for the splice plates is 80 mm. The edge distance therefore becomes 40 mm. The value of k_1 used in the calculation of the bearing capacity is reduced from 2,500 to 2,033 effectively reducing the bearing capacity by almost 20%.

In some cases the connection can be loaded in tension and in compression. The use of bolts loaded in shear in regular bolts would lead to a shift in displacement when the direction of the load changes. It's desirable for slip not to occur so the general detail cannot be used. Alternatives are fitted bolts, injection bolts, prestressed bolts, the use of end plates with bolts loaded in tension or the use of plates welded on site. In this case it was decided to use end plates (Figure 4-17) for economical reasons.

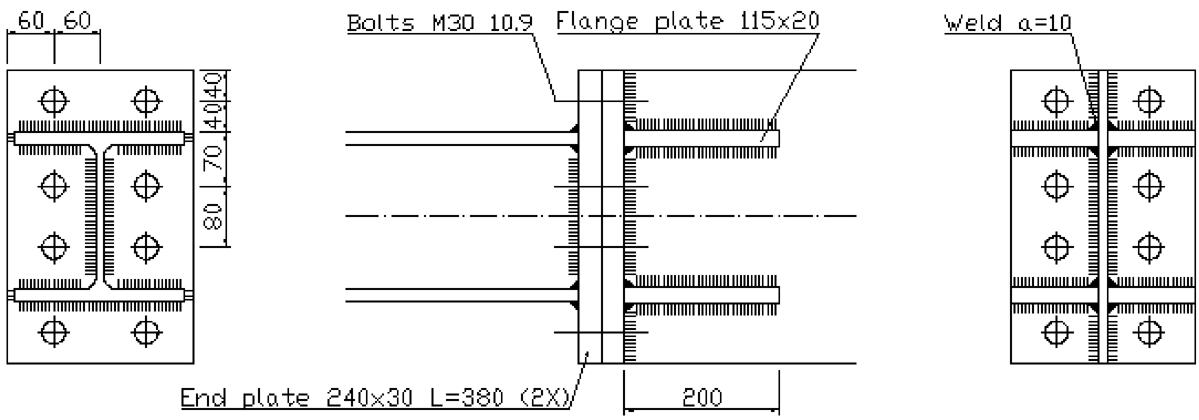


Figure 4-17: Splice with end plates for varying load directions

When a lot of bolts are required the length of the connection could become significant. According to NEN-EN 1993-1-8 3.8 a reduction factor (β_{Lf}) has to be applied to the shear capacity of the connection when the center to center distance of the first and the last bolt in a row is greater than 15xd. For each bolt the maximum amount of bolts that can be applied in a row without the use of a reduction factor is shown in Table 4-9.

	d	p1	15xd	Amount
	[mm]	[mm]	[mm]	of bolts
M27	27	90	405	5
M30	30	100	450	5
M36	36	120	540	5

Table 4-9: Maximum amount of bolts when no reduction factor is applied

For all sections at least 1 detail is designed and checked with the use of the Eurocode. In some cases more than 1 detail has been made because of large differences in the occurring forces. An HEB220 is for instance loaded with a force of 2893 kN (diagonal 1 truss 10) and another one is loaded with a much smaller force of 871 kN (diagonal 3 truss 11). Because of these large possible differences multiple details have been made to obtain an economical design.

Because only the flanges are connected the calculations have been based on flange forces. These forces are based on the acting axial force and the minimum values according to NEN-EN 1993-1-8 6.2.7. This part of the Eurocode states that the splice should be able to transfer 25% of the bending moment capacity in both directions, 25% of the axial force and 2,5% of the axial capacity as a shear force in both directions. These forces do not have to be combined so they can be checked separately. The axial force and bending moment in both directions all result in flange forces. By checking the maximum flange force the connection is automatically checked for the acting normal force and the minimum requirements for the bending moments. The resulting forces in each component of the connection is described with the use of Figure 4-18 - Figure 4-20.

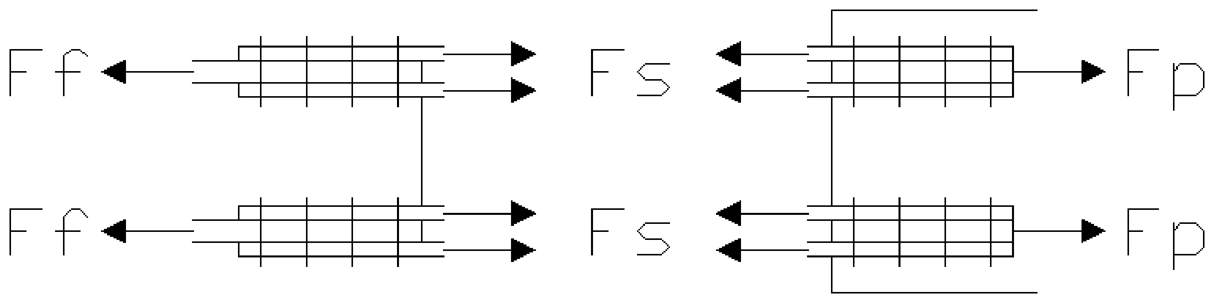


Figure 4-18: Axial force (N) - force transfer

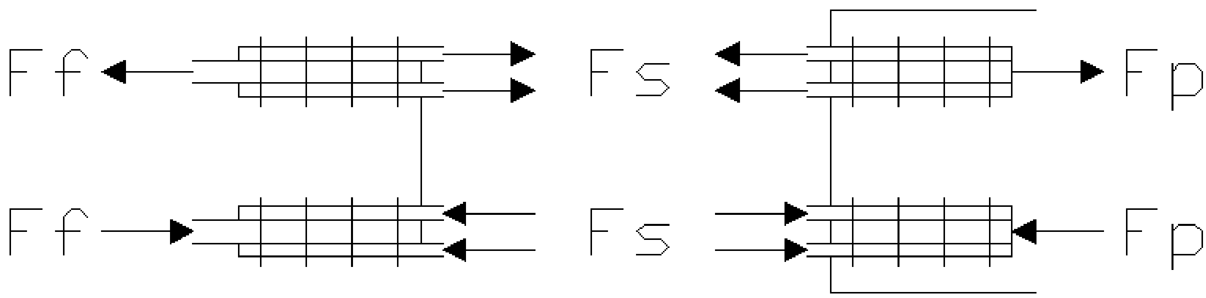


Figure 4-19: Bending moment (M_y) - force transfer

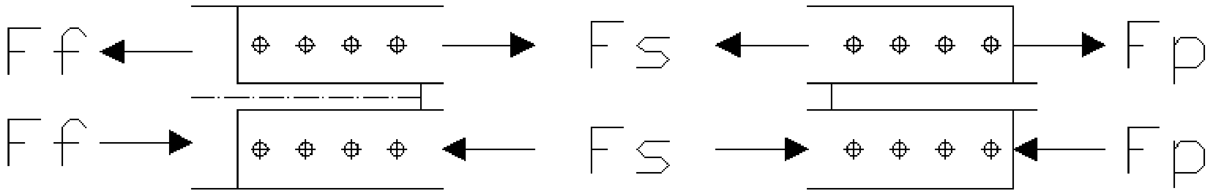


Figure 4-20: Bending moment (M_z) – force transfer

In all situations the following is valid:

- $F_s = F_f/4$ (Each flange is connected with 4 splice plates)
- $F_p = F_f/2$ (Each flange plate is connected with 2 splice plates)

The flange force (F_f) can for each situation be calculated with:

- $F_f = N/2$ (Axial force N)
- $F_f = M_y/z$ (Bending moment M_y with $z = h\text{-}t_f$)
- $F_f = M_z/z$ (Bending moment M_z with $z = b/2$) (F_f is divided over the two flanges)

With the use of these forces the required amount of bolts, size of the welds and the size of the components can be calculated. The shear force requirement states that the connection should be able to transfer 2,5% of the axial capacity. Because this force is relatively small compared to the axial forces transferred it is assumed that all connections are able to transfer this shear force.

The flanges of the section and the splice plates are loaded in tension or compression. The flange plates are loaded in shear. The flanges of the section are sometimes thickened with the use of cover plates. The calculations of the capacities of the flanges and the splice plates have been made with the use of NEN-EN 1993-1-1 6.2.3/6.2.4. The calculation of the capacities of the flange plates have been made with the use of NEN-EN 1993-1-1 6.2.6. The capacity is checked for 3 different failure modes and is explained with the use of Figure 4-21.

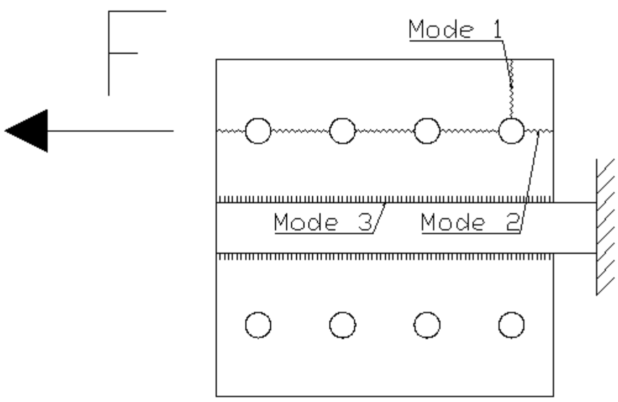


Figure 4-21: Flange plate failure modes

The force F can be a tensile or compressive force. The force is distributed over all the bolts. Tearing of the bolt group is described with the first 2 modes. Mode 1 is the fracture of the flange plate at the bolts, the capacity for this mode is a combination of axial capacity and shear capacity. Mode 2 is also failure of the plate at the bolts, the capacity for this mode is determined with the shear capacity of the net section at the bolts. Mode 3 is failure of the plate at the welds. The flange is not weakened by bolt holes at this part but a combination of shear and bending moment occurs. The bending moment is due to the eccentricity of the bolted connection. The unity check is calculated with the use of the combined stress according to NEN-EN 1993-1-1 6.2.1.

The splices in the chords consist of 2 different splice plates. Because 2 sections are connected the web can also be used. In these cases splice plates for the flanges and web are used. In these splices a combination of axial force, shear force and bending moment occurs. The bending moment is transferred with the use of the splice plates connecting the flanges in the same way as described earlier. The shear force is transferred with the use of the splice plates connecting the web. The remaining axial force is assumed to be divided over all the splice plates. It is assumed that the flanges transfer 70% of the axial force and the web transfers 30% of the axial force.

The welds are of different types. The cover plates are connected to the flanges with the use of fillet welds. The flange plates and gusset plates however are connected with the use of beveled welds. The flange plates are connected with these welds because fillet welds would in some cases obstruct the splice plates. The gusset plates are connected with these welds because fillet welds would become too large (in rare cases fillet welds can be used). The cover plate should be able to reach its full capacity at the position of the first bolt because the combined capacity of the flange and the cover plate are required to resist the acting force. The welds and the cover plate should therefore be designed to allow the full capacity to occur at this position (Figure 4-22). In the figure the top view of a section with cover plates and the side view of only the flange and the cover plate is shown.

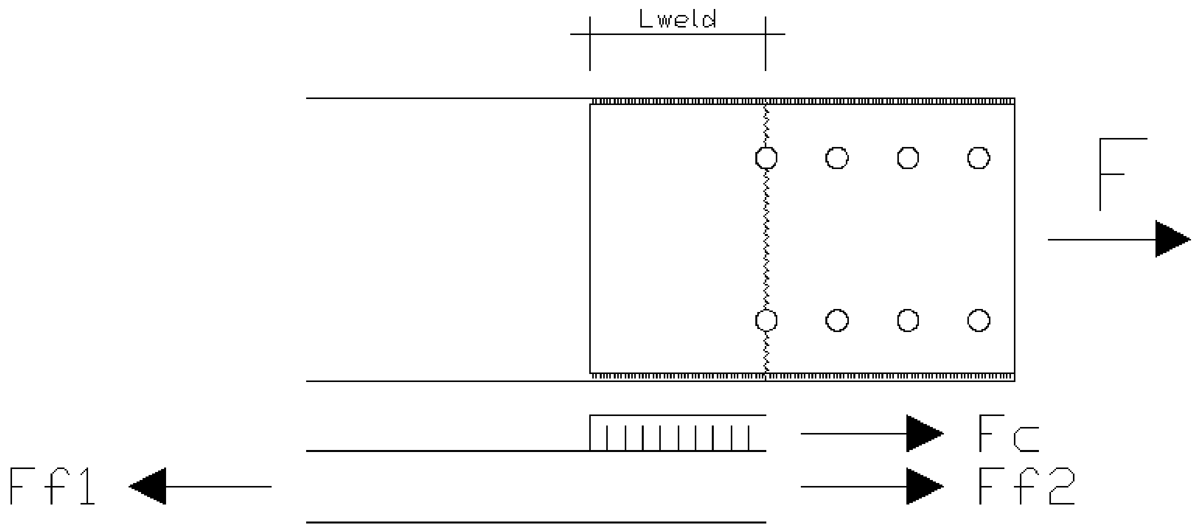


Figure 4-22: Cover plate and active weld length

The welds between the first bolt and the end of the plate (over a length L_{weld}) should be able to resist this force. The calculation of the capacity of the weld is based on this part only. The cover plate is also connected with a weld at the start of the cover plate. The resistance is based on the combined resistance of the weld at the start of the cover plate and the welds at the sides (over a length L_{weld}). This is based on the assumption that the end welds have enough deformation capacity **[18]**. Welds with a throat thickness of 5 mm can be made with a single layer, therefore all cover plates are welded with fillet welds with a throat thickness of 5 mm. The length of the cover plate is based on the occurring force in the cover plate and the length of the side welds that is therefore required.

The splices of the individual members that are connected in a node are described with the use of details. In Figure 4-14 the used details per member are shown. The drawings and the calculations of the capacities of the details are shown in Appendix II.

The thickness of the gusset plates can be dependent on different stresses: Axial stress of the entire plate, shear stress at the welded connection to the chord, tearing of a certain splice and also shear stress between two different splices. For each node the vertical and horizontal reaction forces on the chords are known. These forces need to be transferred from the gusset plates to the chords. It is also possible that the reaction forces are relatively small. This means that the forces from a vertical/diagonal are transferred to another vertical/diagonal with the use of the gusset plate. With the use of all known forces (reaction forces and axial forces in verticals and diagonals) the stresses in different parts of the gusset plate can be calculated. The connections at the nodes where diagonals are connected are made with the use of gusset plates. The node numbers are shown in Figure 4-23.

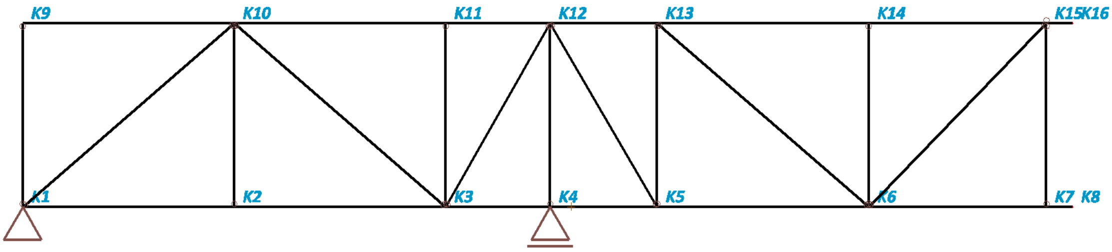


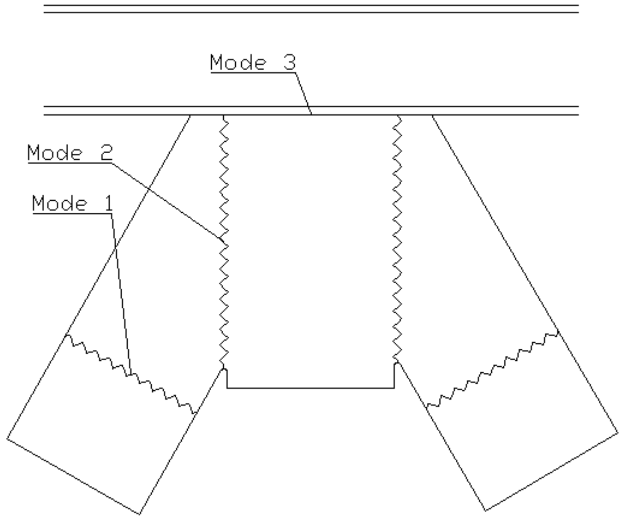
Figure 4-23: Node numbers truss 10

Gusset plates on the bottom chord are referred to as node 1-7, and on the top chord they are referred to as node 9-15. The forces relevant for the gusset plates and for web stiffeners are shown in Table 4-10. In this table different forces are shown. Rx and Rz are the horizontal and vertical forces transferred from the gusset plates to the chords. 'Support' displays the support reactions and the forces transferred from the columns above the truss to the chords. Rx, Rz and the forces in the connected verticals and diagonals are relevant for the design of the gusset plates. The support forces and the forces in the verticals connected with end plates are relevant for the design of web stiffeners.

Node	Members	Rx [kN]	Rz [kN]	Support [kN]
Node 1	S1, S8	-1011	-3426	4227
Node 3	S3, S9, S10	9949	253	N/A
Node 4	S4	N/A	N/A	16151
Node 5	S5, S11	-3871	339	N/A
Node 6	S6, S12, S13	-6863	1153	N/A
Node 9	S1	N/A	N/A	1102
Node 10	S2, S8, S9	-7279	1540	953
Node 11	S3	N/A	N/A	961
Node 12	S4, S10, S11	-934	543	N/A
Node 13	S5, S12	5533	990	577
Node 14	S6	N/A	N/A	131
Node 15	S7, S13	1509	556	96

Table 4-10: Forces relevant for gusset plates and web stiffeners

The calculation of the required thickness is based on multiple cross-sections of the gusset plates. These cross-sections are displayed in Figure 4-24.

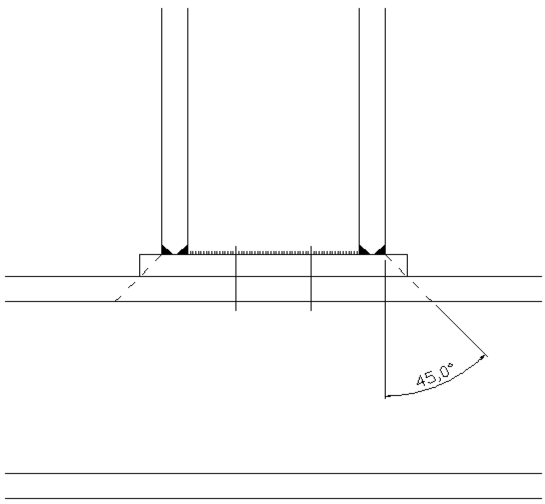


Mode 1 is local failure of the gusset plate at the connections of the diagonals. The capacity for this mode is the axial capacity of the cross-section. Mode 2 is a combination of shear and axial capacity. The resistance of the shear area is calculated first. For mode 3 a combination of axial force, shear force and bending moment has to be transferred. The axial and shear force are based on the reaction forces (Rx and Rz). The bending moment is based on the eccentricity of the center of the cross-section of the gusset plate at the connection to the chord as opposed to the center of the connection.

Figure 4-24: Gusset plate failure modes

The eccentricity in horizontal direction is the distance between the center of the cross-section of the gusset plate and the axis of the connected vertical. The eccentricity in vertical direction is equal to half the beam height. For mode 3 all forces result in a combined stress which has been checked with the use of NEN-EN 1993-1-1 6.2.1. The flange plates stiffen the gusset plate, therefore it is assumed that (local) buckling will not occur before the yield stress has been reached.

Web stiffeners are sometimes required when large forces are applied at a certain point: Support reactions for example. The support forces mentioned earlier are all local forces. They are all modeled as point loads acting on the chords. If the forces are too high the web will yield. The web is encased in concrete so buckling of the web is not an issue. The effective width is shown in Figure 4-25.



The force can be distributed over an angle of 45˚. If the end plate is long enough (distance between the end of the plate and the flange is at least the thickness of the end plate) the effective width is the column height + two times the end plate thickness + two times the flange thickness of the chord. If web stiffeners are required it is assumed that the entire support force is transferred through these stiffeners. The axial force resisting cross-section is than equal to two times the width of the chord times the thickness of the stiffeners.

Figure 4-25: Effective width of the web (compression)

The required plates to make the splices in the current design are shown in Table 4-11.

	Section	Cover plates l x w x t	Splice plates l x w x t	Flange plates l x w x t
Top chord	HD400x382	Flange	1290x170x40	Flange
Top chord	HD400x382	Web	N/A	Web
Bottom chord	HD400x677	Flange	1390x170x40	Flange
Bottom chord	HD400x677	Web	1030x230x50	Web
Diagonal 1	HEB280	N/A	N/A	N/A
Diagonal 2	HD400x463	N/A	1050x170x20	520x190x30
Diagonal 3	HD400x287	N/A	1090x170x25	540x185x30
Diagonal 4	HD400x287	N/A	1090x170x25	540x185x30
Diagonal 5	HD400x216	820x370x8	1090x160x25	540x170x25
Diagonal 6	HEM260	N/A	430x100x10	210x125x30
Vertical 1	HEB280	N/A	610x110x10	300x130x20
Vertical 2	HEB280	N/A	430x110x10	210x130x25
Vertical 3	HEB280	N/A	430x110x10	210x130x25
Vertical 4	HD400x509	N/A	1770x170x35	880x185x60
Vertical 5	HD400x287	N/A	890x170x15	440x185x30
Vertical 6	HEB280	N/A	430x110x10	210x130x25
Vertical 7	HEB280	N/A	430x110x10	210x130x25

Table 4-11: Required cover-, splice-, and flange plates current design (S355)

Even though fairly low unity checks were used in the current design, still some of the members have to be thickened. In this truss only diagonal 5 has to be stiffened. Its unity check is 0,76 based on the plastic capacity of the member. It is clearly visible that bolted connections significantly reduce the effectiveness of certain sections. The length of the cover plate is determined by the capacity of the weld. The cover plates are usually thin (about 10mm) so large fillet welds cannot be used. Instead all cover plates are welded with fillet welds with a throat thickness of 5 mm.

The required plates in the new design are shown in Table 4-12. It is visible that more cover plates are required. This is because of the higher unity checks and the lower f_u/f_y ratio. In all of the cases the length of the cover plates is determined by the weld capacity and in all cases fillet welds are used with a throat thickness of 5 mm. 5 mm is chosen because only 1 layer has to be welded. When the plate has a thickness lower than 7 mm welds with a throat thickness of 4mm have to be used because 5mm would be too large (in this case all cover plates are at least 12 mm thick). The splice and flange plates of the current and the new design have the same length. This makes sense because the length is determined by the amount of bolts used which is the same for both designs. The width and thickness of these plates however are usually smaller in the new design. This is mainly noticeable when looking at the flange plates. Higher savings are expected in these plates because the capacity is mainly based on the yield strength. The capacity of the splice plates is based on the tensile strength which makes large savings less likely. In some cases the plates in the new design are even thicker. This is because the used sections are smaller which decreases the available width for the splice plates.

	Section	Cover plates l x w x t	Splice plates l x w x t	Flange plates l x w x t
Top chord	HD400x216	Flange	890x170x20	Flange
Top chord	HD400x216	Web	710x240x20	Web
Bottom chord	HD400x382	Flange	1390x170x25	Flange
Bottom chord	HD400x382	Web	1030x240x30	Web
Diagonal 1	HEB220	N/A	N/A	N/A
Diagonal 2	HD400x187	N/A	1090x170x15	540x185x20
Diagonal 3	HD400x187	1040x370x12	1090x160x25	540x175x25
Diagonal 4	HD400x187	1040x370x12	1090x160x25	540x175x25
Diagonal 5	HD400x187	1040x370x12	1090x160x25	540x175x25
Diagonal 6	HEB260	N/A	430x100x10	210x125x20
Vertical 1	HEB220	N/A	610x80x10	300x105x15
Vertical 2	HEB220	N/A	430x80x15	210x105x20
Vertical 3	HEB220	N/A	430x80x15	210x105x20
Vertical 4	HD400x382	N/A	1770x170x25	880x190x40
Vertical 5	HD360x147	N/A	970x160x15	480x175x20
Vertical 6	HEB220	N/A	430x80x15	210x105x20
Vertical 7	HEB220	N/A	430x80x15	210x105x20

Table 4-12: Required cover-, splice- and flange plates new design (S460)

The reduced sizes required in the new design results in some weight saving. This saving is mainly due to the reduction in size of the flange plates. The total weight of the cover-, splice- and flange plates and the corresponding saving is shown in Table 4-13. The weight is based on the dimensions of the plates, the amount of plates per section and the unit weight of steel. If cover plates are required 4 of them will be required for the entire section (2 sides and 2 flanges per side). In all cases 16 splice plates are required (4 splice plates per flange), this is not valid for the splice plates connecting the flanges in the chords (8 splice plates are used for the chords). The splice plates connecting the webs of the chords require a different amount of plates. 2 plates per splice are required. The flange plates are only applied for the verticals and diagonals. For each splice 4 flange plates are required so 8 in total per section (2 splices per section).

	Weight [kg]		Saving
	Current	New	[%]
Cover plates	76,2	435,0	570,9
Splice plates	5659,8	4504,6	20,4
Flange plates	1743,4	988,4	43,3
Total	7479,0	6028,0	19,4

Table 4-13: Total weight and weight saving cover-, splice- and flange plates

The total weight of the plates required for the splices is for both designs more than 10% of the weight of all sections. This is mainly due to the long connections that are required. In the column 'saving' the percentage of the cover plates is displayed in red. This is because the weight of the cover plates in the new design is larger than the weight in the current design. The percentage displays how much more weight is used in the new design to strengthen the sections. This percentage is really high (over 6 times more weight required in this case) but the weight is still relatively small compared to the other components. The saving for the splice plates is surprisingly high. The saving is twice as high as the expected saving of 9,3%. In some cases the splice plates weigh more in the new design and in some cases they weigh much less. This difference is mainly caused by the difference in acting force and the difference in used section. As mentioned earlier the splices should be designed for the acting force but also for the minimum requirements. In the new design these minimum requirements are never governing for the design of the cover-, splice- and flange plates. In the current design however these minimum requirements are larger because larger sections are used and in some cases they are governing. Because of this the difference in force increases which results in larger savings. The saving in the flange plates is also larger than expected but this can be explained by the smaller width of the plates. Because smaller sections are used the width of the flange plates can also be smaller. The design of the flange plate is as stated earlier primarily governed by the shear capacity of the plate. The shear capacity is not dependent on the width of the plate so therefore the width can be reduced without the need to use thicker plates. The use of the steel grade S460 enables not only a reduction in thickness but also a reduction of the width. This combination leads to savings greater than the expected value of 23%.

The total weight saving is in this case 19,4%. The saving is primarily influenced by the used splice plates because their weight is relatively large compared to the combined weight of the other plates. The relative weight of the plates compared to the weight of the sections is larger in the new design. This is because cover plates are required more often and the splice plates are relatively heavier because of the lower f_u/f_y ratio.

The required sizes of the gusset plates and stiffeners in the current design and new design are shown in Table 4-14. The length corresponds to the length of the connection of the plates to the chords. The height is not the actual height of the plate but it is used to estimate the surface area of the plate. With this surface area and the given thickness the weight of the gusset plates are estimated. The weight is important for the calculation of the material saving. In all cases the length of the gusset plates are slightly reduced when using S460. This is because smaller sections are connected. The thickness is sometimes smaller and sometimes unchanged. This is mainly because of the governing failure mode. When fracture of a certain cross-section is governing the expected saving is only 9,3%. Because of the decreased size of the gusset plate the governing cross-section will be smaller so the thickness cannot be reduced when fracture is the governing failure mode.

The dimensions of the stiffeners correspond to a single stiffener. When stiffeners are required 4 of them are required (2 per flange). The stiffeners are welded to the beam on all sides. Generally more stiffeners are required when higher strength steel is used. This makes sense because the used sections are smaller but the support reactions are unchanged. The height and width of the stiffeners is in all cases the same because HD400 sections are used. HD400 sections all have the same distance between the flanges (h_w) and the length of the flanges between the radius and the edge (c_f) is also the same.

	Gusset plates lxhxt	Stiffeners hxwxt
Node 1	800x350x25	N/A
Node 2	N/A	N/A
Node 3	1200x850x50	N/A
Node 4	N/A	320x190x60
Node 5	750x1200x50	N/A
Node 6	1200x500x50	N/A
Node 7	N/A	N/A
Node 9	N/A	N/A
Node 10	1250x600x50	N/A
Node 11	N/A	N/A
Node 12	900x1350x50	N/A
Node 13	900x700x50	N/A
Node 14	N/A	N/A
Node 15	700x400x20	N/A

	Gusset plates lxhxt	Stiffeners hxwxt
Node 1	750x350x25	N/A
Node 2	N/A	N/A
Node 3	1200x850x50	N/A
Node 4	N/A	320x190x50
Node 5	700x1200x35	N/A
Node 6	1200x500x35	N/A
Node 7	N/A	N/A
Node 9	N/A	N/A
Node 10	1150x600x35	N/A
Node 11	N/A	N/A
Node 12	900x1350x40	N/A
Node 13	900x700x35	N/A
Node 14	N/A	N/A
Node 15	700x400x15	N/A

Table 4-14: Required gusset plates and stiffeners current (left) and new design (right)

The total weight and weight saving for the gusset plates and stiffeners is shown in Table 4-15.

	Weight [kg]		Saving
	Current	New	[%]
Gusset plates	2106,5	1624,7	22,9
Stiffeners	114,5	95,5	16,7
Total	2221,1	1720,1	22,6

Table 4-15: Total weight and weight saving gusset plates and stiffeners

The saving of the gusset plates is partly due to the reduced length of some of the plates. The length is in some cases reduced by about 10% which is explained by the use of smaller sections. An HEB280 is for instance exchanged for an HEB220. The height of the section is therefore reduced by 60 mm which reduces the required length of the plate. The thickness is also of influence on the total weight saving. In some case the thickness has been reduced a lot. This is because in the current design the gusset plates usually have a thickness of about 50 mm. This results in the use of a lower yield and tensile strength of the plate which increases the weight reduction. Because the chords are encased in concrete more stiffeners are not required in the new design. The thickness can even be reduced due to the higher strength of the material. Normally one would expect an increase in the use of stiffeners. This would be true if the chords were not encased in concrete. In that case local buckling of the web would increase the need for stiffeners.

Due to favorable circumstances the weight saving is higher than expected. A larger saving for the gusset plates is obtained because the difference in yield and tensile strength has become larger due to the thick gusset plates used in the current design. The weight of the gusset plates is relatively large which results in a weight saving of 22,6% for both the gusset plates and the stiffeners.

4.2.2 Truss 11

Truss 11 is a simply supported truss adjacent to truss 10. It is located on the 11th axis of the grid. Because the truss is positioned close to truss 10 the loads are about the same. Because an inward support is not present additional verticals and diagonals are not required and the spacing of 8,1 m can be maintained. In this case 6 verticals and 5 diagonals are required to transfer the forces to the supports. The span of the truss is 40,5 m (Figure 4-6). Element numbers and dimensions used in the model are displayed in Figure 4-26. Again the verticals and diagonals are described from left to right.

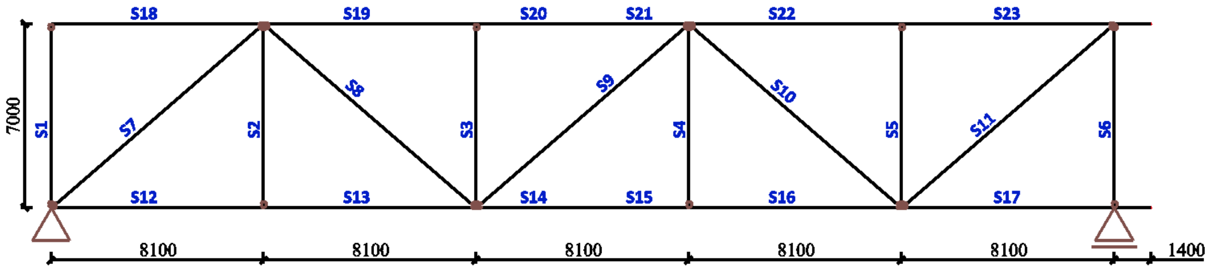


Figure 4-26: Element numbers and dimensions truss 11

4.2.2.1 Sections

In the current design the sections used for the top and bottom chords are exactly the same as the ones used in truss 10. The difference between these trusses is that the top chord in this case is the compressed member instead of the bottom chord. The truss is simply supported at its ends so L_{rep} is equal to the length between the supports (40,5 m). The allowable deflections in this case are u_{end} = 162 mm and u_{add} = 121,5 mm. The camber applied to counteract the permanent deflection is 50 mm. The provided camber is less than the camber used in truss 10. This indicates that the truss deflects less than truss 10 which means that strength will again be the governing factor in the determination of the required cross-sections. The additional deflection caused by the variable loads is 16,9 mm. This value is well below the allowed deflection of 121,5 mm. The local deflection is again limited by u_{end} = 32,4 mm.

The sections in the new design are again based on a HEB220 being the smallest section. The calculations do not differ from the ones used when calculating the relevant unity checks in truss 10. The required forces are shown in (Figure 4-27 - Figure 4-29). From the figures it is visible that the critical cross-section is located just to the right of vertical 2. All the forces are critical in this position. The buckling lengths are the same for the entire top chord. This also applies to the bottom chord. The location of the critical cross-section of the bottom chord is just to the left of vertical 5. In this position a combination of a high tensile force in combination with a high bending moment occurs. This leads to the governing combination of this section.

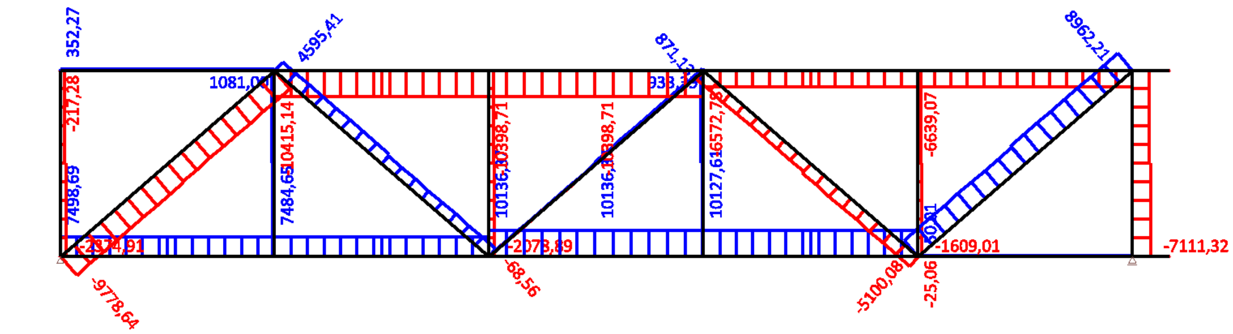


Figure 4-27: Normal force truss 11

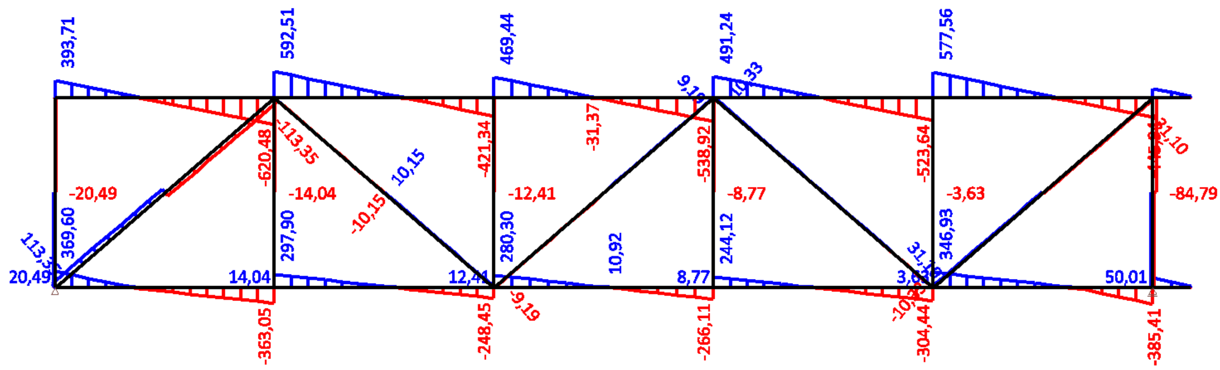


Figure 4-28: Shear force truss 11

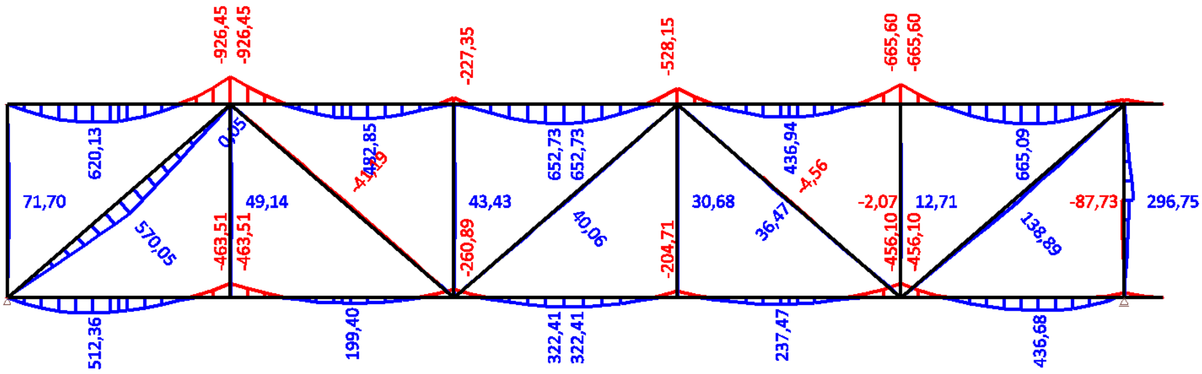


Figure 4-29: Bending moment truss 11

The forces used to determine the required cross-sections are shown in Table 4-16.

	Nmin [kN]	Nmax [kN]
Vertical 1	-2461	-1211
Vertical 2	347	1117
Vertical 3	-2126	-1068
Vertical 4	269	962
Vertical 5	-1635	-595
Vertical 6	-7357	-4755

	Nmin [kN]	Nmax [kN]
Diagonal 1	-10239	-6646
Diagonal 2	2581	4708
Diagonal 3	-81	869
Diagonal 4	-5345	-3089
Diagonal 5	5778	9297

	N [kN]	Vz [kN]	My [kNm]
Top chord	-10525	641	1048
Bottom chord	9929	314	444

Table 4-16: Forces in current design

Again after the new sections had been entered and the acting forces in each member of the truss were recalculated there appeared to be differences in the acting forces (Table 4-17). All forces appear to be decreased slightly which can be related to the decrease in self weight of the truss. This truss also has 1 member which can be loaded in either tension or compression (diagonal 3).

	Nmin [kN]	Nmax [kN]
Vertical 1	-2375	-1119
Vertical 2	322	1081
Vertical 3	-2079	-1016
Vertical 4	252	933
Vertical 5	-1609	-574
Vertical 6	-7111	-4529

	Nmin [kN]	Nmax [kN]
Diagonal 1	-9779	-6248
Diagonal 2	2467	4595
Diagonal 3	-69	871
Diagonal 4	-5100	-2884
Diagonal 5	5484	8962

	N [kN]	Vz [kN]	My [kNm]
Top chord	-10071	593	926
Bottom chord	9675	304	456

Table 4-17: Forces in new design

The calculations are similar to the ones used to calculate the unity checks of truss 10. Buckling lengths are exactly the same, the only thing different is the used sections and the forces. The calculations of the unity checks are shown in Appendix I.

The described sections (Table 4-19) all fulfill the strength requirements written in the Eurocode. The deflection should also be smaller than the described limits. The deflection is shown in Figure 4-30.

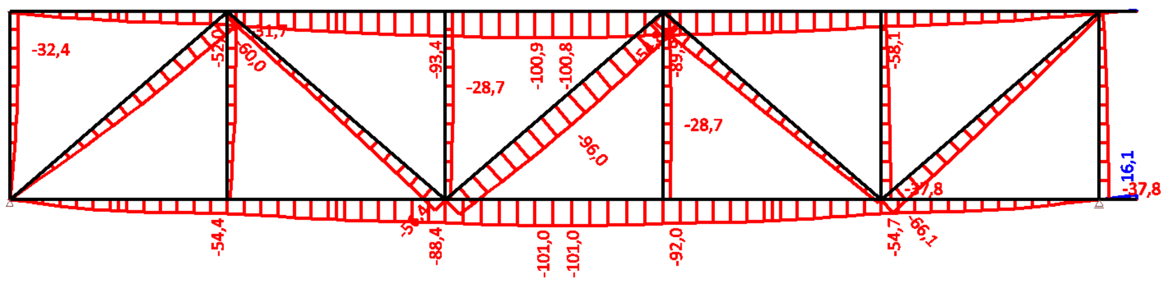


Figure 4-30: Deflection truss 11

The final deflection (u_{end}) is 101,0 mm. The deflection is within the limits so it is not required to use a camber. The deflection caused by the permanent loads is 80,0 mm. The entire permanent deflection can be compensated by applying a camber of 80 mm. Again the additional deflection is the governing factor for the global deflection check. The additional deflection is 21,0 mm, this is well below the limit of 120 mm. The local deflection is checked by using relative displacements (Figure 4-31).

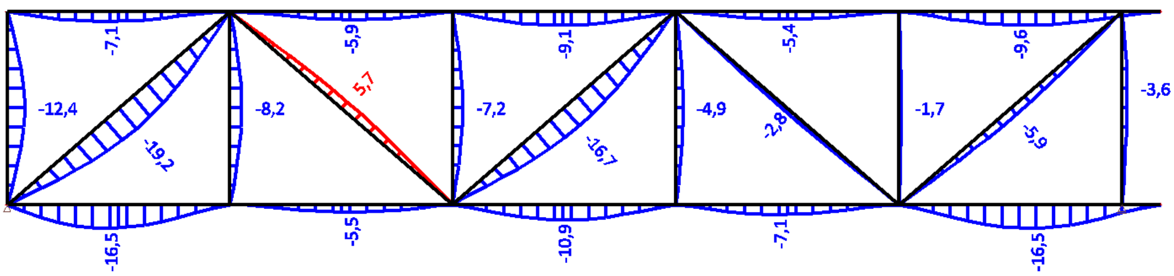


Figure 4-31: Relative displacement truss 11

The maximum relative displacement is 16,5 mm. This is smaller than the limit of 32,4 mm so the local stiffness requirement is met. The truss is stiff enough to limit the global and local deflections.

The rotations are again based on the conservative approach of adding the unity checks for both the global and local deflection. In this case this would lead to an accurate unity check because the maximum rotations due to global and local deflections are at the supports. The unity check is 21/120 + 16,5/32,4 = 0,68. In this case the unity check is far less than the one calculated for truss 10 (0,89). This is because in truss 10 the maximum rotations do not occur at the same position which leads to a unity check that is larger than the actual value. If the stiffness would become governing additional calculations could be made to prove that the rotations are within the limits (if that is indeed the case).

The used sections in the current design are displayed in Table 4-18. The modified weight is used to take account of the difference in unity checks as described earlier. The unity checks are again based on the TGB.

Weight [kg] 62904,5 S355

	Section	Length [m]	Weight [kg/m]	Unity Check	Weight [kg]	
					Direct	Modified
Top chord	HD400x677	41,9	677,8	0,85	28399,8	24139,8
Bottom chord	HD400x384	41,9	382,4	0,72	16022,6	11536,2
Diagonal 1	HD400x463	10,706	462,8	0,81	4954,7	4013,3
Diagonal 2	HEB360	10,706	141,8	0,72	1518,1	1093,0
Diagonal 3	HEB260	10,706	92,9	0,2	994,6	198,9
Diagonal 4	HD400x216	10,706	216,3	0,73	2315,7	1690,5
Diagonal 5	HD400x287	10,706	287,5	0,73	3078,0	2246,9
Vertical 1	HEB280	7	103,1	0,83	721,7	599,0
Vertical 2	HEB280	7	103,1	0,24	721,7	173,2
Vertical 3	HEB280	7	103,1	0,66	721,7	476,3
Vertical 4	HEB280	7	103,1	0,2	721,7	144,3
Vertical 5	HEB280	7	103,1	0,46	721,7	332,0
Vertical 6	HD400x287	7	287,5	0,7	2012,5	1408,8

Modified weight [kg] 48052,4
Average unity 0,76

Table 4-18: Used sections current design (S355)

Weight [kg] 39378,1 S460

	Section	Length [m]	Weight [kg/m]	Unity Check	Weight [kg]	
					Direct	Modified
Top chord	HD400x421	41,9	421,6	0,84	17665,0	14838,6
Bottom chord	HD400x216	41,9	216,3	0,98	9063,0	8881,7
Diagonal 1	HD400x287	10,706	287,5	0,94	3078,0	2893,3
Diagonal 2	HEB280	10,706	103,1	0,77	1103,8	849,9
Diagonal 3	HEB220	10,706	71,5	0,21	765,5	160,8
Diagonal 4	HD360x147	10,706	147,5	0,77	1579,1	1215,9
Diagonal 5	HD400x216	10,706	216,3	0,74	2315,7	1713,6
Vertical 1	HEB220	7	71,5	0,99	500,5	495,5
Vertical 2	HEB220	7	71,5	0,29	500,5	145,1
Vertical 3	HEB220	7	71,5	0,76	500,5	380,4
Vertical 4	HEB220	7	71,5	0,22	500,5	110,1
Vertical 5	HEB220	7	71,5	0,54	500,5	270,3
Vertical 6	HD400x187	7	186,5	0,8	1305,5	1044,4

Modified weight [kg] 32999,7
Average unity 0,84

Table 4-19: Used sections new design (S460)

The used sections in the new design are displayed in Table 4-19. The final savings have been calculated for each individual element to display individual differences if they are present. The savings are displayed in Table 4-20.

Direct saving [%]: 37,4

	Saving [%]	
	Direct	Modified
Top chord	37,8	38,5
Bottom chord	43,4	23,0
Diagonal 1	37,9	27,9
Diagonal 2	27,3	22,2
Diagonal 3	23,0	19,2
Diagonal 4	31,8	28,1
Diagonal 5	24,8	23,7
Vertical 1	30,6	17,3
Vertical 2	30,6	16,2
Vertical 3	30,6	20,1
Vertical 4	30,6	23,7
Vertical 5	30,6	18,6
Vertical 6	35,1	25,9

Modified saving [%]: 31,3

Table 4-20: Weight saving

The modified saving is 31,3% in this case. This is about the same as the modified saving in truss 10. The high saving in truss 10 was explained by the decrease in bending moment at the inward support. In this case it can be explained by the yield strength and imperfection parameters used in the calculations of the unity checks. The top chord is of big influence on the total modified saving because of the large weight it has. The modified saving for the top chord is 38,5% which is very high. In the current design the top chord is an HD400x677 section with a flange thickness of 81,5 mm. Because of this thickness the yield strength must be reduced to 335 N/mm², which in its turn increases the expected saving to 27,2% because the yield strength of HISTAR460 does not have to be reduced. In the TGB (The used Dutch code in the design) buckling curve d is used for both axis in case the flange thickness is larger than 80 mm. In the Eurocode buckling curve d is used when the flange thickness is larger than 100 mm. In this case the modified saving will be larger when the TGB is used. This is mainly because the unity check in the TGB is partly dependent on the imperfection parameter e*. This parameter is linearly dependent on the value of the imperfection factor (a_0, a, b, c and d). In the new design the imperfection factor is 0,21 (a), while the value of the imperfection factor d is 0,76. Because of this the imperfection parameter e* is about 3 times larger in the current design which significantly increases the value of the unity check. The given value for the weight saving is therefore an overestimate because with the current rules a more favorable buckling curve may have been chosen for the section in the current design.

Except for the top chord all the elements appear to have a modified saving averaging to about 23%. The saving is as expected but the influence of the change in acting forces has not yet been looked at. This change is displayed in Table 4-21. The bold values indicate the difference between the governing forces used in the calculations. In some cases this difference is negative indicating that the use of higher strength steel resulted in larger forces in these elements.

	Difference [%]	Saving [%]
Vertical 1	3,5	14,3
Vertical 2	3,2	13,4
Vertical 3	2,2	18,3
Vertical 4	3,0	21,3
Vertical 5	1,6	17,3
Vertical 6	3,3	23,3
Diagonal 1	4,5	27,9
Diagonal 2	2,4	22,2
Diagonal 3	-0,2	19,2
Diagonal 4	4,6	28,1
Diagonal 5	3,6	23,7

Table 4-21: Difference in force and recalculated modified saving

By using these differences in acting forces the modified weights are recalculated for the verticals and diagonals. Now the weight saving is shown if the forces would be exactly the same. These savings are shown in Table 4-21. The average saving for the verticals is 18,0%. This is 5% less than expected but that is due to the greater sensitivity to buckling. The average saving for the diagonals is 24,2%. This average is higher than expected but that is because large sections are used in the current design some of them having flange thicknesses of more than 40 mm. Because of the thick flanges the yield strength is reduced to 335 N/mm² therefore increasing the saving.

Again the saving which included the difference in force is not considered in the cost saving. The cost saving for material is based on the modified saving in weight.

4.2.2.2 Connections

The connections in truss 11 are similar to the ones used in truss 10. Because of the use of a warren truss the amount of connections which require a gusset plate stays limited. Relatively more connections with end plates are used which reduces the fabrication and assembly costs

The calculations of the resistances of the connections are shown in Appendix II. The locations of the splices are shown in Figure 4-32.

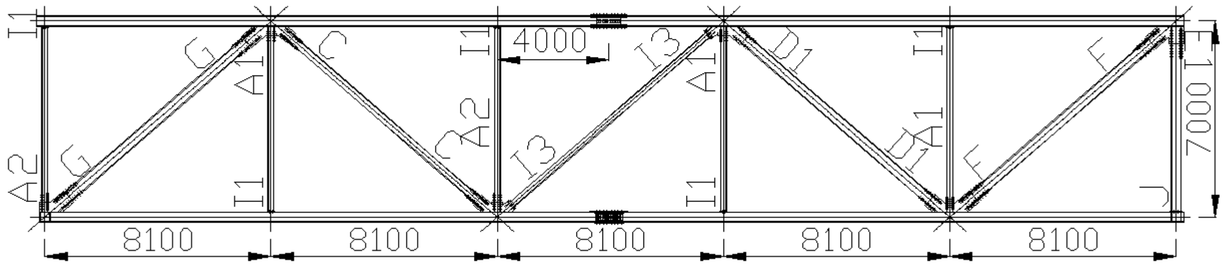


Figure 4-32: Splice locations and details

With the method described earlier the required cross-sections for all the components of the connections have been determined. The plates required for the connections in the current design are shown in Table 4-22.

	Section	Cover plates l x w x t	Splice plates l x w x t	Flange plates l x w x t
Top chord	HD400x677	Flange	1290x170x25	Flange
Top chord	HD400x677	Web	870x240x25	Web
Bottom chord	HD400x382	Flange	1050x170x25	Flange
Bottom chord	HD400x382	Web	870x240x25	Web
Diagonal 1	HD400x463	N/A	1290x170x25	640x190x30
Diagonal 2	HEB360	580x280x8	790x110x25	390x130x25
Diagonal 3	HEB260	N/A	N/A	N/A
Diagonal 4	HD400x216	N/A	890x170x15	440x180x25
Diagonal 5	HD400x287	830x380x8	1090x160x30	540x175x35
Vertical 1	HEB280	N/A	430x110x10	210x130x25
Vertical 2	HEB280	N/A	430x110x10	210x130x25
Vertical 3	HEB280	N/A	430x110x10	210x130x25
Vertical 4	HEB280	N/A	430x110x10	210x130x25
Vertical 5	HEB280	N/A	430x110x10	210x130x25
Vertical 6	HD400x287	N/A	1090x170x25	540x185x30

Table 4-22: Required cover-, splice- and flange plates current design (S355)

Also in this case when relatively low unity checks resulted from the section checks (diagonal 2 0,72 and diagonal 5 0,73) the sections still need to be thickened. Because of the use of gusset plates the effectiveness of the sections (loaded in tension) decreases significantly which is independent of the steel grade used. The effectiveness is dependent on the width and thickness of the flanges because the force is not assumed to be transferred through the web at the location of the bolts, therefore the cross-sectional area of the net section is much smaller than the total cross-sectional area of the section. The HEB360 for example has a cross-sectional area of 18060 mm^2, a width of 300 mm and a flange thickness of 22,5 mm. The section is connected with M27 bolts so the effective width of the flanges is 300-2x30 = 240 mm. The effective area of the 2 flanges is 2x240x22,5 = 10800 mm^2. The effective area is only 60% of the total cross-sectional area of the section which is why the section needs to be thickened at the connections. The height of an HEB360 is 360 mm which makes the height/width ratio 1,2. The smaller sections (HEB300 and smaller) all have an height/weight ratio of 1,0. For these sections the effective area will be relatively larger therefore reducing the need for thickeners. This is also valid for the smaller HD sections (HD400x187-HD400x382). The flange thickness/web thickness ratio is also of influence on the effective area. When the thickness of the flange (t_f) is relatively large the effective area will be relatively large as well. For these kind of connections it is therefore favorable to use sections with a small h/b ratio and a large t_f/t_w ratio. The smaller HD sections mentioned earlier for example have an h/b ratio of about 1,0 and an t_f/t_w ratio of about 1,6. Larger sections have more unfavorable ratio's which leads to a smaller effectiveness for these sections when loaded in tension.

The plates required for the new design are shown in Table 4-23.

	Section	Cover plates l x w x t	Splice plates l x w x t	Flange plates l x w x t
Top chord	HD400x421	Flange	1290x170x20	Flange
Top chord	HD400x421	Web	870x240x20	Web
Bottom chord	HD400x216	900x370x10	1050x160x25	Flange
Bottom chord	HD400x216	605x240x8	870x240x25	Web
Diagonal 1	HD400x287	N/A	1050x170x20	520x185x30
Diagonal 2	HEB280	650x260x10	790x100x25	390x125x20
Diagonal 3	HEB220	N/A	N/A	N/A
Diagonal 4	HD360x147	N/A	790x160x10	390x175x20
Diagonal 5	HD400x216	900x370x10	1050x160x25	520x175x25
Vertical 1	HEB220	N/A	610x80x10	300x105x15
Vertical 2	HEB220	N/A	430x80x15	210x105x20
Vertical 3	HEB220	N/A	610x80x10	300x105x15
Vertical 4	HEB220	N/A	430x80x15	210x105x20
Vertical 5	HEB220	N/A	430x80x15	210x105x20
Vertical 6	HD400x187	N/A	1090x170x25	540x185x20

Table 4-23: Required cover-, splice- and flange plates new design (S460)

It is visible that cover plates are required for the bottom chord. This is because the tension in this chord is larger than the tension in the top chord of truss 10. Because of this high tension the net section is too small to be able to resist the load. The calculation of the resistance has been split in a flange part and a part for the web. For both the flanges and the web cover plates are required. Although 4 cover plates have to be welded for each side of the connections it will definitely be a more economical solution than upsizing the component because the total length is over 40 meters. The same amount of cover plates are required in both designs for the verticals and diagonals. In the new design however these plates are thicker and longer. It was expected that cover plates are more often required when stronger steel grades are used. In this case this statement is also valid but it means that more material needs to be added to provide the load bearing capacity. The increased thickness implies that the cover plates have to transfer a relatively larger part of the load. This is confirmed by the length of the plate because longer welds are required to transfer the loads to the cover plates.

The weight saving is shown in Table 4-24.

	Weight [kg]		Saving
	Current	New	[%]
Cover plates	120,0	298,7	248,8
Splice plates	4359,5	3655,0	16,2
Flange plates	1043,3	723,3	30,7
Total	5522,8	4677,0	15,3

Table 4-24: Total weight and weight saving cover-, splice- and flange plates

Again the weight saving for the flange plates is substantial. The total saving however is much lower. This is mainly because cover plates are also required in the bottom chord for the new design which in turn will increase the fabrication costs of the truss.

The connections at the nodes where diagonals are connected are made with the use of gusset plates. The node numbers are shown in Figure 4-33.

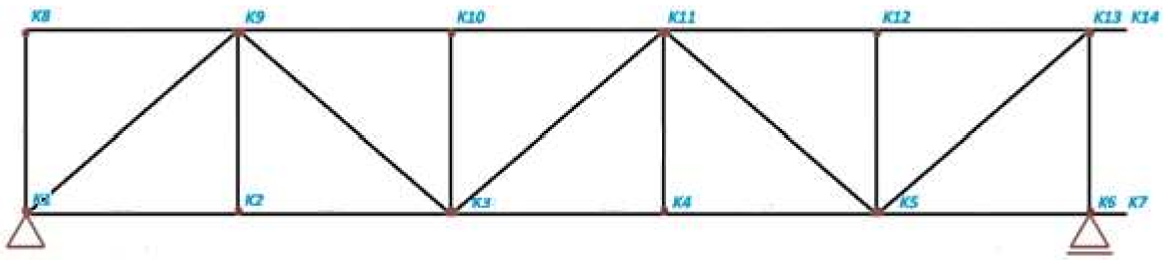


Figure 4-33: Node numbers truss 11

Gusset plates on the bottom chord are referred to as node 1-6, and on the top chord they are referred to as node 8-13. The forces relevant for the gusset plates and for web stiffeners are shown in Table 4-25. In this table different forces are shown. Rx and Rz are the horizontal and vertical forces transferred from the gusset plates to the chords. 'Support' displays the support reactions and the forces transferred from the columns above the truss to the chords. Rx, Rz and the forces in the connected verticals and diagonals are relevant for the design of the gusset plates. The support forces and the forces in the verticals connected with end plates are relevant for the design of web stiffeners.

Node	Members	Rx [kN]	Rz [kN]	Support [kN]
Node 1	S1, S7	7317	8811	9543
Node 3	S3, S8, S9	2899	-434	N/A
Node 5	S5, S10, S11	-10094	-528	N/A
Node 6	S6	N/A	N/A	7608
Node 8	S1	N/A	N/A	900
Node 9	S2, S7, S8	-10437	-990	875
Node 10	S3	N/A	N/A	773
Node 11	S4, S9, S10	3840	-1030	471
Node 12	S5	N/A	N/A	145
Node 13	S6, S11	-6639	497	203

Table 4-25: Forces relevant for gusset plates and web stiffeners

The required sizes of the gusset plates and stiffeners in the current design and new design are shown in Table 4-26. In both cases stiffeners are required at the supports. This is because the weaker chord (the chord in tension) is the bottom chord. Because the truss is simply supported at the ends of the truss the load is more evenly distributed over the supports which results in large forces at both supports.

	Gusset plates lxhxt	Stiffeners hxwxt
Node 1	1000x500x60	320x190x30
Node 2	N/A	N/A
Node 3	1100x450x30	N/A
Node 4	N/A	N/A
Node 5	1250x600x50	N/A
Node 6	N/A	320x190x25
Node 8	N/A	N/A
Node 9	1400x600x50	N/A
Node 10	N/A	N/A
Node 11	1200x450x30	N/A
Node 12	N/A	N/A
Node 13	900x750x60	N/A

	Gusset plates lxhxt	Stiffeners hxwxt
Node 1	1000x500x55	320x190x35
Node 2	N/A	N/A
Node 3	1100x450x25	N/A
Node 4	N/A	N/A
Node 5	1250x600x40	N/A
Node 6	N/A	320x190x30
Node 8	N/A	N/A
Node 9	1250x600x40	N/A
Node 10	N/A	N/A
Node 11	1200x450x25	N/A
Node 12	N/A	N/A
Node 13	900x750x40	N/A

Table 4-26: Required gusset plates and stiffeners current (left) and new design (right)

The total weight and weight saving for the gusset plates and stiffeners is shown in Table 4-27.

	Weight [kg]		Saving
	Current	New	[%]
Gusset plates	1421,2	1101,9	22,5
Stiffeners	124,1	105,0	15,4
Total	1545,3	1206,9	21,9

Table 4-27: Total weight and weight saving gusset plates and stiffeners

The weight reduction for all the required plates is quite substantial, but in some cases additional plates (cover plates) are required for the sections to be able to resist the loads. The thickness of gusset plates can in some cases significantly be reduced which allows easier connection of small and large sections in a single node.

4.3 Cost saving

The cost saving can be divided in 4 parts. The material cost saving can be related to the weight saving, however the use of heavy sections results in an increase in €/t which could increase the cost saving. It is expected that the fabrication of beams and connections in S460 results in higher costs per ton steel. This is because of the reduced machinability of this steel grade. The required materials however are decreased in size compared to the ones required in the current design (S355). This means that smaller sections have to be sawn and drills have to go through a smaller thickness. The transportation costs are dependent on the total weight that needs transportation and the size of structural components. In this case the trusses are assembled on site so the trucks are loaded to their full capacity. When higher strength steel is used the total weight decreases thus decreasing the transportation costs. Cost saving for the assembly is not expected to be high. The connections still require the same amount of bolts so the amount of hours necessary to assemble the trusses do not change. It is possible to use a different crane with a smaller capacity because of the decreased weight.

4.3.1 Material

The material costs are calculated by using Table 3-4. The actual weight has to be used to calculate the costs so the modified weights cannot be used. This would lead to an overestimate of the cost saving because the unity checks in the current design are lower than the ones in the new design. The connections are designed by using high unity checks just like the sections in the new design. Therefore the weight of the members in the new design are not modified. The weight of the members in the current design are scaled by using the values of their unity checks and the values of the unity checks of the corresponding members in the new design. The weight of the members are multiplied with the value of their unity checks and divided by the unity checks in the corresponding members. This basically means that the modified weight of the current design is divided by the unity checks in the new design. This results in an approximation of the required weight of the members in the current design that would lead to the same unity checks as the members in the new design. The unity checks calculated by using the TGB are again used except for the chords in both trusses. In these trusses the sections HD400x677 and HD400x382 are used in the current design. As stated earlier the TGB uses different imperfection factors and formulas than the Eurocode which resulted in larger savings. For these chords the unity checks in the current design are recalculated by using the Eurocode and compared to the ones calculated in the new design. For the other members the unity checks are not altered because in these cases the TGB and the Eurocode use the same values and formulas.

The bottom chord in truss 10 results in a modified saving of 38,1% (Table 4-5). The unity checks of the bottom chords in the current design and the new design were 0,83 (current design) and 0,91 (new design) (Table 4-3 and Table 4-4). The Eurocode results in a unity check of 0,84 for the current design and 0,91 for the new design. The unity check in the current design has increased more so the modified weight saving is slightly underestimated. By using the new unity checks the modified saving turns out to be 38,9%, this is slightly more than the earlier calculated saving. The saving is high but this was already explained by the reduced stiffness of the section. The modified saving for truss 11 was 28,1%. In this case the Eurocode also results in different unity checks. In the current design the unity check is 0,74 according to the TGB, when using the Eurocode however the unity check would be 0,59 which is significantly smaller. The unity check for the top chord in the new design also decreases (from 0,94 to 0,89). The value for the current design has decreased much more which would decrease the modified saving. The modified saving is in this case 14,7% which is 13,4% smaller than the previously calculated value.

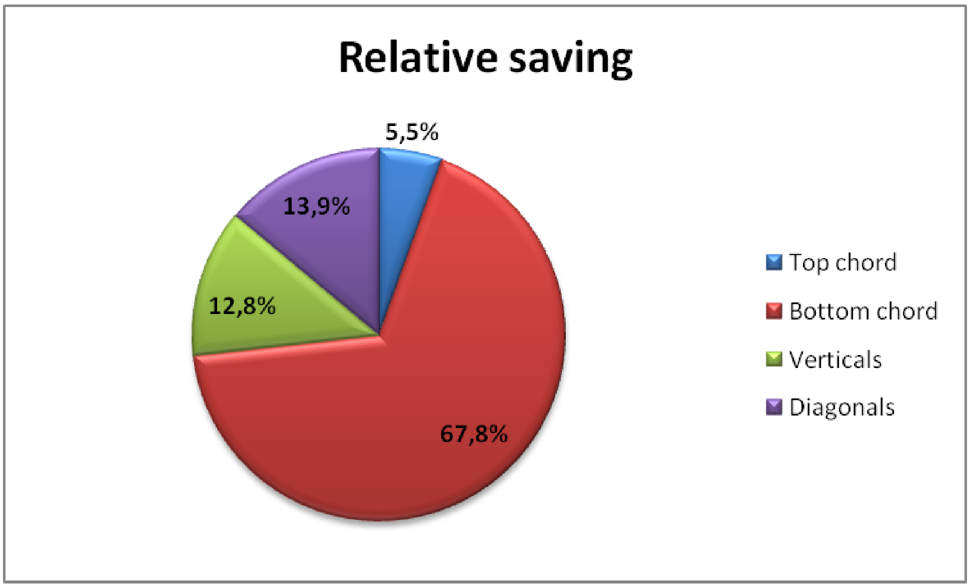
The top chord in truss 11 results in a modified saving of 38,5% (Table 4-20). This is much higher than expected because the reduced stiffness did not have that much of an effect on the acting forces. The axial force in the compressed chords of truss 10 and 11 are about the same, the bending moment however is smaller in the top chord of truss 11. Still the unity check is relatively high. This is because buckling is a more important factor in the top chord of truss 11. The top chord has a buckling length of 8100mm while the critical cross-section in the bottom chord of truss 10 has a buckling length of 4100mm. Because of the lower buckling length higher stresses are allowed to occur in the bottom chord. The unity checks of the top chords in the current design and in the new design were 0,85 (current design) and 0,84 (new design) (Table 4-18 and Table 4-19). The recalculation with use of the Eurocode results in a unity check of 0,78 for the current design and 0,87 for the new design. The unity check for the current design has decreased while the unity check for the new design has increased. This indicates that the modified saving is much lower than calculated. The new modified saving is 30,6%. This is 7,9% lower than before. This big difference is caused by the different imperfection factors and formulas used in the Eurocode. Because the top chord in truss 11 is more prone to buckling the difference is also larger than the difference for the bottom chord in truss 10. Still the value is above the expected 23% but this can be explained by the reduction in acting forces and by the reduced yield strength of 335 N/mm² which needs to be used in the calculation of the unity check in the current design. Because of this reduced yield strength the expected saving is raised to 27,2% which is closer to the calculated value of 27,0%. For the bottom chord the modified saving is 23,0%. The unity check in the current design changes from 0,72 to 0,61 when the Eurocode is applied. For the new design it changes from 0,98 to 0,88 which is also a significant reduction. In all cases when members are subjected to combined tension and bending the Eurocode results in smaller values for the unity checks. In this case the modified saving changes to 18,4%. The saving has decreased because the unity check for the current design decreased relatively more than the unity check for the new design.

By using the new modified weights for the current design the costs for each member have been calculated. The costs and cost saving for truss 10 are displayed in Table 4-28.

	Costs [€]		Saving
	Current	New	[%]
Top chord	9936	8913	10,3
Bottom chord	28421	15757	44,6
Diagonal 1	797	739	7,3
Diagonal 2	3155	2047	35,1
Diagonal 3	1819	1541	15,3
Diagonal 4	1895	1551	18,2
Diagonal 5	2561	2047	20,1
Diagonal 6	1181	879	25,6
Vertical 1	580	483	16,7
Vertical 2	631	483	23,4
Vertical 3	544	483	11,3
Vertical 4	4292	2744	36,1
Vertical 5	1392	1063	23,6
Vertical 6	560	483	13,7
Vertical 7	613	483	21,2
Total	58377	39695	32,0
Verticals	8611	6222	27,7
Diagonals	11409	8803	22,8

Table 4-28: Costs and cost saving truss 10

Because the size of the required section for the bottom chord has decreased much in the new design the section is cheaper per ton even with the grade extra of 50€/t. This results in a cost saving of 44,6% which amounts to almost 13.000€. The total cost saving is 32,0% resulting in a cost reduction of almost 19.000€. Because of the size of the bottom chord it was already determined that is has a large influence on the weight saving. Because the bottom chord is also cheaper the influence on the cost saving is even greater. The influence on the total cost saving of each type of member is displayed in Graph 4-1. In this graph the influence of the top chord, bottom chord, verticals and diagonals is shown. The verticals and diagonals are grouped because their individual influence is very small. The individual differences however cannot be ignored. The verticals have savings of 11,3% up to 36,1%. These differences are partly the result of the difference in weight saving described earlier, but they are also the result of the difference in costs for certain sections. Vertical 3 (11,3% saving) is more expensive per ton when executed in S460 because relatively small sections are used in both the current and the new design. Vertical 4 on the other hand (36,1%) is cheaper per ton which results in a very large cost saving. The same applies for the diagonals where the savings range from 7,3% up to 35,1%.



Graph 4-1: Relative saving truss 10

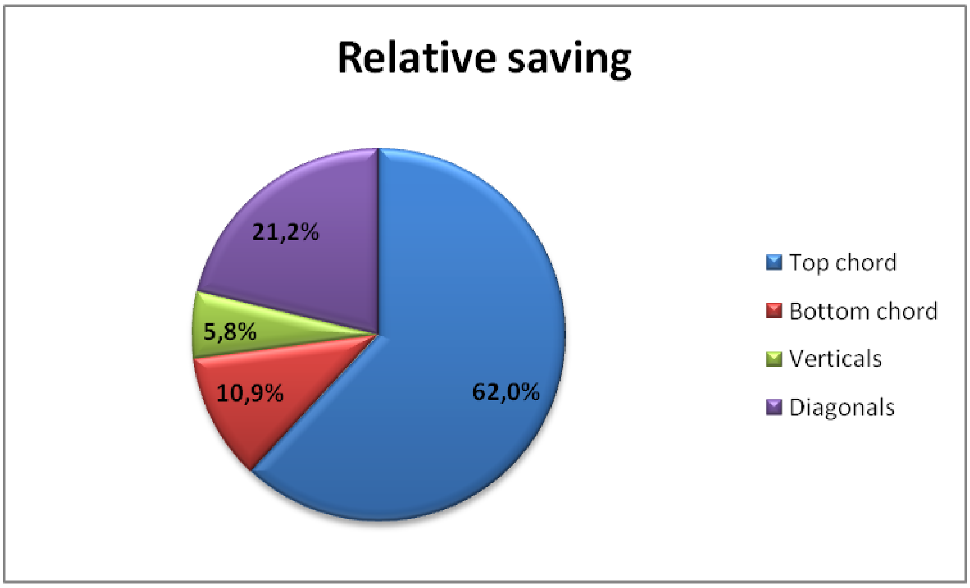
In Graph 4-1 it can be seen that the influence of the bottom chord on the total cost saving is more than 67%. This indicates that the use of stronger steel grades is really effective when an inward support is present. This is because the use of higher strength steel enables the designer to reduce the stiffness of the chords effectively decreasing the hogging moment above the inward support. The magnitude of the relative saving of the bottom chord is in these cases dependent on the length and height of the truss which determine the stiffness and relative weights of the chords.

The costs and cost saving for truss 11 are displayed in Table 4-29. The total cost saving is 25,1% which amounts to over 14.000€. The cost saving is as expected not that high as the previously calculated cost saving in truss 10. In simply supported trusses the stiffness of the top and bottom chords do not have a large influence on the acting forces as opposed to the chords of a cantilever truss. Because of this the saving will always be lower. In this case the costs per ton for the sections used in truss 11 is less favorable than the ones used in truss 10. This is especially noticeable when looking at the cost saving of the top chord. Because of the use of the section HD400x421 the advantage of less costs per ton no longer applies (Table 3-4). This reduces the cost saving slightly as opposed to the weight saving. The same applies to the verticals. The basic price and size extra's have not changed when calculating the costs of the verticals. This means that every section in the new design is 50€/t more expensive than the ones used in the current design. This increase in price is about 5% of the total material costs which results in relatively low savings, so the average cost saving of 17,9% for the verticals is this low because of the increased material costs.

	Costs [€]		Saving
	Current	New	[%]
Top chord	28772	20050	30,3
Bottom chord	10829	9290	14,2
Diagonal 1	4632	3155	31,9
Diagonal 2	1349	1071	20,6
Diagonal 3	871	739	15,2
Diagonal 4	2141	1627	24,0
Diagonal 5	2960	2374	19,8
Vertical 1	557	483	13,2
Vertical 2	549	483	12,1
Vertical 3	577	483	16,2
Vertical 4	604	483	20,0
Vertical 5	566	483	14,6
Vertical 6	1717	1338	22,1
Total	56123	42057	25,1
Verticals	4569	3753	17,9
Diagonals	11953	8964	25,0

Table 4-29: Costs and cost saving truss 11

The relative saving of each type of member is shown in Graph 4-2.



Graph 4-2: Relative saving truss 11

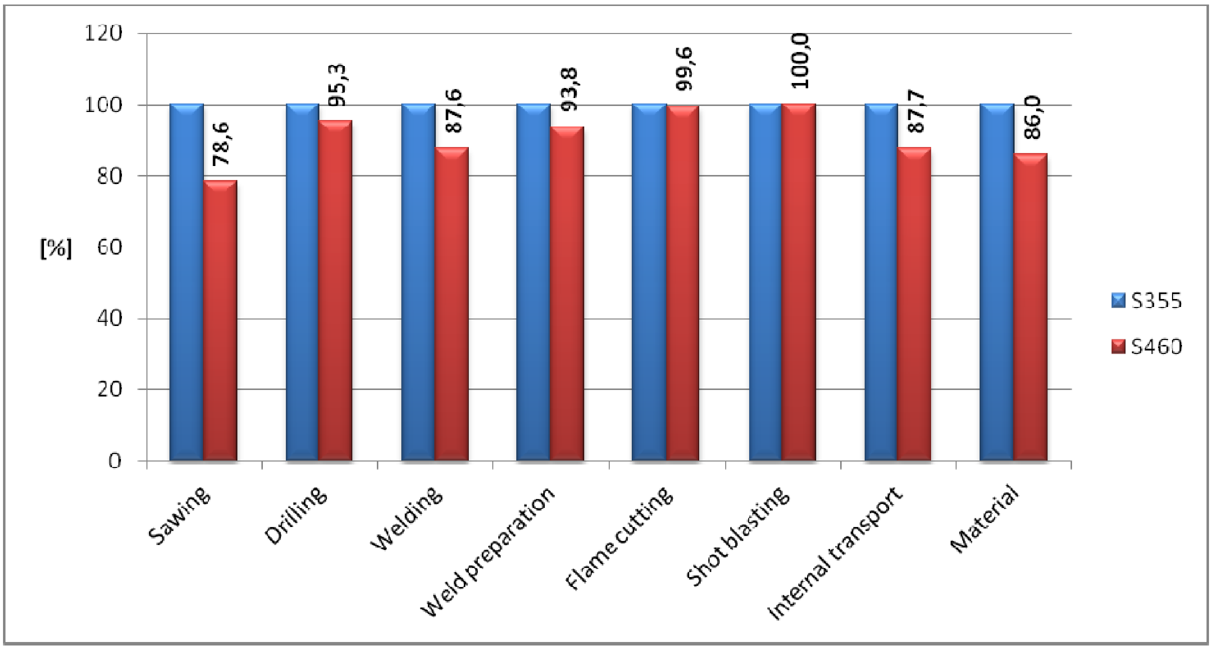
Again the influence of the compressed chord (top chord) is very high (over 60%). This is caused by the large weight of the top chord (almost 50% of the total weight) and the saving which is higher than the average saving. The relative saving of the verticals is very low. This is due to their relatively low weight and the low cost saving. Because of the length of the diagonals large sections are sometimes required. Because of this the relative saving is larger. It is visible that the relative saving of the diagonals is larger for truss 11 than for truss 10 even though 1 less diagonal is used. This is because larger forces are present in the diagonals of truss 11. Because of these large forces larger sections are required thus increasing the relative saving.

4.3.2 Fabrication

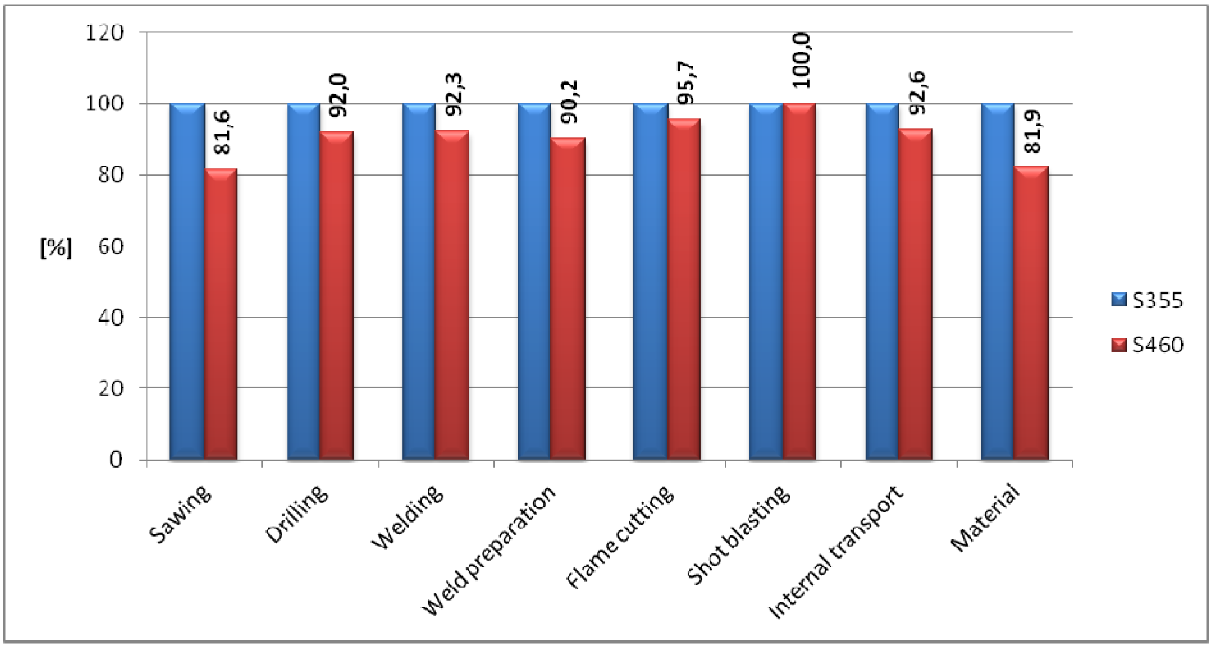
The fabrication costs depend on the amount of hours required to prepare the parts for final assembly at the construction site. The costs are then calculated with the use of hourly fees. For each individual component the required hours for each individual process is known. By doing this the influence on the total fabrication costs for all types of components and for the individual processes are known. The components are: Sections, cover plates, splice plates, flange plates, gusset plates and stiffeners.

The processes used are: Sawing, drilling, welding, weld preparation, flame cutting, shot blasting and internal transport. The costs of the plates is also included in the fabrication costs and is referred to as 'materials'. All the components are shot blasted (SA 2,5). The total costs of the internal transport is based on the total weight of all materials (sections and plates). The sections are sawn in the correct lengths and after that the holes for the bolts are drilled (after the cover plates are applied). Cover plates are made with the use of flame cutting and after that they are welded to the flanges of the section. Splice plates are made with the use of flame cutting and holes for the bolts are drilled. The same applies for the flange plates but these plates also require welding. The gusset plates are all of a special shape so flame cutting is required. One edge has to be cut diagonally from both sides so the welded connection can be made. Finally the stiffeners are made with the use of flame cutting. They also require weld preparation and welding.

Although fabrication costs are assumed to be 5% higher for S460 the total costs for each process are lower when S460 is used. The 5% increase is based on elements with the same size. The use of higher strength steel results in a size reduction which in its turn reduces the fabrication costs. The Reduction is shown in Graph 4-3 for truss 10 and in Graph 4-4 for truss 11.



Graph 4-3: Cost reduction for the different processes (truss 10)



Graph 4-4: Cost reduction for the different processes (truss 11)

The cost reduction is at most about 20% for a single process. Even though the weight reduction is large the cost reduction for fabrication is small. The cost reduction is shown in Table 4-30.

	Cost saving [%]	
	Truss 10	Truss 11
Sawing	21,4	18,4
Drilling	4,7	8,0
Welding	12,4	7,7
Weld preparation	6,2	9,8
Flame cutting	0,4	4,3
Shot blasting	0,0	0,0
Internal transport	12,3	7,4
Material	14,0	18,1
Total	11,3	9,6

Table 4-30: Cost saving for different processes (truss 10 and 11)

The cost saving for flame cutting is small. The costs for flame cutting gusset plates and stiffeners are about the same in both designs. However in the new design more cover plates are required therefore increasing the costs for flame cutting. The reduced thickness of splice and flange plates in turn reduces these costs resulting in a small cost saving.

As mentioned earlier the costs for the internal transport is based on the total weight of all the components. A reduction in weight of 50% however does not mean that the internal transportation costs will be 50% less because there is not a linear relation between the weight and the time required for internal transportation. Because of this the transportation costs of higher strength steel are assumed to be more expensive per ton. It was shown that the modified weight saving for the sections of truss 10 is 31,9%. Because the sections have a relatively large weight this still results in a significant cost saving in internal transport. As shown earlier the cost reduction of the sections is

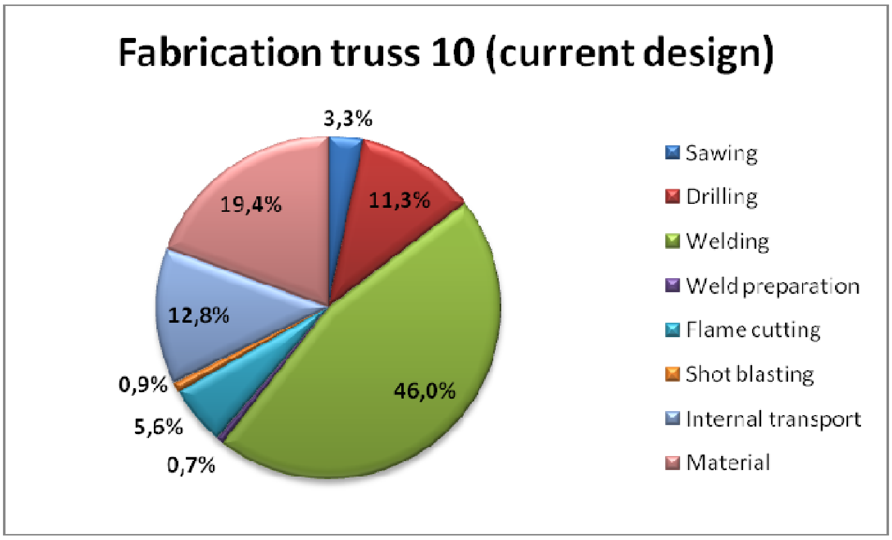
33,2% for truss 10. The costs for the plates however have only been reduced by 14,0% for truss 10 and 18,1% for truss 11. This is mainly because the stiffness of the plates has no effect on the occurring forces which are all nearly the same as in the current design. Some of the plates are designed with the use of the yield strength (flange plates, stiffeners and some of the gusset plates) and some of them are designed with the use of the tensile strength (flange plates, splice plates and some of the gusset plates). Because the forces are about the same and the governing strength differs for different types of plates the saving should be between the expected saving of 9,3% (tensile strength governing) and 23% (yield strength governing). This is indeed the case so the cost saving of the plates is as expected. The saving is actually a bit larger than expected when the individual components are examined. This is mainly noticeable when looking at the gusset plates. Very often thicknesses of 50 mm are required when the steel grade S355 is used. This means that the yield strength and the tensile strength have to be reduced which effectively increases the cost saving because this is not valid for plates in the new design (almost all plates are 40 mm thick or less).

The cost savings for operations such as sawing and drilling are not extremely high. This is because these operations all require a certain amount of time to set up the machines and to position the materials for the operations. A decreased thickness will result in less time required for drilling for example but the positioning of plates and set-up of the machine is not going to be faster. The saving for weld preparation is strongly dependent on the required stiffeners. In some cases thick stiffeners are required to support local loads. These stiffeners significantly increase the costs for weld preparation. In this case additional stiffeners are not required in the new design because (local) buckling of the web cannot occur due to the concrete encasement of the sections. If this was not the case the costs for stiffeners would very likely increase for the new design therefore also reducing the cost saving of weld preparation.

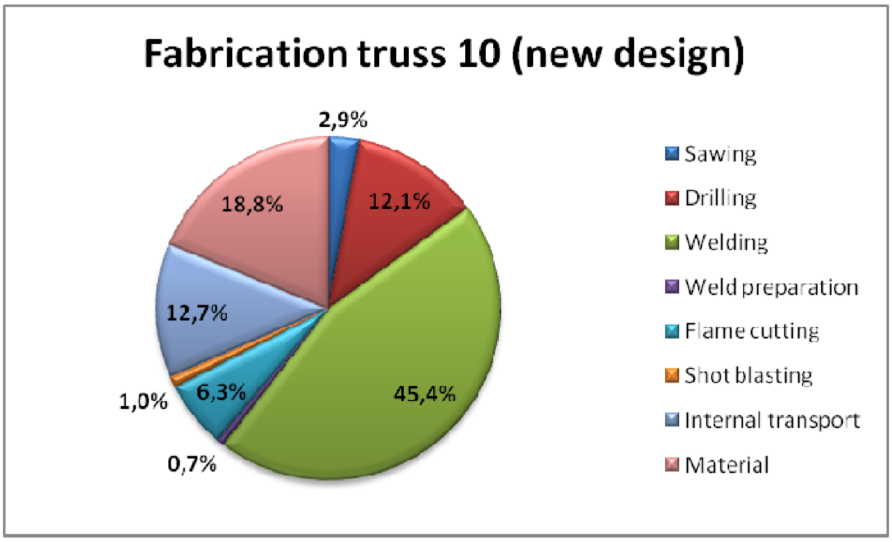
The cost saving for shot blasting is 0,0%. This is because all sections are shot blasted with a fixed speed. The length of the individual components remains unchanged so the time required for shot blasting stays the same.

It is visible that there can be large differences in savings when comparing truss 10 to truss 11. For truss 10 additional cover plates are required in order to satisfy the section checks at the connections. For truss 11 this is only the case for the bottom chord. This leads to the difference in costs for flame cutting. The difference in cost saving for weld preparation can be related to the required stiffeners which have as mentioned earlier a large influence on the costs for weld preparation.

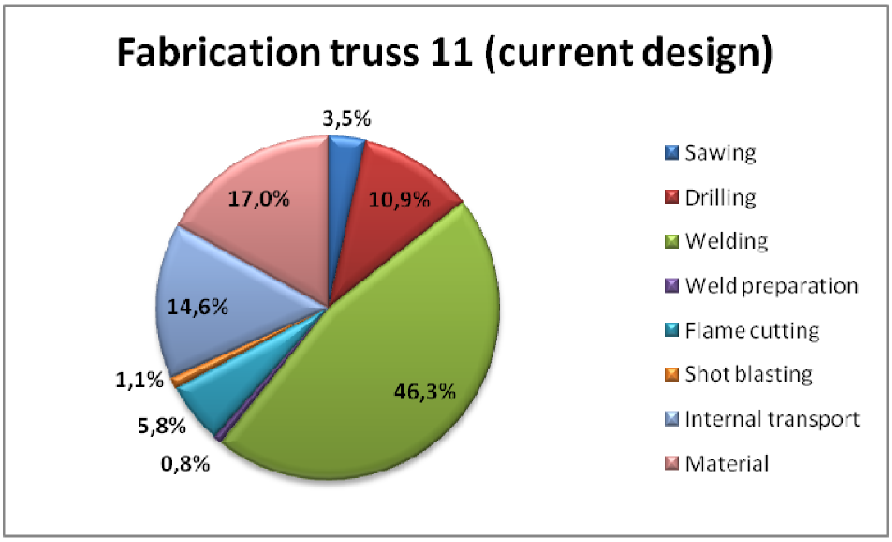
The relative costs of the current and new design of both trusses are shown in Graph 4-5 – Graph 4-8. It is visible that the relative costs do not change much (also not for different truss types). This is because most of the processes result in a saving close to the total saving which indicates that the relative saving will not change much. Other operations such as flame cutting have a saving that is not close to the total saving. The relative costs will therefore differ more. This is however slightly noticeable because the relative costs for these operations is very small. It is visible that welding and internal transport result in almost 2/3rd of the total fabrication costs. Because the cost saving for these operations is not large the total cost saving is not large either. This is however mainly because the welds have similar strengths when S355 or S460 is used which leads to about the same weld sizes.



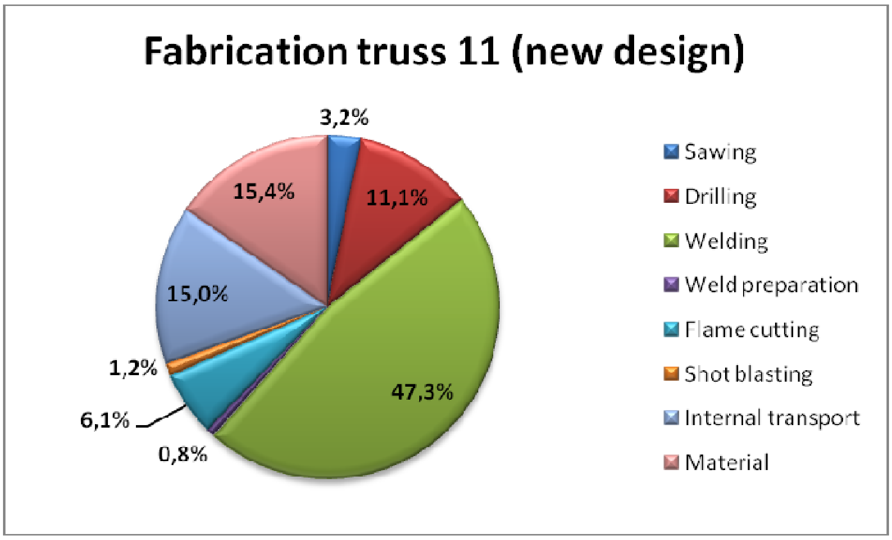
Graph 4-5: Relative costs current design (truss 10)



Graph 4-6: Relative costs new design (truss 10)



Graph 4-7: Relative costs current design (truss 11)



Graph 4-8: Relative costs new design (truss 11)

As shown in Table 4-30 the total cost reduction for fabrication is 11,3% for truss 10 and 9,6% for truss 11. These percentages are based on approximations of the required time for the different processes for each individual component. The costs required for welding and internal transport can however be accurately calculated which would decrease any errors in the estimations because these costs already cover about 2/3rd of the total fabrication costs. Any differences in the actual and assumed fabrication costs will therefore be small so the fabrication costs calculation is assumed to be valid for all cases.

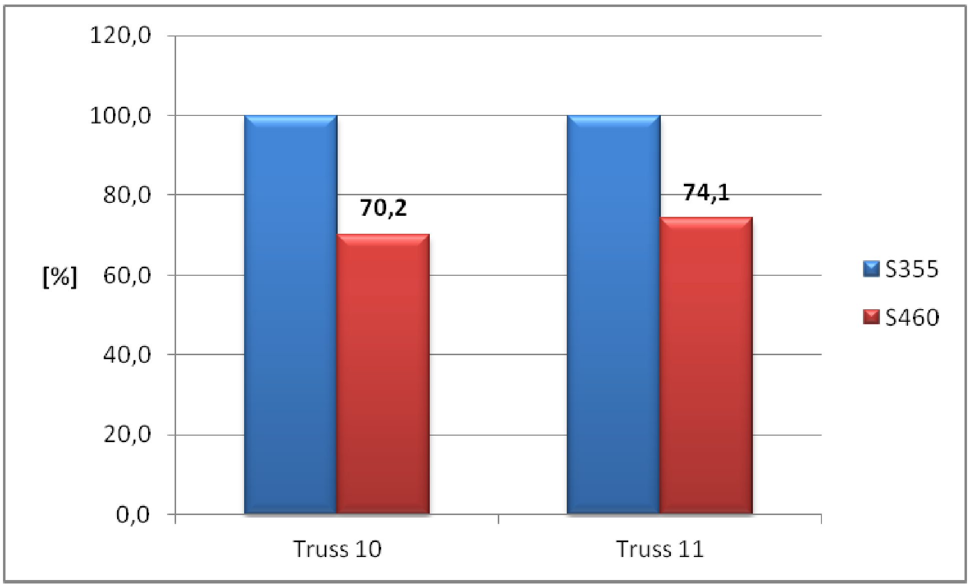
4.3.3 Transportation

As mentioned earlier there is a linear relationship between the weight and the transportation costs. The transportation costs also depend on the distance travelled. In this case the distance is between 60km and 80km so the transportation costs per truck are €337. The size is irrelevant so the costs only depend on the weight. The amount of trucks required is the total weight divided by 20t. The amount of trucks required for both trusses and the costs are shown in Table 4-31.

	Weight [t]	Trucks Amount	Costs [€]
Truss 10 current	65,6	3,3	1105
Truss 10 new	46,0	2,3	775
Truss 11 current	59,5	3,0	1002
Truss 11 new	44,0	2,2	742

Table 4-31: Transportation costs truss 10 and 11 for both designs

The amount of trucks required is not an integer. This is because single parts of the entire structure were reviewed. When a truck is not loaded to its capacity other parts of the structure could be placed on this truck as well so the costs of that truck could be divided over these parts, hence the use of decimals. The costs are relatively small compared to the material and fabrication costs. This is mainly due to the small distance to the construction site and also because special transportation is not required. The costs are related to the weight so the cost saving is equal to the weight saving of the entire truss including all the components required for the connections. The cost saving is 29,8% for the transportation of truss 10. Because in this case the costs are related to the weight the expected saving would be 23%. Because of the favorable effects of the use of higher strength steel the weight saving is higher than this value and because of that so is the cost saving. For truss 11 the cost saving is smaller (25,9%) and this is because of the reduced weight saving possible in this case. The relative costs of the new designs of both trusses compared to the current designs are shown in Graph 4-9.



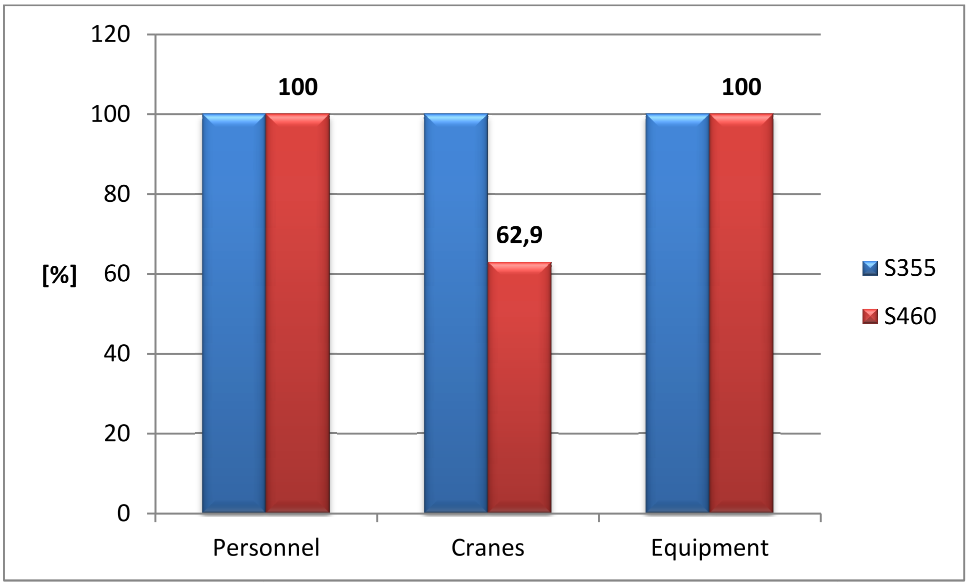
Graph 4-9: Cost reduction for transportation (truss 10 and 11)

4.3.4 Assembly

The assembly costs can be split in different parts: Costs for the construction crew and costs for the equipment required. For each part of the construction a certain amount of builders and equipment is required. In this case the parts of the trusses are assembled on site with the use of a single crane. The entire truss is lifted in place with the use of a heavy crane and a relatively light crane which is used to stabilize the truss and if necessary to carry a bit of the load. The highest point of one of the trusses is at 21m above ground level (for truss 10 and 11 this is 14m). At the supports boom lifts are required in order to properly install the trusses. Boom lifts with a working height of 22m are used.

The crane required to assemble the truss is based on the heaviest part it needs to lift. The heaviest and longest sections used are the chords of both trusses. As mentioned earlier the chords are split in 3 parts in order to be able to transport them. The distance between the position of the heaviest section and the crane is assumed to be 20m in all cases. The heaviest part of truss 10 executed with the use of S355 is 14,2t. According to Table 3-6 a 160t crane is required. For S460 the heaviest part is 8,0t which requires an 80t crane. The installation of the entire truss is done with the use of 2 cranes. A heavy crane which lifts most or all of the load and a 50t crane which is used to stabilize the truss. The 50t crane is also able to lift a part of the load. The distance between the crane and the center of the truss is about 26m. The weight of truss 10 is 65,6t for the current design and 46,0t for the new design. The current design of truss 10 leads to the use of a 700t crane. The new design leads to the use of a 400t crane. The capacity at 26m of this crane is 45,5t but as stated earlier the 50t crane can carry the remaining load.

In order to calculate the costs the time required for the assembly and the installation of the crane has to be known. It is assumed that the assembly of a single truss takes an entire day. The installation of a truss is assumed to take half a day. The calculation of the costs for the cranes are based on the daily costs. The costs for the boom lifts are based on the weekly costs because the assembly and installation of all the trusses will take several weeks. The boom lifts are part of the equipment other than cranes required at the construction site. This equipment is necessary regardless of the steel grade used. The costs for the equipment will therefore be the same for both designs. Although the truss weighs less when using higher strength steel the time required for assembly and installation is assumed to be the same. The costs for personnel will therefore also stay the same. The only costs that change are the costs for the cranes. The cost reduction for truss 10 is shown in Graph 4-10.



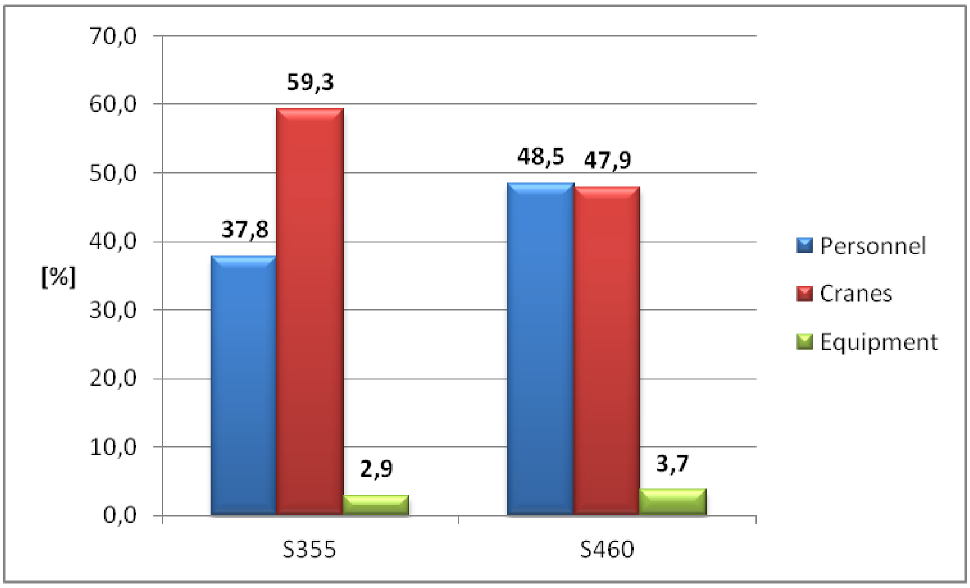
Graph 4-10: Assembly cost reduction (truss 10)

The reduction of the costs for cranes is 37,1%. This is very high and it is all the result of the weight reduction due to the use of higher strength steel. The lighter cranes cost far less than the heavier ones. Because of the weight reduction instead of a 160t and a 700t crane, an 80t and a 400t crane can be used. The difference in costs for these cranes is shown in Table 4-32.

	Current [€/d]	New [€/d]	Saving [%]
Assembly	2000	1160	42,0
Installation	5600	3440	38,6

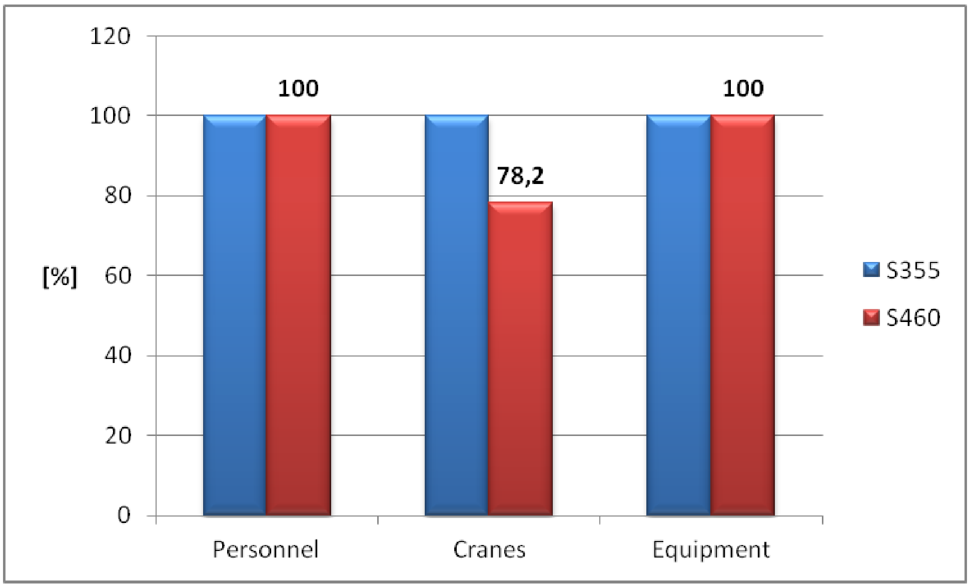
Table 4-32: Cost saving for cranes (truss 10)

Because the cost saving of these cranes is very high the total saving for the cranes is also high. The use of cranes is relatively expensive which is shown in Graph 4-11. It is visible that the costs for personnel become more significant when higher strength steel is used. This is because about the same amount of hours is required to built the construction so if other costs decrease these costs become more significant. The total saving for truss 10 is 22,0% which is due to the reduced costs for the required cranes. The costs for the equipment are relatively low. This is because only the costs for the boom lifts are taken into account. In reality other equipment is used as well for instance bolt fastening equipment.



Graph 4-11: Relative assembly costs (truss 10)

For truss 11 the heaviest part in the current design is 14,2t which requires a 160t crane. For the new design the heaviest part is 8,8t which requires a 120t crane. This is 0,8t heavier than the heaviest part of truss 10. In this case a 120t crane has to be used. This small increase in weight already results in the use of a much heavier crane. The total weight of truss 11 is 59,5t. In this case a 500t crane can be used which has a capacity of 58,1t at a distance of 26m. This is not enough for the entire truss but the smaller 50t crane can easily lift the remaining weight. The new design results in a total weight of 44,0t which requires a 400t crane. The cost reduction for truss 11 is shown in Graph 4-12.



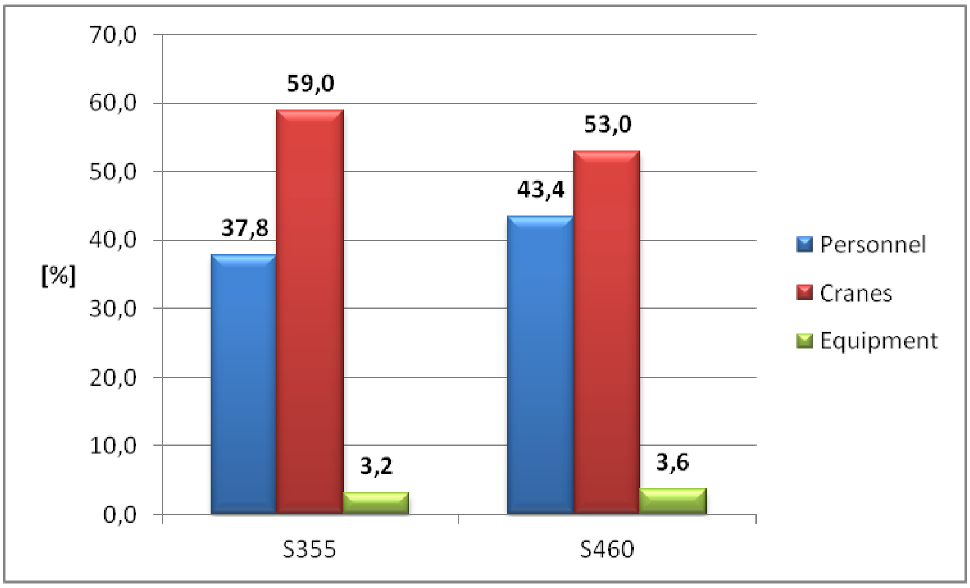
Graph 4-12: Assembly cost reduction (truss 11)

For cranes the cost reduction is 21,8%. This is a lot smaller than the percentage found for truss 10 but that is caused by the reduced weight reduction for this truss. Again smaller cranes can be used: A 160t crane is in this case replaced by a 120t crane and a 500t crane by a 400t crane. The cost difference for these cranes is shown in .

	Current [€/d]	New [€/d]	Saving [%]
Assembly	2000	1560	22,0
Installation	4600	3440	25,2

Table 4-33: Cost saving for cranes (truss 11)

The cost saving for these cranes is far less than the saving possible for truss 10. This is because the heaviest piece is heavier than the one in truss 10 and the decrease of the total weight of this truss which results in a smaller crane for the current design but not for the new design. The relative costs are shown in .



Graph 4-13: Relative assembly costs (truss 11)

In this case the cranes remain relatively more expensive. Obviously the other costs become more significant but not as much as for truss 10. The total cost reduction for truss 11 is 12,9% which is far less than the possible cost reduction found for truss 10.

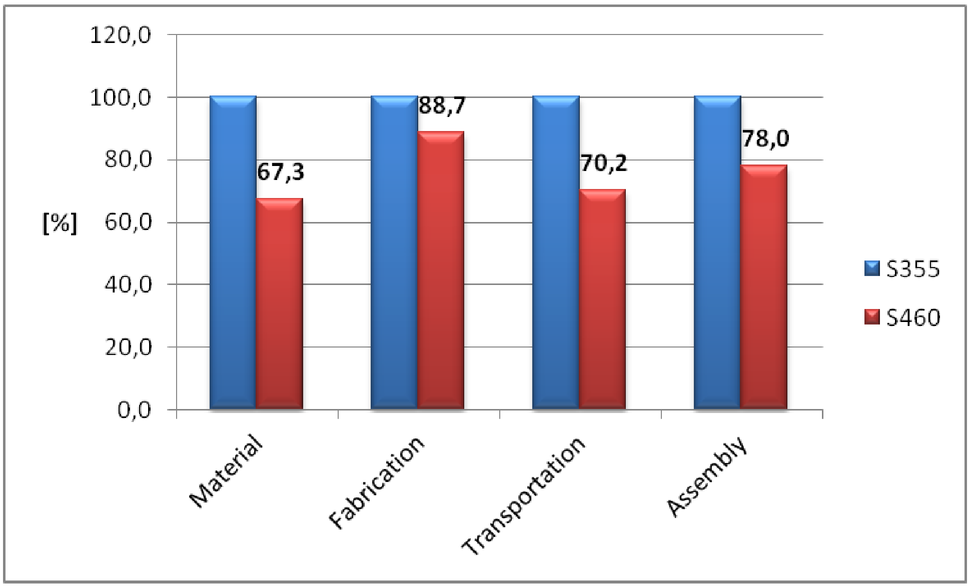
4.3.5 Conclusion

The cantilever truss enables great material savings in the chords, savings can also be expected when stronger steel grades are used in the verticals and diagonals. These savings can be relatively low and depend on the magnitude of the forces and the dimensions of the sections.

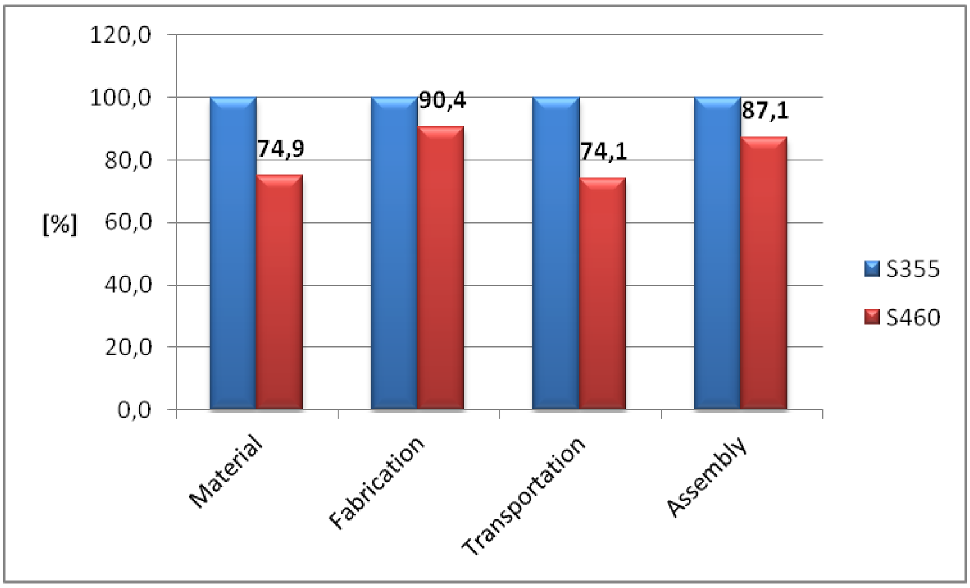
The fabrication costs for the new design are 11,3% less than the costs for the current design. This is mainly due to the reduced welding costs. The fabrication cost saving is not as high as the material cost saving but this can be expected since these costs depend on more than the weight of the individual components. Operations such as drilling will not give great savings when higher strength steel is used, not just because of the increased hardness of the material but mainly due to the costs that are always required regardless of the steel grade used (positioning for example).

The assembly costs can only be reduced by reducing the weight of the structure. The connections will not vary much when using higher strength steel so assembly of the individual components will take the same time. The height of the structure is not changed by the use of higher strength steel (maybe by a few centimeters but in this case that is negligible) so the same boom lifts are required, also the equipment required for fastening bolts for example stays the same so the equipment costs are also unchanged. The only thing that remains are the cranes. A large saving is possible by using higher strength steel because the weight reduction will lead to the use of cheaper cranes.

The transportation cost saving is in this case also very high. This is because the cost saving is linearly related to the weight of the structure which was decreased significantly with the use of S460. The cost reduction of these categories is shown in Graph 4-14 for truss 10 and in Graph 4-15 for truss 11.

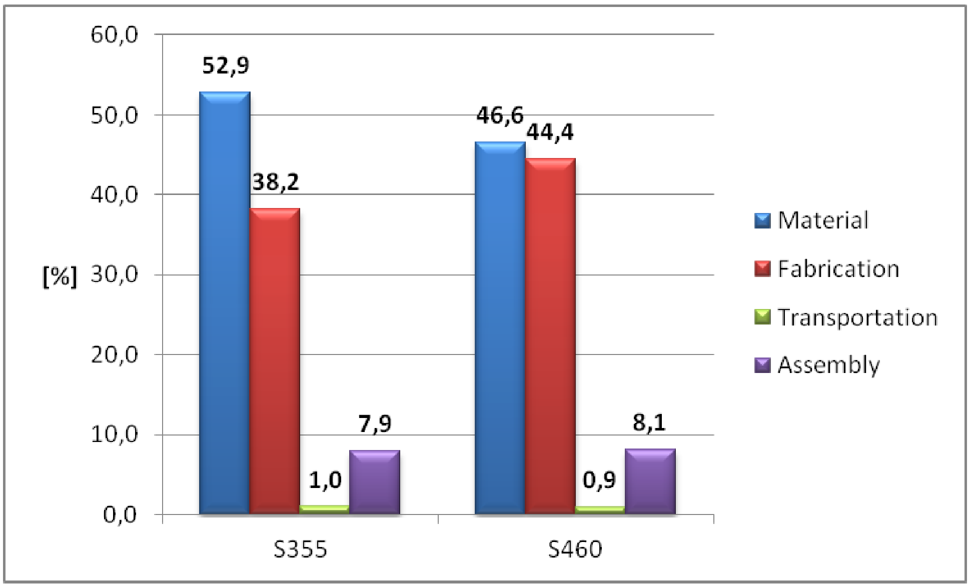


Graph 4-14: Cost reduction truss 10

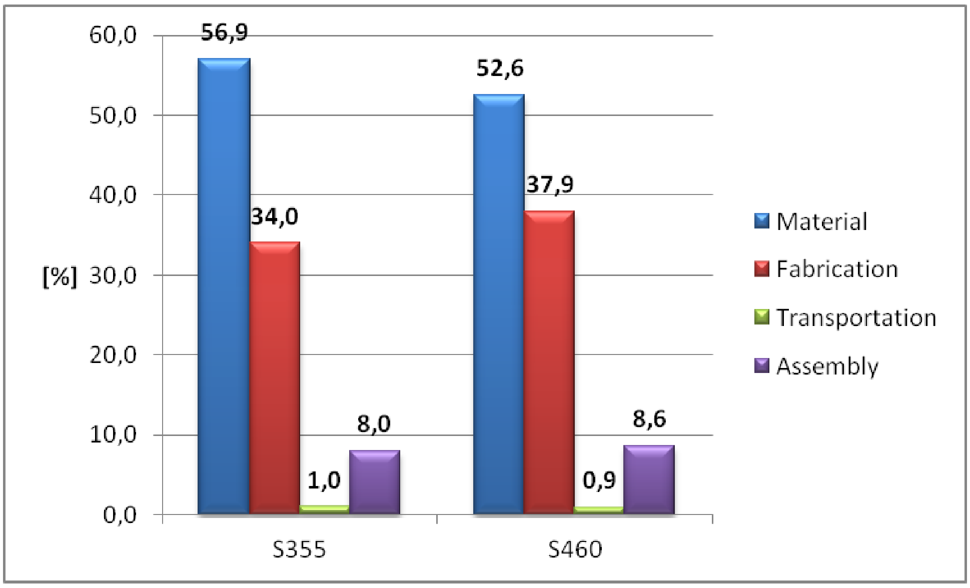


Graph 4-15: Cost reduction truss 11

It is visible that the costs for the used sections (material) are reduced the most for truss 10. This is because large savings were possible in the chords. The fabrication costs have not been reduced as much but that is because of the increased hardness of the material and because the weight is of less influence on the cost saving. It is also interesting to show the relative costs of materials, fabrication, assembly and transportation (Graph 4-16 for truss 10 and Graph 4-17 for truss 11).



Graph 4-16: Relative costs (truss 10)



Graph 4-17: Relative costs (truss 11)

Because heavy trusses are used in the current design the material costs of the sections are relatively the highest. When higher strength steel is used the weight and the costs of the sections significantly decreases. Fabrication costs also decrease but not as much as the material costs which makes the fabrication costs relatively more expensive in the new design than in the current design. The assembly costs are relatively a bit higher because only the costs for cranes can be reduced so the cost saving is not as high as the material cost saving. The transportation costs decreased significantly because of the large weight reduction. Because of this the relative costs have not changed much.

The total cost saving is 23,6% for truss 10 and 18,8% for truss 11. The difference of about 5% is because different truss types are used. Cantilever trusses therefore result in an increased cost saving.

It is also interesting to look at the influence of the cover plates on the total saving of the individual members. The material costs for the cover plates, the costs for flame cutting and the costs for welding the cover plates have been added to the costs for the section itself. This has been done for the members with cover plates in the current and in the new design. The total costs of the sections are shown in Table 4-34 for truss 10 and in Table 4-35 for truss 11.

Member	Costs [€]		Saving
	Current	New	[%]
Diagonal 3	1819	1866	-2,6
Diagonal 4	1895	1876	1,0
Diagonal 5	2791	2372	15,0

Table 4-34: Effect of cover plates on material saving (truss 10)

Member	Costs [€]		Saving
	Current	New	[%]
Diagonal 2	1512	1253	17,1
Diagonal 5	3195	2645	17,2

Table 4-35: Effect of cover plates on material saving (truss 11)

Diagonal 3 and 4 in truss 10 only need stiffening in the new design. It is visible that the costs for the sections in both designs is about the same. The costs for the current design have however been modified to make an accurate comparison. In this case the modification excludes the possible need of cover plates. As shown for diagonal 5 in truss 10 and diagonal 2 and 5 in truss 11 (sections in both designs require cover plates) the cost saving is still substantial.

By changing the steel grade a reduction in weight is possible. Based on the cost comparison between the members described in Table 4-34 and Table 4-35 an accurate conclusion cannot be made. Therefore for all sections in the new design which require cover plates a new design has been made. The design is based on the condition that no cover plates should be required. This leads to the use of larger sections. A comparison for the sections in truss 10 has been made and is shown in Table 4-36. In this table new 1 indicates the sections with cover plates and new 2 indicates the sections without cover plates.

Member	New 1	Costs	New 2	Costs
		[€]		[€]
Diagonal 3	HD400x187	1866	HD400x262	2159
Diagonal 4	HD400x187	1876	HD400x262	2173
Diagonal 5	HD400x187	2372	HD400x237	2594

Table 4-36: Effect of upsizing the member

It is shown that the costs for upsizing the member are larger than the costs for stiffening the member. It is therefore more economical to use smaller sections with cover plates than to use larger sections which do not need cover plates. The additional costs for cover plates are at most 20% of the costs for the relevant section. The upsized member should therefore have been at most 20% heavier than the relevant section.

4.3.6 Additional costs

Not all costs are factored in the calculation of the final costs. There are for instance many more connections in a single truss than the ones described and many additional columns and beam above or below the truss. The additional connections would result in increased fabrication and assembly costs. The graphs describing the relative costs (Graph 4-16 and Graph 4-17) show that the material costs are relatively the most expensive. In reality one would expect the fabrication costs to be relatively more expensive. This would probably be the case if all the connections were factored in the cost calculation. The objective of this paper is to examine the effect of the use of S460 in structures and therefore all the other connections and columns and beams are not factored in the calculations.

As described earlier the carbon equivalent (CEV) could be of influence on the fabrication costs. This is also dependent on the combined thickness of the connections. The largest combined thickness will be obtained for the welds that connect the gusset plates to the chords. In this case the combined thickness is the thickness of the gusset plate + 2 times the thickness of the flange. The maximum combined thicknesses are:

- 223 mm for the current design (CEV = 0,47)
- 146 mm for the new design (CEV = 0,47)

For both designs the carbon equivalent is the same. In both cases a weld consumable in class D is required in combination with a heat input of 2,0 kj/mm to prevent the need for preheating. If a heat input of 1,5 kj/mm is used a preheating temperature of 20˚ is required. For the current design the carbon equivalent is valid for S355J2. If S355M was used for the sections the carbon equivalent would be 0,45 and no preheating would be required with a heat input of 1,5 kj/mm. The use of S460 will lead to increased fabrication costs when compared to S355M.

5 Columns: BP4, high-rise office building

5.1 Introduction

In London England the construction of a high-rise office has recently been completed. The structure is 24 stories high. The structure consist of a concrete core and steel-concrete composite floors which are supported by 29 steel columns and the concrete core (Figure 5-1).

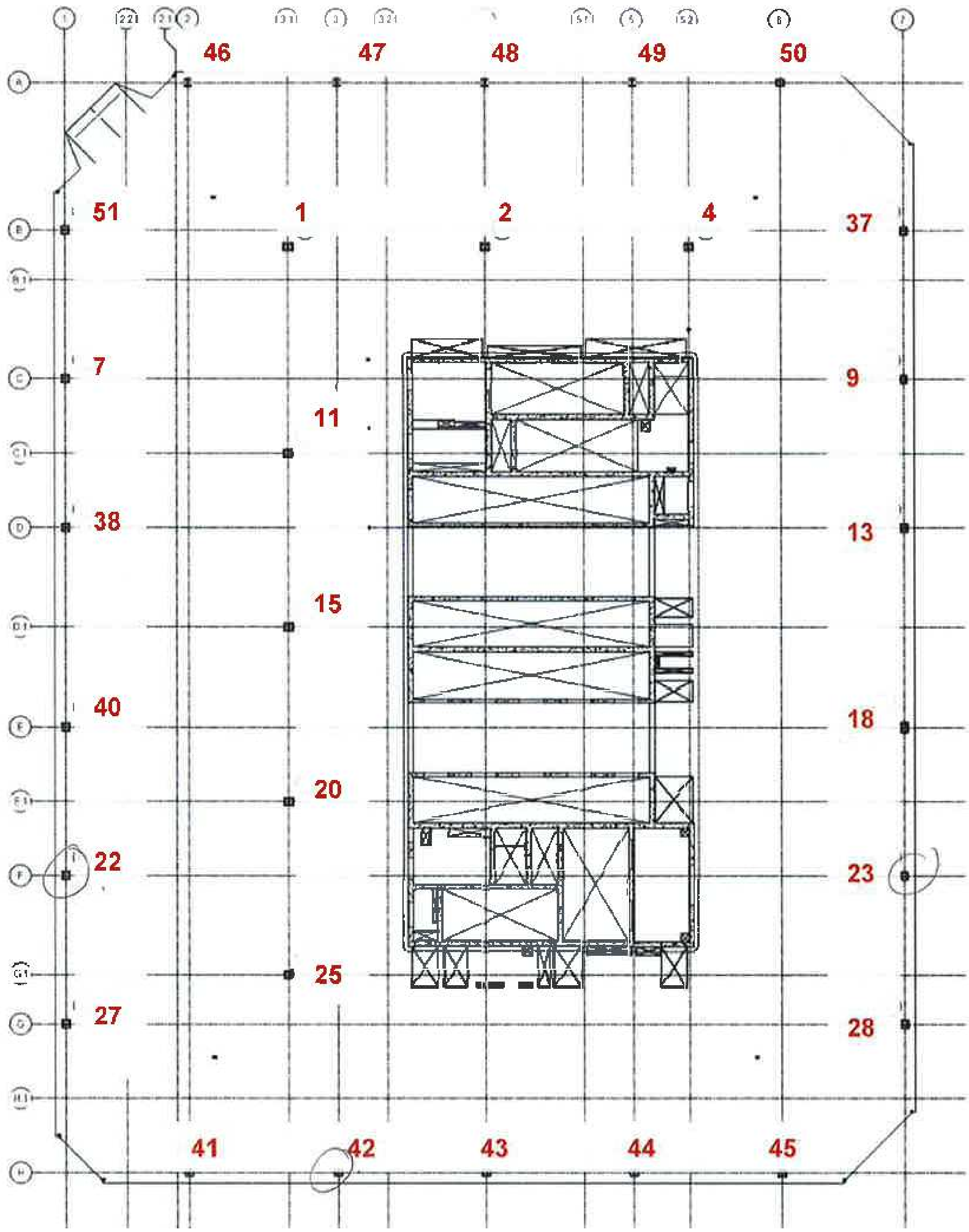


Figure 5-1: Column numbers and locations

All floors are mainly designated as office floors. In some locations space has been reserved for installations. Between the 12th and 13th floor there is a mezzanine. The distance between the columns is 9 m in both directions. The columns are used to support beams which support the steel-concrete composite floors or the façade of the building. The façade of the building is not supported in horizontal direction by the columns so no direct bending moments can occur in the columns due to the loads on the façade. All columns are connected to the concrete core which is located in the center of the building. Almost all columns are supported in horizontal direction at all levels. The columns can therefore be designed as axially loaded simply supported columns. Bending moments can only occur due to the eccentricity of the connected beams.

The columns are designed with the use of S355. The distance between the floors is not much (mostly about 4-5m) so buckling will not lead to a great reduction of the axial capacity of the columns. The largest section used is UC356x406x634. This is a British section and it is equivalent to the HD400x634 section. The used section is the largest section available in the UC range. Because this is the largest column used additional material is required for the columns to be able to resist the loads at the lower levels. To increase the cross-sectional area and the stiffness about the weak axis of the columns plates parallel to the webs are welded to the flanges (Figure 5-2 left). This is mainly because the cross-sectional area is not large enough. The increase in buckling resistance is small because of the small buckling lengths. Even if the largest HD sections were used (HD400x1299) the resistance of the bottom columns would still not be large enough because of the high loads present. The UC356x406x634 sections are stiffened with plates up to 100mm thick. In some cases even these sections will not suffice. If that is the case box sections are used with a height and width of 600mm and a thickness of 100mm for the flanges and webs (Figure 5-2 right). This is only necessary for 2 columns at the ground floor.

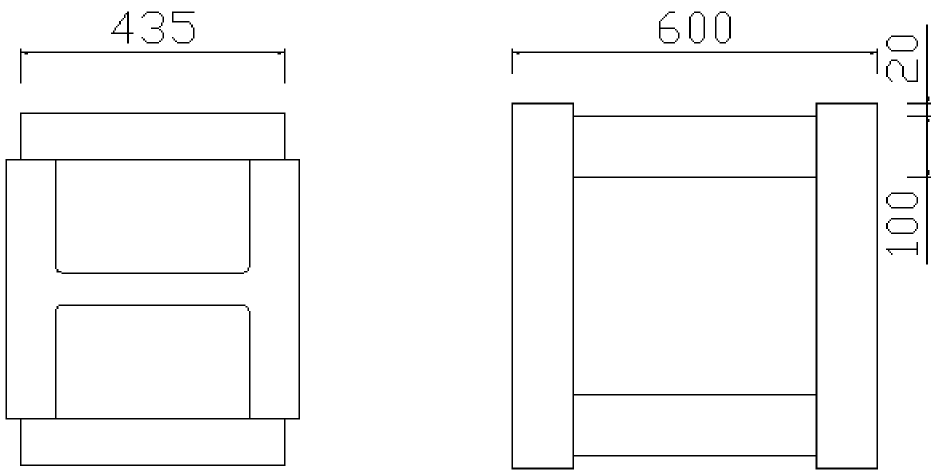


Figure 5-2: Box sections

Some of the columns are not horizontally supported at floors 1 and 12M. Floor 12M is a mezzanine which is located between floors 12 and 13. Because of the missing support the unsupported column length can be up to 10,46 m. For these columns HD sections will be ineffective because buckling about the weak axis will significantly reduce the bearing capacity of these columns. Therefore box sections built up from HD sections and 2 plates welded to the sides (parallel to the web) are also considered.

Columns that are not supported at floor 1 are: Column 1, 2, 4, 7, 11, 37, 38 and 46-51. Columns that are not supported at floor 12M are: Column 1, 2, 4, 7, 11, 15, 25, 27, 38, 40-44, 46-49 and 51. The height between the floors is not constant. For most floors the height is 3,95 m (c.t.c.). Column lengths and their level heights are shown in Table 5-1. The level heights are based on the ground floor being at 0,0 m which in reality is not the case.

Level	Column length	Level height
Roof	N/A	115,920
level 23	6,890	109,030
level 22	7,000	102,030
level 21	6,040	95,990
level 20	4,350	91,640
level 19	3,950	87,690
level 18	3,950	83,740
level 17	3,950	79,790
level 16	3,950	75,840
level 15	3,950	71,890
level 14	3,950	67,940
level 13	4,010	63,930
level 12M	4,410	59,520
level 12	5,410	54,110
level 11	4,350	49,760
level 10	3,950	45,810
level 9	3,950	41,860
level 8	3,950	37,910
level 7	3,950	33,960
level 6	3,950	30,010
level 5	3,950	26,060
level 4	5,200	20,860
level 3	5,380	15,480
level 2	5,020	10,460
level 1	4,700	5,760
Ground floor	5,760	0,000

Table 5-1: Column lengths and level heights

Almost all medium to heavily loaded columns have a buckling length between 4-5 m which makes the use of S460 probably very effective. At the top of the structure relatively small loads are present in the columns but the buckling lengths are 6-7 m. When S460 is used this would result in very slender columns which would make the use of a stronger steel grade less effective because slender columns are more prone to buckling.

The used sections in the current design are shown in Appendix III.

5.2 Column design

The occurring forces in each column were not given therefore an exact determination of the required sections cannot be made. The columns are mainly loaded in compression and this is the basis for the determination of the required sections. All sections used in the current design are shown in Table 5-2 and Table 5-3. Their relevant (maximum) buckling lengths are also given for 2 situations:

1) "Short" columns: Supported at consecutive floors
2) "Long" columns: Unsupported at floor 1 or 12M

For these situations the buckling resistances have been calculated according to Appendix III. The buckling resistance is the axial capacity of the section multiplied by the reduction factor for buckling.

Name	Buckling length [mm]	Governing Floor	Nb,Rd [kN]
UC356x406x634 + 2x100PL	5380	3rd	48442
UC356x406x634 + 2x75PL	5760	Ground	44073
UC356x406x634 + 2x60PL	5380	3rd	40160
UC356x406x634 + 2x50PL	5380	3rd	37097
UC356x406x634 + 2x40PL	5380	3rd	34015
UC356x406x634 + 2x30PL	5380	3rd	30900
UC356x406x634 + 2x20PL	4350	11th	29435
UC356x406x634	5380	3rd	20916
UC356x406x551	5410	12th	17977
UC356x406x467	7000	22nd	12778
UC356x406x393 + 2x50PL	5380	3rd	26435
UC356x406x393 + 2x40PL	5410	12th	23782
UC356x406x393 + 2x30PL	3950	5th	22738
UC356x406x393	7000	22nd	10619
UC356x406x340	7000	22nd	9100
UC356x406x287	7000	22nd	7861
UC356x406x235	7000	22nd	6368
UC356x368x202	6890	23rd	5257
UC305x305x240	6890	23rd	5220
UC305x305x198	7000	22nd	4152
UC305x305x158	6890	23rd	3308
UC254x254x167	6890	23rd	2862
UC254x254x132	6890	23rd	2205
UC203x203x86	6890	23rd	1014

Table 5-2: Column lengths and buckling resistances ("short" columns)

Although 3,95 m is the most commonly used floor height almost all columns have a governing buckling length of 5 m or larger. The relative slenderness (λ_{rel}) varies from 0,334 (the column with the smallest buckling length) to 1,688 (the smallest column with a large buckling length). For almost all columns the relative slenderness is between 0,2 and 1,2. This particular range is mainly of interest for columns in high-rise buildings.

Name	Buckling length [mm]	Governing Floor	Nb,Rd [kN]
SHS600x100	10460	Ground	51142
UC356x406x634 + 2x100PL	10460	Ground	37531
UC356x406x634 + 2x75PL	10460	Ground	34617
UC356x406x634 + 2x60PL	10460	Ground	30567
UC356x406x634 + 2x50PL	10460	Ground	27563
UC356x406x634 + 2x40PL	10460	Ground	24529
UC356x406x634	9820	12th	12639
UC356x406x551	9820	12th	10768
UC356x406x393 + 2x50PL	10460	Ground	19508
UC356x406x393 + 2x40PL	9820	12th	18234
UC356x406x340	9820	12th	6290

Table 5-3: Column lengths and buckling resistances ("long" columns)

When the columns are not supported at a certain floor the relative slenderness becomes larger therefore reducing the buckling resistance of the sections. In this case the relative slenderness varies from 0,649 to 1,200. All the values are in the range described earlier.

The relative slenderness is an important factor used in the calculation of the reduction factor (χ) for buckling. Combined with an imperfection factor the reduction factor can be calculated. There are 5 different imperfection factors and for each factor an buckling curve can be drawn which shows the reduction factor as a function of the relative slenderness only. These curves are displayed in Figure 5-3.

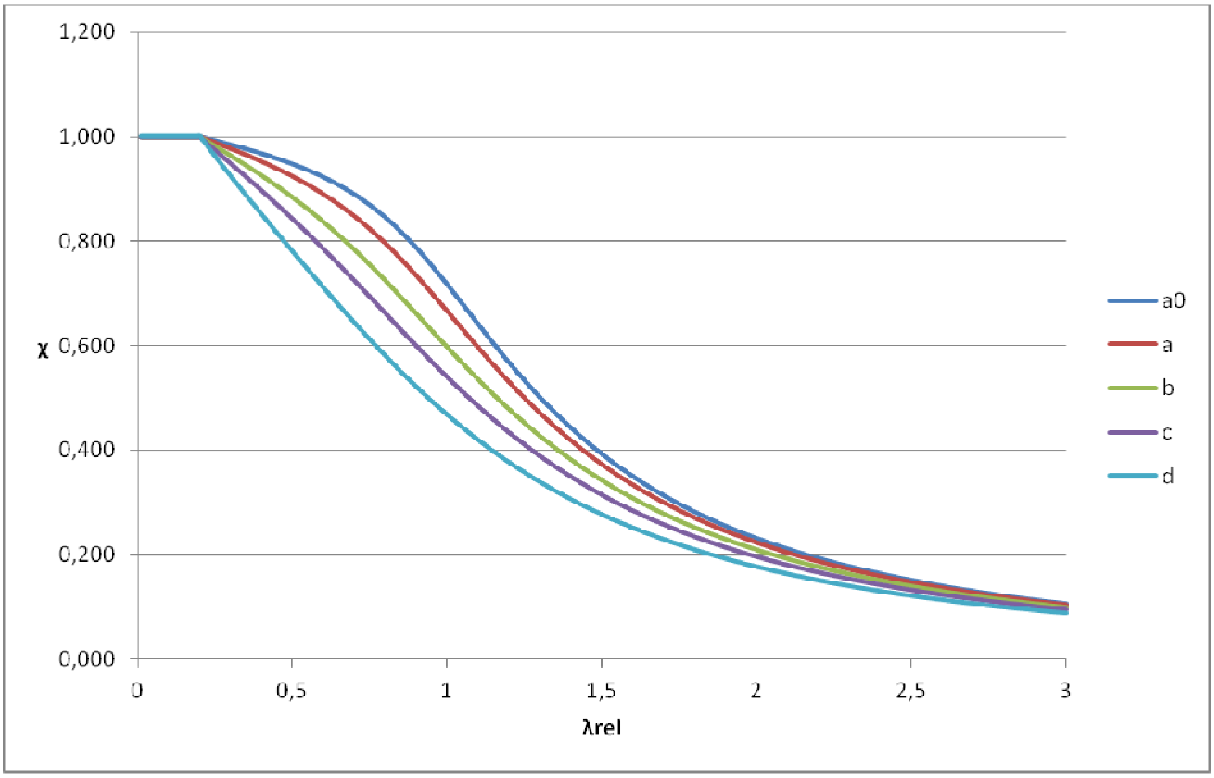


Figure 5-3: Buckling curves

The use of a certain buckling curve depends on the section used and the steel grade. For steel grades up to and including S420 the following curves should be used:

- a0: Never
- a: Buckling about the strong axis for all I-sections and HE360+ sections
- b: Buckling about the weak axis for all I-sections and HE360+ sections, buckling about both axis for welded box sections and buckling about the strong axis for all other H-sections
- c: Buckling about the weak axis for all H-sections with thicknesses of maximum 100 mm
- d: Buckling about both axis for all H-sections with a thickness of more than 100 mm

buckling curve d is applicable for HD400x900 and larger sections. For the steel grade S460 the following curves should be used:

- a0: Buckling about both axis for all I-sections and HE360+ sections
- a: Buckling about both axis for all H-sections with thicknesses of maximum 100 mm
- b: Welded box sections
- c: Buckling about both axis for all sections with a thickness of more than 100 mm
- d: Buckling about the weak axis for welded H-sections

In high-rise buildings steel HD (or the British UC) sections can be used as columns because of their large axial capacity and stiffness. For S355 this means that buckling curve c or d must be used depending on the thickness of the section. Buckling curve b is generally not used because it is used for buckling about the strong axis and in this case the buckling lengths about both axis are the same which means that buckling about the weak axis is always governing the design. For welded box sections buckling curve b may be used. When however a box section consists of an H-section with 2 plates welded to the sides buckling curve c should be used for buckling about the weak axis.

When H-sections are used buckling curve c is used for S355 and curve a is used for S460. A comparison between these curves for the most relevant range (0,2-1,2) is shown in Figure 5-4.

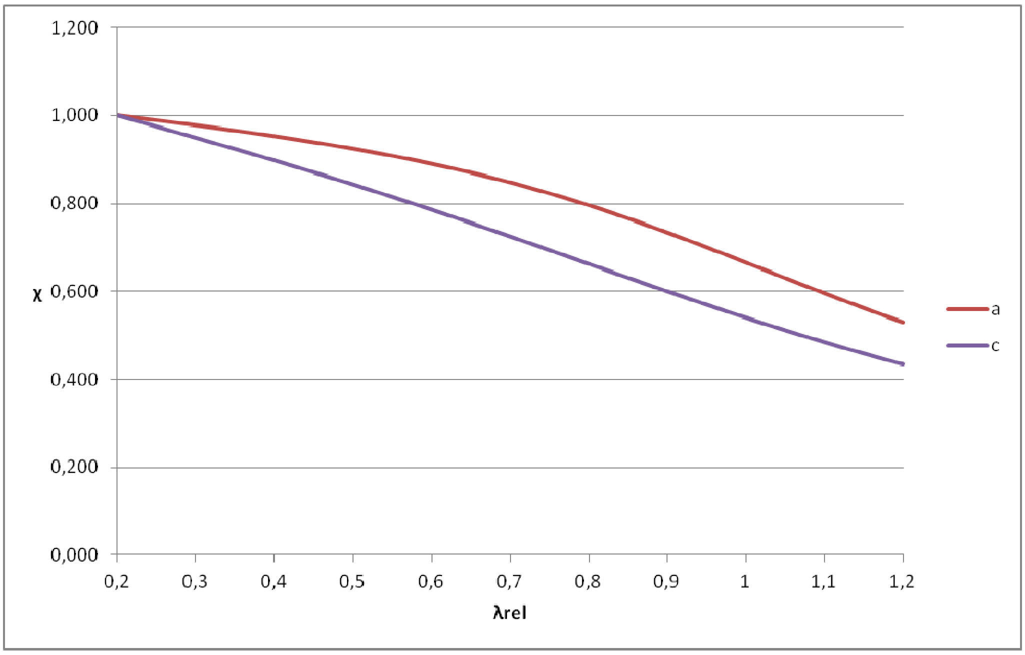


Figure 5-4: Buckling curve a and c

When buckling is not of importance (χ=1) and similar sections with steel grades S355 and S460 steel are compared the section with grade S460 will have a 460/355 = 1,296 times larger capacity than the one with the grade S355. When similar sections are used and buckling is of importance the increase in capacity could be even larger. The capacity depends on the relative slenderness which is dependent on the stiffness of the section, the buckling length and the axial capacity of the section. When a stronger steel grade is used the axial capacity increases but the stiffness and the buckling lengths does not increase. Therefore the relative slenderness will be larger when a stronger steel grade is used which results in a smaller reduction factor. When buckling is not of importance the expected weight saving is 23%. When buckling is of importance (which is the case for every column in this building) the saving could be larger or smaller dependent on the increase in relative slenderness. In order to determine if the saving is larger or smaller the horizontal distance between the 2 buckling curves should be examined. For most of the values in the described range this horizontal distance is about 0,2. When the relative slenderness of a certain section (S355) is 0,9 this will lead to a reduction factor of about 0,6. To obtain the same reduction factor when S460 is used the relative slenderness will be 1,1. Based on the difference between the relative slenderness of certain sections in both steel grades the weight saving can be estimated. When the difference is smaller than 0,2 the saving will probably be larger than 23%, when the difference is however larger the saving will probably be smaller than 23%.

When box sections are used buckling curve b may be used when the section is an SHS section or when the capacity in the weak direction of the used H-section is larger than the capacity in the strong direction. In these cases the expected saving will be smaller because the allowable difference between the relative slenderness of the box section with steel grade S355 and a section with steel grade S460 will be smaller. This is especially the case when box sections are used with the steel grade S460. In this case the expected saving is smaller than 23% because for both steel grades the buckling curve b must be used. The design with S460 will in this case always lead to a larger relative slenderness and thus a smaller reduction factor reducing the effectiveness of the use of S460 in box sections.

The welds connecting the plates to the flanges are based on the minimum requirements as stated in the AISC (Figure 5-5). The minimum required weld size is also calculated with the use of initial imperfections (Appendix III). The calculated sizes are however smaller than the minimum required sizes. For the SHS section and the UC356x406x634 section welds with an effective throat of 13 mm are required (v = 15 mm). For the UC356x406x392 section an effective throat of 10 mm is required (v = 12 mm).

Plate thickness of thicker part jointed (In.)	Min. eff. throat (In.)	Plate thickness of thicker part jointed (mm)	Min. eff. throat (mm)
To 1/4 inclusive	1/8	To 6 inclusive	3
Over 1/4 to 1/2	3/16	Over 6 to 13	5
Over 1/2 to 3/4	1/4	Over 13 to 19	6
Over 3/4 to 1-1/2	5/16	Over 19 to 38	8
Over 1-1/2 to 2-1/4	3/8	Over 38 to 57	10
Over 2-1/4 to 6	1/2	Over 57 to 150	13
Over 6	5/8	Over 150	16

Figure 5-5: Minimum weld sizes

5.2.1 Sections

The used columns in the current design are shown in Table 5-2 and Table 5-3. The columns in the new design are based on the axial bearing capacity of these columns. This capacity should be at least the value shown in the tables. If the bearing capacity is larger than the column in the current design could be replaced by the column in the new design if the calculation is based on the axial force only. In reality bending moment and shear forces could also occur in the columns but these are unknown. Even though the bearing capacity will be larger the column could not be satisfactory when certain bending moments are present. This is because the reduced size of the columns significantly reduces the stiffness and the bending moment resistance of the columns which in turn could lead to too high stresses resulting from a combination of compression and bending. Therefore the weight and cost saving is not scaled (the modified saving is not used) and the saving is assumed to be an upper limit of the possible saving. When bending moments are present the saving will be smaller than the calculated saving based on compression only.

The sections used for "short" columns in the new design are shown in Table 5-4. The calculation of the bearing capacity is shown in Appendix III.

Section	Buckling length [mm]	Governing Floor	Nb,Rd [kN]
HD400x677 + 2x40PL	5380	3rd	49186
HD400x677 + 2x30PL	5760	Ground	43890
HD400x677 + 2x20PL	5380	3rd	40387
HD400x818	5380	3rd	39889
HD400x744	5380	3rd	36071
HD400x634	5380	3rd	31027
HD400x592	4350	11th	30998
HD400x463	5380	3rd	22295
HD400x382	5410	12th	18225
HD400x347	7000	22nd	13536
HD400x551	5380	3rd	26736
HD400x509	5410	12th	24605
HD400x463	3950	5th	24605
HD400x287	7000	22nd	11056
HD400x262	7000	22nd	10036
HD400x216	7000	22nd	8171
HD400x187	7000	22nd	6974
HD360x162	6890	23rd	5773
HD360x147	6890	23rd	5222
HD360x134	7000	22nd	4628
HD360x134	6890	23rd	4721
HD320x127	6890	23rd	3264
HD320x97,6	6890	23rd	2478
HD260x68,2	6890	23rd	1362

Table 5-4: Column lengths and buckling resistances ("short" columns)

It can be seen that less box sections are used in the new design. This is because larger HD sections can be used and the stronger steel grade significantly increases the buckling resistance. The relative slenderness varies from 0,510 (the largest box section) up to 1,579 (the smallest H-section HD260x68,2). It is possible to replace the box sections in the new design by large HD sections but the weight of these sections would be larger and for these sections buckling curve c must be used which significantly reduces the effectiveness of these sections. The box sections are made with the use of HD400x677 sections because this still enables the use of a yield strength of 460 N/mm^2.

The sections used for "long" columns are shown in Table 5-5. In this case a lot of box sections are used because the stiffness in the weak direction of H-sections is too small to provide the required capacity. The relative slenderness varies from 0,759 (the SHS section) up to 1,422 (the smallest H-section HD400x287). In some cases plates with a thickness larger than 40 mm have to be used. For these plates the yield strength is 430 N/mm^2. In these cases the buckling resistances have been calculated with the use of this yield strength for the entire section.

Section	Buckling length [mm]	Governing Floor	Nb,Rd [kN]
SHS600x80	10460	Ground	51546
HD400x677 + 2x55PL	10460	Ground	38306
HD400x677 + 2x45PL	10460	Ground	34691
HD400x677 + 2x40PL	10460	Ground	33679
HD400x677 + 2x30PL	10460	Ground	29151
HD400x677 + 2x20PL	10460	Ground	24558
HD400x509	9820	12th	13119
HD400x463	9820	12th	11712
HD400x382 +2x40PL	10460	Ground	20829
HD400x382 +2x25PL	9820	12th	18390
HD400x287	9820	12th	6865

Table 5-5: Column lengths and buckling resistances ("long" columns)

The smallest HD section is an HD400x54,1. This section is not used because according to the Eurocode the section is in class 4. Class 4 sections have to be designed by taking into account local buckling. In this case the buckling resistance (global buckling) is barely larger than the resistance of the original section. It is therefore assumed that this section does not satisfy this condition when local buckling is taken into account. As a consequence the weight saving will be smaller than expected because a larger section has to be used whose buckling resistance is much larger.

Long columns will lead to the use of box sections instead of H-sections because of their increased stiffness in the weak direction. The use of these box sections will lead to a decrease in material costs but an increase in fabrication costs. It is therefore not yet known if these sections should be preferred instead of H-sections. When the loads are small (at the top columns of a building for instance) the use of S460 could lead to sections which are in class 4. These sections are not effective because local buckling limits their capacity.

The weight saving of the sections is shown in Table 5-6 for “short” columns and in Table 5-7 for “long” columns. In the tables the difference between the relative slenderness in the new design and in the current design is shown. The difference is in this case always less than 0,2 for H-sections which implies that the saving is expected to be larger than 23% which is not always the case. This is because the difference of 0,2 is based on the stiffness of the section only. The relative slenderness is defined as: $\lambda_{rel} = \sqrt{(N_{pl,rd}/N_{cr})}$ where $N_{pl,rd} = A x f_y$ and $N_{cr} = \pi^2 x EI/L_k^2$. If for a column with steel grade S355 and a column with steel grade S460 (or HISTAR460) the plastic axial capacities ($N_{pl,rd}$) are the same, a difference of 0,2 for the relative slenderness will indeed lead to a weight saving of approximately 23%. The plastic axial capacities will however always differ. A difference of 0,2 for the relative slenderness will still lead to the same reduction factor, but these reduction factors are multiplied with different axial capacities which results in weight savings larger or smaller than expected. In some case the difference is negative. A negative value means that the relative slenderness in the new design is smaller. The negative values are unexpected because they indicate an increase in the radius of gyration ($\sqrt{I/A}$) compared to the used sections in the current design. This increase is the result of more favorable dimensions of the smaller HD sections compared to the smaller UC sections. As a result the plastic axial capacity of the sections is sometimes smaller or the same but the stiffness is always larger which leads to a decrease in the relative slenderness.

Current section	Weight [kg/m]	New section	Weight [kg/m]	Weight saving [%]	λ Difference
UC356x406x634 + 2x100PL	1316,8	HD400x677 + 2x40PL	954,1	27,5	0,087
UC356x406x634 + 2x75PL	1146,1	HD400x677 + 2x30PL	885,0	22,8	0,189
UC356x406x634 + 2x60PL	1043,7	HD400x677 + 2x20PL	815,9	21,8	0,193
UC356x406x634 + 2x50PL	975,4	HD400x818	818,8	16,1	0,284
UC356x406x634 + 2x40PL	907,1	HD400x744	744,3	17,9	0,270
UC356x406x634 + 2x30PL	838,8	HD400x634	634,3	24,4	0,263
UC356x406x634 + 2x20PL	770,5	HD400x592	592,6	23,1	0,189
UC356x406x634	633,9	HD400x463	462,8	27,0	0,131
UC356x406x551	550,4	HD400x382	382,4	30,5	0,135
UC356x406x467	467,0	HD400x347	347,0	25,7	0,166
UC356x406x393 + 2x50PL	691,3	HD400x551	550,6	20,3	0,337
UC356x406x393 + 2x40PL	631,6	HD400x509	509,5	19,3	0,320
UC356x406x393 + 2x30PL	572,0	HD400x463	462,8	19,1	0,218
UC356x406x393	393,0	HD400x287	287,5	26,8	0,167
UC356x406x340	339,9	HD400x262	262,7	22,7	0,164
UC356x406x287	287,1	HD400x216	216,3	24,7	0,139
UC356x406x235	234,7	HD400x187	186,5	20,5	0,139
UC356x368x202	201,9	HD360x162	161,9	19,8	0,143
UC305x305x240	240,1	HD360x147	147,5	38,6	-0,018
UC305x305x198	198,1	HD360x134	133,9	32,4	-0,031
UC305x305x158	158,1	HD360x134	133,9	15,3	-0,050
UC254x254x167	167,1	HD320x127	126,6	24,2	0,032
UC254x254x132	132,0	HD320x97,6	97,7	26,0	0,023
UC203x203x86	86,0	HD260x68,2	68,2	20,8	-0,109

Table 5-6: Weight saving “short” columns

Current section	Weight [kg/m]	New section	Weight [kg/m]	Weight saving [%]	λ Difference
SHS600x100	1570,0	SHS600x80	1256,0	20,0	0,110
UC356x406x634 + 2x100PL	1316,8	HD400x677 + 2x55PL	1057,7	19,7	0,077
UC356x406x634 + 2x75PL	1146,1	HD400x677 + 2x45PL	988,6	13,7	0,105
UC356x406x634 + 2x60PL	1043,7	HD400x677 + 2x40PL	954,1	8,6	0,230
UC356x406x634 + 2x50PL	975,4	HD400x677 + 2x30PL	885,0	9,3	0,256
UC356x406x634 + 2x40PL	907,1	HD400x677 + 2x20PL	815,9	10,0	0,291
UC356x406x634	633,9	HD400x509	509,5	19,6	0,225
UC356x406x551	550,4	HD400x463	462,8	15,9	0,222
UC356x406x393 + 2x50PL	691,3	HD400x382 +2x40PL	621,0	10,2	0,139
UC356x406x393 + 2x40PL	631,6	HD400x382 +2x25PL	531,5	15,9	0,233
UC356x406x340	339,9	HD400x287	287,5	15,4	0,222

Table 5-7: Weight saving "long" columns

The total weight saving is shown for columns 1-51 (29 columns) and for the different sections. When examining the weight saving for the column numbers the total decrease in load can be calculated based on an estimated design load. Because the unit weight of columns is always relatively small compared to the applied loads the decrease in load is not expected to be large. The weight saving per column is shown in Table 5-8.

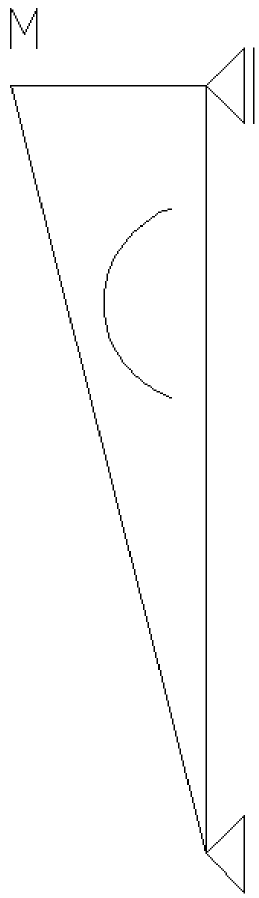
The weight saving averages to about 22% which is quite close to the expectation of 23%. The expected value was calculated without taking account of a reduced resistance due to buckling. The total saving implies that buckling has almost the same influence on the resistance for grade S355 and S460. This is not entirely true. The relative slenderness will usually increase when stronger steel grades are used but because of the more favorable buckling curve which may be used and the better yield strength ratio for thick sections the total saving is still close to the expected value.

The decreased weight results in a decreased load. This is mainly noticeable at the bottom of the columns. As shown in Table 5-3 the SHS600x100 section has a buckling resistance of 51546 kN. The design load is therefore estimated to be 40000 kN (bending moments could be present so about 80% of the buckling resistance is used). Table 5-8 shows the total decrease in force which is related to the decrease in weight. For column 1 this decrease is 224 kN which is only 0,5% of the estimated design load. The design load for the columns and the foundations are barely (if at all) affected by the decreased weight of the columns so it is assumed that additional savings due to the reduced load are not possible.

Column	Current Weight [kg]	New Weight [kg]	Saving [%]	Force reduction [kN]
1	103089	80696	21,7	224
2	83417	66971	19,7	164
4	74455	58995	20,8	155
7	60609	47873	21,0	127
9	59131	44871	24,1	143
11	92971	73458	21,0	195
13	69554	54358	21,8	152
15	80407	63422	21,1	170
18	76777	59387	22,7	174
20	81776	64420	21,2	174
22	60609	47105	22,3	135
23	58754	45041	23,3	137
25	78463	61087	22,1	174
27	60049	45695	23,9	144
28	62508	47348	24,3	152
37	64931	50492	22,2	144
38	68065	54909	19,3	132
40	65473	50606	22,7	149
41	50592	38273	24,3	123
42	57968	44318	23,5	137
43	57131	43757	23,4	134
44	57675	44049	23,6	136
45	51791	39502	23,7	123
46	51610	40655	21,2	110
47	47359	36844	22,2	105
48	47359	36844	22,2	105
49	42882	33135	22,7	97
50	51692	40293	22,1	114
51	63466	50313	20,7	132
Total	1880562 kg	1464715 kg	22,1 %	4158 kN

Table 5-8: Weight and weight saving per column and force reduction

The columns used in the current design have been replaced by columns with at least the same buckling resistance in the new design. Therefore the saving based on axial compression only could be larger than 22,1%. Possible bending moments have however not been included. When bending moments are present the axial capacity of the sections decreases. In this case bending moments can only occur due to the eccentricity of beam-column connection. For perimeter columns this bending moment is the largest because internal beams are connected on one side of the column only. For internal columns the bending moments can be smaller because on both sides of the columns beams can be connected. The spacing between the columns is constant in this case so the difference in force transferred from the beams to the column can be much smaller than the difference in force transferred to perimeter columns. Therefore the saving for the perimeter columns could be less than the saving for internal columns.



In order to check the influence of the bending moments on the design compression resistance a simplified model of the columns is used. The beam reaction at the top of the column multiplied by the eccentricity of the connection results in the bending moment. This bending moment is assumed to act on the column below only (Figure 5-7). This is a conservative assumption because in reality a part of the bending moment will also be transferred through the column above the connection (the columns are continuous). The column are check for a combination of compression and bending moment (stability check). Lateral torsional buckling could also be significant for long columns. This is checked with a reduction factor (similar to the reduction factor for axial compression). The reduction factor does not have to be applied for box sections because these cross-sections are not susceptible for lateral torsional buckling. Lateral torsional buckling is mainly an issue for slender beams. Columns however are relatively short when compared to beams and they are much stiffer, it is therefore expected that reduction factors for lateral torsional buckling will be close to 1,0.

The calculations of the occurring bending moments and the relevant reduction factors are shown in Appendix III.

Figure 5-6: Bending moment in a column

With the use of NEN-EN 1993-1-1 section 6.3.3 the unity check for combined compression and bending can be calculated. When the occurring bending moment is known the remaining axial capacity can also be calculated by using a unity check of 1,0. Multiple factors used in the calculation of the unity check are however dependent on the design compressive force and the buckling resistance of the sections. An iterative method is therefore used to calculate the remaining buckling resistance. The calculation of this remaining resistance is shown in Appendix III. It is shown that for box sections and stocky H-sections (stocky columns have a small relative slenderness) the reduction in resistance is small. For “short” columns the maximum allowable compressive force is shown in Table 5-9.

Section (current)	Nb,Rd	Nmax	Relative	Section (new)	Nb,Rd	Nmax	Relative
	[kN]	[kN]	[%]		[kN]	[kN]	[%]
UC356x406x634 + 2x100PL	48442	46870	96,8	HD400x677 + 2x40PL	49186	48763	99,1
UC356x406x634 + 2x75PL	44073	42728	96,9	HD400x677 + 2x30PL	43890	43889	100,0
UC356x406x634 + 2x60PL	40160	40160	100,0	HD400x677 + 2x20PL	40387	40386	100,0
UC356x406x634 + 2x50PL	37097	37097	100,0	HD400x818	39889	39172	98,2
UC356x406x634 + 2x40PL	34015	34015	100,0	HD400x744	36071	35758	99,1
UC356x406x634 + 2x30PL	30900	30899	100,0	HD400x634	31027	30609	98,7
UC356x406x634 + 2x20PL	29435	29434	100,0	HD400x592	30998	29912	96,5
UC356x406x634	20916	20652	98,7	HD400x463	22295	22022	98,8
UC356x406x551	17977	16793	93,4	HD400x382	18225	16998	93,3
UC356x406x467	12778	11833	92,6	HD400x347	13536	12582	93,0
UC356x406x393 + 2x50PL	26435	26059	98,6	HD400x551	26736	26321	98,4
UC356x406x393 + 2x40PL	23782	23219	97,6	HD400x509	24605	23642	96,1
UC356x406x393 + 2x30PL	22738	22712	99,9	HD400x463	24605	23970	97,4
UC356x406x393	10619	10208	96,1	HD400x287	11056	10640	96,2
UC356x406x340	9100	8590	94,4	HD400x262	10036	9499	94,6
UC356x406x287	7861	6428	81,8	HD400x216	8171	6584	80,6
UC356x406x235	6368	5676	89,1	HD400x187	6974	5980	85,7
UC356x368x202	5257	5164	98,2	HD360x162	5773	5642	97,7
UC305x305x240	5220	4553	87,2	HD360x147	5222	4156	79,6
UC305x305x198	4152	3580	86,2	HD360x134	4628	3714	80,3
UC305x305x158	3308	2980	90,1	HD360x134	4721	4198	88,9
UC254x254x167	2862	2341	81,8	HD320x127	3264	2670	81,8
UC254x254x132	2205	1757	79,7	HD320x97,6	2478	1826	73,7
UC203x203x86	1014	584	57,6	HD260x68,2	1362	695	51,0

Table 5-9: Maximum allowable compressive force "short" columns current (left) and new design (right)

For certain box sections the remaining resistance is 100% of the design buckling resistance. The resistance has not decreased in these cases because the resistance in the weak direction is governing the design. For box sections the resistance in the weak direction does not have to be reduced when a bending moment is present in the strong direction only. Only the resistance in the strong direction has to be reduced for box sections. In these cases the remaining resistance in the strong direction is apparently still larger than the design buckling resistance in the weak direction. For the heaviest box sections the stiffness in the weak direction is almost as large or even larger than the stiffness in the strong direction. In these cases the resistance in the strong direction will be governing and therefore the remaining resistance is smaller than the design buckling resistance. Still this decrease is at most only 3%. For box sections and stocky H-sections the difference in design strength between sections used in the current and new design is still very small. The percentages are in most cases about the same which implies that the use of higher strength steel in heavily loaded (stocky) columns is not less advantageous when bending moments are applied.

For slender columns (H-sections) the resistance is 10-20% smaller than the design buckling resistance. This is because the bending moment also has an influence on the resistance in the weak direction. For the smallest column used the influence is significant (effectively decreasing the resistance of the section by almost 50% for the smallest column in the new design). For slender columns it is also visible that the use of higher strength steel (increased slenderness) decreases the effectiveness of the used sections. When designing columns with S460 the columns will be more slender than when S355 is used. This increased slenderness results in a smaller efficiency of the section when bending moments are applied as a smaller percentage of the design buckling resistance can be used for compressive forces. However in all cases (except for the UC305x305x240 section) the design resistance for sections used in the new design is still larger than the resistance of the sections used in the current design.

For "long" columns the design resistance is shown in Table 5-10.

Section (current)	Nb,Rd	Nmax	Relative	Section (new)	Nb,Rd	Nmax	Relative
	[kN]	[kN]	[%]		[kN]	[kN]	[%]
SHS600x100	51142	50568	98,9	SHS600x80	51546	51383	99,7
UC356x406x634 + 2x100PL	37531	36664	97,7	HD400x677 + 2x55PL	38306	37517	97,9
UC356x406x634 + 2x75PL	34617	34088	98,5	HD400x677 + 2x45PL	34691	34691	100,0
UC356x406x634 + 2x60PL	30567	30566	100,0	HD400x677 + 2x40PL	33679	33678	100,0
UC356x406x634 + 2x50PL	27563	27562	100,0	HD400x677 + 2x30PL	29151	29150	100,0
UC356x406x634 + 2x40PL	24529	24529	100,0	HD400x677 + 2x20PL	24558	24558	100,0
UC356x406x634	12639	12424	98,3	HD400x509	13119	12909	98,4
UC356x406x551	10768	10268	95,4	HD400x463	11712	11225	95,8
UC356x406x393 + 2x50PL	19508	19146	98,1	HD400x382 +2x40PL	20829	20506	98,4
UC356x406x393 + 2x40PL	18234	17937	98,4	HD400x382 +2x25PL	18390	18389	100,0
UC356x406x340	6290	6085	96,7	HD400x287	6865	6661	97,0

Table 5-10: Maximum allowable compressive force "long" columns current (left) and new design (right)

Even for the long columns the resistance does not have to be reduced much. This is because large sections are used which reduces the slenderness of the columns. The bending moment is also relatively small compared to the bending moment resistance of the sections. Because the bending moments can in this case only occur due to the eccentricity of the connected beams the bending moment will increase (slightly) when larger sections are used because the eccentricity will increase. The bending moment resistance is however much larger than this design bending moment, therefore the bending moment will not have a significant effect on the resistance of large sections.

When combined compression and bending occurs in a section the design buckling resistance will decrease because of the additional bending moment. For the used sections in the new design this usually does not lead to smaller resistances than resistances of the replaced sections in the current design. The is possibly only smaller when slender sections are used. For heavy sections the bending moments will have a small influence on the design resistance regardless of the steel grade used. The use of higher strength steel will therefore still result in savings similar to the ones shown in Table 5-8. The cost reduction is therefore not altered because only 1 section has to be upsized (HD360x147). This section is relatively light and occurs only once in the entire structure. The effect of upsizing this section will therefore be negligible.

5.2.2 Connections

All columns have a length of 2 stories. This length depends on the height of the individual stories. In this case 13 different sections can be used for a single column (12 to support the floors and 1 for the roof structure). Because many different sections are used the amount of different splices is very large. A total of 372 column splices are required in the building. The use of the different sections results in 123 unique splices. Many of the splices only occur once in the entire building. Because the amount of different splices is large only a few of them are examined. There are 4 different types of splices in the building:

- Box section – Box section
- Box section – H-section
- H-section – H-section (t_f > 50 mm)
- H-section – H-section

Because the perimeter of the columns should be kept as small as possible (more free space) connection plates outside the section are not used. The box sections can therefore not be bolted and need to be welded on site (Figure 5-8).

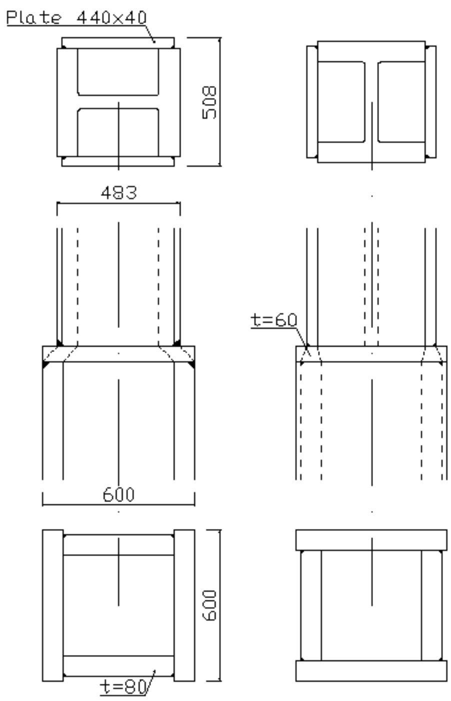


Figure 5-7: Box section – Box section

Box section – Box section

In some cases full contact between both sections is not possible due to different sizes of the sections. In these cases additional plates are welded to the bottom column (shop weld). These plates allow full contact bearing. The columns should also be connected with the use of welds. Beveled welds are used to connect the columns. The bottom column can be welded to a plate (if necessary) in the factory. The top column has to be welded on site. The size of the welds depend on the maximum design stress that can occur. The design stress is calculated with the use of the minimum requirements according to the Eurocode. An elastic distribution of the stress is used. With the use of thickness ratios the required weld size can be calculated. In all cases the web cannot be welded because it is inaccessible. The stress distribution is therefore based on the flanges and side plates only (the web is excluded from the calculation). The calculations of the required weld sizes are shown in Appendix IV.

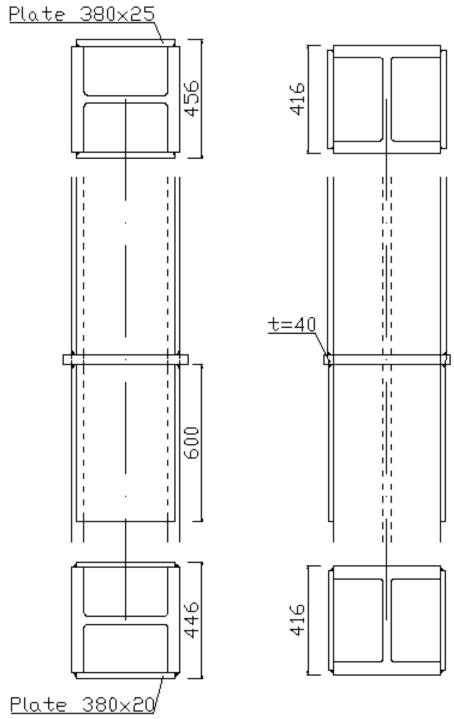


Figure 5-8: Box section – H-section

Box section – H-section

When a box section is connected to an H-section a discontinuity occurs. The side plates are no longer present. This abrupt change is not acceptable and the force from the side plates need to be transferred to the flanges of the H-section. This is accomplished by welding plates to the sides of the H-section at the location of the splice (Figure 5-9). The force in the side plates of the box section are transferred through bearing to the side plates of the H-section. The side plates of the H-sections are welded with fillet welds to the flanges. The entire force must be transferred to the flanges by these welds. The design of the welds and the plate is therefore not based on the minimum requirements but on the actual stresses in the section. The (beveled) welds connecting the sections are designed in the same way as the welds for the box section – box section connections.

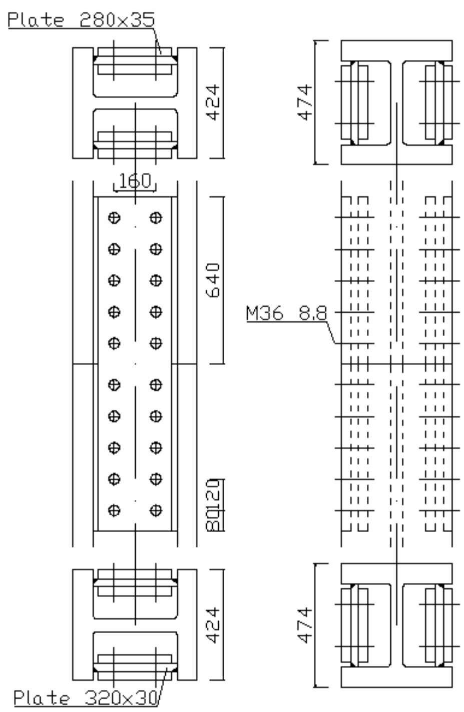


Figure 5-9: H-section – H-section (t_f > 50)

H-section – H-section (t_f > 50 mm)

For connections with H-sections 2 different types are used dependent on the flange thickness of the thickest member. If the flange thickness is larger than 50 mm they are not connected with the use of flange plates anymore. Instead plates are welded between the flanges parallel to the web (Figure 5-10). These welded plates are connected with the use of splice plates. The splice plates and the welded plates are designed with the use of the minimum required forces. Because the length of the welded plates is large the plates are assumed to be clamped to the flanges for forces in plane of the welded plate (design axial force). The axial force is divided over the 2 plates. The bending moment in the weak direction can be translated to forces with the use of a lever. The lever is in this case the center to center distance of the welded plates. For the bending moment in the strong direction a different approach is used. The welded plates are in this case always assumed to be loaded in tension. The compressive force is provided by one of the flanges (depending on the direction of the bending moment). The flanges transfer the compressive force through bearing (Figure 5-10).

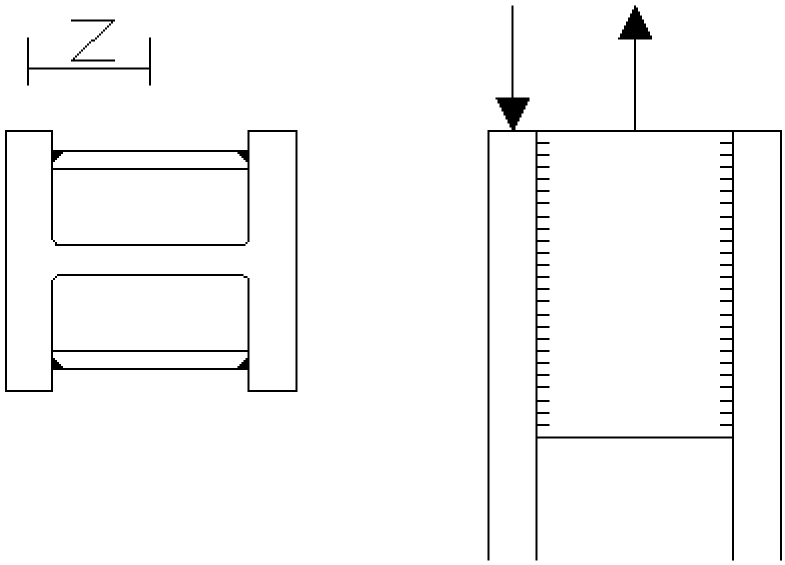


Figure 5-10: Bending moment (M_y) transfer

The force in the welded plates due to bending about the strong axis is: $F = M_y/z$. This force is divided over the 2 welded plates. The welded plate is connected with 2 bolt rows. Due to the clamped ends of the plate the internal force distribution is statically undetermined. The distribution of bending moments can however be determined with the use of certain ratios. Because 2 bolts are used next to each other with edge distance e2 and spacing p2 the plate can be modeled as shown in Figure 5-11.

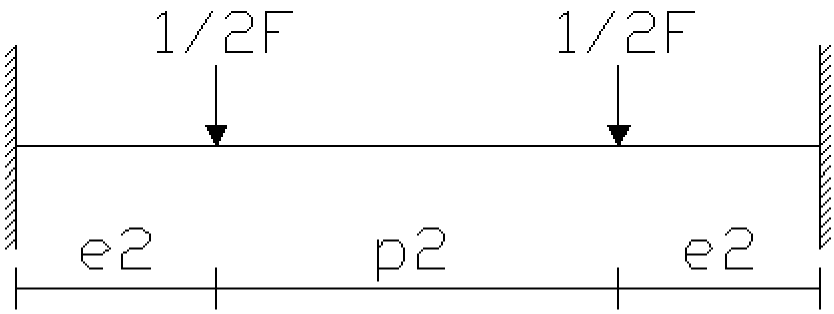


Figure 5-11: Forces and restraints welded plates

This is a symmetrical system so it can also be described by the model shown in Figure 5-12. The assumed bending moments are also shown. M_w is the bending moment at the location of the welds and M_b is the bending moment at the location of the bolts.

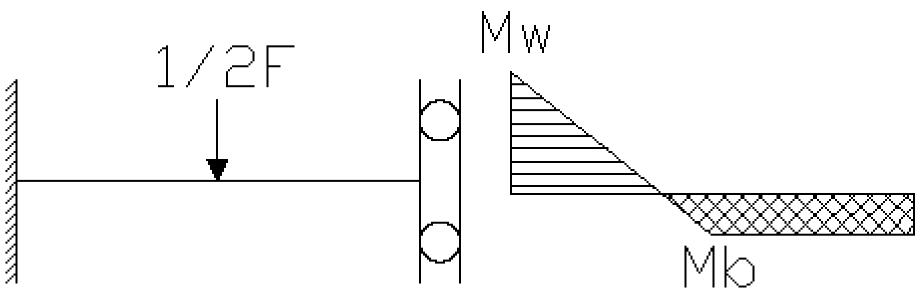


Figure 5-12: Simplified model and bending moment distribution welded plates

The restraints used in the simplified model imply that the rotation at the edge and at midspan should be 0. With the use of these boundary conditions the bending moment distribution can be calculated. It is first assumed that the bending stiffness of the plate is constant for the entire span. The bolt holes are therefore neglected for the calculation of the bending moment distribution. Rotations can be calculated with the use of the reduced bending moment diagram. This is the bending moment diagram divided by the bending stiffness. Because the bending stiffness is assumed to be constant this diagram has the same shape as the bending moment diagram. By definition the area of the reduced bending moment diagram over a certain length is equal to the total rotation over this length. Because the rotation at both ends should be 0 the total area of the reduced bending moment diagram should be 0 as well. This means that the area of the bending moment diagram marked with the horizontal lines (Figure 5-12) should be equal to the area of the bending moment diagram marked with the diagonal lines.

The total bending moment M = F/2xe2 and $M_w + M_b = M$. F is in this case the force transferred by a single welded plate. The distance between the weld and the point where M = 0 is equal to: $M_w$xe2/M. The area of the part marked with horizontal lines can be described with:

$A_w = M_w^2 xe2/(2xM)$

$A_b = (M-M_w)^2 xe2/(2xM) + (M-M_w)xp2/2$ $\quad\quad\quad$ $(M_b = M-M_w)$

$A_w = A_b \rightarrow M_w^2 xe2/(2xM) = M_w^2 xe2/(2xM) + M^2 xe2/(2xM) – 2xM_w xMxe2/(2xM) – M_w xp2/2 + Mxp2/2$

This can be rewritten to: $Mxe2/2 – M_w xe2 – M_w xp2/2 + Mxp2/2 = 0$

$M_w x(e2+p2/2) = Mx(e2/2+p2/2) \rightarrow M_w = Mx((e2/2+p2/2)/(e2+p2/2))$

Now the shear force and bending moments at the location of the welds and the bolts are known and the section checks can be made (the net section is used for the check at the bolts).

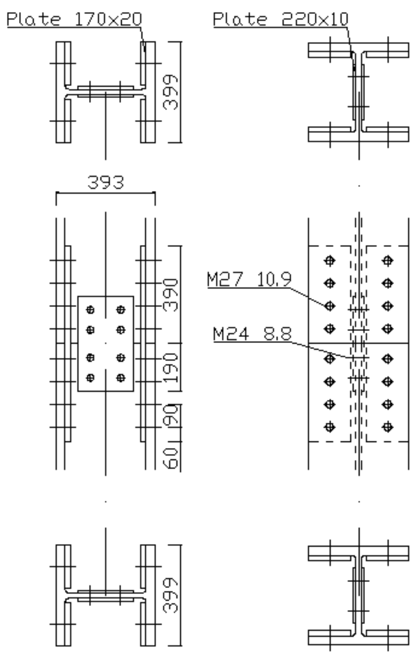


Figure 5-13: H-section – H-section

H-section – H-section

For H-sections with a flange thickness of 50 mm or less flange and web plates are used. The webs are connected with the use of two splice plates. The flanges are connected with the use of 2 splice plates on the inside of the section only. The bolts connecting the flanges are therefore loaded in 1 shear plane only. It can therefore be guaranteed that the shear force is transferred through the shaft of the bolts which results in an increased resistance. 10.9 bolts are in this case also useful because the factor α_v is 0,6 for all classes (instead of 0,5 for class 10.9). The shear resistance can be calculated with the use of:

$$F_{v,Rd} = \frac{\alpha_v * f_{ub} * A}{\gamma_{M2}}$$

The area A is in this case the area of the shaft: $A = \pi x d^2/4$. For M27 bolts class 8.8 loaded in double shear through the threaded section it was shown that the capacity per shear plane $F_{v,Rd}$ =

176,3 kN → 352,6 kN per bolt. For M27 class 10.9 bolts loaded in single shear through the shaft the capacity per shear plane $F_{v,Rd}$ = 0,6x1000x πx27^2/(4x1,25x1000) = 274,8 kN. The use of class 10.9 results in a large capacity per bolt even though the bolts are loaded in 1 shear plane only.

The flanges are assumed to transfer 70% of the axial force and the bending moments in both directions. The shear force in the weak direction of the section is also transferred through the flanges. The web is assumed to transfer 30% of the axial force and the shear force in the strong direction.

The calculations of the resistances of the different connections are shown in Appendix IV.

8 different splices are examined. The sections used for the examined splices in the current design are shown in Table 5-11.

Splice	Top	Bottom
1	UC356x406x634 + 2x100PL	SHS600x100
2	UC356x406x634 + 2x60PL	SHS600x100
3	UC356x406x634 + 2x60PL	UC356x406x634 + 2x60PL
4	UC356x406x634 + 2x30PL	UC356x406x634 + 2x30PL
5	UC356x406x551	UC356x406x634
6	UC356x406x393 + 2x40PL	UC356x406x551
7	UC356x406x393	UC356x406x393
8	UC356x368x202	UC356x406x235

Table 5-11: Splice numbers and the connected sections (current design)

The forces occurring in these splices are shown in Table 5-12.

Splice	F,Ed [kN]	My,Ed [kNm]	Mz,Ed [kNm]
1	27350	69	165
2	44682	26	0
3	41412	9	70
4	29693	85	398
5	20493	65	13
6	17499	195	13
7	14367	60	12
8	3130	442	127

Table 5-12: Forces in the examined splices

Splices 1 and 2 only occur once in the entire building. They are examined because the dimensions of the top and bottom sections differ a lot. If this is the case full contact bearing cannot occur when simply placing the top section on top of the bottom section. In order to achieve full contact bearing an additional end plate is required. For connections which only contain UC sections (current design) or HD sections (new design) additional plates are not required because full contact bearing can always be achieved if the top column is not larger than the bottom column. The required welds and plates for the splices (Box section – Box section) in the current design are shown in Table 5-13.

Splice	Weld flange	Weld plate	Plate
	[mm]	[mm]	l x w x t
1	28	35	600x600x70
2	29	23	600x600x70
3	29	23	N/A
4	29	13	N/A

Table 5-13: Required welds and plates Box section – Box section splices (current design)

The top column is always welded on site to the bottom column. If an additional plate is required to provide full contact bearing it is welded in the factory to the bottom column. The same welds are then applied to connect the plate to the bottom column.

Splice 5 is a connection between 2 UC sections so an additional plate is not required. The flange thickness of both sections is larger than 50 mm so welded plates are used to connect the sections. Per splice 2 welded plates and 4 splice plates are required. The required plates and welds are shown in Table 5-14.

Splice	Welded plate	Weld	Splice plate
	l x w x t	[mm]	l x w x t
5	390x320x35	18	780x280x25

Table 5-14: Required welds and plates H-section- H-section splices (t_f > 50 mm) (current design)

Splice 6 is a connection between a Box section and an H-section. An additional plate with a thickness of 40 mm is used. A plate is welded to the H-section to allow the transfer of the force present in the side plates of the box section to the H-section.

Splice	Welded plate	Weld	Weld flange	Weld plate
	l x w x t	[mm]	[mm]	[mm]
6	600x415x30	15	16	13

Table 5-15: Required welds and plates Box section – H-section splices (current design)

Splices 7 and 8 are connections between 2 UC sections. The thickness is not larger than 50 mm so flange and web plates are used to connect the columns. Welds are therefore not required. 4 splice plates are used to connect the flanges and 2 splice are used to connect the webs. The required plates are shown in .

Splice	Flange plate	Web plate
	l x w x t	l x w x t
7	780x170x20	380x220x10
8	780x160x25	380x220x10

Table 5-16: Required plates H-section – H-section splices (current design)

The splices in the new design are similar the ones used in the current design. Due to the use of different sections the type of splice could differ. The sections used for the examined splices in the new design are shown in Table 5-17.

Splice	Top	Bottom
1	HD400x677 + 2x20PL	SHS600x80
2	HD400x677 + 2x20PL	SHS600x80
3	HD400x677 + 2x20PL	HD400x677 + 2x20PL
4	HD400x634	HD400x634
5	HD400x382	HD400x463
6	HD400x382 + 2x25PL	HD400x382
7	HD400x287	HD400x287
8	HD360x162	HD400x187

Table 5-17: Splice numbers and the connected sections (new design)

It is visible that one less Box section – Box section splice is used which reduces the amount of site welds. The same force are used for the calculation of the required resistances. These calculations lead to the following welds and plates for splice 1 to 3 (Table 5-18).

Splice	Weld flange	Weld plate	Plate
	[mm]	[mm]	l x w x t
1	36	19	600x600x60
2	37	11	600x600x60
3	36	11	N/A

Table 5-18: Required welds and plates Box section – Box section splices (new design)

The welds connecting the flanges are larger than the ones used in the current design. This is because the side plates of the box sections are smaller than the ones used in the current design. The flanges need to transfer a larger force to the bottom column hence the increase of the required weld size. Because of the reduced thickness of the side plates the welds connecting these plates are smaller than the ones used in the current design.

The weld sizes are not based on the allowable shear stress but they are based on the allowable tensile stress. A different formula is applied according to NEN-EN 1993-1-8 4.5.3.2: $\sigma \leq 0{,}9xf_u/1{,}25$. The application of a correlation factor for welding is no longer required and therefore the welds for S460 can be a bit smaller than the welds for S355.

Splices 4 and 5 are connections between 2 HD sections with flanges that are thicker than 50 mm. The required plates and welds are shown in Table 5-19.

Splice	Welded plate	Weld	Splice plate
	l x w x t	[mm]	l x w x t
4	640x320x30	19	1280x280x35
5	390x320x30	18	780x280x25

Table 5-19: Required welds and plates H-section- H-section splices (t_f > 50 mm) (new design)

It is visible that the required welds and plates for splice 5 are similar to the ones used in the current design. This implies that the use of S460 is not beneficial when these kind of connections are made.

The required welds and plates for splice 6 are shown in Table 5-20.

Splice	Welded plate	Weld	Weld flange	Weld plate
	l x w x t	[mm]	[mm]	[mm]
6	600x380x20	11	18	10

Table 5-20: Required welds and plates Box section – H-section splices (new design)

In this case the use of S460 leads to reduced plate and weld sizes. This is because the side plates are smaller and therefore a smaller force is transferred by these side plates resulting in smaller plate sizes for the connection.

Splices 7 and 8 are connections between 2 HD sections. The connection is made with the use of flange and web plates only. The required plates are shown in Table 5-21.

Splice	Flange plate	Web plate
	l x w x t	l x w x t
7	780x170x20	380x220x10
8	780x160x25	380x220x10

Table 5-21: Required plates H-section – H-section splices (new design)

The design with the use of S460 results in the use of exactly the same plates used in the current design. This is because the same forces are transferred and the tensile strength of the material is used to calculate the resistance. The difference in tensile strength for S355 and S460 is smaller than the difference in yield strength and because the plates are already thin a further reduction of thickness could lead to a reduction of 1 to 2 mm in thickness which is not practical.

5.3 Cost saving

The cost saving is again split up in material, fabrication, transportation and assembly costs. The material costs cannot directly be calculated with Table 3-4 because built up sections are also used. The costs for the columns (without connections) therefore already includes some of the fabrication costs. The fabrication costs can be calculated with the same method used for the trusses. The same is valid for the transportation costs. The assembly costs differ however. In this case some of the connections are welded on site which is of influence on the assembly costs. Certain types of connections can therefore be cheaper and recommendable for column splices. This depends on the combination of fabrication and assembly costs. As site welding is expensive these kind of connections should be kept to a minimum. In this case they are used because box sections cannot be bolted to other box sections without the use of plates welded to the exterior of the columns. Since the use of these plates is in this case not desirable the box sections have to be welded on site.

5.3.1 Material

The material cost saving in this case also includes a bit of fabrication costs. Generally a price per ton is used for steel sections. For box sections this can still be used but the required welds and plates should also be taken into account. This is done with the use of a fixed speed of 10 m/hr for the welds (valid for one layer). For the plates the price per ton is increased with 100 €/t to take account of the fabrication costs required to cut the plates. The costs per ton for all the sections are shown in Table 5-22.

Section	Costs
	€/t
SHS600x100	1140
UC356x406x634 + 2x100PL	1816
UC356x406x634 + 2x75PL	1769
UC356x406x634 + 2x60PL	1733
UC356x406x634 + 2x50PL	1706
UC356x406x634 + 2x40PL	1674
UC356x406x634 + 2x30PL	1637
UC356x406x634 + 2x20PL	1593
UC356x406x634	1130
UC356x406x551	1130
UC356x406x467	1085
UC356x406x393 + 2x50PL	1629
UC356x406x393 + 2x40PL	1597
UC356x406x393 + 2x30PL	1557
UC356x406x393	975
UC356x406x340	975
UC356x406x287	975
UC356x406x235	975
UC356x368x202	975
UC305x305x240	940
UC305x305x198	940
UC305x305x158	940
UC254x254x167	935
UC254x254x132	935
UC203x203x86	925

Section	Costs
	€/t
SHS600x80	1275
HD400x818	1180
HD400x744	1180
HD400x677 + 2x55Pl	1783
HD400x677 + 2x45Pl	1748
HD400x677 + 2x40Pl	1729
HD400x677 + 2x30Pl	1686
HD400x677 + 2x20Pl	1636
HD400x634	1180
HD400x592	1180
HD400x551	1180
HD400x509	1180
HD400x463	1134
HD400x382 + 2x40Pl	1696
HD400x382 + 2x25Pl	1623
HD400x382	1025
HD400x347	1025
HD400x287	1025
HD400x262	1025
HD400x216	1025
HD400x187	1025
HD360x162	1025
HD360x147	1030
HD360x134	1030
HD320x127	990
HD320x97,6	995
HD260x68,2	980

Table 5-22: Costs per ton for the sections in the current (left) and new design (right)

The costs for the fabrication of the plates and welds are included in these prices. For the fabrication of the plates the costs were increased with 100 €/t. The costs for the welds depend in the size of the welds. These costs can be described as costs per meter. For the plates the costs can also be described as costs per meter. These costs depend on the section used and the thickness of the plates. When both plate and welding costs are described as costs per meter they can be transformed into costs per ton by simply dividing the costs per meter by the unit weight of the section in tons (t/m). By adding up all the components the final costs per ton are known (Table 5-22).

The cost saving for the individual sections is shown in Table 5-23 for “short” columns and in Table 5-24 for “long” columns.

Current section	Costs [€/m]	New section	Costs [€/m]	Cost saving [%]
UC356x406x634 + 2x100PL	2391	HD400x677 + 2x40PL	1650	31,0
UC356x406x634 + 2x75PL	2027	HD400x677 + 2x30PL	1492	26,4
UC356x406x634 + 2x60PL	1809	HD400x677 + 2x20PL	1335	26,2
UC356x406x634 + 2x50PL	1664	HD400x818	965	42,0
UC356x406x634 + 2x40PL	1518	HD400x744	878	42,2
UC356x406x634 + 2x30PL	1373	HD400x634	748	45,5
UC356x406x634 + 2x20PL	1227	HD400x592	699	43,1
UC356x406x634	716	HD400x463	525	26,7
UC356x406x551	623	HD400x382	392	37,1
UC356x406x467	507	HD400x347	356	29,8
UC356x406x393 + 2x50PL	1126	HD400x551	650	42,3
UC356x406x393 + 2x40PL	1008	HD400x509	601	40,4
UC356x406x393 + 2x30PL	891	HD400x463	525	41,1
UC356x406x393	383	HD400x287	294	23,2
UC356x406x340	332	HD400x262	269	19,0
UC356x406x287	280	HD400x216	221	20,9
UC356x406x235	229	HD400x187	192	16,3
UC356x368x202	197	HD360x162	166	15,7
UC305x305x240	226	HD360x147	151	32,9
UC305x305x198	186	HD360x134	138	25,8
UC305x305x158	149	HD360x134	138	7,1
UC254x254x167	156	HD320x127	126	19,5
UC254x254x132	123	HD320x97,6	97	21,3
UC203x203x86	80	HD260x68,2	67	16,0

Table 5-23: Cost saving "short" columns

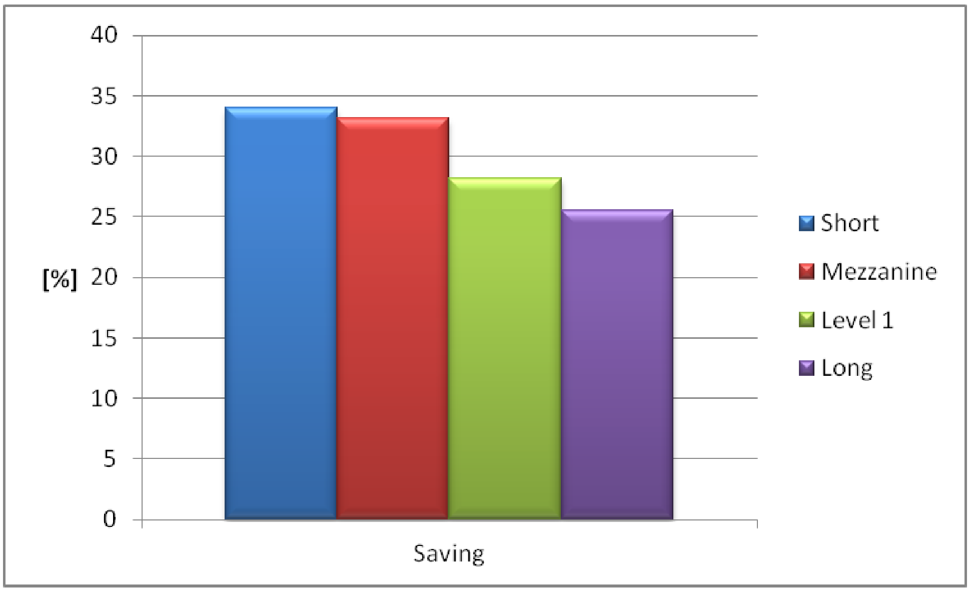
Current section	Costs [€/m]	New section	Costs [€/m]	Cost saving [%]
SHS600x100	1790	SHS600x80	1602	10,5
UC356x406x634 + 2x100PL	2391	HD400x677 + 2x55PL	1886	21,1
UC356x406x634 + 2x75PL	2027	HD400x677 + 2x45PL	1728	14,7
UC356x406x634 + 2x60PL	1809	HD400x677 + 2x40PL	1650	8,8
UC356x406x634 + 2x50PL	1664	HD400x677 + 2x30PL	1492	10,3
UC356x406x634 + 2x40PL	1518	HD400x677 + 2x20PL	1335	12,1
UC356x406x634	716	HD400x509	601	16,2
UC356x406x551	623	HD400x463	525	15,7
UC356x406x393 + 2x50PL	1126	HD400x382 +2x40PL	1053	6,5
UC356x406x393 + 2x40PL	1008	HD400x382 +2x25PL	863	14,4
UC356x406x340	332	HD400x287	294	11,3

Table 5-24: Cost saving "long" columns

The cost saving differs from 6,5 % to 45,5 % for different sections. This difference is very large and is caused by the different costs between box sections and H-sections. The UC356x406x643 + 2x30PL section can be replaced by an HD400x634 section in S460 which is essentially the same section without the additional plates. The section therefore weighs and costs less per meter which results in this significant cost reduction. The box section in the current design has a buckling resistance of 30900 kN at a buckling length of 5380 mm. If an H-section were to be used this section should have been an HD400x1202 section. These sections cost 1130 €/t which results in 1358 €/m. This is slightly less expensive than the box section used. The box sections however weigh less which would result in reduced transportation costs. If the buckling lengths are small (about 4 m) the use of large HD sections will become the cheaper solution because the stiffness in the weak direction would matter less.

The other differences are caused by significant differences in weight saving and by the price per ton of certain section. In some cases the section in S460 will result in a higher price per ton and sometimes the price per ton will be lower. When the latter is the case and the weight saving is significant the cost saving will be even more significant because of the reduced costs per ton.

With the use of the section overview shown In Appendix III the total costs per column can be calculated. These costs are shown in Table 5-25. It is visible that the saving for the columns vary from about 20% up to 35%. This difference is mainly caused by the use of “long” columns in some of the columns. For “long” columns the cost saving is relatively small when compared to the saving for “short” columns. When a column consist of only “short” columns the total saving is therefore larger than when “long” columns are also present. Combined with the difference in bearing resistances at the same level of these columns (probably because the columns are more heavily loaded), which results in different savings of each individual column at the same height, the total saving can vary this much. This also shows that the use of stronger steel grades is more favorable when used in parts of the construction where the buckling lengths are relatively small. This can also be seen in Graph 5-1. In this graph different terms are used. “Short” displays the cost saving for columns which are supported at all floors, “Mezzanine” displays the saving for columns not supported at the mezzanine. The same applies for “Level 1” and “long” displays the cost saving for columns that are not supported at level 1 and the mezzanine.



Graph 5-1: Cost saving for different column types

Column	Current Costs [€]	New Costs [€]	Saving [%]
1	158277	115896	26,8
2	129887	94726	27,1
4	114512	77161	32,6
7	80782	61202	24,2
9	76934	54034	29,8
11	133330	97798	26,6
13	102101	67059	34,3
15	120427	76048	36,9
18	117500	73843	37,2
20	123879	82213	33,6
22	80782	55162	31,7
23	75997	49950	34,3
25	117546	76989	34,5
27	79772	53466	33,0
28	84593	57417	32,1
37	90638	63348	30,1
38	94577	71391	24,5
40	88970	60138	32,4
41	58554	41764	28,7
42	75843	51531	32,1
43	75026	50957	32,1
44	72233	49026	32,1
45	65908	43083	34,6
46	61095	48945	19,9
47	54678	43034	21,3
48	54678	43034	21,3
49	48776	38745	20,6
50	64703	48294	25,4
51	85403	65124	23,7
Total	2587402 €	1811376 €	30,0 %

Table 5-25: Costs and cost saving per column

The total cost saving based on axially compressed members only is in this case 30% which leads to an amount of over 700.000 € for the entire building. When larger loads are present (taller buildings) the cost saving could even increase because the difference in yield strength between S355 and HISTAR460 becomes larger when larger sections are used.

5.3.2 Fabrication and assembly

The fabrication and assembly costs are reviewed because the final costs of a certain connection is the sum of these 2 costs. The different connection types used all have different fabrication and assembly costs. Connections welded on site will have larger assembly costs than bolted connections. The fabrication costs could however be smaller since additional plates and drilling of holes is not required. The combination of both costs determines the final costs of the connection. Therefore all connections are compared based on the summation of the fabrication and assembly costs.

The fabrication costs are determined for sawing, drilling, welding, flame cutting, weld preparation and flattening of the column ends. The material costs for the required plates is also included. Because only a few splice details are examined the costs for shot blasting and internal transport are assumed to be constant. They are assumed to be the same regardless of the steel grade and connection type used.

All sections are sawn and the column ends are flattened to allow full contact bearing. Holes (if required) are drilled in the sections and the required plates. The plates are then cut and welded (if required) to the sections. Box section – Box section connections require weld preparation of the top column end and if a plate is required between the column ends the bottom column needs weld preparation as well. For Box section – H-section the same operations are required. The welded plates in the heavy H-section – H-section connections require weld preparation as well.

The assembly costs are determined for the connections only. Any possible savings in crane use for example are not reviewed. The costs of the connections depend on the location of the connections. There is a difference in costs for a connection at the 2nd floor or the same connection at the 21st floor. This is because it takes more time to lift the required components in the right position. Because connections can be used multiple times at different levels an average lift time is assumed. It is assumed that all connections require this average lift time which leads to the use of certain basic costs that are always required regardless of the type of connection. The assembly costs are therefore a summation of these basic costs and the actual time required to connect the columns. For bolted connections the required time depends on the weight of the splice plates (handling) and the amount of bolts required in the connection. For the examined connections the amount of bolts required are about the same and the weight of the splice plates does not differ much. The costs for bolting the connections on site are therefore assumed to be constant independent of the connection type. Welded connections require a certain time dependent on the amount of layers required. The required time is calculated with a reduced welding speed as opposed to the welding speed use for the fabrication process. The reduced welding speed is chosen to take account of the disadvantage of welding on site: A welding tent could be required or the weather interferes with the possibility to lay down quality welds.

With the use of the required fabrication and assembly time the total costs can be estimated. For the connections used in the current design the cost estimations are shown in Table 5-26. It is visible that the relative costs for fabrication differ per connection type. Splices 1 to 4 are Box section – Box section connections and they are welded on site. The assembly costs are large in this case and therefore the relative fabrication costs are small for these kind of splices. For splices with bolted connections (splice 5, 7 and 8) the fabrication costs are much more than the assembly costs because bolted connections are easier to assemble and therefore cost much less. The fabrication costs of splice 5 (H-section – H-section with $t_f > 50$ mm) is high compared to the fabrication costs of the other splices. The welded plates require 2 beveled welds per plate. 4 welded plates are required per splice which results in large fabrication costs because the required welding time will be large.

Splice	Fabrication	Assembly	Total
	[€]	[€]	[€]
1	2054	3675	5729
2	1863	2805	4668
3	1121	2805	3926
4	1120	2010	3130
5	2231	450	2681
6	1980	990	2970
7	1340	450	1790
8	1305	450	1755

Table 5-26: Costs of the different connection details (current design)

According to the model used to calculate the fabrication and assembly costs the difference between bolted and welded connections is not really noticeable for the fabrication costs. The assembly costs differ a lot depending on the type of the connection. It is visible that welded connections will cost more than bolted connections. This is mainly because of the increased costs required for welding on site. The increased fabrication costs for the heavy H-section – H-section splices is compensated by the reduced assembly costs. This makes this type of connection more suitable for heavy H-sections than welded connections on site.

The connections used in the new design are similar to the ones used in the current design. The required plates for bolted connections are in some cases even the same. Welded connections require about the same weld sizes as the connections used in the current design (larger flange welds but smaller side plate welds). The use of thinner side plates still could result in reduced welding costs. The estimated fabrication and assembly costs are shown in Table 5-27.

Splice	Fabrication	Assembly	Total
	[€]	[€]	[€]
1	1821	2670	4491
2	1795	2595	4390
3	1126	2460	3586
4	3030	450	3480
5	2197	450	2647
6	1936	990	2926
7	1323	450	1773
8	1305	450	1755

Table 5-27: Costs of the different connection details (new design)

In this case the fabrication costs differs a lot depending on the connection type used. The fabrication costs for the heavy H-section – H-section connections (splice 4 and 5) are again the most. The costs for splice 4 are significantly higher. This is because a long connection is required to transfer the forces. This results in the use of long welded plates which significantly increases the time required for welding. The reduced assembly costs however still results in smaller total costs than the welded connections (splice 1 to 3).

The cost saving per connection detail is shown in .

Splice	Current [€]	New [€]	Saving [%]
1	5729	4491	21,6
2	4668	4390	6,0
3	3926	3586	8,7
4	3130	3480	11,2
5	2681	2647	1,3
6	2970	2926	1,5
7	1790	1773	0,9
8	1755	1755	0,0

Table 5-28: Fabrication and assembly cost saving

Cost savings (if at all present) are very small for connections between 2 H-sections. This is because bolted connections are used for these kind of connections. In this case the columns are all loaded in compression and this force is transferred through bearing. The connections are therefore designed according to minimum requirements. These requirements result in about the same (tensile) forces for connections in S355 and S460. For bolted connections the same amount of bolts are therefore required. Because the tensile strength of splice plates is used in the design of most connections only a small reduction in thickness is possible when using S460. This however leads to thicknesses that are not practical (23 mm instead of 25 mm for example). S355 plates can always be used in bolted connections (also in the new design) so additional fabrication costs will not occur. The use of S460 for the section will not lead to increased fabrication costs as well because of the reduced size and thickness of these sections as opposed to the sections used in the current design. Welded connections lead to slightly larger cost savings because the total amount of welding time required can be reduced when using S460. The reduced welding time required is mainly noticeable for the site welds and could significantly reduce the required time and thus costs for assembly. For splice 4 the saving is shown in red. This means that the costs have increased. In this case the type of connections has changed. In the current design splice 4 is a connection between 2 box sections. In the new design both box sections have been replaced by 2 heavy H-sections. This change results in increased fabrication costs but a reduction in assembly costs. The increase in fabrication costs is however larger than the decrease in assembly costs resulting in increased total costs.

5.3.3 Transportation

The transportation costs are again assumed to be linearly related to the weight of the entire structure. In Table 5-8 it was shown that the total weight reduction of all columns is 22,1%. The weight of the required plates for the connections is however not included in the total weight reduction. The connections require similar plates as the ones used in the current design. In some cases the size of the plates does not even change when using S460. The weight reduction for the required plates will therefore be far less than 22,1%. The total weight of the plates is however much smaller than the total weight of all sections. The total weight saving will therefore decrease by not that much. It is assumed that the weight saving including all the required plates is about 20%. The transportation costs are therefore assumed to be 20% less also. This is based on the same method used for calculating the transportation costs for the trusses in the Delft project. In this method the costs are based on the amount of trucks required and the distance travelled. Because the amount of trucks required is linearly dependent on the weight so are the costs.

5.3.4 Conclusion

The use of S460 for the columns results in great material savings. This is mainly noticeable for the heavily loaded columns. Sections with the steel grade S355 will not be able to resist the large compressive forces so additional plates are required to strengthen these sections. When S460 is used side plates will be required less often. This depends on the magnitude of the forces and the unsupported length of the columns. For slender columns the possible saving is much smaller than for stocky columns. This is because the plastic axial capacity of slender columns has to be reduced (much) more to take account of buckling than the plastic axial capacity of stocky columns. In office buildings unsupported column lengths are usually 4-5 m. At the bottom levels of these buildings large forces have to be transferred. The use of H-sections will be very effective in these cases because large sections would be required. These large sections are very stiff and the buckling resistance will therefore be close to the axial plastic capacity which means that these sections are very effective. The use of box sections is therefore not required and the savings can therefore be substantial. This is shown for the box sections used in the new design. Only 3 box sections are used for "short" columns. The buckling resistances have been calculated for a buckling length of 5 m and equivalent H-sections are shown with the relevant costs (Table 5-29).

Box section	Nb,Rd [kN]	Costs [€/m]	H-section	Nb,Rd [kN]	Costs [€/m]
HD400x677 + 2x40PL	50070	1650	HD400x1202	53968	1418
HD400x677 + 2x30PL	45749	1492	HD400x1086	48270	1281
HD400x677 + 2x20PL	41334	1335	HD400x990	43635	1168

Table 5-29: Cost comparison box sections and H-sections "short" columns

All the equivalent H-sections weigh more but cost less per meter so material costs can be further reduced if H-sections are used instead of box sections for short columns. The fabrication, transportation and assembly costs will however increase with the use of these H-sections because connections with welded plates are used and the weight is increased.

In some cases the unsupported column length could be much larger than 4-5 m. This could for instance happen at the ground floor of such a building at the location of the entrance. When the unsupported column length is large the use of box sections becomes more attractive because they are very effective in these cases. The use of H-sections would lead to buckling resistances much smaller than the plastic axial capacity because of the increased slenderness at longer unsupported lengths. All the box sections used for "long" columns are examined. The buckling resistances are calculated for a buckling length of 10 m. The equivalent H-sections and the relevant costs are shown in Table 5-30.

The equivalent H-sections cost about the same as the box sections. The weight of the sections is however much larger than the weight of the box sections. This will lead to increased transportation costs. The increased weight combined with the welded plates required to connect the heavy H-sections will also result in increased fabrication and assembly costs. The use of H-sections for longer columns is therefore not advisable because it would ultimately lead to increased costs as opposed to box sections.

Box section	Nb,Rd	Costs	H-section	Nb,Rd	Costs
	[kN]	[€/m]		[kN]	[€/m]
SHS600x80	52899	1602	N/A		
HD400x677 + 2x55PL	39744	1886	N/A		
HD400x677 + 2x45PL	36119	1728	N/A		
HD400x677 + 2x40PL	35240	1650	N/A		
HD400x677 + 2x30PL	30656	1492	HD400x1299	32683	1533
HD400x677 + 2x20PL	25972	1335	HD400x1202	29695	1418
HD400x382 +2x40PL	21877	1053	HD400x818	22267	965
HD400x382 +2x25PL	18027	863	HD400x744	19822	878

Table 5-30: Cost comparison box sections and H-sections “long” columns

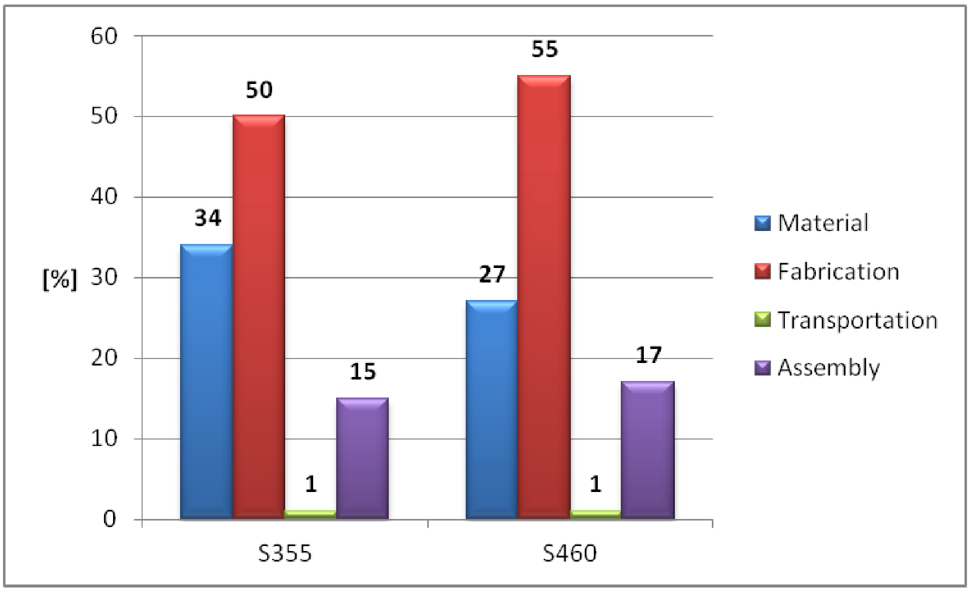
Fabrication and assembly costs will not change much. The use of H-sections instead of box sections however leads to an increase in fabrication and assembly costs. If the use of H-sections results in large material savings as opposed to box sections this increase in costs could be acceptable. It is shown that for unsupported lengths of about 10 m the use of H-sections is ineffective. If the unsupported length is smaller (about 5 m) the use of H-sections becomes very effective because the material savings can be substantial. The increased fabrication and assembly costs will easily be compensated by the reduced material costs. Office buildings will usually have column lengths of 4-5 m so the use of H-sections is recommendable. Box sections should only be used when long columns are required or when the capacity of the largest H-section available is too small. For S355 this happen more often than for S460. The largest HD-section available is the HD400x1299 section. This is a very heavy section with 140 mm thick flanges. For S355 this leads to the use of a yield strength of 295 N/mm^2 and buckling curve d. For S460 (HISTAR) the yield strength is 450 N/mm^2 and buckling curve c should be used. For a column with a length of 5 m the design buckling strength is:

- 38605 kN for S355
- 58608 kN for S460

The capacity of the column in S460 is over 50% larger than the capacity of the one in S355. Larger building heights can be achieved with the use of S460 without the need for box sections.

The most cost saving is achieved when box sections can be replaced by H-sections. It is shown that the material cost saving can be over 40%. Additional costs for fabrication, transportation and assembly are by far compensated. Longer columns or short columns with a small load (at the top of the building) can also be replaced by columns with the use of S460. This will lead to a material cost reduction of about 10-20%. The fabrication and assembly costs will in these cases stay about the same. The transportation costs will also be reduced by about 20%.

A relative cost estimation is shown for both designs (current and new). This cost estimation is based on a total of 34% material costs, 50% fabrication costs, 15% assembly costs and 1% transportation costs. The relative costs estimation is shown in Graph 5-2.



Graph 5-2: Relative cost estimation both designs

Because the fabrication and assembly costs stay about the same their relative costs will increase when S460 is used. The relative material costs will in its turn decrease. The percentages shown in Graph 5-2 are not based on real calculated values but they are estimated values used to show what the changes in relative costs could look like.

Assuming that the material costs are about 1/3rd of the total costs of a construction the use of S460 would lead to a reduction of 10% on the total costs. This is based on the material cost reduction of 30% only. Any differences in fabrication and assembly costs are considered small enough to be negligible and any transportation cost reduction will also be negligible because the transportation costs are small compared to the other costs.

5.3.5 Additional costs

Some of the costs are not described and factored in the calculation of the cost reduction. All columns are used to support 2-4 beams. These beams are connected to the columns with the use of fin plates. The costs of these connections have not been examined. It is however possible to connect the beams with the use of fin plates with the steel grade S355. The thickness of the plates will therefore not change and also the amount of bolts required stays the same. The only thing that could differ is the size of the welds. Fin plates are connected with the use of fillet welds. The strength of fillet welds is based on the design tensile strength of the connected parts. The strength of the fillet weld ($f_{w,u,d}$) is determined with the use of the following formula: $f_{w,u,d} = f_u/(\sqrt{3} x \beta_w x 1,25)$. β_w is the correlation factor and is dependent on the steel grade used. β_w is 0,9 for S355 and 1,0 for S460. The values for $f_{w,u,d}$ are shown in Table 5-31 for different thicknesses.

Grade	Thickness [mm]	
	40	150
S355	251,5	241,2
S460	249,4	244,8

Table 5-31: Design strength of fillet welds $f_{w,u,d}$

The design strengths of the fillet welds are for both steel grades about the same. The fabrication costs will therefore remain the same regardless of the steel grade used.

The use of S460 results in larger possible stresses in the columns. These stresses can be translated to strains with the use of Hooke's law: σ = Exε. The Young's modulus is the same for both steel grades: E = 210000 N/mm^2. The use of S460 will lead to larger strains. Due to the strain the columns will shorten. The building is about 120 m tall. With the use of the occurring stresses the total shortening of the entire building can be calculated. For simplicity it is assumed that all columns are loaded to 75% of the yield strength. The total shortening will then be:

- 0,75x120x1000x355/210000 = 152 mm for S355
- 0,75x120x1000x460/210000 = 197 mm for S460.

For the new design the total shortening will be about 45 mm more than the shortening for the current design. To compensate this shortening the columns can be made a bit longer (every column 2 cm taller will completely compensate the shortening). In this can division plates are used at certain levels to compensate for any differences between column heights. They can also be used to compensate for the shortening of the columns. The total shortening for the new design is 0,16% This is considered small enough to neglect any differences in costs.

The welded columns are sometimes very thick. In these cases preheating could be required. It was already shown that for both designs no preheating is required when a weld consumable in class D in combination with a heat input of 2 kj/mm is used. If preheating would be required the fabrication costs would increase but the assembly costs would increase even more because preheating on site would require more effort.

6 Concluding summary and design recommendations

The effectiveness of higher strength steel in buildings is dependent on the design of the load bearing elements. Large material savings can be expected for mainly axially loaded members of a construction. The construction of the Delft railway station consists of large trusses which carry the loads of multiple stories of the building. Such truss systems are used more often lately. The use of higher strength steel is very beneficial in these structures and leads to material cost savings of about 25-32% dependent on the type of truss used.

Large savings for the chords greatly influence the total material cost saving. The used calculation method is of influence on the design and thus calculated cost savings for the chords. In this case a computer program was used to calculate the occurring forces in each section. The calculation method used by this program also includes the stiffness of the chords for the distribution of the forces. This leads to larger bending moments in the chords than one would find when performing a manual calculation because in a manual calculation the stiffness of the chords is considered to be negligible. Partly due to the bending moments large savings were possible. If a different calculation method were to be used the bending moment would probably be smaller and would therefore not lead to a significant increase in cost reduction.

The verticals and diagonals are considered to be loaded by an axial force only. The expected weight reduction for such members is 23%. The material cost reduction is closely related to the weight reduction and is dependent on the size of the used sections. In some cases the reduced size of sections could lead to reduced costs per ton even if a higher strength steel grade is used. For such sections the cost saving is very large. For the verticals and diagonals a material cost saving of about 20-25% is found.

In this case bolted connections were used to connect al the diagonals and verticals to the chords. For members loaded in tension this could lead to the use of cover plates to strengthen the sections at the connections. These kind of plates are more often required when S460 is used and increase the required fabrication costs. All plates used for the connections can also be designed with the use of S460. This also leads to some cost reductions. Although the material has an increased hardness the reduced thickness of all the components could lead to some fabrication cost savings. This saving is about 10% based on the model used to calculate the fabrication costs. The model does however not include the costs for conservation and not all connection details are included in the calculation. The amount of stiffeners required is the same for both designs because webs of the relevant sections are all encased in concrete. If this is not the case local or global buckling of the web prevents the full utilization of the axial capacity of the web at the connections. Because designing with S460 leads to decreased thicknesses buckling will have an increased influence on the design of connections. Stiffeners would therefore be required more often and would increase the fabrication costs.

The fabrication costs are dependent on the hardness and size of the used sections and plates. One could say that the decreased thickness and increased hardness of the materials when S460 is used compensate each other when fabrication costs are considered. Fabrication costs for designs with S355 and S460 will therefore be about the same.

The assembly of the trusses for both designs will require about the same time because the amount of connections stays the same. Cost savings are therefore only possible for the required equipment. The use of S460 leads to a reduced weight. Smaller cranes can therefore be used for the assembly and installation of the trusses. The cost reduction for cranes can therefore be significant because daily costs can be very high. In this case these cranes will be required for several weeks which leads to large cost savings for the use of cranes only.

The reduced weight of the sections also leads to reduced transportation costs. The transportation costs are however small compared to the other costs. These costs are assumed to be linearly related to the weight of the structure. Because large weight savings are possible the fabrication cost reduction will usually be about 20-30%.

The high-rise office (BP4) consists of a lot of box sections. These box sections are required because the largest H-sections cannot provide the required load bearing capacity. When S460 is used box sections will be required less often. The main advantage of the use of S460 is when the unsupported column lengths are small. For office buildings this length will be about 4-5 m. The use of S460 is mainly preferable for heavier loaded columns. When box sections are required when using S355 this will not necessarily be the case for a design with the use of S460. Box sections could therefore be replaced by H-sections. In these cases the material cost savings can be over 40%. This is because the use of box sections results in increased fabrication costs for the sections and thus higher costs per ton. When replacing H-sections the material cost saving will be about 20%. This is a bit below the expected weight saving but that can be explained by the increased cost for S460 sections. When long columns are required the cost saving will be about 10%. This is because the buckling is of great influence on the design of the sections. When S460 is applied the increased slenderness results in a small possible size reduction of the sections.

Fabrication and assembly costs are slightly influenced by the use of S460. Any differences will only be noticeable for welded splices. This is because smaller welds can be used for the design with S460. The decrease in size of the welds is also due to the decreased size of the sections.

The transportation costs are again assumed to be linearly related to the weight. The transportation costs for the columns are in this case decreased by about 20%.

The use of S460 could lead to large material and transportation cost savings. Fabrication costs are hardly influenced by the use of S460. Assembly costs can be reduced but this is dependent on the weight of the construction only. Any construction time savings are not expected to occur. An overview of the expected cost reduction is shown in Table 6-1. When S460 is used the material costs will decrease relative to the total costs. The relative fabrication costs will however increase and therefore an increased amount of time required per ton should be for a cost calculation.

	Saving [%]
Material	10 - 40
Fabrication	-5 - 5
Transportation	20 - 30
Assembly	0 - 20

Table 6-1: Expected cost saving

6.1 Further research

The results obtained are only applicable for members in large trusses and columns. These members are mainly loaded in tension or compression which leads to an optimal use of the applied material. Possible savings for beams were not examined because it was assumed that large reductions were not possible. When steel-concrete composite floors are used the beams are not loaded by a bending moment anymore but by an axial (tensile) force. The concrete on top of the beams provides resistance against a compressive force. Together with the tensile force in the beams this results in a bending moment. Bending moments can therefore be decoupled into a compressive force in the concrete and a tensile force in the steel beams. In these cases the use of S460 could lead to material cost savings. The stiffness of the composite beams should be large enough to limit the deflections. If the stiffness governs the design the use of S460 would not be beneficial.

Fire resistance is an important part of the engineering process. The use of S460 results in size reductions of the used columns or truss members. The effect of this size reduction on the fire resistance has however not been investigated and could lead to additional costs required for designs with the use of S460 to provide enough fire resistance.

Conservation of the sections is also of importance. Conservation and finishing of the sections is generally assumed to account for 10% of the total costs. The use of higher strength steel leads to an increase in carbon equivalent. This could lead to increased conservation costs.

6.1.1 Beams

The possible cost savings for beams was not examined. Stiffness could govern the design for which the use of S460 would not lead to any cost reductions. If however the stiffness is not governing the design any section (S355) could be replaced by another section (S460) with the same bending moment capacity. In Table 6-2 some HEA sections are shown with equivalent sections in S460.

Section S355	My,Rd [kNm]	Weight [kg/m]	Section S460	My,Rd [kNm]	Weight [kg/m]	Saving [%]
HE200A	152	42,3	HE200A	179	42,3	0,0
HE300A	447	88,3	HE280A	466	76,4	13,5
HE400A	910	125	HE360A	960	112	10,4
HE500A	1260	155	HE450A	1332	140	9,7

Table 6-2: Possible beam weight reduction

For the smallest sections a weight reduction will not be possible. S460 should therefore not be used for the design of girders. It is shown that a weight reduction of about 10% is possible for the larger sections. This reduction is however based on the bending moment capacity of the beams. In reality lateral torsional buckling should also be considered. The use of smaller sections results in an increased slenderness. The sections are therefore more susceptible for lateral torsional buckling and the resistance of sections in S460 should therefore be reduced more than sections in S355. A 10% weight reduction is therefore an overestimate of the possible weight reduction. The costs of S460 are about 5% more than the costs for S355. Combined with the low weight reduction the use of S460 will very likely never lead to decreased costs for beams.

6.1.2 Fire resistance

The fire resistance of sections is partly dependent on the section factor (A_m/V) where A_m is the surface area of the section per unit length and V is the volume of the section per unit length. For all sections in a construction a critical steel temperature can be calculated. An increased temperature results in a lower yield strength and Young's modulus of the steel. The critical temperature is the temperature for which the design resistance of a certain section becomes smaller than the design load. A large section factor results in a large temperature development in the steel. For fire resistance it is therefore beneficial to keep the section factor as small as possible. However when designing sections with the use of S460 leads to size reductions. The volume always decreases more than the surface area so smaller sections have a larger section factor than bigger sections. The use of S460 results in a faster increase of the steel temperature and therefore additional costs should be made to provide fire safety.

Additional costs required for conservation and providing fire resistance have not been calculated. The calculated cost savings are therefore an overestimate because additional costs for S460 have not been included.

6.2 Design recommendations

The use of S460 enables great cost savings for the design of axially loaded members. For large trusses used to support multiple floors in buildings some simple design recommendations can be given. It was shown that the cantilevered truss resulted in a larger cost saving than the simply supported truss. In this case the location of the support could not have been chosen differently. If the designer of a building which uses this kind of support system (trusses) is free to place 2 columns anywhere beneath the truss it is more favorable to place both columns slightly inward (Figure 6-1). When doing so the maximum force in the diagonals decreases and smaller sections may be used.

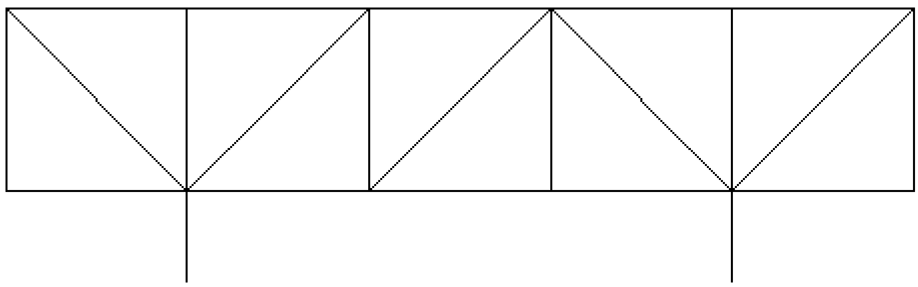


Figure 6-1: Recommended truss design for bolted connections

The type of connection used in the trusses is also of influence on the design. When bolted connections are required (this is the case when an entire truss cannot be transported in one or a few separate pieces) the connections will be similar to the ones used in the railway station (Delft). These connections significantly reduce the effectiveness of tensile members because in most cases cover plates are required. When a connection is loaded in compression cover plates will never be required. The force is transferred through the flanges only. The flanges have an area of about 80% of the area of the total section. This means that only 80% of the plastic axial capacity of compressed members can be transferred. The full capacity of a compressed member can therefore be utilized if the reduction factor for buckling is 0,8 or less. This will usually be the case for diagonal members (buckling lengths of about 5-8 m). When bolted connections are used the use of compressed members will lead to reduced costs as opposed to members loaded in tension (Figure 6-1).

When large pieces can be transported welded connections can be recommended. Assembly costs will decrease but more significantly the costs for the required sections will also decrease as opposed to the costs for bolted trusses. When welded connections are used the resistance of tensile members does not have to be reduced at the connections. This means that the full plastic capacity of sections can be used to resist the occurring forces. This leads to sections smaller than the ones required for compressed members because a reduction for buckling does not have to be made. So dependent on the type of connection different designs can be recommended. For welded trusses the use of tensile members leads to the least amount of costs (Figure 6-2).

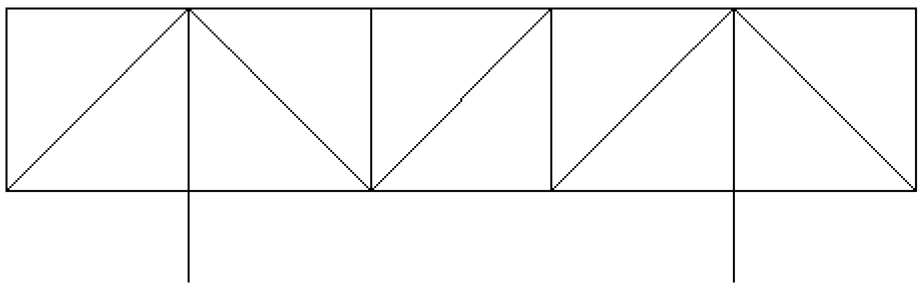


Figure 6-2: Recommended truss design for welded connections

The connections can be made with the use of S355 and S460. It is assumed that the total costs will differ slightly dependent on the steel grade used. The use of S460 will ultimately lead to the lightest construction. The weight of the trusses are however always much smaller than the total design load transferred by the trusses. Any weight savings achieved by using S460 will therefore not lead to any cost reduction for the foundation for example.

The use of S460 is highly recommendable in heavily loaded stocky columns. In high-rise buildings the buckling lengths will usually be small 4-5 m). Great reductions for buckling are therefore not required and the use of H-sections is therefore recommended. Box sections will result in larger costs when used in these situations. Box sections should only be used when the largest available H-sections is not large enough to provide enough load bearing capacity (no choice but to use box sections for a steel design). When buckling lengths are large (about 10 m or more) the use of H-sections is not recommended because buckling about the weak axis will significantly reduce the effectiveness of these sections. The use of H-sections could still be possible but they would have to be large. Instead box sections should be used for long columns because of the large stiffness in both directions. For column lengths of about 10 m the costs for a box sections and an H-section will not differ much. The increased weight of the H-section will however result in increased transportation costs. The large H-sections would also require expensive connections therefore making them less favorable to use as long columns.

The use of S460 will only lead to great cost reductions for truss members and columns. The use of this steel grade for beams is very ineffective and will likely not result in any cost reduction at all. It is therefore recommended to use S460 or stronger steel grades in axially loaded members only.

7 References

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8 Appendices

8.1 Appendix I

This appendix displays the verifications of the members of both trusses with the use of the Eurocode. The top and bottom chords require an extensive calculation. The verticals and diagonals can be checked with the use of simpler calculations. All calculations require the use of γ_{M0} or γ_{M1} which are both equal to 1 and are therefore left out of the calculations. As mentioned earlier in the paper the lateral torsional buckling length is 810mm. This length is considered small enough to prevent lateral torsional buckling so in any case the reduction factor χ_{LT} = 1.

The unity checks of the chords have been calculated by using the sections HD400x216, HD400x382 and HD400x421. A distinction can be made between chords loaded in tension and chords loaded in compression. The chords loaded in tension are both HD400x216 sections. For all sections the first necessary step is to classify the used section. The classification of the sections has to be done with the use of a ratio (c/t) which has to be compared to maximum allowable values for certain classes displayed in Table 8-1.

Classification	S460		
1	51,46	23,59	6,43
2	59,32	27,16	7,15
3	88,63	30,02	10,01
	Bending	Compression	Flange

Table 8-1: Maximum c/t ratios with f_y = 460 N/mm²

The classification of each section has been determined by using these ratios. The classification of the sections is shown in Table 8-2.

Section	Flange			Web			Class
	cf	tf	c/t	cw	tw	c/t	
	[mm]	[mm]		[mm]	[mm]		
HD400x421	173	52,6	3,29	290	32,8	8,84	1
HD400x382	173	48,0	3,60	290	29,8	9,73	1
HD400x287	173	36,6	4,73	290	22,6	12,83	1
HD400x216	173	27,7	6,25	290	17,3	16,76	1
HD400x187	173	24,0	7,21	290	15,0	19,33	3
HD360x147	164	19,8	8,28	290	12,3	23,58	3
HEB280	111	18	6,17	196	10,5	18,67	1
HEB260	101	17,5	5,77	177	10	17,70	1
HEB220	87,3	16	5,46	152	9,5	16,00	1

Table 8-2: Classification of the used sections

All chords are in class 1 which allows a plastic calculation.

The stability checks have been done with the use of more favorable buckling curves because it is allowed to use a lower imperfection factor when using HISTAR460.

HD400x216:

The chords loaded in tension are checked for the shear force first. The maximum shear force in is 575 kN (truss 10). The resistance is calculated with the use of a shear area. The calculation of the shear area has been done according to the guidelines provided in NEN-EN-1993-1-1 6.2.6: $A_v = A-2xbxt_f+(t_w+2xr)xt_f \geq \eta xh_w xt_w$

27550-2x394x27,7+(17,3+2x15)x27,7 = 7032 mm^2 1,2x290x17,3 = 6012 mm^2 → A_v = 7032 mm^2

- $V_{Rd} = A_v x f_y/(\sqrt{3}x1000)$ = 1868 kN Unity check: 575/1868 = 0,31

Because the shear unity check is below 0,5 the effect of the shear force can be neglected.

The calculation of the unity check only requires the use of the axial force and the bending moment. There are 2 situations: N_{Ed} = 6630 kN and M_{Ed} = 931 kNm (truss 10) and N_{Ed} = 9675 kN and M_{Ed} = 456 kNm (truss 11).

- $N_{Rd} = Axf_y/1000$ = 12673 kN Unity check: 9675/12673 = 0,76
- $M_{Rd} = W_{pl}xf_y/1000000$ = 1961 kNm Unity check: 931/1961 = 0,47

Combination of N and M (truss 10):

- $n = N_{Ed}/N_{Rd}$ = 6630/12673 = 0,52
- $a = (A-2xbxt_f)/A$ = (27550-2x394x27,7)/27550 = 0,21
- $M_{N,Rd} = M_{Rd}x(1-n)/(1-0,5xa)$ = 1961x(1-0,52)/(1-0,5x0,21) = 1043 kNm
- Unity check: 931/1043 = 0,89

Combination of N and M (truss 11):

- $n = N_{Ed}/N_{Rd}$ = 9675/12673 = 0,76
- $a = (A-2xbxt_f)/A$ = (27550-2x394x27,7)/27550 = 0,21
- $M_{N,Rd} = M_{Rd}x(1-n)/(1-0,5xa)$ = 1961x(1-0,76)/(1-0,5x0,21) = 518 kNm
- Unity check: 456/518 = 0,88

The section HD400x216 is able to resist the occurring forces in both trusses.

The check of the compressed chords (HD400x382 and HD400x421) is more extensive because of the compressive force. The governing unity check will in this case be the combination of compression and bending. The combination of compression and bending is checked according to NEN-EN 1993-1-1 6.3.3:

$N_{Ed}/(\chi_y x N_{Rk}) + k_{yy} x M_{Ed}/(\chi_{LT} x M_{Rk}) \leq 1$ $N_{Ed}/(\chi_z x N_{Rk}) + k_{zy} x M_{Ed}/(\chi_{LT} x M_{Rk}) \leq 1$

The factors describing the combination of compression and bending (k_{yy} and k_{zy}) are dependent on the value of the compressive force and (dependent on the system used (sway/non-sway)) the values of the bending moment at the vertical supports and midspan. These interaction factors are determined with the use of method 2 (appendix B in NEN-EN 1993-1-1). Table B1 and B3 are used which in this case results in $k_{zy} = 0,6xk_{yy}$.

The shear force in the HD400x382 section is 576 kN, in the HD400x421 section this is 593 kN. For the HD400x216 section a shear force of 575 kN results in a unity check of 0,31. Because the sections used for the compressed chords are larger and the shear force in these sections are only slightly larger it can be assumed that the shear force unity check will in both cases be below 0,5. In both compressed chords the shear force can be neglected. This means that the unity check of combined axial force and bending is governing for the section check.

HD400x382:

The forces occurring in the governing cross section of truss 10 are: N_{Ed} = 10714 kN, V_{Ed} = 576 kN and M_{Ed} = 1547 kNm. The cross-sectional area (A) = 48710 mm^2 → N_{Rk} = 48710x460/1000 = 22407 kN. The plastic section modulus (W_{pl}) = 7965x10^3 mm^3 → M_{Rk} = 7965x10^{3}x460/1000000 = 3664 kNm. The moment of inertia (I_y) = 141300x10^4 mm^4 and (I_z) = 53620x10^4 mm^4.

Combination of N and M (truss 10):

- $n = N_{Ed}/N_{Rd}$ = 10714/22407 = 0,48
- $a = (A-2xbxt_f)/A$ = (48710-2x406x48)/48710 = 0,20
- $M_{N,Rd} = M_{Rd}x(1-n)/(1-0,5xa)$ = 3664x(1-0,48)/(1-0,5x0,20) = 2124 kNm
- Unity check: 1547/2124 = 0,73

Stability checks:

- $L_{k,y}$ = 4100 mm	$L_{k,z}$ = 4100 mm
- α_y = 0,21 (curve a)	α_z = 0,21 (curve a)
- $N_{cr,y} = \pi^2 xExI_y/(L_{k,y}{}^2x1000)$ = 174218 kN	$N_{cr,z} = \pi^2 xExI_z/(L_{k,z}{}^2x1000)$ = 66112 kN
- $\lambda_y = \sqrt{}(N_{Rk}/N_{cr,y})$ = 0,359	$\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z})$ = 0,582
- $\phi_y = 0,5x(1+\alpha_y x(\lambda_y-0,2)+\lambda_y{}^2)$ = 0,581	$\phi_z = 0,5x(1+\alpha_z x(\lambda_z-0,2)+\lambda_z{}^2)$ = 0,710
- $\chi_y = 1/(\phi_y+\sqrt{}(\phi_y{}^2-\lambda_y{}^2))$ = 0,963	$\chi_z = 1/(\phi_z+\sqrt{}(\phi_z{}^2-\lambda_z{}^2))$ = 0,897
- Unity check: $N_{Ed}/(\chi_y xN_{Rk})$ = 0,50	Unity check: $N_{Ed}/(\chi_z xN_{Rk})$ = 0,53

The interaction factors are dependent on these unity checks and the factor C_{my}. In this case the nodes aren't fixed in their position which leads to the use of the sway mode for which C_{my} = 0,9.

- $k_{yy} = C_{my}x(1+(\lambda_y-0,2)x\ N_{Ed}/(\chi_y xN_{Rk})$ = 0,971
- $k_{zy} = 0,6xk_{yy}$ = 0,583
- Unity check: $N_{Ed}/(\chi_y xN_{Rk}) + k_{yy}xM_{Ed}/(\chi_{LT}xM_{Rk})$ = 0,91
- Unity check: $N_{Ed}/(\chi_z xN_{Rk}) + k_{zy}xM_{Ed}/(\chi_{LT}xM_{Rk})$ = 0,78

HD400x421:

The forces occurring in the governing cross section of truss 11 are: N_{Ed} = 10071 kN, V_{Ed} = 593 kN and M_{Ed} = 926 kNm. The cross-sectional area (A) = 53710 mm^2 → N_{Rk} = 53710x460/1000 = 24707 kN. The plastic section modulus (W_{pl}) = 8880x10^3 mm^3 → M_{Rk} = 8880x10^{3}x460/1000000 = 4085 kNm. The moment of inertia (I_y) = 159600x10^4 mm^4 and (I_z) = 60080x10^4 mm^4.

Combination of N and M (truss 11):

- $n = N_{Ed}/N_{Rd} = 10071/24707 = 0{,}41$
- $a = (A\text{-}2xbxt_f)/A = (53710\text{-}2x409x52{,}6)/53710 = 0{,}20$
- $M_{N,Rd} = M_{Rd}x(1\text{-}n)/(1\text{-}0{,}5xa) = 4085x(1\text{-}0{,}41)/(1\text{-}0{,}5x0{,}20) = 2687$ kNm
- Unity check: 926/2687 = 0,34

Stability checks:

- $L_{k,y} = 8100$ mm
- $\alpha_y = 0{,}21$ (curve a)
- $N_{cr,y} = \pi^2 xExI_y/(L_{k,y}{}^2x1000) = 50418$ kN
- $\lambda_y = \sqrt{}(N_{Rk}/N_{cr,y}) = 0{,}700$
- $\phi_y = 0{,}5x(1+\alpha_y x(\lambda_y\text{-}0{,}2)+\lambda_y{}^2) = 0{,}798$
- $\chi_y = 1/(\phi_y+\sqrt{}(\phi_y{}^2\text{-}\lambda_y{}^2)) = 0{,}848$
- Unity check: $N_{Ed}/(\chi_y xN_{Rk}) = 0{,}48$

$L_{k,z} = 8100$ mm
$\alpha_z = 0{,}21$ (curve a)
$N_{cr,z} = \pi^2 xExI_z/(L_{k,z}{}^2x1000) = 18979$ kN
$\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z}) = 1{,}141$
$\phi_z = 0{,}5x(1+\alpha_z x(\lambda_z\text{-}0{,}2)+\lambda_z{}^2) = 1{,}250$
$\chi_z = 1/(\phi_z+\sqrt{}(\phi_z{}^2\text{-}\lambda_z{}^2)) = 0{,}568$
Unity check: $N_{Ed}/(\chi_z xN_{Rk}) = 0{,}72$

The interaction factors are dependent on these unity checks and the factor C_{my}. In this case the nodes aren't fixed in their position which leads to the use of the sway mode for which $C_{my} = 0{,}9$.

- $k_{yy} = C_{my}x(1+(\lambda_y\text{-}0{,}2)x\ N_{Ed}/(\chi_y xN_{Rk}) = 1{,}116$
- $k_{zy} = 0{,}6xk_{yy} = 0{,}670$
- Unity check: $N_{Ed}/(\chi_y xN_{Rk}) + k_{yy}xM_{Ed}/(\chi_{LT}xM_{Rk}) = 0{,}73$
- Unity check: $N_{Ed}/(\chi_z xN_{Rk}) + k_{zy}xM_{Ed}/(\chi_{LT}xM_{Rk}) = 0{,}87$

Both compressed chords are able to resist the occurring forces.

The unity checks of the verticals and diagonals are calculated with the use of axial forces only. The calculations used are valid for sections of class 1, 2 and 3 which corresponds to all sections used. The factor $\gamma_{M2} = 1{,}25$. For members in tension 2 checks were made: Yielding of the section and fracture of the section. In reality only yielding of the section has to be checked when dimensioning the members. In this case the fracture of the section is also reviewed because bolted connections are used. This is done to easily predict the need of cover plates at the connections. If the unity checks are close to 1 for tensile members then these sections will probably need to be thickened at the connections to compensate for the bolt holes. For each section a single calculation with the use of the largest tensile force is made. For members in compression 2 checks were made: buckling about the strong and weak axis or, when 2 different combinations of force and buckling length occur, buckling about the weak axis for both combinations. The forces displayed are the maximum forces present for the given section. This prevents the need to calculate the resistance for each member.

HEB220, Max tension = 2893 kN, Max compression = 2571 kN (short), 1344 kN (long):

- $A = 9100\ mm^2$
- $I_y = 8091x10^4\ mm^4$
- $I_z = 2843x10^4\ mm^4$

Tension checks:

- $N_{pl,Rd} = Axf_y/1000 = 4186$ kN — Unity check: 2893/4186 = 0,69
- $N_{t,u,d} = 0{,}9xAxf_u/(1{,}25x1000) = 3538$ kN — Unity check: 2893/3538 = 0,82

Compression checks:

- $L_{k,z} = 3500$ mm ($N_{Ed} = 2571$ kN)
- $\alpha_z = 0{,}21$ (curve a)
- $N_{cr,z} = \pi^2xExI_z/(L_{k,z}^2x1000) = 4810$ kN
- $\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z}) = 0{,}933$
- $\phi_z = 0{,}5x(1+\alpha_zx(\lambda_z-0{,}2)+\lambda_z^2) = 1{,}012$
- $\chi_z = 1/(\phi_z+\sqrt{}(\phi_z^2-\lambda_z^2)) = 0{,}712$
- Unity check: $N_{Ed}/(\chi_zxN_{Rk}) = 0{,}86$

- $L_{k,z} = 5353$ mm ($N_{Ed} = 1344$ kN)
- $\alpha_z = 0{,}21$ (curve a)
- $N_{cr,z} = \pi^2xExI_z/(L_{k,z}^2x1000) = 2056$ kN
- $\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z}) = 1{,}427$
- $\phi_z = 0{,}5x(1+\alpha_zx(\lambda_z-0{,}2)+\lambda_z^2) = 1{,}647$
- $\chi_z = 1/(\phi_z+\sqrt{}(\phi_z^2-\lambda_z^2)) = 0{,}405$
- Unity check: $N_{Ed}/(\chi_zxN_{Rk}) = 0{,}79$

HEB260, No tension, Max compression = 2697 kN:

- $A = 11840\ mm^2$
- $I_y = 14920x10^4\ mm^4$
- $I_z = 5135x10^4\ mm^4$

Compression checks:

- $L_{k,y} = 4880$ mm
- $\alpha_y = 0{,}21$ (curve a)
- $N_{cr,y} = \pi^2xExI_y/(L_{k,y}^2x1000) = 12985$ kN
- $\lambda_y = \sqrt{}(N_{Rk}/N_{cr,y}) = 0{,}648$
- $\phi_y = 0{,}5x(1+\alpha_yx(\lambda_y-0{,}2)+\lambda_y^2) = 0{,}757$
- $\chi_y = 1/(\phi_y+\sqrt{}(\phi_y^2-\lambda_y^2)) = 0{,}871$
- Unity check: $N_{Ed}/(\chi_yxN_{Rk}) = 0{,}57$

- $L_{k,z} = 4880$ mm
- $\alpha_z = 0{,}21$ (curve a)
- $N_{cr,z} = \pi^2xExI_z/(L_{k,z}^2x1000) = 4469$ kN
- $\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z}) = 1{,}104$
- $\phi_z = 0{,}5x(1+\alpha_zx(\lambda_z-0{,}2)+\lambda_z^2) = 1{,}204$
- $\chi_z = 1/(\phi_z+\sqrt{}(\phi_z^2-\lambda_z^2)) = 0{,}593$
- Unity check: $N_{Ed}/(\chi_zxN_{Rk}) = 0{,}83$

HEB280, Max tension = 4595 kN, No compression:

- $A = 13140\ mm^2$
- $I_y = 19270x10^4\ mm^4$
- $I_z = 6595x10^4\ mm^4$

Tension checks:

- $N_{pl,Rd} = Axf_y/1000 = 6044$ kN — Unity check: 2893/4186 = 0,76
- $N_{t,u,d} = 0{,}9xAxf_u/(1{,}25x1000) = 5109$ kN — Unity check: 2893/3538 = 0,90

HD360x147, No tension, Max compression = 6226 kN (short), 5100 kN (long):

- $A = 18790\ mm^2$
- $I_y = 46290x10^4\ mm^4$
- $I_z = 16720x10^4\ mm^4$

Compression checks:

- $L_{k,z} = 3500$ mm ($N_{Ed} = 6226$ kN) — $L_{k,z} = 5353$ mm ($N_{Ed} = 5100$ kN)
- $\alpha_z = 0{,}21$ (curve a) — $\alpha_z = 0{,}21$ (curve a)
- $N_{cr,z} = \pi^2 xExI_z/(L_{k,z}^2 x1000) = 28289$ kN — $N_{cr,z} = \pi^2 xExI_z/(L_{k,z}^2 x1000) = 12094$ kN
- $\lambda_z = \sqrt{(N_{Rk}/N_{cr,z})} = 0{,}553$ — $\lambda_z = \sqrt{(N_{Rk}/N_{cr,z})} = 0{,}845$
- $\phi_z = 0{,}5x(1+\alpha_z x(\lambda_z-0{,}2)+\lambda_z^2) = 0{,}690$ — $\phi_z = 0{,}5x(1+\alpha_z x(\lambda_z-0{,}2)+\lambda_z^2) = 0{,}925$
- $\chi_z = 1/(\phi_z+\sqrt{(\phi_z^2-\lambda_z^2)}) = 0{,}907$ — $\chi_z = 1/(\phi_z+\sqrt{(\phi_z^2-\lambda_z^2)}) = 0{,}769$
- Unity check: $N_{Ed}/(\chi_z xN_{Rk}) = 0{,}79$ — Unity check: $N_{Ed}/(\chi_z xN_{Rk}) = 0{,}77$

HD400x187, Max tension = 8493 kN, Max compression = 7831 kN:

- $A = 23760\ mm^2$
- $I_y = 60180x10^4\ mm^4$
- $I_z = 23920x10^4\ mm^4$

Tension checks:

- $N_{pl,Rd} = Axf_y/1000 = 10930$ kN — Unity check: 2893/4186 = 0,78
- $N_{t,u,d} = 0{,}9xAxf_u/(1{,}25x1000) = 9238$ kN — Unity check: 2893/3538 = 0,92

Compression checks:

- $L_{k,y} = 5353$ mm — $L_{k,z} = 5353$ mm
- $\alpha_y = 0{,}21$ (curve a) — $\alpha_z = 0{,}21$ (curve a)
- $N_{cr,y} = \pi^2 xExI_y/(L_{k,y}^2 x1000) = 43529$ kN — $N_{cr,z} = \pi^2 xExI_z/(L_{k,z}^2 x1000) = 17302$ kN
- $\lambda_y = \sqrt{(N_{Rk}/N_{cr,y})} = 0{,}501$ — $\lambda_z = \sqrt{(N_{Rk}/N_{cr,z})} = 0{,}795$
- $\phi_y = 0{,}5x(1+\alpha_y x(\lambda_y-0{,}2)+\lambda_y^2) = 0{,}657$ — $\phi_z = 0{,}5x(1+\alpha_z x(\lambda_z-0{,}2)+\lambda_z^2) = 0{,}878$
- $\chi_y = 1/(\phi_y+\sqrt{(\phi_y^2-\lambda_y^2)}) = 0{,}924$ — $\chi_z = 1/(\phi_z+\sqrt{(\phi_z^2-\lambda_z^2)}) = 0{,}799$
- Unity check: $N_{Ed}/(\chi_y xN_{Rk}) = 0{,}78$ — Unity check: $N_{Ed}/(\chi_z xN_{Rk}) = 0{,}90$

HD400x216, Max tension = 8962 kN, No compression:

- $A = 27550\ mm^2$
- $I_y = 71140x10^4\ mm^4$
- $I_z = 28250x10^4\ mm^4$

Tension checks:

- $N_{pl,Rd} = Axf_y/1000 = 12673$ kN — Unity check: 2893/4186 = 0,71
- $N_{t,u,d} = 0{,}9xAxf_u/(1{,}25x1000) = 10711$ kN — Unity check: 2893/3538 = 0,84

HD400x287, No tension, Max compression = 9779 kN:

- $A = 36630\ mm^2$
- $I_y = 99710x10^4\ mm^4$
- $I_z = 38780x10^4\ mm^4$

Compression checks:

- $L_{k,y} = 5353$ mm — $L_{k,z} = 5353$ mm
- $\alpha_y = 0,21$ (curve a) — $\alpha_z = 0,21$ (curve a)
- $N_{cr,y} = \pi^2 xExI_y/(L_{k,y}{}^2 x1000) = 72121$ kN — $N_{cr,z} = \pi^2 xExI_z/(L_{k,z}{}^2 x1000) = 28050$ kN
- $\lambda_y = \sqrt{}(N_{Rk}/N_{cr,y}) = 0,483$ — $\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z}) = 0,775$
- $\phi_y = 0,5x(1+\alpha_y x(\lambda_y-0,2)+\lambda_y{}^2) = 0,647$ — $\phi_z = 0,5x(1+\alpha_z x(\lambda_z-0,2)+\lambda_z{}^2) = 0,861$
- $\chi_y = 1/(\phi_y+\sqrt{}(\phi_y{}^2-\lambda_y{}^2)) = 0,929$ — $\chi_z = 1/(\phi_z+\sqrt{}(\phi_z{}^2-\lambda_z{}^2)) = 0,810$
- Unity check: $N_{Ed}/(\chi_y x N_{Rk}) = 0,62$ — Unity check: $N_{Ed}/(\chi_z x N_{Rk}) = 0,72$

HD400x382, No tension, Max compression = 15039 kN:

- $A = 48710\ mm^2$
- $I_y = 141300x10^4\ mm^4$
- $I_z = 53620x10^4\ mm^4$

Compression checks:

- $L_{k,y} = 3500$ mm — $L_{k,z} = 3500$ mm
- $\alpha_y = 0,21$ (curve a) — $\alpha_z = 0,21$ (curve a)
- $N_{cr,y} = \pi^2 xExI_y/(L_{k,y}{}^2 x1000) = 239070$ kN — $N_{cr,z} = \pi^2 xExI_z/(L_{k,z}{}^2 x1000) = 90721$ kN
- $\lambda_y = \sqrt{}(N_{Rk}/N_{cr,y}) = 0,306$ — $\lambda_z = \sqrt{}(N_{Rk}/N_{cr,z}) = 0,497$
- $\phi_y = 0,5x(1+\alpha_y x(\lambda_y-0,2)+\lambda_y{}^2) = 0,558$ — $\phi_z = 0,5x(1+\alpha_z x(\lambda_z-0,2)+\lambda_z{}^2) = 0,655$
- $\chi_y = 1/(\phi_y+\sqrt{}(\phi_y{}^2-\lambda_y{}^2)) = 0,976$ — $\chi_z = 1/(\phi_z+\sqrt{}(\phi_z{}^2-\lambda_z{}^2)) = 0,925$
- Unity check: $N_{Ed}/(\chi_y x N_{Rk}) = 0,69$ — Unity check: $N_{Ed}/(\chi_z x N_{Rk}) = 0,73$

8.2 Appendix II

This appendix displays the verifications of the connections of both trusses with the use of the Eurocode. The calculations of the splices in the chords are described first, then the calculations of the general splice details and finally the gusset plates are checked.

The design of the chord splices is reviewed separately because the web can also be connected with the use of splice plates. Because of this the forces can be conveniently divided over the flanges and the web. In all cases it is assumed that the flanges transfer 70% of the axial force and the entire bending moment, the web is assumed to transfer 30% of the axial force and the entire shear force. The shear force also results in bending moments in the splice plates of the web due to the eccentricity of the force. This bending moment is equal to the shear force times the distance between the center of the bolt group (at one side of the splice) and the center of the splice. In all cases a spacing of 10 mm is used between the beam ends.

In Table 4-9 it can be seen that for every bolt size used a reduction factor should be applied when there are more than 5 bolts in a single row. This reduction factor is described with the following formula: β_{Lf} = 1-(L_j-15d)/(200d) and 0,75 ≤ β_{Lf} ≤ 1,0. In this case reduction factors should only be applied for the splice in the bottom chord of truss 10. 6 M36 bolts per row are used to connect the flanges. L_j is in that case 550 mm. This leads to a reduction factor of 0,9986. 6 M27 bolts are used per row to connect the web. A smaller spacing is however used which leads to a connection length of 400 mm. 15xd = 15x27 = 405 mm. In this case no reduction factor has to be applied for the bolts connecting the web.

All members in compression are checked with the use of the yield strength:

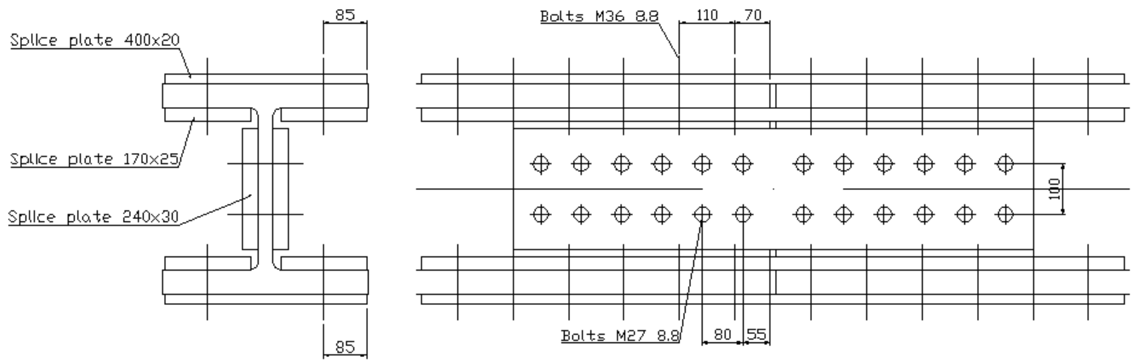
- $N_{c,u,d} = Axf_y$ with
- $A = b_fxt_f$ for the flanges
- $A = 2xb_pxt_p$ for the splice plates outside the section
- $A = 4xb_pxt_p$ for the splice plates inside the section

All members in tension are checked with the use of the tensile strength:

- $N_{t,u,d} = 0{,}9xA_{net}xfu/\gamma_{M2}$ $\gamma_{M2} = 1{,}25$ with
- $A_{net} = b_fxt_f - 2xd_0xt_f$ for the flanges
- $A_{net} = 2xb_pxt_p - 4xd_0xt_p$ for the splice plates outside the section
- $A_{net} = 4xb_pxt_p - 4xd_0xt_p$ for the splice plates inside the section

The check of the splice plates connecting the web has been made with the use of the yield criterion because an axial force, shear force and bending moment can occur in these plates.

TRUSS 10 BOTTOM CHORD SPLICE:



N_{Ed} = 10730 kN V_{Ed} = 455 kN M_{Ed} = 1092 kNm

Splice plates connecting the flanges transfer 70% of the axial force and 100% of the bending moment

Splice plates connecting the web transfer 30% of the axial force and 100% of the shear force.

- Flanges: F_{Ed} = 0,7x10730/2+1092x1000/(416-48) = 6723 kN (Compression only)
- Web: F_{Ed} = 0,3x10730 = 3219 kN V_{Ed} = 455 kN M_{Ed} = 455x0,26 = 118,3 kNm

Bolts:

- Flange shear capacity: $F_{v,u,d}$ = 0,9986x12x2x313,7 = 7518 kN
- Flange bearing capacity: $F_{b,u,d}$ = 12x2x23,26x20 = 11165 kN (Splice plates are governing)
- Web shear capacity (single bolt): $F_{v,u,d}$ = 2x176,3 = 352,6 kN
- Web bearing capacity (single bolt): $F_{b,u,d}$ = 17,82x29,8 = 531 kN (Web is governing)

Flanges:

- $N_{c,u,d}$ = 406x48x460/1000 = 8964 kN

Flange splice plates (outside section):

- $N_{c,u,d}$ = 2x400x20x460/1000 = 7360 kN

Flange splice plates (inside section):

- $N_{c,u,d}$ = 4x170x25x460/1000 = 7820 kN

Web splice plates:

- Normal stress = 3219x1000/(2x240x30) = 223,5 N/mm^2
- Shear stress = 455x1000/(2x(240-2x30)x30) = 42,1 N/mm^2
- Bending stress = 118,3x1000000/(0,25x2x240^{2}x30-2x2x30x30x50) = 173,0 N/mm^2
- Combined stress = √((223,5+173,0)^2+3x42,1^2) = 403,2 N/mm^2 ≤ 460 N/mm^2

Maximum force in bolts (web):

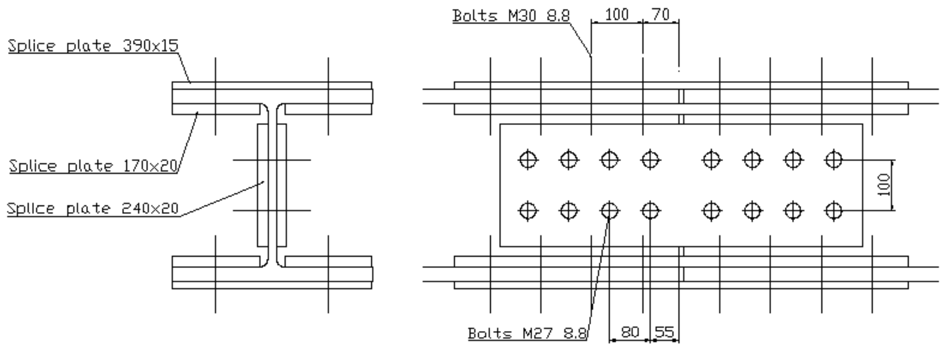
NUMBER OF BOLTS	=	12		
VERT. FORCE	=	455	kN	(= POSITIVE)
HORZ. FORCE	=	-3219	kN	(→ = POSITIVE)
MOMENT	=	118,3	kNm	(CLOCKWISE = POSITIVE)

CENTRE OF GRAVITY						
X =	200	mm	$\Sigma (x^2)$	=	224000	mm²
Y =	50	mm	$\Sigma (y^2)$	=	30000	mm²
			$\Sigma (x^2+y^2)$	=	254000	mm²

BOLT	X	Y	X T.O.V. COG	Y T.O.V. COG	x²	y²	Fv	Fh	Fr
	mm	mm			mm²	mm²	kN	kN	kN
1	0	0	-200	-50	40000	2500	131,07	-291,54	319,64
2	80	0	-120	-50	14400	2500	93,81	-291,54	306,26
3	160	0	-40	-50	1600	2500	56,55	-291,54	296,97
4	240	0	40	-50	1600	2500	19,29	-291,54	292,17
5	320	0	120	-50	14400	2500	-17,97	-291,54	292,09
6	400	0	200	-50	40000	2500	-55,23	-291,54	296,72
7	0	100	-200	50	40000	2500	131,07	-244,96	277,82
8	80	100	-120	50	14400	2500	93,81	-244,96	262,31
9	160	100	-40	50	1600	2500	56,55	-244,96	251,40
10	240	100	40	50	1600	2500	19,29	-244,96	245,72
11	320	100	120	50	14400	2500	-17,97	-244,96	245,62
12	400	100	200	50	40000	2500	-55,23	-244,96	251,11

- $F_{u,d}$ = 320 kN ≤ 352,6 kN

TRUSS 10 TOP CHORD SPLICE:



N_{Ed} = 6977 kN V_{Ed} = 149 kN M_{Ed} = 342 kNm

Splice plates connecting the flanges transfer 70% of the axial force and 100% of the bending moment

Splice plates connecting the web transfer 30% of the axial force and 100% of the shear force.

- Flanges: F_{Ed} = 0,7x6977/2+342x1000/(375-27,7) = 3427 kN
- Web: F_{Ed} = 0,3x6977 = 2093 kN V_{Ed} = 149 kN M_{Ed} = 149x0,18 = 26,8 kNm

Bolts:

- Flange shear capacity: $F_{v,u,d}$ = 8x2x215,4 = 3446 kN
- Flange bearing capacity: $F_{b,u,d}$ = 8x22,91x27,7 = 5077 kN (Flanges are governing)
- Web shear capacity (single bolt): $F_{v,u,d}$ = 2x176,3 = 352,6 kN
- Web bearing capacity (single bolt): $F_{b,u,d}$ = 17,82x17,3 = 308 kN (Web is governing)

Flanges:

- $N_{t,u,d}$ = 0,9x(394-2x33)x27,7x540/(1,25x1000) = 3532 kN

Flange splice plates (outside section):

- $N_{t,u,d}$ = 2x0,9x(390-2x33)x15x540/(1,25x1000) = 3779 kN

Flange splice plates (inside section):

- $N_{t,u,d}$ = 4x0,9x(170-33)x20x540/(1,25x1000) = 4261 kN

Web splice plates:

- Normal stress = 2093x1000/(2x(240-2x30)x20) = 290,7 N/mm^2
- Shear stress = 149x1000/(2x(240-2x30)x20) = 20,7 N/mm^2
- Bending stress = 26,8x1000000/(0,25x2x240^{2}x20-2x2x30x20x50) = 58,8 N/mm^2
- Combined stress = $\sqrt{}$((290,7+58,8)^2+3x20,7^2) = 351,4 N/mm^2 ≤ 460 N/mm^2

Maximum force in bolts (web):

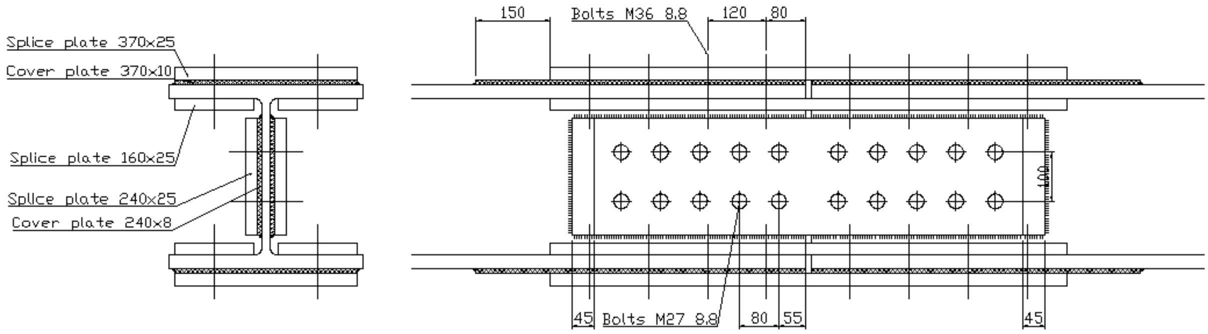
NUMBER OF BOLTS	=	8		
VERT. FORCE	=	149	kN	(= POSITIVE)
HORZ. FORCE	=	2093	kN	(→ = POSITIVE)
MOMENT	=	26,8	kNm	(CLOCKWISE = POSITIVE)

CENTRE OF GRAVITY			$\Sigma (x^2)$	=	64000	mm^2
X =	120	mm	$\Sigma (y^2)$	=	20000	mm^2
Y =	50	mm	$\Sigma (x^2+y^2)$	=	84000	mm^2

BOLT	X	Y	X T.O.V. COG	Y T.O.V. COG	x^2	y^2	Fv	Fh	Fr
	mm	mm			mm^2	mm^2	kN	kN	kN
1	0	0	-120	-50	14400	2500	56,91	245,67	252,18
2	80	0	-40	-50	1600	2500	31,39	245,67	247,67
3	160	0	40	-50	1600	2500	5,86	245,67	245,74
4	240	0	120	-50	14400	2500	-19,66	245,67	246,46
5	0	100	-120	50	14400	2500	56,91	277,58	283,35
6	80	100	-40	50	1600	2500	31,39	277,58	279,35
7	160	100	40	50	1600	2500	5,86	277,58	277,64
8	240	100	120	50	14400	2500	-19,66	277,58	278,27

- $F_{u,d}$ = 283 kN ≤ 308 kN

TRUSS 11 BOTTOM CHORD SPLICE:



N_{Ed} = 10336 kN V_{Ed} = 11 kN M_{Ed} = 322 kNm

Splice plates connecting the flanges transfer 70% of the axial force and 100% of the bending moment

Splice plates connecting the web transfer 30% of the axial force and 100% of the shear force.

- Flanges: F_{Ed} = 0,7x10336/2+332x1000/(375-27,7) = 4545 kN
- Web: F_{Ed} = 0,3x10336 = 3101 kN V_{Ed} = 11 kN M_{Ed} = 11x0,22 = 2,4 kNm

Bolts:

- Flange shear capacity: $F_{v,u,d}$ = 8x2x313,7 = 5019 kN
- Flange bearing capacity: $F_{b,u,d}$ = 8x2x26,58x15 = 6379 kN (Splice plates are governing)
- Web shear capacity (single bolt): $F_{v,u,d}$ = 2x176,3 = 352,6 kN
- Web bearing capacity (single bolt): $F_{b,u,d}$ = 17,82x17,3 = 308 kN (Web is governing)

Flanges:

- $N_{t,u,d}$ = 0,9x(394-2x39)x(27,7+10)x540/(1,25x1000) = 4632 kN

Flange splice plates (outside section):

- $N_{t,u,d}$ = 2x0,9x(370-2x39)x25x540/(1,25x1000) = 5676 kN

Flange splice plates (inside section):

- $N_{t,u,d}$ = 4x0,9x(160-39)x25x540/(1,25x1000) = 4704 kN

Web:

- $N_{t,u,d}$ = 0,9x(320-2x30)x(17,3+2x8)x540/(1,25x1000) = 3366 kN

Web splice plates:

- $N_{t,u,d}$ = 2x0,9x(240-2x30)x25x540/(1,25x1000) = 3499 kN

Maximum force in bolts (web):

NUMBER OF BOLTS = 10
VERT. FORCE = 11 kN (= POSITIVE)
HORZ. FORCE = 3101 kN (→ = POSITIVE)
MOMENT = 2,4 kNm (CLOCKWISE = POSITIVE)

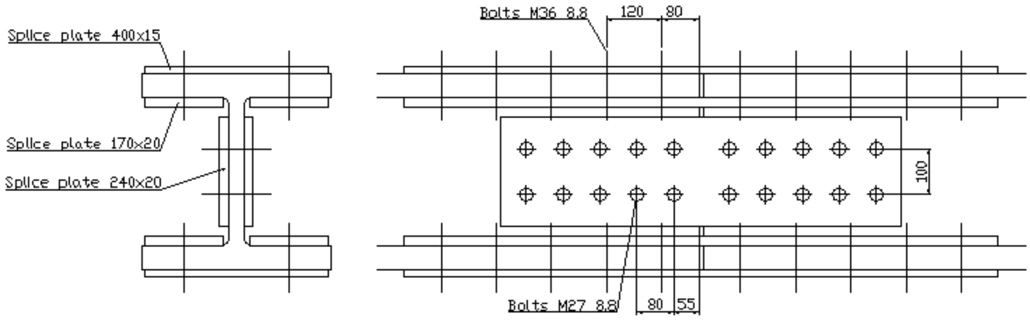
CENTRE OF GRAVITY
X = 160 mm
Y = 50 mm

$\Sigma (x^2)$ = 128000 mm^2
$\Sigma (y^2)$ = 25000 mm^2
$\Sigma (x^2+y^2)$ = 153000 mm^2

BOLT	X	Y	X T.O.V. COG	Y T.O.V. COG	x^2	y^2	Fv	Fh	Fr
	mm	mm	COG	COG	mm^2	mm^2	kN	kN	kN
1	0	0	-160	-50	25600	2500	3,61	309,32	309,34
2	80	0	-80	-50	6400	2500	2,35	309,32	309,32
3	160	0	0	-50	0	2500	1,10	309,32	309,32
4	240	0	80	-50	6400	2500	-0,15	309,32	309,32
5	320	0	160	-50	25600	2500	-1,41	309,32	309,32
6	0	100	-160	50	25600	2500	3,61	310,88	310,91
7	80	100	-80	50	6400	2500	2,35	310,88	310,89
8	160	100	0	50	0	2500	1,10	310,88	310,89
9	240	100	80	50	6400	2500	-0,15	310,88	310,88
10	320	100	160	50	25600	2500	-1,41	310,88	310,89

- $F_{u,d}$ = 311 kN ≤ 308 kN NOT OK but the difference is small and plastic deformation allows the remaining force to flow through the flanges.

TRUSS 11 TOP CHORD SPLICE:



N_{Ed} = 10619 kN V_{Ed} = 31 kN M_{Ed} = 653 kNm

Splice plates connecting the flanges transfer 70% of the axial force and 100% of the bending moment

Splice plates connecting the web transfer 30% of the axial force and 100% of the shear force.

- Flanges: F_{Ed} = 0,7x10619/2+653x1000/(425-52,6) = 5470 kN (Compression only)
- Web: F_{Ed} = 0,3x10619 = 3186 kN V_{Ed} = 31 kN M_{Ed} = 31x0,22 = 6,8 kNm

Bolts:

- Flange shear capacity: $F_{v,u,d}$ = 10x2x313,7 = 6274 kN
- Flange bearing capacity: $F_{b,u,d}$ = 10x2x23,26x20 = 11165 kN (Splice plates are governing)
- Web shear capacity (single bolt): $F_{v,u,d}$ = 2x176,3 = 352,6 kN
- Web bearing capacity (single bolt): $F_{b,u,d}$ = 17,82x32,8 = 584 kN (Web is governing)

Flanges:

- $N_{c,u,d}$ = 409x52,6x460/1000 = 9896 kN

Flange splice plates (outside section):

- $N_{c,u,d}$ = 2x400x15x460/1000 = 5520 kN

Flange splice plates (inside section):

- $N_{c,u,d}$ = 4x170x20x460/1000 = 6256 kN

Web splice plates:

- Normal stress = 3186x1000/(2x240x20) = 311,8 N/mm^2
- Shear stress = 31x1000/(2x(240-2x30)x20) = 4,3 N/mm^2
- Bending stress = 6,8x1000000/(0,25x2x240^{2}x20-2x2x30x20x50) = 15,0 N/mm^2
- Combined stress = √((311,8+15,0)^2+3x4,3^2) = 346,9 N/mm^2 ≤ 460 N/mm^2

Maximum force in bolts (web):

NUMBER OF BOLTS = 10

VERT. FORCE = 31 kN (= POSITIVE)

HORZ. FORCE = -3186 kN (→ = POSITIVE)

MOMENT = 6,8 kNm (CLOCKWISE = POSITIVE)

CENTRE OF GRAVITY						
X =	160	mm	Σ (x²)	=	128000	mm²
Y =	50	mm	Σ (y²)	=	25000	mm²
			Σ (x²+y²)	=	153000	mm²

BOLT	X	Y	X T.O.V.	Y T.O.V.	x²	y²	Fv	Fh	Fr
	mm	mm	COG	COG	mm²	mm²	kN	kN	kN
1	0	0	-160	-50	25600	2500	10,21	-320,82	320,98
2	80	0	-80	-50	6400	2500	6,66	-320,82	320,89
3	160	0	0	-50	0	2500	3,10	-320,82	320,84
4	240	0	80	-50	6400	2500	-0,46	-320,82	320,82
5	320	0	160	-50	25600	2500	-4,01	-320,82	320,85
6	0	100	-160	50	25600	2500	10,21	-316,38	316,54
7	80	100	-80	50	6400	2500	6,66	-316,38	316,45
8	160	100	0	50	0	2500	3,10	-316,38	316,39
9	240	100	80	50	6400	2500	-0,46	-316,38	316,38
10	320	100	160	50	25600	2500	-4,01	-316,38	316,40

- $F_{u,d} = 321\ kN \leq 352{,}6\ kN$

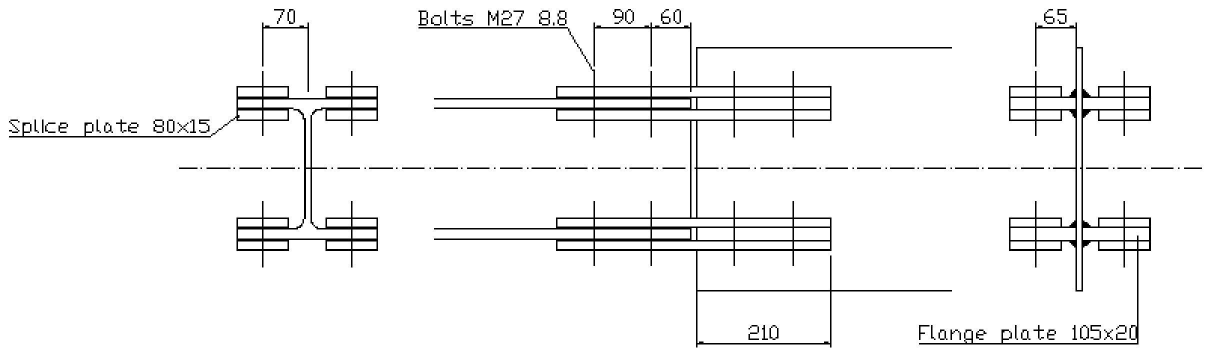
In the following part the resistances of the different general details are displayed. Splice plates are only used to connect the flanges. Therefore the resistance of the connection can be described with the use of flange forces. The resistance is calculated for: the flanges, the bolts, the splice plates and the flange plates. The resistance of the bolts is calculated in the same way as this was done for the splices in the chords. In this case only for detail H a reduction factor should be applied. The length of the connection is 720 mm. This results in a reduction factor of 0,975. The resistance of the flanges is based on the cross-sectional area of the flanges only ($A = 2xb_fxt_f$). For members in tension the net section is used. Per splice 8 splice plates and 4 flange plates are used.

In some cases cover plates are required to strengthen the section. This is only necessary for members in tension. When a cover plate is applied its thickness is based on the required net section. Thicknesses are in this case 8-12 mm. Thinner plates are not used because fillet welds with a throat thickness of 5 mm are applied to connect the cover plates to the flanges.

The force that is transferred by the cover plate is determined with the use of the net section. The total net section is the net section of the flange and the net section of the cover plate. The force is assumed to be evenly distributed over this net section which results in certain force in the cover plate which is equal to the total force in the net section times the net section of the cover plate divided by the total net section. The force obtained is the force that needs to be transferred through the cover plate. In order to do so this force should already be present in the cover plate at the location of the first bolt. The force is transferred by the 2 side welds with an effective length of L_{weld} as shown in Figure 4-22. The required length of L_{weld} can then be calculated which determines the total length of the cover plate.

For the splice in the bottom chord of truss 11 cover plates are required for both the flanges and the web. The net section of the cover plates (on the flanges) is: 370x10-2x39x10 = 2920 mm^2. The total net section is: 2920+394x27,7-2x39x27,7 = 11673 mm^2. The force transferred through the cover plate is therefore: 4545x2920/11673 = 1137 kN. This force should be transferred by the side welds. The length between the end and the first bolt is 460 mm. The total resistance of the side welds then becomes: $2xL_{weld}xaxf_{w,u,d}$ = 1147 kN. For all other cover plates similar calculations were performed and the corresponding lengths for which the welds just satisfy the requirements are shown in the figures.

DETAIL A1: HEB220 WITH FLANGE PLATES (LIGHT):



Bolts:

- Shear capacity: $F_{v,u,d}$ = 8x2x176,3 = 2821 kN
- Bearing capacity: $F_{b,u,d}$ = 8x15,81x16 = 2024 kN (Flanges are governing)

Flanges:

- $N_{c,u,d}$ = 2x220x16x460/1000 = 3238 kN
- $N_{t,u,d}$ = 0,9x(2x220x16-4x30x16)x540/(1000x1,25) = 1991 kN

Splice plates:

- $N_{c,u,d}$ = 8x80x15x460/1000 = 4416 kN
- $N_{t,u,d}$ = 0,9x(8x80x15-8x30x15)x540/(1000x1,25) = 2333 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((40-30/2)x20x540/(1000x1,25) + (150-30x1,5)x20x460/(√3x1000)) = 3095 kN
- $V_{eff,u,d,2}$ = 4x(210-2x30)x20x460/(√3x1000) = 3187 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(210x20x4))^2 + (65x1000x6/(4x210^{2}x20))^2) = 3043 kN

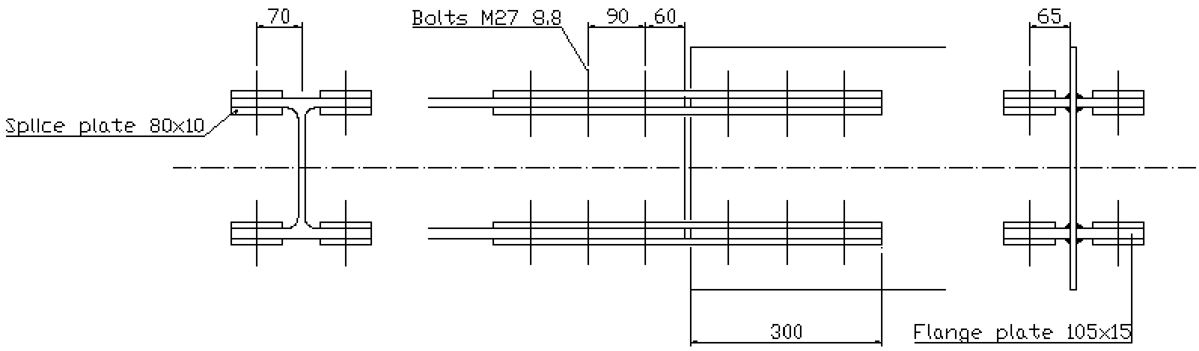
Capacity connection:

- $F_{c,u,d}$ = 2024 kN (Bearing capacity of the flanges)
- $F_{t,u,d}$ = 1991 kN (Fracture of the flanges)

Welds flange plates – gusset plate (a = 9):

- Shear stress = 2024x1000/(8x210x9) = 133,9 N/mm^2
- Bending stress = 65x2024x1000x3/(4x√2x210^{2}x9) = 175,8 N/mm^2
- Combined stress = √(4x175,8^2+3x133,9^2)/√3 = 243,1 N/mm^2 ≤ 249,4 N/mm^2

DETAIL A2: HEB220 WITH FLANGE PLATES (HEAVY):



<u>Bolts:</u>

- Shear capacity: $F_{v,u,d}$ = 12x2x176,3 = 4231 kN
- Bearing capacity: $F_{b,u,d}$ = 12x15,81x15 = 2846 kN (Flange plates are governing)

<u>Flanges:</u>

- $N_{c,u,d}$ = 2x220x16x460/1000 = 3238 kN
- $N_{t,u,d}$ = 0,9x(2x220x16-4x30x16)x540/(1000x1,25) = 1991 kN

<u>Splice plates:</u>

- $N_{c,u,d}$ = 8x80x10x460/1000 = 2944 kN
- $N_{t,u,d}$ = 0,9x(8x80x10-8x30x10)x540/(1000x1,25) = 1555 kN

<u>Flange plates:</u>

- $V_{eff,u,d,1}$ = 4x((40-30/2)x15x540/(1000x1,25) + (240-30x2,5)x15x460/(√3x1000)) = 3277 kN
- $V_{eff,u,d,2}$ = 4x(300-3x30)x15x460/(√3x1000) = 3346 kN
- $F_{V+M,u,d} = 460/\sqrt{(3x(1000/(4x300x15))^2 + (65x1000x6/(4x300^2x15))^2)}$ = 3823 kN

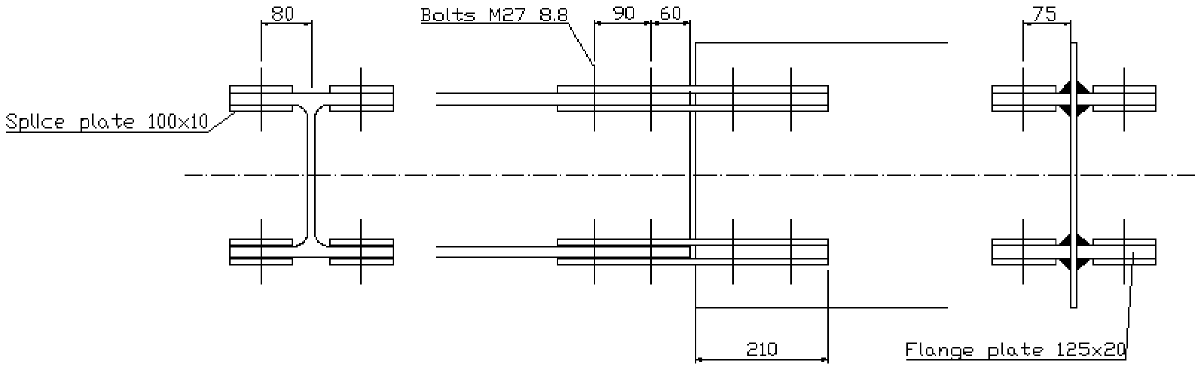
<u>Capacity connection:</u>

- $F_{c,u,d}$ = 2846 kN (Bearing capacity of the flange plates)
- $F_{t,u,d}$ = 1555 kN (Fracture of the splice plates)

<u>Welds flange plates – gusset plate (a = 7):</u>

- Shear stress = 2846x1000/(8x300x7) = 169,4 N/mm^2
- Bending stress = 65x2846x1000x3/(4x√2x300^{2}x7) = 155,7 N/mm^2
- Combined stress = $\sqrt{(4x155,7^2+3x169,4^2)}/\sqrt{3}$ = 247,0 N/mm^2 ≤ 249,4 N/mm^2

DETAIL B:HEB260 WITH FLANGE PLATES:



Bolts:

- Shear capacity: $F_{v,u,d}$ = 8x2x176,3 = 2821 kN
- Bearing capacity: $F_{b,u,d}$ = 8x19,44x17,5 = 2722 kN (Flanges are governing)

Flanges:

- $N_{c,u,d}$ = 2x260x17,5x460/1000 = 4186 kN
- $N_{t,u,d}$ = 0,9x(2x260x17,5-4x30x17,5)x540/(1000x1,25) = 2722 kN

Splice plates:

- $N_{c,u,d}$ = 8x100x10x460/1000 = 3680 kN
- $N_{t,u,d}$ = 0,9x(8x100x10-8x30x10)x540/(1000x1,25) = 2177 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((50-30/2)x20x540/(1000x1,25) + (150-30x1,5)x20x460/(√3x1000)) = 3440 kN
- $V_{eff,u,d,2}$ = 4x(210-2x30)x20x460/(√3x1000) = 3187 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x210x20))^2 + (75x1000x6/(4x210^{2}x20))^2) = 2805 kN

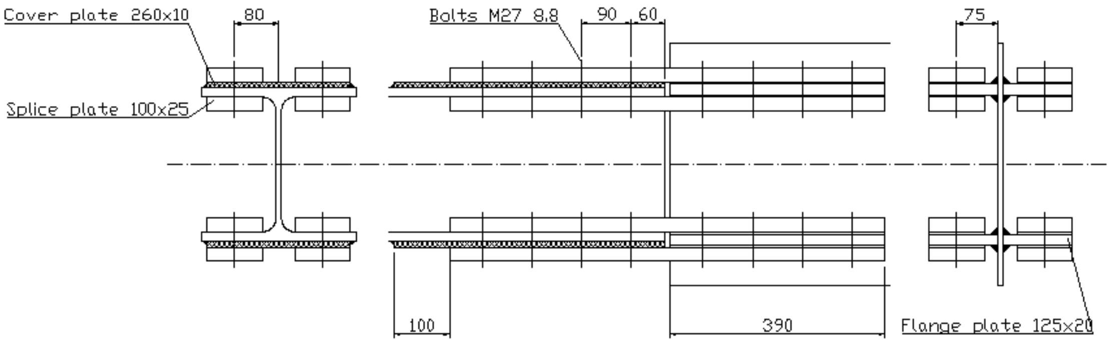
Capacity connection:

- $F_{c,u,d}$ = 2722 kN (Bearing capacity of the flanges)
- $F_{t,u,d}$ = 2177 kN (Fracture of the splice plates)

Welds flange plates – gusset plate (a = 14):

- Shear stress = 2722x1000/(8x210x14) = 115,7 N/mm^2
- Bending stress = 75x2722x1000x3/(4x√2x210^{2}x14) = 175,3 N/mm^2
- Combined stress = √(4x175,3^2+3x115,7^2)/√3 = 233,2 N/mm^2 ≤ 249,4 N/mm^2

DETAIL C: HEB280 WITH FLANGE PLATES:



Bolts:

- Shear capacity: $F_{v,u,d}$ = 16x2x176,3 = 5642 kN
- Bearing capacity: $F_{b,u,d}$ = 16x19,44x20 = 6221 kN (Flange plates are governing)

Flanges + cover plates:

- $N_{c,u,d}$ = 2x(280x18+260x10)x460/1000 = 7029 kN
- $N_{t,u,d}$ = 0,9x(2x(280x18+260x10)-4x30x28)x540/(1000x1,25) = 4634 kN

Splice plates:

- $N_{c,u,d}$ = 8x100x25x460/1000 = 9200 kN
- $N_{t,u,d}$ = 0,9x(8x100x25-8x30x25)x540/(1000x1,25) = 5443 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((50-30/2)x20x540/(1000x1,25) + (330-30x3,5)x20x460/(√3x1000)) = 5990 kN
- $V_{eff,u,d,2}$ = 4x(390-4x30)x20x460/(√3x1000) = 5737 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x390x20))^2 + (75x1000x6/(4x390^{2}x20))^2) = 6896 kN

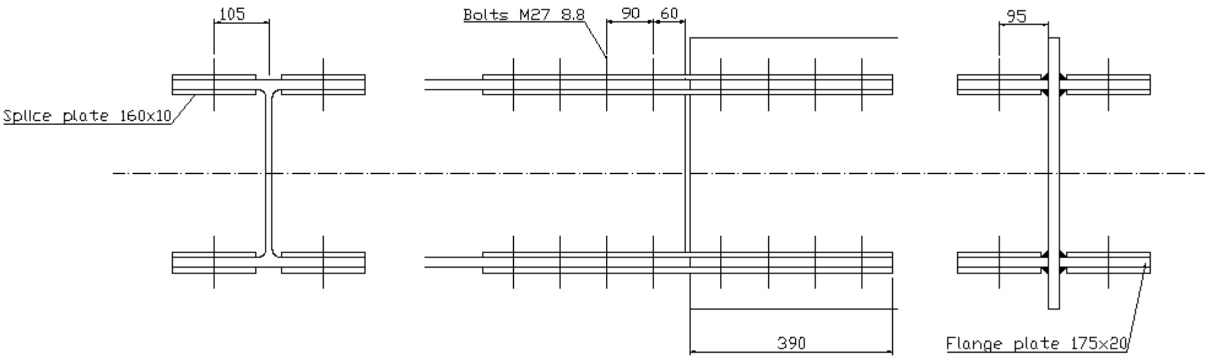
Capacity connection:

- $F_{c,u,d}$ = 5642 kN (Shear capacity of the bolts)
- $F_{t,u,d}$ = 4634 kN (Fracture of the flanges + cover plates)

Welds flange plates – gusset plate (a = 10):

- Shear stress = 5642x1000/(8x390x10) = 180,8 N/mm^2
- Bending stress = 75x5642x1000x3/(4x√2x390^{2}x10) = 147,5 N/mm^2
- Combined stress = √(4x147,5^2+3x180,8^2)/√3 = 248,4 N/mm^2 ≤ 249,4 N/mm^2

DETAIL D1: HD360X147 WITH FLANGE PLATES (LIGHT):



Bolts:

- Shear capacity: $F_{v,u,d}$ = 16x2x176,3 = 5642 kN
- Bearing capacity: $F_{b,u,d}$ = 16x19,44x19,8 = 6159 kN (Flanges are governing)

Flanges:

- $N_{c,u,d}$ = 2x370x19,8x460/1000 = 6740 kN
- $N_{t,u,d}$ = 0,9x(2x370x19,8-4x30x19,8)x540/(1000x1,25) = 4773 kN

Splice plates:

- $N_{c,u,d}$ = 8x160x10x460/1000 = 5888 kN
- $N_{t,u,d}$ = 0,9x(8x160x10-8x30x10)x540/(1000x1,25) = 4044 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((80-30/2)x20x540/(1000x1,25) + (330-30x3,5)x20x460/(√3x1000)) = 7027 kN
- $V_{eff,u,d,2}$ = 4x(390-4x30)x20x460/(√3x1000) = 5737 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x390x20))^2 + (95x1000x6/(4x390^{2}x20))^2) = 6333 kN

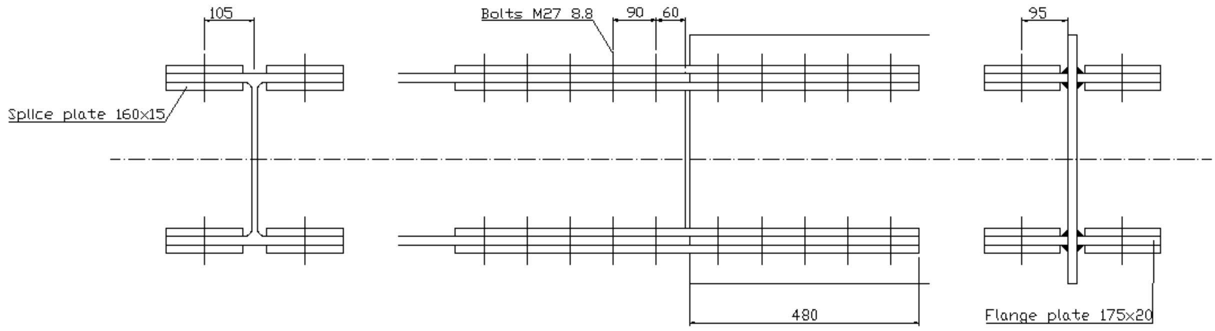
Capacity connection:

- $F_{c,u,d}$ = 5642 kN (Shear capacity of the bolts)
- $F_{t,u,d}$ = 4044 kN (Fracture of the splice plates)

Welds flange plates – gusset plate (a = 12):

- Shear stress = 5642x1000/(8x390x12) = 150,7 N/mm^2
- Bending stress = 95x5642x1000x3/(4x√2x390^{2}x12) = 155,7 N/mm^2
- Combined stress = √(4x155,7^2+3x150,7^2)/√3 = 234,6 N/mm^2 ≤ 249,4 N/mm^2

DETAIL D2: HD360X147 WITH FLANGE PLATES (HEAVY):



Bolts:

- Shear capacity: $F_{v,u,d}$ = 20x2x176,3 = 7052 kN
- Bearing capacity: $F_{b,u,d}$ = 20x19,44x19,8 = 7698 kN (Flanges are governing)

Flanges:

- $N_{c,u,d}$ = 2x370x19,8x460/1000 = 6740 kN
- $N_{t,u,d}$ = 0,9x(2x370x19,8-4x30x19,8)x540/(1000x1,25) = 4773 kN

Splice plates:

- $N_{c,u,d}$ = 8x160x15x460/1000 = 8832 kN
- $N_{t,u,d}$ = 0,9x(8x160x15-8x30x15)x540/(1000x1,25) = 6065 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((80-30/2)x20x540/(1000x1,25) + (420-30x4,5)x20x460/(√3x1000)) = 8302 kN
- $V_{eff,u,d,2}$ = 4x(480-5x30)x20x460/(√3x1000) = 7011 kN
- $F_{V+M,u,d} = 460/\sqrt{(3x(1000/(4x480x20))^2 + (95x1000x6/(4x480^2x20))^2)}$ = 8411 kN

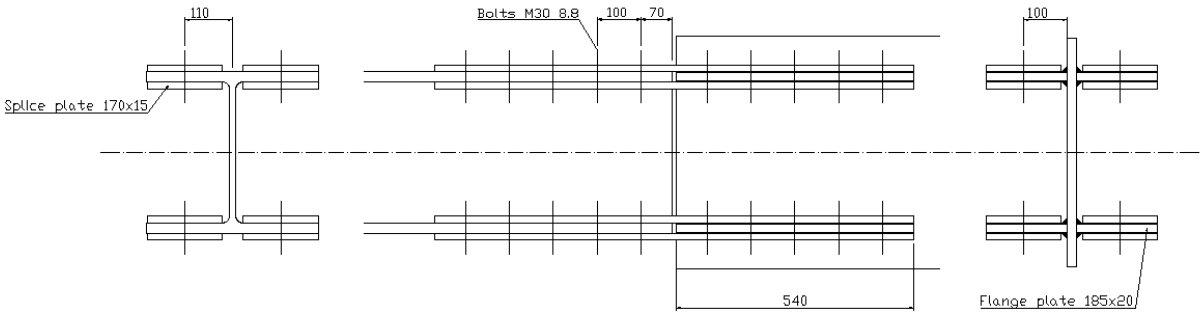
Capacity connection:

- $F_{c,u,d}$ = 6740 kN (Yielding of the flanges)
- $F_{t,u,d}$ = 4773 kN (Fracture of the flanges)

Welds flange plates – gusset plate (a = 10):

- Shear stress = 6740x1000/(8x480x10) = 175,5 N/mm^2
- Bending stress = 95x6740x1000x3/(4x√2x480^{2}x10) = 147,4 N/mm^2
- Combined stress = √(4x147,4^2+3x175,5^2)/√3 = 244,5 N/mm^2 ≤ 249,4 N/mm^2

DETAIL E1: HD400X187 WITH FLANGE PLATES (LIGHT):



Bolts:

- Shear capacity: $F_{v,u,d}$ = 20x2x215,4 = 8616 kN
- Bearing capacity: $F_{b,u,d}$ = 20x22,91x20 = 9164 kN (Flange plates are governing)

Flanges:

- $N_{c,u,d}$ = 2x391x24x460/1000 = 8633 kN
- $N_{t,u,d}$ = 0,9x(2x391x24-4x33x24)x540/(1000x1,25) = 6065 kN

Splice plates:

- $N_{c,u,d}$ = 8x170x15x460/1000 = 9384 kN
- $N_{t,u,d}$ = 0,9x(8x170x15-8x33x15)x540/(1000x1,25) = 6392 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((85-33/2)x20x540/(1000x1,25) + (470-33x4,5)x20x460/(√3x1000)) = 9198 kN
- $V_{eff,u,d,2}$ = 4x(540-5x33)x20x460/(√3x1000) = 7967 kN
- $F_{V+M,u,d} = 460/\sqrt{(3x(1000/(4x540x20))^2 + (100x1000x6/(4x540^2x20))^2)}$ = 9657 kN

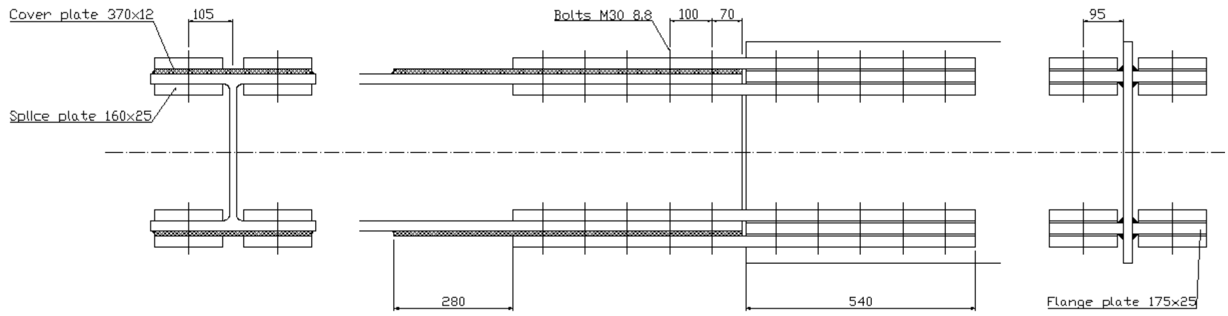
Capacity connection:

- $F_{c,u,d}$ = 7967 kN (Shear capacity of the flange plates)
- $F_{t,u,d}$ = 6065 kN (Fracture of the flanges)

Welds flange plates – gusset plate (a = 10):

- Shear stress = 7967x1000/(8x540x10) = 184,4 N/mm^2
- Bending stress = 100x7967x1000x3/(4x√2x540^{2}x10) = 144,9 N/mm^2
- Combined stress = $\sqrt{(4x144,9^2+3x184,4^2)}/\sqrt{3}$ = 249,0 N/mm^2 ≤ 249,4 N/mm^2

DETAIL E2: HD400X187 WITH FLANGE PLATES (HEAVY):



Bolts:

- Shear capacity: $F_{v,u,d}$ = 20x2x215,4 = 8616 kN
- Bearing capacity: $F_{b,u,d}$ = 20x22,91x25 = 11455 kN (Flange plates are governing)

Flanges + cover plates:

- $N_{c,u,d}$ = 2x(391x24+370x12)x460/1000 = 12718 kN
- $N_{t,u,d}$ = 0,9x(2x(391x24+370x12)-4x33x36)x540/(1000x1,25) = 8902 kN

Splice plates:

- $N_{c,u,d}$ = 8x160x25x460/1000 =14720 kN
- $N_{t,u,d}$ = 0,9x(8x160x25-8x33x25)x540/(1000x1,25) = 9876 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((80-33/2)x25x540/(1000x1,25) + (470-33x4,5)x25x460/(√3x1000)) = 11282 kN
- $V_{eff,u,d,2}$ = 4x(540-5x33)x25x460/(√3x1000) = 9959 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x540x25))^2 + (95x1000x6/(4x540^{2}x25))^2) = 12246 kN

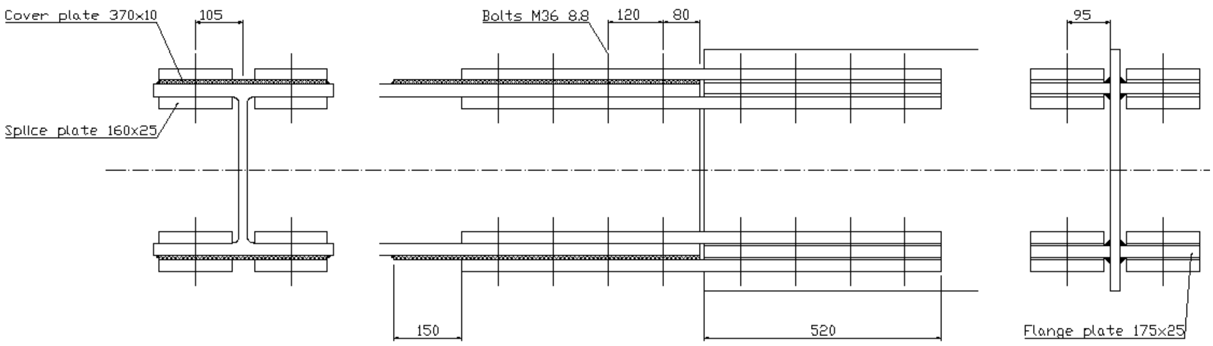
Capacity connection:

- $F_{c,u,d}$ = 8616 kN (Shear capacity of the bolts)
- $F_{t,u,d}$ = 8616 kN (Shear capacity of the bolts)

Welds flange plates – gusset plate (a = 11):

- Shear stress = 8616x1000/(8x540x11) = 181,3 N/mm^2
- Bending stress = 95x8616x1000x3/(4x√2x540^{2}x11) = 135,3 N/mm^2
- Combined stress = √(4x135,3^2+3x181,3^2)/√3 = 249,0 N/mm^2 ≤ 239,4 N/mm^2

DETAIL F: HD400X216 WITH FLANGE PLATES:



Bolts:

- Shear capacity: $F_{v,u,d}$ = 16x2x313,7 = 10038 kN
- Bearing capacity: $F_{b,u,d}$ = 16x26,58x25 = 10634 kN (Flange plates are governing)

Flanges + cover plates:

- $N_{c,u,d}$ = 2x(394x27,7+370x10)x460/1000 = 13445 kN
- $N_{t,u,d}$ = 0,9x(2x(394x27,7+370x10)-4x39x37,7)x540/(1000x1,25) = 9077 kN

Splice plates:

- $N_{c,u,d}$ = 8x160x25x460/1000 =14720 kN
- $N_{t,u,d}$ = 0,9x(8x160x25-8x39x25)x540/(1000x1,25) = 9409 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((80-39/2)x25x540/(1000x1,25) + (440-39x3,5)x25x460/(√3x1000)) = 10674 kN
- $V_{eff,u,d,2}$ = 4x(520-4x39)x25x460/(√3x1000) = 9667 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x520x25))^2 + (95x1000x6/(4x520^{2}x25))^2) = 11670 kN

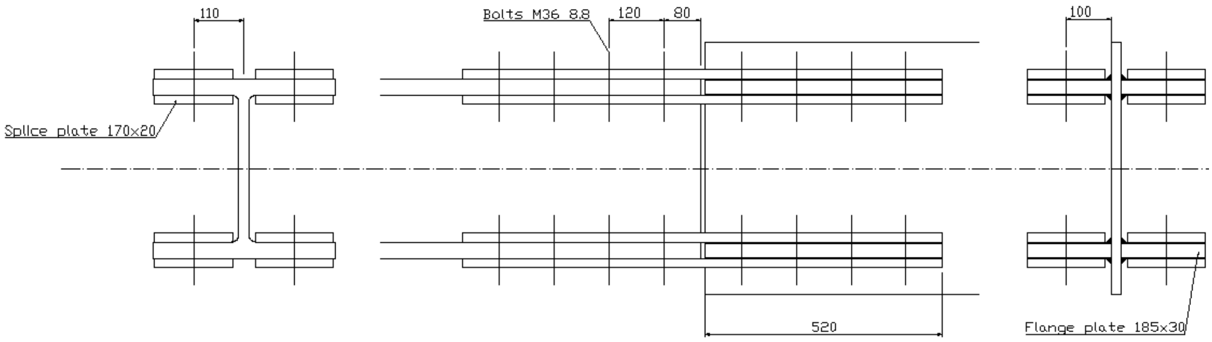
Capacity connection:

- $F_{c,u,d}$ = 9667 kN (Shear capacity of the flange plates)
- $F_{t,u,d}$ = 9077 kN (Fracture of the flanges + cover plates)

Welds flange plates – gusset plate (a = 13):

- Shear stress = 9667x1000/(8x520x13) = 178,8 N/mm^2
- Bending stress = 95x9667x1000x3/(4x√2x520^{2}x13) = 138,6 N/mm^2
- Combined stress = √(4x138,6^2+3x178,8^2)/√3 = 239,9 N/mm^2 ≤ 239,4 N/mm^2

DETAIL G: HD400X287 WITH FLANGE PLATES:



Bolts:

- Shear capacity: $F_{v,u,d}$ = 16x2x313,7 = 10038 kN
- Bearing capacity: $F_{b,u,d}$ = 16x26,58x30 = 12761 kN (Flange plates are governing)

Flanges:

- $N_{c,u,d}$ = 2x393x36,6x460/1000 = 13233 kN
- $N_{t,u,d}$ = 0,9x(2x393x36,6-4x39x36,6)x540/(1000x1,25) = 8965 kN

Splice plates:

- $N_{c,u,d}$ = 8x170x20x460/1000 = 12512 kN
- $N_{t,u,d}$ = 0,9x(8x170x20-8x39x20)x540/(1000x1,25) = 8149 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((80-39/2)x30x540/(1000x1,25) + (440-39x3,5)x30x460/(√3x1000)) = 12809 kN
- $V_{eff,u,d,2}$ = 4x(520-4x39)x30x460/(√3x1000) = 11601 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x520x30))^2 + (105x1000x6/(4x520^{2}x30))^2) = 13580 kN

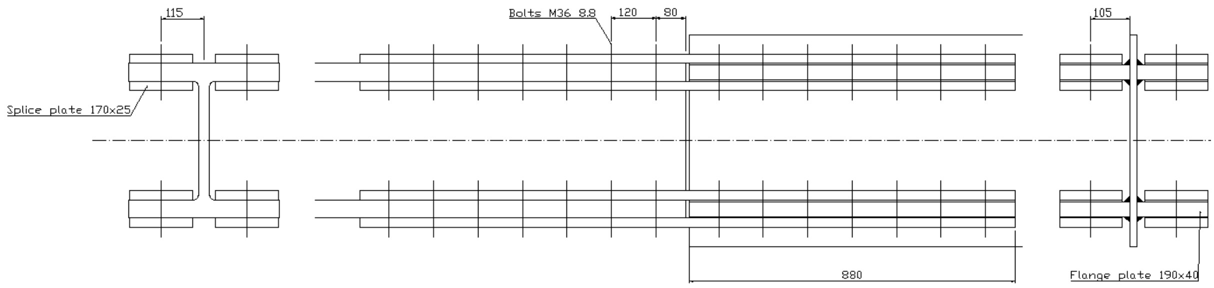
Capacity connection:

- $F_{c,u,d}$ = 10038 kN (Shear capacity of the bolts)
- $F_{t,u,d}$ = 8149 kN (Fracture of the splice plates)

Welds flange plates – gusset plate (a = 14):

- Shear stress = 10038x1000/(8x520x14) = 172,4 N/mm^2
- Bending stress = 105x10038x1000x3/(4x√2x520^{2}x14) = 147,7 N/mm^2
- Combined stress = √(4x147,7^2+3x172,4^2)/√3 = 242,4 N/mm^2 ≤ 249,4 N/mm^2

DETAIL H: HD400X382 WITH FLANGE PLATES:



Bolts:

- Shear capacity: $F_{v,u,d}$ = 28x2x313,7x0,975 = 17128 kN
- Bearing capacity: $F_{b,u,d}$ = 28x26,58x40 = 29775 kN (Flange plates are governing)

Flanges:

- $N_{c,u,d}$ = 2x406x48x460/1000 = 17929 kN
- $N_{t,u,d}$ = 0,9x(2x406x48-4x39x48)x540/(1000x1,25) = 12243 kN

Splice plates:

- $N_{c,u,d}$ = 8x170x25x460/1000 = 15640 kN
- $N_{t,u,d}$ = 0,9x(8x170x25-8x39x25)x540/(1000x1,25) = 10187 kN

Flange plates:

- $V_{eff,u,d,1}$ = 4x((85-39/2)x40x540/(1000x1,25) + (800-39x6,5)x40x460/(√3x1000)) = 27750 kN
- $V_{eff,u,d,2}$ = 4x(880-7x39)x40x460/(√3x1000) = 25793 kN
- $F_{V+M,u,d}$ = 460/√(3x(1000/(4x880x40))^2 + (105x1000x6/(4x880^{2}x40))^2) = 34558 kN

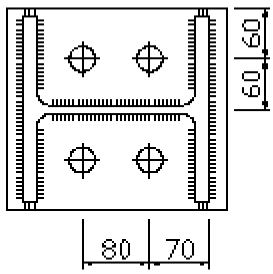
Capacity connection:

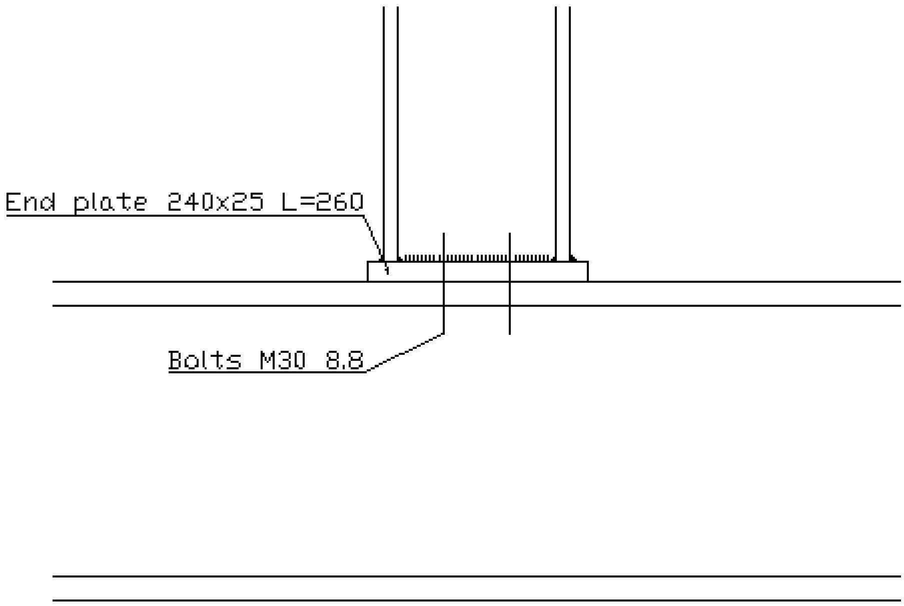
- $F_{c,u,d}$ = 15640 kN (Yielding of the splice plates)
- $F_{t,u,d}$ = 10187 kN (Fracture of the splice plates)

Welds flange plates – gusset plate (a = 11):

- Shear stress = 15640x1000/(8x880x11) = 202,0 N/mm^2
- Bending stress = 105x15640x1000x3/(4x√2x880^{2}x11) = 102,2 N/mm^2
- Combined stress = √(4x102,2^2+3x202,0^2)/√3 = 233,9 N/mm^2 ≤ 249,4 N/mm^2

DETAIL I1: HEB220 WITH END PLATES (LIGHT):





Welds section – end plate (a= 5 for flanges and web):

- $F_{t,u,d}$ = (2x(2x220-9,5-2x18)x5 + 2x152x5)x249,4x√3/(√2x1000) = 1669 kN

Bolts: ($F_{t,u,d}$ = 561x0,9x800/(1,25x1000) = 323 kN)

T-section end plate:

- m1 = 60-9,5/2-0,8x5x√2 = 50 mm　　m2 = 70-16-0,8x5x√2 = 48 mm
- e2 = n = 60 mm　　p = 80 mm
- λ1 = 50/(50+60) = 0,453　　λ2 = 48/(50+60) = 0,441 → α = 6
- l_{eff1} = 2xπx50 = 312 mm　　l_{eff2} = πx50+80 = 236 mm
- l_{eff3} = 6x50 = 298 mm　　l_{eff4} = 0,5x80+(6-2)x50-0,625x60) = 201 mm
- $M_{pl,rd}$ = 201x25^{2}x460/4000000 = 14,4 kNm　　β = 4x14,4x1000000/(50x2x323x1000) = 1,80
- γ = 60/50 = 1,21 → Failure mode 2
- $F_{t,u,d}$ = 2x((2x14,4x1000 + 60x2x323)/(50+60)) = 1235 kN

T-section flange chord:

- $m = 60–17{,}3/2-15 = 39$ mm — $e = (394-120)/2 = 137$ mm
- $n = 1{,}25x39 = 49$ mm — $p = 80$ mm
- $l_{eff1} = 2x\pi x39 = 245$ mm — $l_{eff2} = 4x39+1{,}25x137 = 327$ mm
- $l_{eff3} = \pi x39+80 = 203$ mm — $l_{eff4} = 2x39+0{,}625x137+0{,}5x80 = 204$ mm
- $M_{pl,rd} = 203x27{,}7^2x460/4000000 = 17{,}9$ kNm — $\beta = 4x17{,}9x1000000/(39x2x323x1000) = 2{,}84$
- Failure mode 3 → $F_{t,u,d} = 4x323 = 1292$ kN

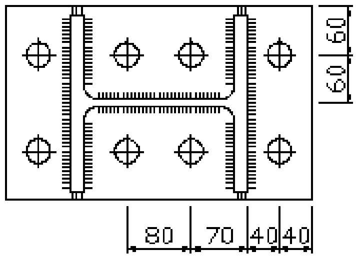
Bearing chord:

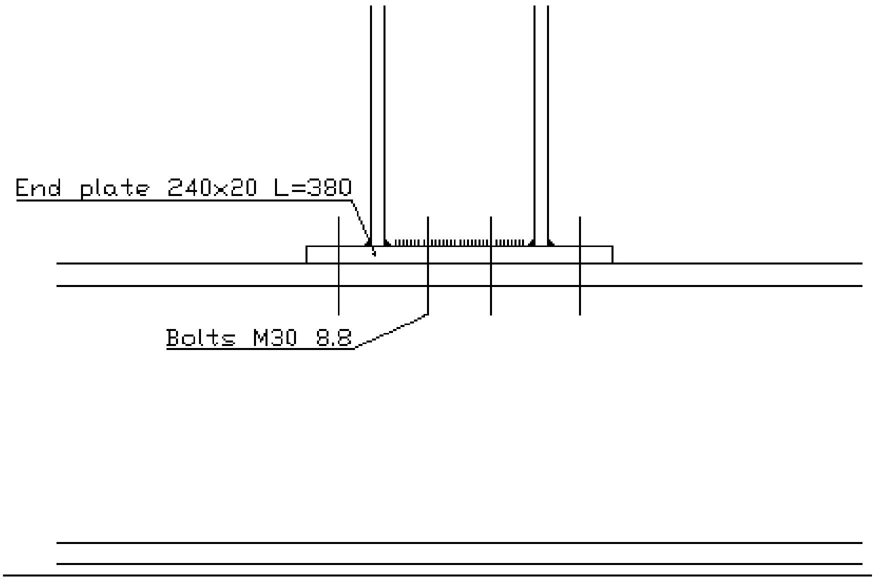
- $F_{c,u,d} = (260+2x20+2x27{,}7)x17{,}3x460/1000 = 2828$ kN (HD400x216)
- $F_{c,u,d} = (260+2x20+2x48)x29{,}8x460/1000 = 5428$ kN (HD400x382)

Capacity connection:

- $F_{c,u,d} = 2828$ kN (Yielding of the chords web)
- $F_{t,u,d} = 1235$ kN (Mode 2 failure of the end plate)

DETAIL I2: HEB220 WITH END PLATES (HEAVY):





Welds section – end plate (a= 7 for flanges and a = 5 for web):

- $F_{t,u,d}$ = (2x(2x220-9,5-2x18)x7 + 2x152x5)x249,4x√3/(√2x1000) = 2151kN

Bolts: ($F_{t,u,d}$ = 561x0,9x800/(1,25x1000) = 323 kN)

T-section end plate (inside section):

- m1 = 60-9,5/2-0,8x5x√2 = 50 mm m2 = 70-16-0,8x7x√2 = 46 mm
- e2 = n = 60 mm p = 80 mm
- λ1 = 50/(50+60) = 0,453 λ2 = 46/(50+60) = 0,420 → α = 6
- l_{eff1} = 2xπx50 = 312 mm l_{eff2} = πx50+80 = 236 mm
- l_{eff3} = 6x50 = 298 mm l_{eff4} = 0,5x80+(6-2)x50-0,625x60) = 201 mm
- $M_{pl,rd}$ = 201x20^{2}x460/4000000 = 9,2 kNm β = 4x9,2x1000000/(50x2x323x1000) = 1,15
- γ = 60/50 = 1,21 → Failure mode 2
- $F_{t,u,d}$ = 2x((2x9,2x1000 + 60x2x323)/(50+60)) = 1039 kN

T-section end plate (outside section):

- $m = (380\text{-}220)/4\text{-}0,8x7x\sqrt{2} = 32$ mm $e1 = 40$ mm
- $e2 = 60$ mm $n = 1,25x32 = 40$ mm
- $l_{eff1} = 2x\pi x32 = 202$ mm $l_{eff2} = \pi x32+120 = 221$ mm
- $l_{eff3} = \pi x32+2x60 = 221$ mm $l_{eff4} = 4x32+1,25x40 = 178$ mm
- $l_{eff5} = 60+2x32+0,625x40 = 149$ mm $l_{eff6} = 0,5x240 = 120$ mm
- $l_{eff7} = 0,5x120+2x32+0,625x40 = 149$ mm $M_{pl,rd} = 120x20^2x460/4000000 = 5,5$ kNm
- $\beta = 4x5,5x1000000/(32x2x323x1000) = 1,06$ $\gamma = 40/32 = 1,25$ → Failure mode 2
- $F_{t,u,d} = 2x((2x5,5x1000 + 40x2x323)/(32+40)) =$ 1023 kN

T-section flange chord (inner bolts):

- $m = 60–17,3/2\text{-}15 = 39$ mm $e = (394\text{-}120)/2 = 137$ mm
- $n = 1,25x39 = 49$ mm $p = 80$ mm
- $l_{eff1} = 2x\pi x39 = 245$ mm $l_{eff2} = 4x39+1,25x137 = 327$ mm
- $l_{eff3} = 2x80 = 160$ mm $l_{eff4} = 80$ mm
- $M_{pl,rd} = 80x27,7^2x460/4000000 = 7,1$ kNm $\beta = 4x7,1x1000000/(39x2x323x1000) = 1,12$
- $\gamma = 49/39 = 1,26$ → Failure mode 2
- $F_{t,u,d} = 2x((2x7,1x1000 + 49x2x323)/(39+49)) =$ 1042 kN

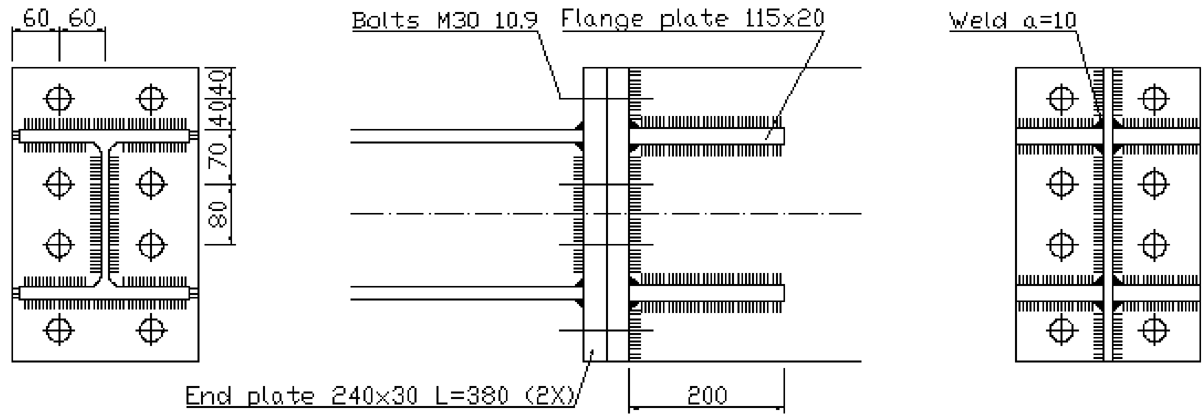
T-section flange chord (outer bolts):

- $m = 60–17,3/2\text{-}15 = 39$ mm $e = (394\text{-}120)/2 = 137$ mm
- $n = 1,25x39 = 49$ mm $p = 110$ mm
- $l_{eff1} = 2x\pi x39 = 245$ mm $l_{eff2} = 4x39+1,25x137 = 327$ mm
- $l_{eff3} = \pi x39+110 = 233$ mm $l_{eff4} = 2x39+0,625x137+0,5x110 = 219$ mm
- $M_{pl,rd} = 219x27,7^2x460/4000000 = 19,3$ kNm $\beta = 4x19,3x1000000/(39x2x323x1000) = 3,06$
- Failure mode 3 → $F_{t,u,d} = 4x323 = 1292$ kN

Capacity connection:

- $F_{c,u,d} = 2828$ kN (Yielding of the chords web)
- $F_{t,u,d} = 2062$ kN (Mode 2 failure of the end plate)

DETAIL I3: HEB220 WITH END PLATES (GUSSET PLATE):



Welds section – end plate (a= 9 for flanges and for web):

- $F_{t,u,d}$ = (2x(2x220-9,5-2x18)x9 + 2x152x9)x249,4x√3/(√2x1000) = 3005 kN

Bolts: ($F_{t,u,d}$ = 561x0,9x1000/(1,25x1000) = 404 kN)

T-section end plate (inside section):

- m1 = 60-9,5/2-0,8x9x√2 = 45 mm m2 = 70-16-0,8x9x√2 = 44 mm
- e2 = 60 mm n = 1,25x45 = 56 mm p = 80 mm
- λ1 = 45/(45+56) = 0,444 λ2 = 44/(45+56) = 0,432 → α = 6
- l_{eff1} = 2xπx45 = 283 mm l_{eff2} = πx45+80 = 222 mm
- l_{eff3} = 6x45 = 270 mm l_{eff4} = 0,5x80+(6-2)x45-0,625x60) = 183 mm
- $M_{pl,rd}$ = 183x30^{2}x460/4000000 = 18,9 kNm β = 4x18,9x1000000/(45x2x404x1000) = 2,08
- Failure mode 3 → $F_{t,u,d}$ = 4x404 = 1616 kN

T-section end plate (outside section):

- m = (380-220)/4-0,8x9x√2 = 30 mm e1 = 40 mm
- e2 = 60 mm n = 1,25x30 = 37 mm
- l_{eff1} = 2xπx30 = 187 mm l_{eff2} = πx30+120 = 214 mm
- l_{eff3} = πx30+2x60 = 214 mm l_{eff4} = 4x30+1,25x40 = 169 mm
- l_{eff5} = 60+2x30+0,625x40 = 145 mm l_{eff6} = 0,5x240 = 120 mm
- l_{eff7} = 0,5x120+2x30+0,625x40 = 145 mm
- $M_{pl,rd}$ = 120x30^{2}x460/4000000 = 12,4 kNm β = 4x12,4x1000000/(29x2x404x1000) = 2,06
- Failure mode 3 → $F_{t,u,d}$ = 4x404 = 1616 kN

T-section end plate (between flange plates):

- $m1 = 60\text{-}10/2\text{-}0{,}8x9x\sqrt{2} = 45$ mm $\quad$ $m2 = 70\text{-}20\text{-}0{,}8x9x\sqrt{2} = 40$ mm
- $e2 = 60$ mm $\quad$ $n = 1{,}25x45 = 56$ mm $\quad$ $p = 80$ mm
- $\lambda1 = 45/(45+56) = 0{,}444$ $\quad$ $\lambda2 = 40/(45+56) = 0{,}395 \rightarrow \alpha = 2x\pi$
- $l_{eff1} = 2x\pi x45 = 282$ mm $\quad$ $l_{eff2} = \pi x45+80 = 221$ mm
- $l_{eff3} = 2x\pi x45 = 282$ mm $\quad$ $l_{eff4} = 0{,}5x80+(2x\pi\text{-}2)x45\text{-}0{,}625x60) = 194$ mm
- $M_{pl,rd} = 194x30^2x460/4000000 = 20{,}1$ kNm $\quad$ $\beta = 4x20{,}1x1000000/(45x2x404x1000) = 2{,}22$
- Failure mode 3 $\rightarrow F_{t,u,d} = 4x404 = 1616$ kN

T-section end plate (outside flange plates):

Similar to T-section end plate (between flange plates) but with smaller m1 so:

- Failure mode 3 $\rightarrow F_{t,u,d} = 4x404 = 1616$ kN

Flange plates:

- $F_{V+M,u,d} = 460/\sqrt{(3x(1000/(4x200x20))^2 + (52{,}5x1000x6/(4x200^2x20))^2)} = 3838$ kN

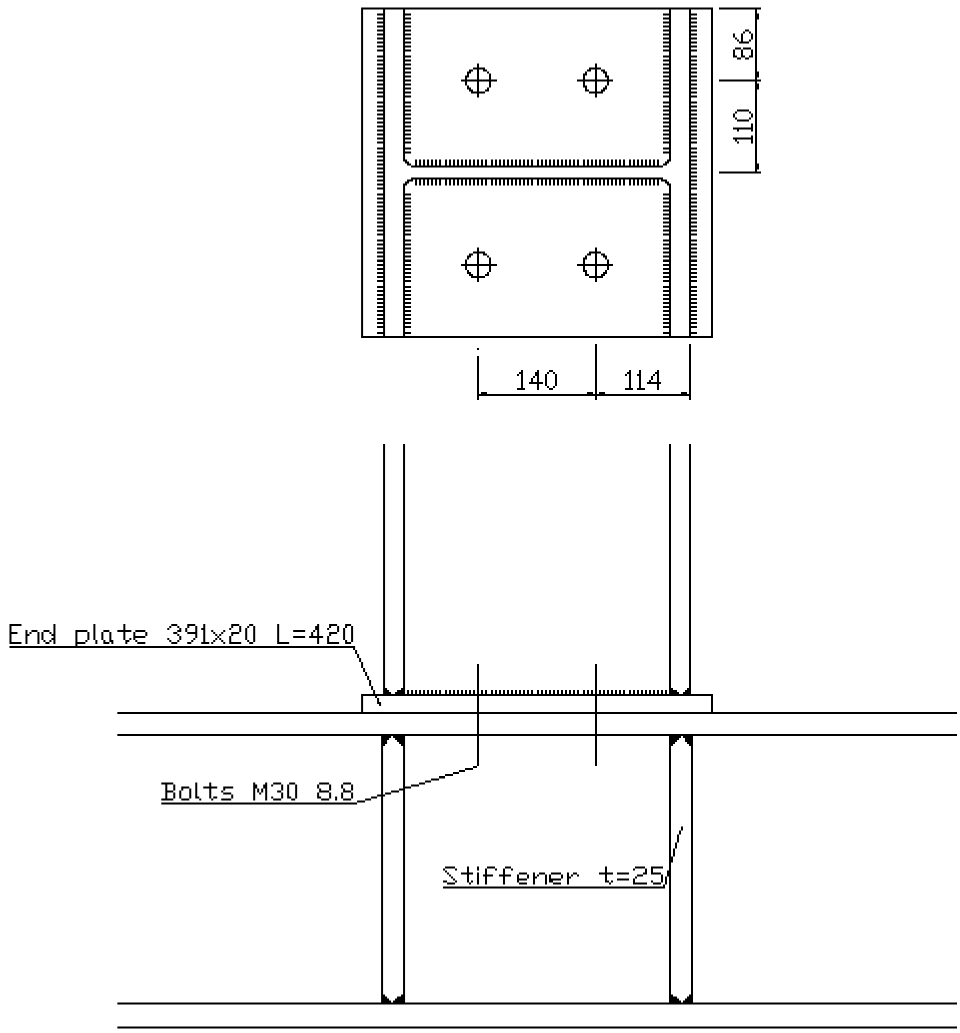
Capacity connection:

- $F_{c,u,d}$ = N/A
- $F_{t,u,d} = 3005$ kN (Failure of the welds)

Welds flange plates – gusset plate (a = 10):

- Shear stress = $3005x1000/(8x200x11) = 145{,}3$ N/mm^2
- Bending stress = $52{,}5x3005x1000x3/(4x\sqrt{2}x200^2x10) = 161{,}8$ N/mm^2
- Combined stress = $\sqrt{(4x161{,}8^2+3x145{,}3^2)}/\sqrt{3} = 236{,}7$ N/mm^2 $\leq 249{,}4$ N/mm^2

DETAIL J: HD400x187 WITH END PLATES:



Welds flanges and web:

- Stress in section = 7111x1000/23760 = 299,3 N/mm^2
- v = 1,1x299,3x24/(460x2) = 9 mm → 11 mm (flanges)
- v = 1,1x299,3x15/(460x2) = 6 mm → 8 mm (web)

Unstiffened resistance:

- $F_{r,d}$ = (368+2x20+2x27,7)x17,3x460/1000 = 3685 kN

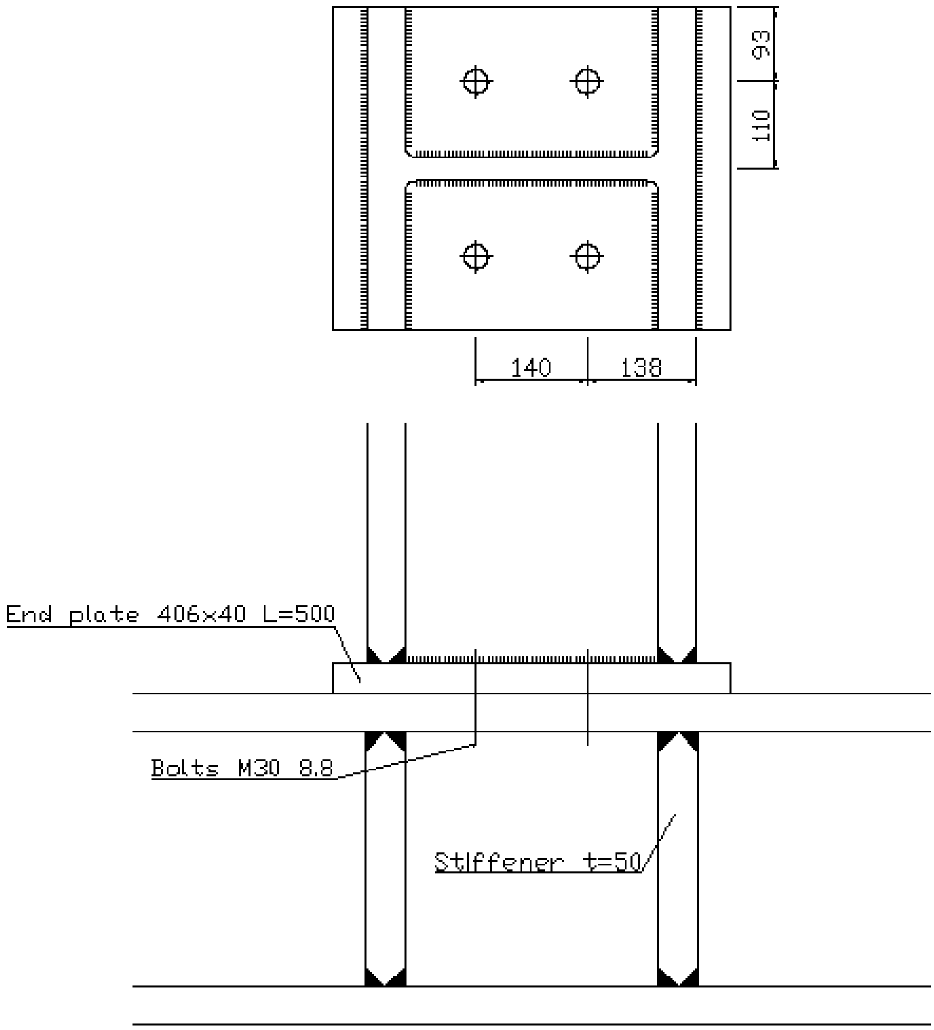
Stiffeners:

- Stress in stiffener = 7808x1000/(2x25x394) = 396,3 N/mm^2
- v = 1,1x396,3x25/(460x2) = 12 mm → 14 mm

Capacity stiffeners:

- $F_{r,d}$ = 2x394x25x460/1000 = 9062 kN

DETAIL K: HD400x382 WITH END PLATES:



Welds flanges and web:

- Stress in section = 15039x1000/48710 = 308,7 N/mm^2
- v = 1,1x308,7x48/(460x2) = 18 mm → 20 mm (flanges)
- v = 1,1x308,7x29,8/(460x2) = 11 mm → 13 mm (web)

Unstiffened resistance:

- $F_{r,d}$ = (416+2x40+2x48)x29,8x460/1000 = 8115 kN

Stiffeners:

- Stress in stiffener = 16151x1000/(2x50x406) = 397,8 N/mm^2
- v = 1x397,8x50/(430x2) = 23 mm → 25 mm

Capacity stiffeners:

- $F_{r,d}$ = 2x406x50x430/1000 = 17458 kN

In the next part the gusset plates are checked for the connection forces and the forces of the individual members. Different failure modes can be governing (Figure 4-24) dependent on the magnitude of the occurring forces and the shape of the gusset plates. When the forces in the connected members are large it is possible that only a part of the gusset plate fails. This could be prevented by changing the shape of the gusset plate. This was however not permitted by the designer of the building so the only parameter that can be changed is the thickness of the gusset plates. At the location of the welds a combination of axial force, shear force and bending moment can occur. The bending moment occurs due to the eccentricity of the connection. The center of the gusset plate is at a horizontal distance (ex) and a vertical distance (ez) from the center of the connection. The center of the connection is the point where all the system lines of the individual members intersect. The bending moment is equal to: Rx x ez + Rz x ex. For truss 10 the reaction forces and the corresponding bending moments are shown in Table 8-3. For truss 11 they are shown in Table 8-4.

Node	Members	Rx	Rz	ex	ez	M
		[kN]	[kN]	[mm]	[mm]	[kNm]
Node 1	S1, S8	-1011	-3426	-171	208	376
Node 3	S3, S9, S10	9949	253	110,5	208	2097
Node 5	S5, S11	-3871	339	81,5	208	778
Node 6	S6, S12, S13	-6863	1153	46,5	208	1374
Node 10	S2, S8, S9	-7279	1540	-53,5	-187,5	1282
Node 12	S4, S10, S11	-934	543	-2,5	-187,5	174
Node 13	S5, S12	5533	990	-177,5	-187,5	1213
Node 15	S7, S13	1509	556	166,5	-187,5	190

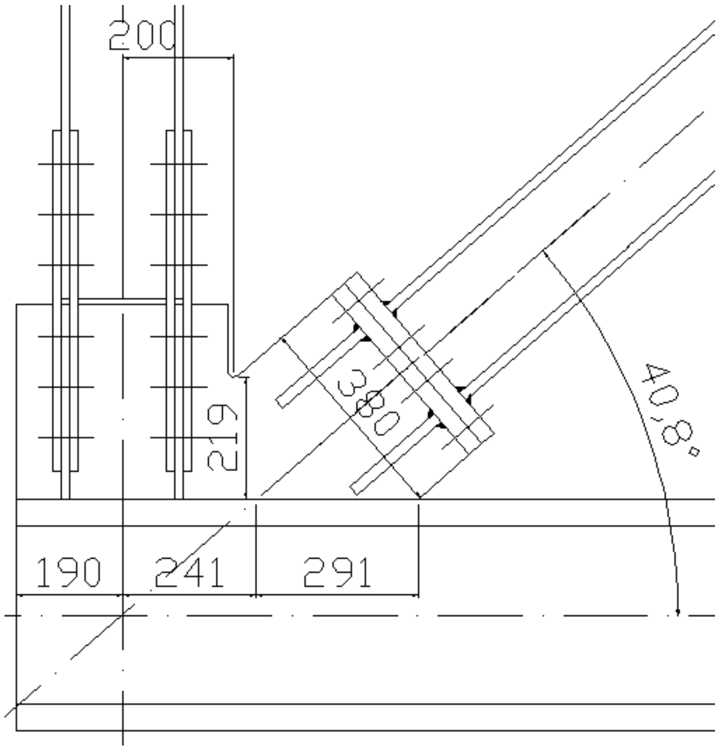
Table 8-3: Reaction forces and bending moments truss 10

Node	Members	Rx	Rz	ex	ez	M
		[kN]	[kN]	[mm]	[mm]	[kNm]
Node 1	S1, S8	7317	8811	-224	187,5	602
Node 3	S3, S9, S10	2899	-434	22,5	187,5	534
Node 5	S5, S11	-10094	-528	-3,5	187,5	1891
Node 9	S1	-10437	-990	42,5	-212,5	2176
Node 11	S3	3840	-1030	-53,5	-212,5	761
Node 13	S5, S12	-6639	497	193,5	-212,5	1507

Table 8-4: Reaction forces and bending moments truss 11

The combination of axial force, shear force and bending moment is checked with the use of the yield criterion. The welds are checked by comparing the total thickness of the welds to the thickness of the plate.

TRUSS 10 NODE 1:



Rx = 1011 kN Rz = 3426 kN M = 376 kNm Fd1 = 0 kN Fv = 2571 kN Fd2 = 2893 kN

Gusset plate thickness = 25 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 722 mm:

- Diagonal 2 (F = 2893 kN): $N_{u,d}$ = 380x25x460/1000 = 4370 kN

Shear:

- Diagonal 2 (Fv = sin(40,8)x2893 = 1890 kN): $V_{u,d}$ = 219x25x460/(√3x1000) = 1454 kN

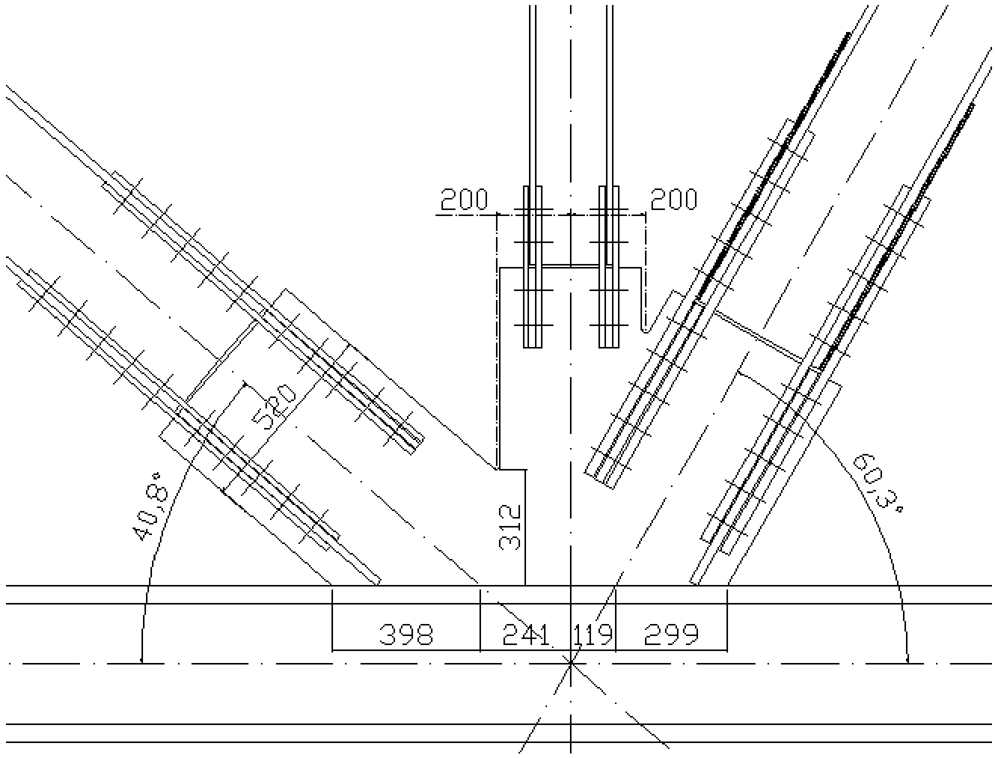
A part of the vertical force of diagonal 2 needs to go through the chord but this is not governing:

- Normal stress = 3426x1000/(722x25) = 189,8 N/mm^2
- Shear stress = 1011x1000/(722x25) = 56,0 N/mm^2
- Bending stress = 376x1000000x6/(722^{2}x25) = 172,9 N/mm^2
- Combined stress = √((189,8+172,9)^2+3x56,0^2) = 375,5 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 14 mm → 12 mm effective):

- Normal stress =189,8x25/(2x12) = 197,7 N/mm^2
- Shear stress = 56,0x25/(2x12) = 58,3 N/mm^2
- Bending stress = 172,9x25/(2x12) = 180,1 N/mm^2
- Combined stress = √((197,7+180,1)^2+3x58,3^2)/√3 = 225,8 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 10 NODE 3:



Rx = 9949 kN Rz = 253 kN M = 2097 kNm Fd1 = 7831 kN Fv = 1883 kN Fd2 = 8493 kN

Gusset plate thickness = 50 mm → f_y = 430 N/mm², $f_{w,u,d}$ = 244,8 N/mm², L = 1057 mm:

- Diagonal 1 (F = 7831 kN): $N_{u,d}$ = 520x50x430/1000 = 11180 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x7831 = 5117 kN): $V_{u,d}$ = 312x50x430/(√3x1000) = 3873 kN

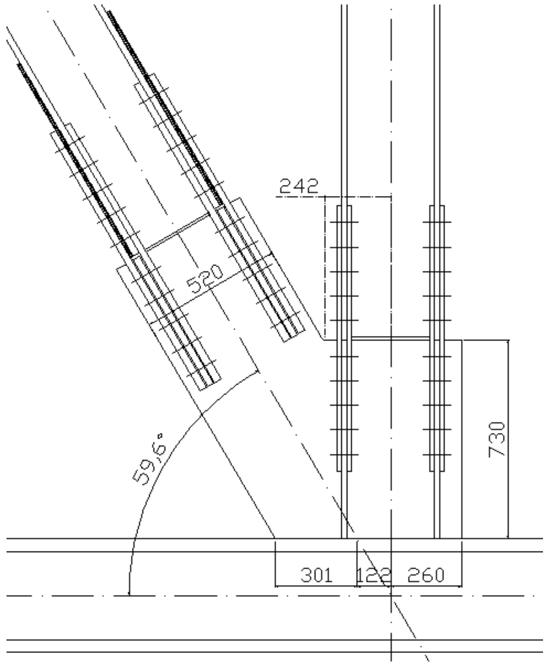
A part of the vertical force of diagonal 1 needs to go through the chord but this is not governing:

- Normal stress = 253x1000/(1057x50) = 4,8 N/mm²
- Shear stress = 9949x1000/(1057x50) = 188,2 N/mm²
- Bending stress = 2097x1000000x6/(1057^2x50) = 225,3 N/mm²
- Combined stress = $\sqrt{(4{,}8+225{,}3)^2+3\times188{,}2^2}$ = 399,0 N/mm² ≤ 430 N/mm²

Welds gusset plate – chord (v = 26 mm → 24 mm effective):

- Normal stress = 4,8x50/(2x24) = 5,0 N/mm²
- Shear stress = 188,2x50/(2x24) = 196,1 N/mm²
- Bending stress = 225,3x50/(2x24) = 234,7 N/mm²
- Combined stress = $\sqrt{(5{,}0+234{,}7)^2+3\times196{,}1^2}/\sqrt{3}$ = 240,0 N/mm² ≤ 244,8 N/mm²

TRUSS 10 NODE 5:



Rx = 3871 kN Rz = 339 kN M = 778 kNm Fd1 = 7855 kN Fv = 6226 kN Fd2 = 0 kN

Gusset plate thickness = 35 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 683 mm:

- Diagonal 1 (F = 7855 kN): $N_{u,d}$ = 520x35x460/1000 = 8372 kN

Shear:

- Diagonal 1 (Fv = sin(59,6)x7855 = 6775 kN): $V_{u,d}$ = 730x35x460/(√3x1000) = 6786 kN

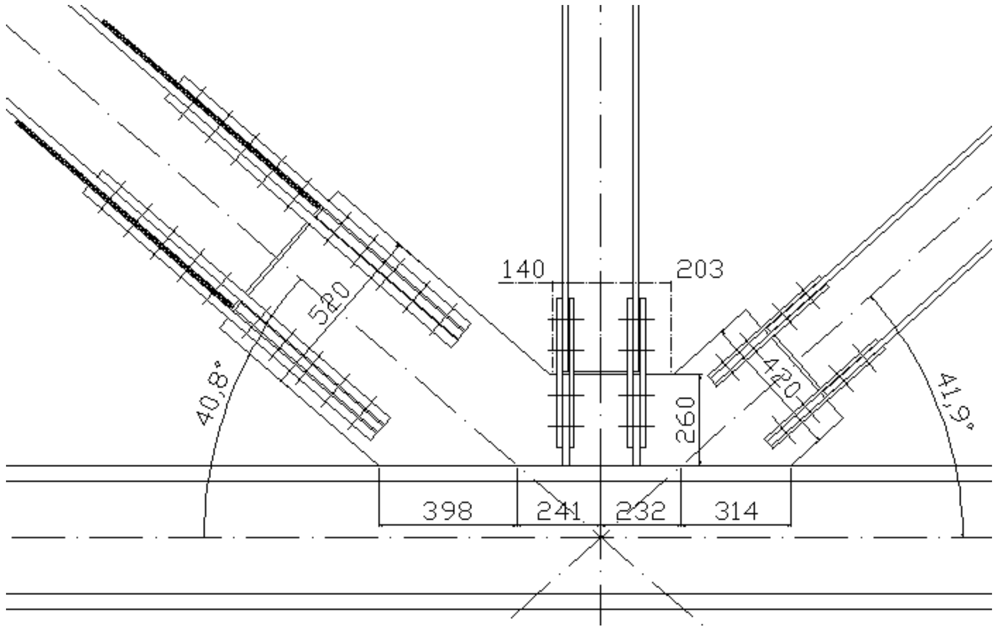
The vertical force of diagonal 1 can be transferred through the gusset plate:

- Normal stress = 339x1000/(683x35) = 14,2 N/mm^2
- Shear stress = 3871x1000/(683x35) = 161,9 N/mm^2
- Bending stress = 778x1000000x6/(683^{2}x35) = 285,7 N/mm^2
- Combined stress = √((14,2+285,7)^2+3x161,9^2) = 410,6 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 20 mm → 18 mm effective):

- Normal stress = 14,2x35/(2x18) = 13,8 N/mm^2
- Shear stress = 161,9x35/(2x18) = 157,4 N/mm^2
- Bending stress = 285,7x35/(2x18) = 277,8 N/mm^2
- Combined stress = √((13,8+277,8)^2+3x157,4^2)/√3 = 230,5 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 10 NODE 6:



Rx = 6863 kN Rz = 1153 kN M = 1374 kNm Fd1 = 7347 kN Fv = 1521 kN Fd2 = 2697 kN

Gusset plate thickness = 35 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 1185 mm:

- Diagonal 1 (F = 7347 kN): $N_{u,d}$ = 520x35x460/1000 = 8372 kN
- Diagonal 2 (F = 2697 kN): $N_{u,d}$ = 420x35x460/1000 = 6762 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x7347 = 4801 kN): $V_{u,d}$ = 260x35x460/(√3x1000) = 2417 kN
- Diagonal 2 (Fv = sin(41,9)x2697 = 1801 kN): $V_{u,d}$ = 260x35x460/(√3x1000) = 2417 kN

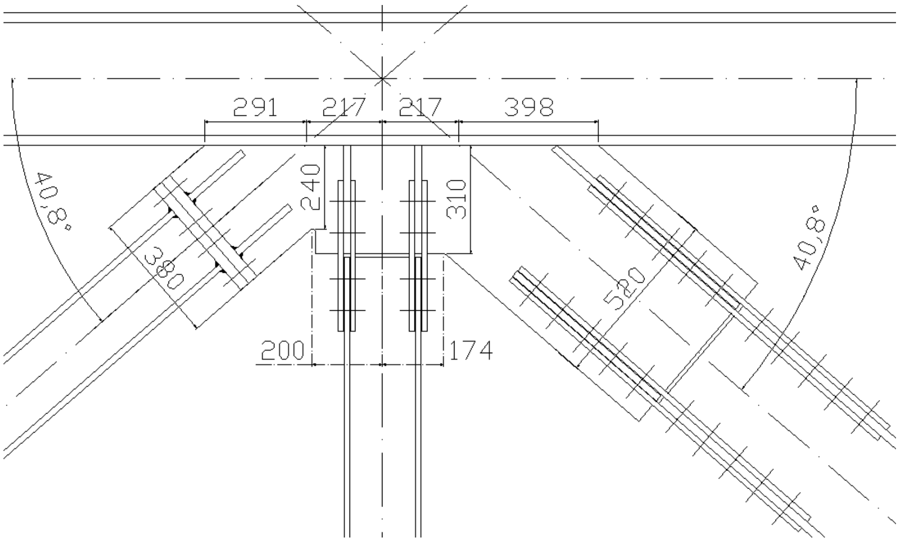
A part of the vertical force of diagonal 1 needs to go through the chord but this is not governing:

- Normal stress = 1153x1000/(1185x35) = 27,8 N/mm^2
- Shear stress = 6863x1000/(1185x35) = 165,5 N/mm^2
- Bending stress = 1374x1000000x6/(1185^{2}x35) = 167,7 N/mm^2
- Combined stress = √((27,8+167,7)^2+3x165,5^2) = 346,9 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 18 mm → 16 mm effective):

- Normal stress = 27,8x35/(2x16) = 30,4 N/mm^2
- Shear stress = 165,5x35/(2x16) = 181,0 N/mm^2
- Bending stress = 167,7x35/(2x16) = 183,4 N/mm^2
- Combined stress = √((30,4+183,4)^2+3x181,0^2)/√3 = 219,1 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 10 NODE 10:



Rx = 7279 kN Rz = 1540 kN M = 1282 kNm Fd1 = 2893 kN Fv = 1671 kN Fd2 = 7831 kN

Gusset plate thickness = 35 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 1123 mm:

- Diagonal 1 (F = 2893 kN): $N_{u,d}$ = 380x35x460/1000 = 6118 kN
- Diagonal 2 (F = 7831 kN): $N_{u,d}$ = 520x35x460/1000 = 8372 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x2893 = 1890 kN): $V_{u,d}$ = 240x35x460/(√3x1000) = 2231 kN
- Diagonal 2 (Fv = sin(40,8)x7831 = 5117 kN): $V_{u,d}$ = 310x35x460/(√3x1000) = 2882 kN

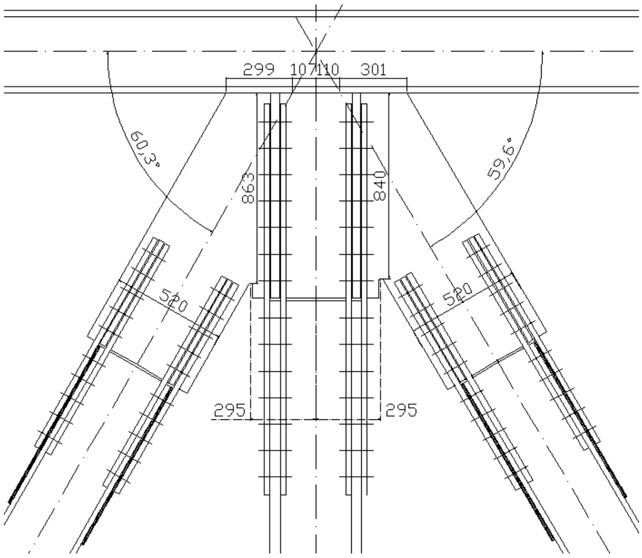
A part of the vertical force of diagonal 2 needs to go through the chord but this is not governing:

- Normal stress = 1540x1000/(1123x35) = 39,2 N/mm^2
- Shear stress = 7279x1000/(1123x35) = 185,2 N/mm^2
- Bending stress = 1282x1000000x6/(1123^{2}x35) = 174,3 N/mm^2
- Combined stress = √((39,2+174,3)^2+3x185,2^2) = 385,3 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 18 mm → 16 mm effective):

- Normal stress = 39,2x35/(2x16) = 42,9 N/mm^2
- Shear stress = 185,2x35/(2x16) = 202,6 N/mm^2
- Bending stress = 174,3x35/(2x16) = 190,7 N/mm^2
- Combined stress = √((42,9+190,7)^2+3x202,6^2)/√3 = 243,3 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 10 NODE 12:



Rx =934 kN Rz = 543 kN M = 174 kNm Fd1 = 8493 kN Fv = 15039 kN Fd2 = 7855 kN

Gusset plate thickness = 40 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 817 mm:

- Diagonal 1 (F = 8493 kN): $N_{u,d}$ = 520x40x460/1000 = 9568 kN
- Diagonal 2 (F = 7855 kN): $N_{u,d}$ = 520x40x460/1000 = 9568 kN

Shear:

- Diagonal 1 (Fv = sin(60,3)x8493 = 7377 kN): $V_{u,d}$ = 863x40x460/(√3x1000) = 9168 kN
- Diagonal 2 (Fv = sin(59,6)x7855 = 6775 kN): $V_{u,d}$ = 840x40x460/(√3x1000) = 8924 kN

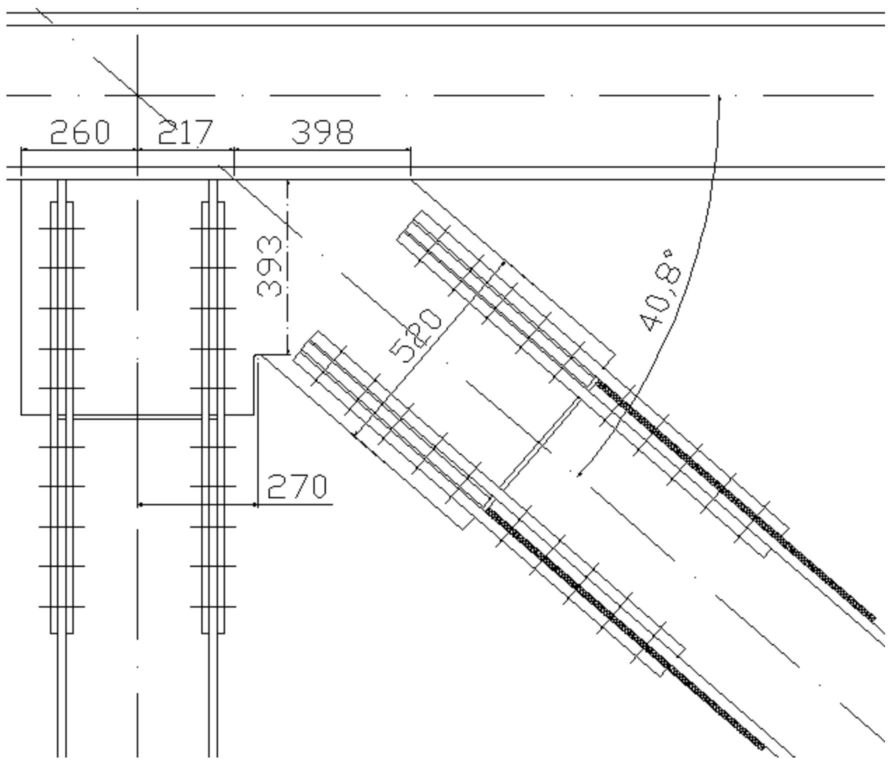
The vertical force of diagonal 1 and 2 can be transferred through the gusset plate:

- Normal stress = 543x1000/(817x40) = 16,6 N/mm^2
- Shear stress = 934x1000/(817x40) = 28,6 N/mm^2
- Bending stress = 174x1000000x6/(817^{2}x40) = 39,0 N/mm^2
- Combined stress = √((16,6+39,0)^2+3x28,6^2) = 74,5 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (a = 8 mm):

- Normal/shear stress (axial force) = 543x1000/(2x8x817x√2) = 29,4 N/mm^2
- Shear stress = 934x1000/(2x8x817) = 71,5 N/mm^2
- Normal/shear stress (moment) = 174x1000000x3/(8x817^{2}x√2) = 69,1 N/mm^2
- Combined stress = √((29,4+71,5)^2+3x(29,4+71,5+69,1)^2)/√3 = 179,3 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 10 NODE 13:



Rx = 5533 kN Rz = 990 kN M = 1213 kNm Fd1 = 0 kN Fv = 6226 kN Fd2 = 7347 kN

Gusset plate thickness = 35 mm → f_y = 460 N/mm^2, $f_{w,u,d}$ = 249,4 N/mm^2, L = 875 mm:

- Diagonal 2 (F = 7347 kN): $N_{u,d}$ = 520x35x460/1000 = 8372 kN

Shear:

- Diagonal 2 (Fv = sin(40,8)x7347 = 4801 kN): $V_{u,d}$ = 393x35x460/(√3x1000) = 4801 kN

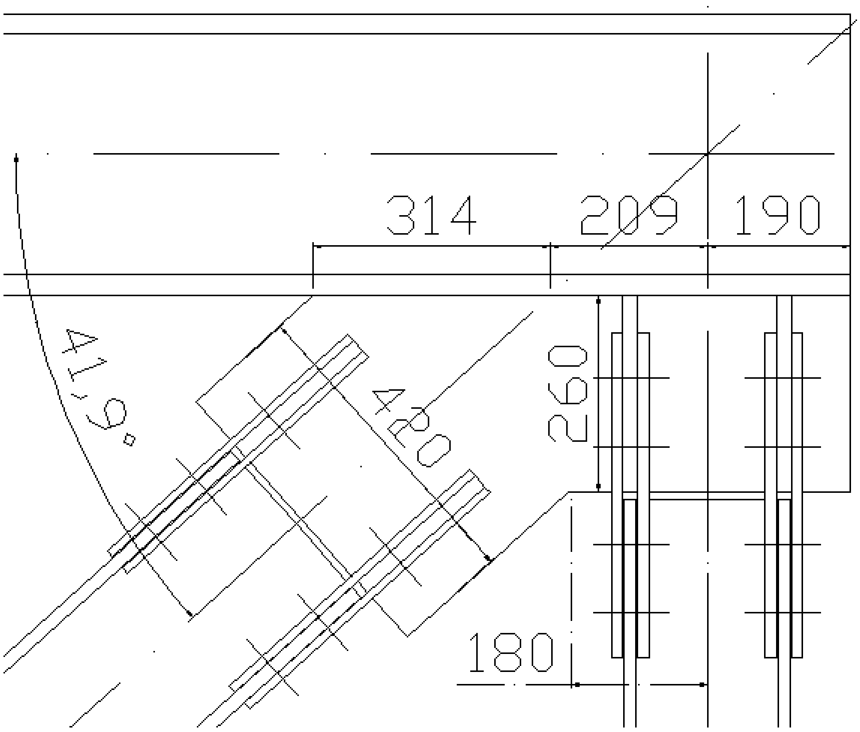
The vertical force of diagonal 2 needs to go through the chord but this is not governing:

- Normal stress = 990x1000/(875x35) = 32,3 N/mm^2
- Shear stress = 5533x1000/(875x35) = 180,7 N/mm^2
- Bending stress = 1213x1000000x6/(875^{2}x35) = 271,6 N/mm^2
- Combined stress = √((32,2+271,6)^2+3x180,7^2) = 436,3 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 20 mm → 18 mm effective):

- Normal stress = 32,3x35/(2x18) = 31,4 N/mm^2
- Shear stress = 180,7x35/(2x18) = 175,7 N/mm^2
- Bending stress = 271,6x35/(2x18) = 264,1 N/mm^2
- Combined stress = √((31,4+264,1)^2+3x175,7^2)/√3 = 244,9 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 10 NODE 15:



Rx = 1509 kN Rz = 556 kN M = 190 kNm Fd1 = 2697 kN Fv = 1084 kN Fd2 = 0 kN

Gusset plate thickness = 15 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 713 mm:

- Diagonal 1 (F = 2697 kN): $N_{u,d}$ = 420x15x460/1000 = 2898 kN

Shear:

- Diagonal 1 (Fv = sin(41,9)x2697 = 1801 kN): $V_{u,d}$ = 260x15x460/(√3x1000) = 1036 kN

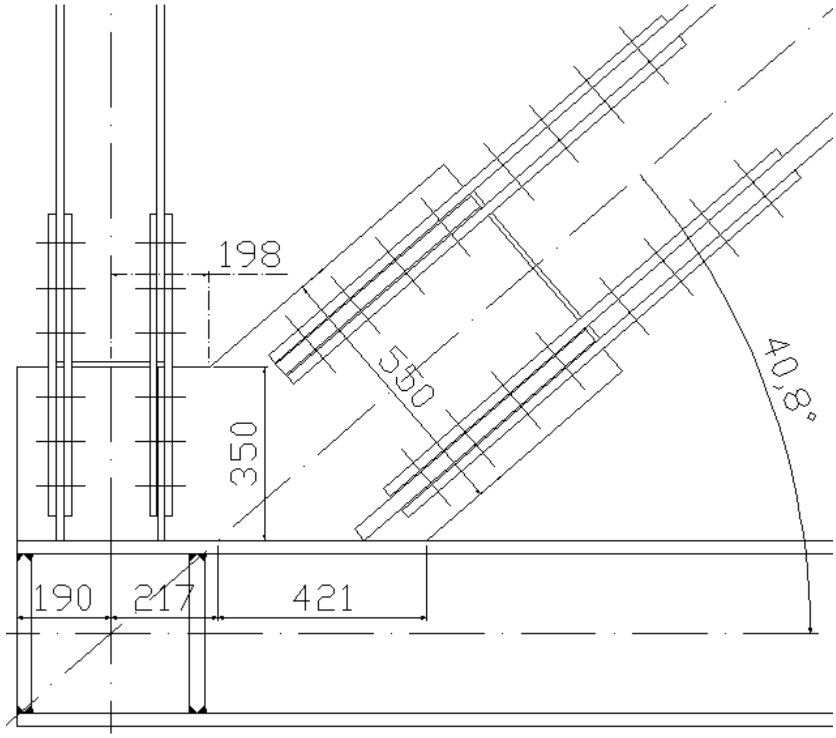
A part of the vertical force of diagonal 1 needs to go through the chord but this is not governing:

- Normal stress = 556x1000/(713x15) = 52,0 N/mm^2
- Shear stress = 1509x1000/(713x15) = 141,1 N/mm^2
- Bending stress = 190x1000000x6/(713^{2}x15) = 149,8 N/mm^2
- Combined stress = √((52,0+149,8)^2+3x141,1^2) = 316,9 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (a = 9 mm):

- Normal/shear stress (axial force) = 556x1000/(2x8x713x√2) = 30,6 N/mm^2
- Shear stress = 1509x1000/(2x9x713) = 117,6 N/mm^2
- Normal/shear stress (moment) = 190x1000000x3/(9x713^{2}x√2) = 88,3 N/mm^2
- Combined stress = √((30,6+88,3)^2+3x(30,6+117,6+88,3)^2)/√3 = 246,2 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 11 NODE 1:



Rx = 7317 kN   Rz = 8811 kN   M = 602 kNm   Fd1 = 0 kN   Fv = 2375 kN   Fd2 = 9779 kN

Gusset plate thickness = 55 mm → f_y = 430 N/mm², $f_{w,u,d}$ = 244,8 N/mm², L = 828 mm:

- Diagonal 2 (F = 9779 kN): $N_{u,d}$ = 550x55x430/1000 = 13008 kN

Shear:

- Diagonal 2 (Fv = sin(40,8)x9779 = 6390 kN): $V_{u,d}$ = 350x55x430/(√3x1000) = 4779 kN

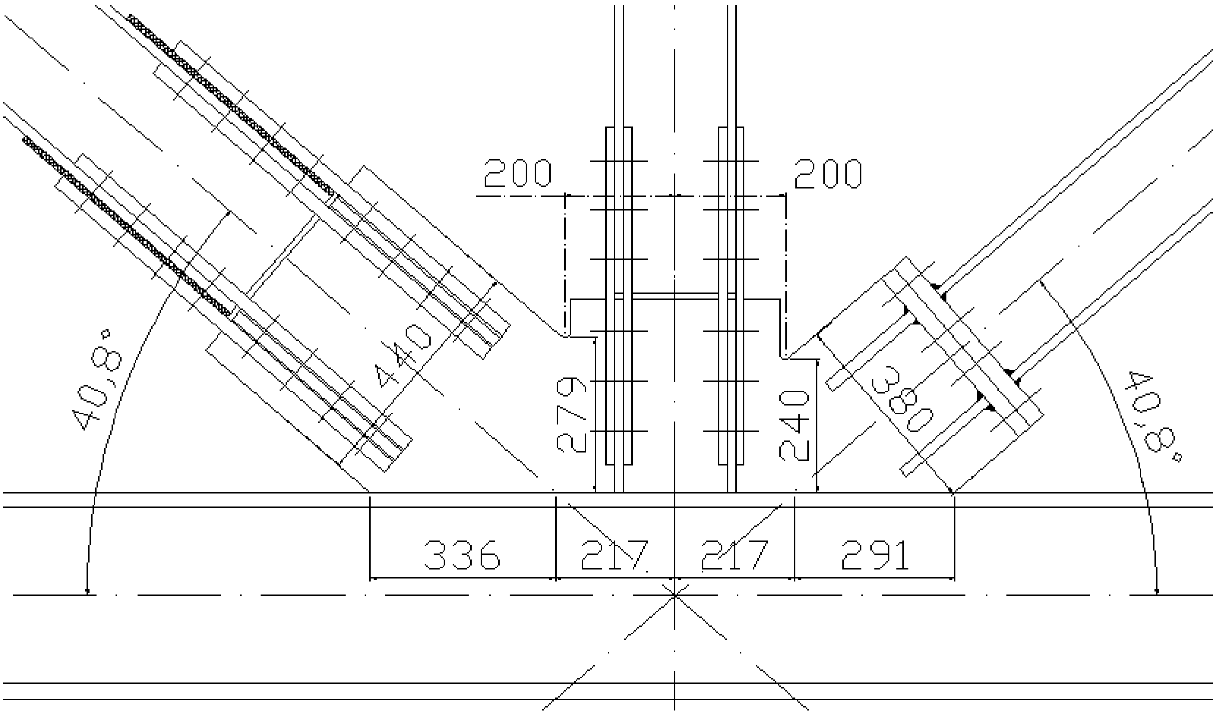
A part of the vertical force of diagonal 2 needs to go through the chord but this is not governing:

- Normal stress = 8811x1000/(828x55) = 193,5 N/mm²
- Shear stress = 7317x1000/(828x55) = 160,7 N/mm²
- Bending stress = 602x1000000x6/(828²x55) = 95,7 N/mm²
- Combined stress = √((193,5+95,7)²+3x160,7²) = 401,4 N/mm² ≤ 430 N/mm²

Welds gusset plate – chord (v = 30 mm → 28 mm effective):

- Normal stress = 193,5x55/(2x28) = 190,0 N/mm²
- Shear stress = 160,7x55/(2x28) = 157,8 N/mm²
- Bending stress = 95,7x55/(2x28) = 94,0 N/mm²
- Combined stress = √((190,0+94,0)²+3x157,8²)/√3 = 227,6 N/mm² ≤ 244,8 N/mm²

TRUSS 11 NODE 3:



Rx = 2899 kN Rz = 434 kN M = 534 kNm Fd1 = 4595 kN Fv = 2079 kN Fd2 = 871 kN

Gusset plate thickness = 25 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 1061 mm:

- Diagonal 1 (F = 4595 kN): $N_{u,d}$ = 440x25x460/1000 = 5060 kN
- Diagonal 2 (F = 871 kN): $N_{u,d}$ = 380x25x460/1000 = 4370 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x4595 = 3002 kN): $V_{u,d}$ = 279x25x460/(√3x1000) = 1852 kN
- Diagonal 2 (Fv = sin(40,8)x871 = 569 kN): $V_{u,d}$ = 240x25x460/(√3x1000) = 1593 kN

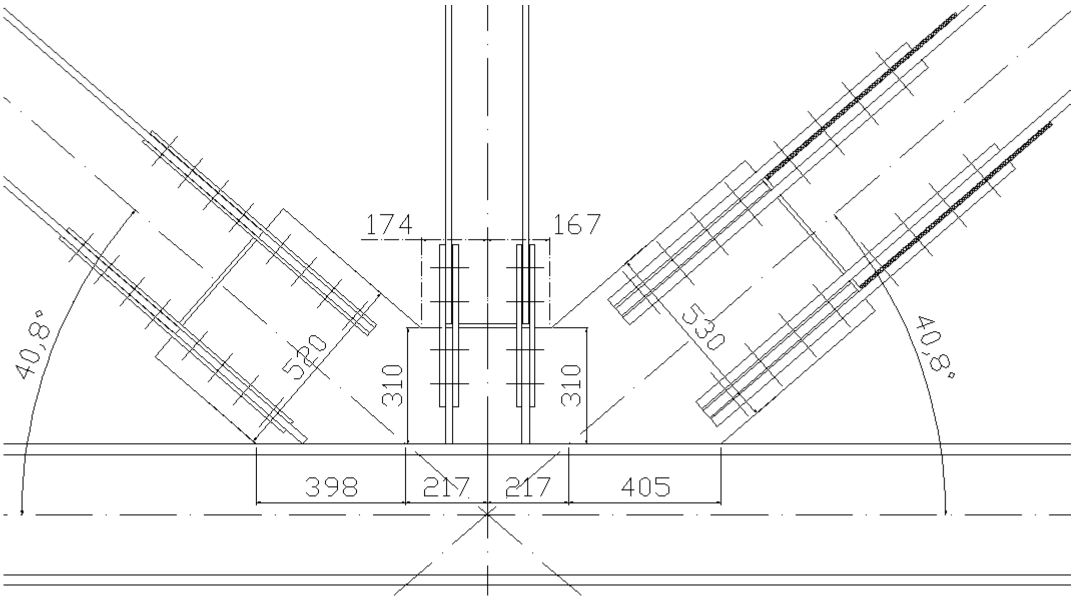
A part of the vertical force of diagonal 1 needs to go through the chord:

- Normal stress = (3002-1852)x1000/((336+217-200)x25) = 130,3 N/mm^2
- Shear stress = 2899x1000/(1061x25) = 109,3 N/mm^2
- Combined stress = √(130,3^2+3x109,3^2) = 229,8 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 10 mm → 8 mm effective):

- Normal stress = 130,3x25/(2x8) = 203,6 N/mm^2
- Shear stress = 109,3x25/(2x8) = 170,8 N/mm^2
- Combined stress = √(203,6^2+3x170,8^2)/√3 = 207,3 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 11 NODE 5:



Rx = 10094 kN   Rz = 528 kN   M = 1891 kNm Fd1 = 5100 kN Fv = 1609 kN   Fd2 = 8962 kN

Gusset plate thickness = 40 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 1237 mm:

- Diagonal 1 (F = 5100 kN): $N_{u,d}$ = 520x40x460/1000 = 9568 kN
- Diagonal 2 (F = 8962 kN): $N_{u,d}$ = 530x40x460/1000 = 9752 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x5100 = 3332 kN): $V_{u,d}$ = 310x40x460/(√3x1000) = 3293 kN
- Diagonal 2 (Fv = sin(40,8)x8962 = 5856 kN): $V_{u,d}$ = 310x40x460/(√3x1000) = 3293 kN

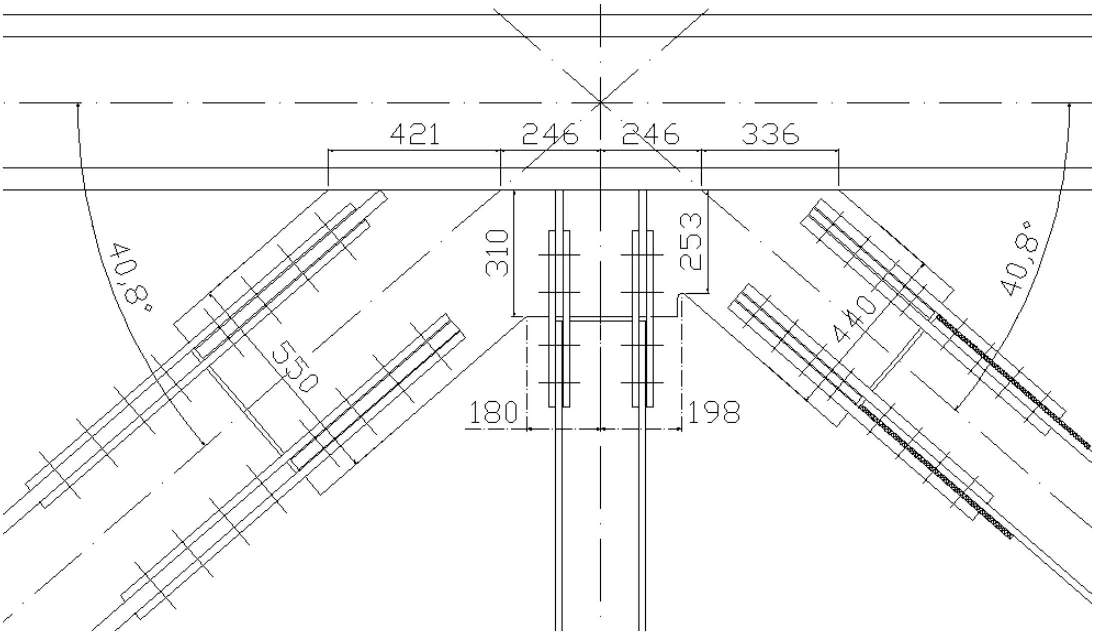
A part of the vertical force of diagonal 1/2 needs to go through the chord but this is not governing:

- Normal stress = 528x1000/(1237x40) = 10,7 N/mm^2
- Shear stress = 10094x1000/(1237x40) = 204,0 N/mm^2
- Bending stress = 1891x1000000x6/(1237^{2}x40) = 185,3 N/mm^2
- Combined stress = √((10,7+185,3)^2+3x204,0^2) = 404,1 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 22 mm → 20 mm effective):

- Normal stress = 10,7x40/(2x20) = 10,7 N/mm^2
- Shear stress = 204,0x40/(2x20) = 204,0 N/mm^2
- Bending stress = 185,3x40/(2x20) = 185,3 N/mm^2
- Combined stress = √((10,7+185,3)^2+3x204,0^2)/√3 = 233,3 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 11 NODE 9:



Rx = 10437 kN Rz = 990 kN M = 2176 kNm Fd1 = 9779 kN Fv = 1081 kN Fd2 = 4595 kN

Gusset plate thickness = 40 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 1249 mm:

- Diagonal 1 (F = 9779 kN): $N_{u,d}$ = 550x40x460/1000 = 10120 kN
- Diagonal 2 (F = 4595 kN): $N_{u,d}$ = 440x40x460/1000 = 8096 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x9779 = 6390 kN): $V_{u,d}$ = 310x40x460/(√3x1000) = 3293 kN
- Diagonal 2 (Fv = sin(40,8)x4595 = 3002 kN): $V_{u,d}$ = 253x40x460/(√3x1000) = 2688 kN

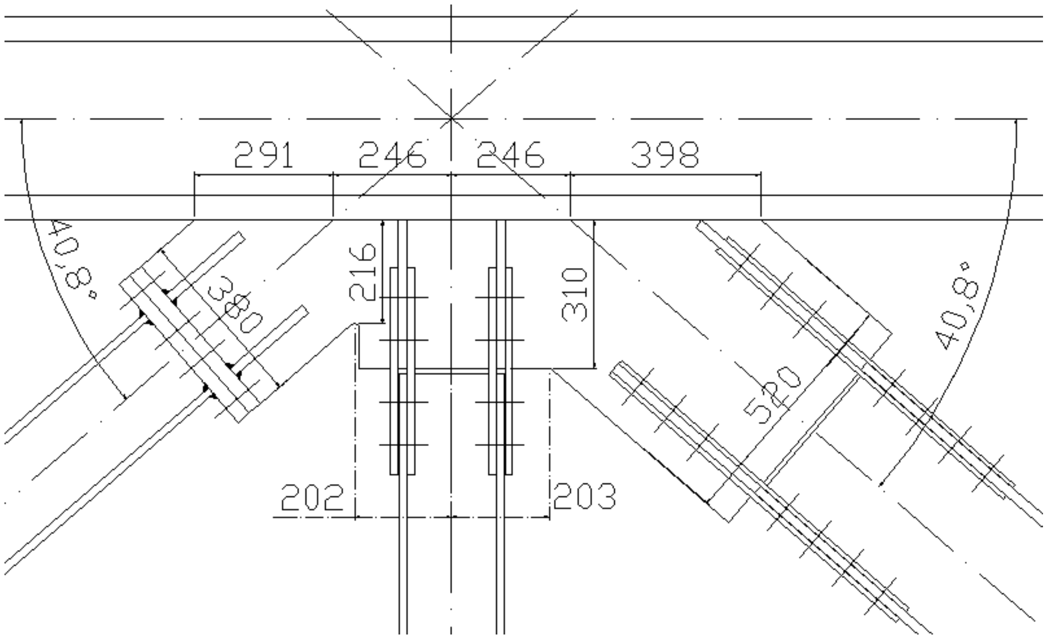
A part of the vertical force of diagonal 1/2 needs to go through the chord but this is not governing:

- Normal stress = 990x1000/(1249x40) = 19,8 N/mm^2
- Shear stress = 10437x1000/(1249x40) = 208,9 N/mm^2
- Bending stress = 2176x1000000x6/(1249^{2}x40) = 209,2 N/mm^2
- Combined stress = √((19,8+209,2)^2+3x208,9^2) = 428,2 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 22 mm → 20 mm effective):

- Normal stress = 19,8x40/(2x20) = 19,8 N/mm^2
- Shear stress = 208,9x40/(2x20) = 208,9 N/mm^2
- Bending stress = 209,2x40/(2x20) = 209,2 N/mm^2
- Combined stress = √((19,8+209,2)^2+3x208,9^2)/√3 = 247,2 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 11 NODE 11:



Rx = 3840 kN Rz = 1030 kN M = 761 kNm Fd1 = 871 kN Fv = 933 kN Fd2 = 5100 kN

Gusset plate thickness = 25 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 1181 mm:

- Diagonal 1 (F = 871 kN): $N_{u,d}$ = 380x25x460/1000 = 4370 kN
- Diagonal 2 (F = 5100 kN): $N_{u,d}$ = 520x25x460/1000 = 5980 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x871 = 569 kN): $V_{u,d}$ = 216x25x460/(√3x1000) = 1434 kN
- Diagonal 2 (Fv = sin(40,8)x5100 = 3332 kN): $V_{u,d}$ = 310x25x460/(√3x1000) = 2058 kN

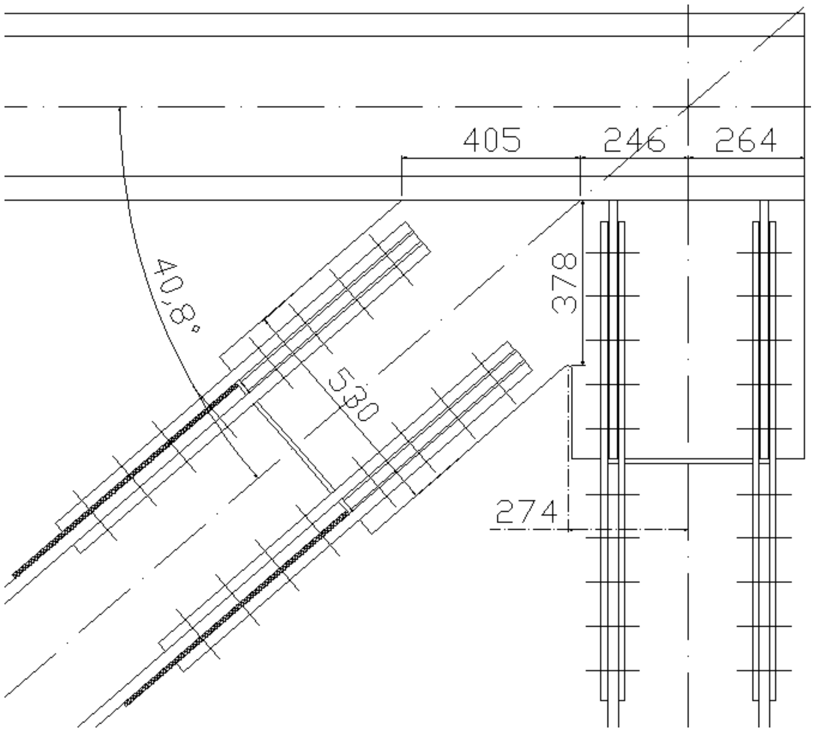
A part of the vertical force of diagonal 2 needs to go through the chord but this is not governing:

- Normal stress = 1030x1000/(1181x25) = 34,9 N/mm^2
- Shear stress = 3840x1000/(1181x25) = 130,1 N/mm^2
- Bending stress = 761x1000000x6/(1181^{2}x25) = 130,9 N/mm^2
- Combined stress = √((34,9+130,9)^2+3x130,1^2) = 279,7 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 12 mm → 10 mm effective):

- Normal stress = 34,9x25/(2x10) = 43,6 N/mm^2
- Shear stress = 130,1x25/(2x10) = 162,6 N/mm^2
- Bending stress = 130,9x25/(2x10) = 163,7 N/mm^2
- Combined stress = √((43,6+163,7)^2+3x162,6^2)/√3 = 201,9 N/mm^2 ≤ 249,4 N/mm^2

TRUSS 11 NODE 13:



Rx = 6639 kN Rz = 497 kN M = 1507 kNm Fd1 = 8962 kN Fv = 7111 kN Fd2 = 0 kN

Gusset plate thickness = 40 mm → f_y = 460 N/mm^2 , $f_{w,u,d}$ = 249,4 N/mm^2 , L = 915 mm:

- Diagonal 1 (F = 8962 kN): $N_{u,d}$ = 530x40x460/1000 = 9752 kN

Shear:

- Diagonal 1 (Fv = sin(40,8)x8962 = 5956 kN): $V_{u,d}$ = 378x40x460/(√3x1000) = 4016 kN

A part of the vertical force of diagonal 1 needs to go through the chord but this is not governing:

- Normal stress = 497x1000/(915x40) = 13,6 N/mm^2
- Shear stress = 6639x1000/(915x40) = 181,4 N/mm^2
- Bending stress = 1507x1000000x6/(915^{2}x40) = 270,0 N/mm^2
- Combined stress = √((13,6+270,0)^2+3x181,4^2) = 423,2 N/mm^2 ≤ 460 N/mm^2

Welds gusset plate – chord (v = 22 mm → 20 mm effective):

- Normal stress = 13,6x40/(2x20) = 13,6 N/mm^2
- Shear stress = 181,4x40/(2x20) = 181,4 N/mm^2
- Bending stress = 270,0x40/(2x20) = 270,0 N/mm^2
- Combined stress = √((13,6+270,0)^2+3x181,4^2)/√3 = 244,4 N/mm^2 ≤ 249,4 N/mm^2

8.3 Appendix III

This appendix displays the calculation of the buckling resistances of the sections in the current and new design. The buckling resistances of the sections are dependent on the cross-sectional area (A), the moment of inertia (I_y and I_z) and the buckling length (L_k). For H-sections all parameters are known. For box sections the cross-sectional area and the moment of inertia about both axis has to be calculated. A space of 20 mm is required to weld the plates to the sections. The height of the plates (h_p) is therefore the height of the section – 40 mm. The total cross-sectional area is than calculated:

$A = A_{section} + 2 x h_p x t$ where t is the thickness of the plates.

The moment of inertia about the strong axis:

$I_y = I_{y,section} + 2 x t x h_p^3/12$

The moment of inertia about the weak axis:

$I_z = I_{z,section} + 2 x h_p x t^3/12 + 2 x h_p x t x (b/2+t/2)^2$ where b is the width of the section.

If the thickness of the plates is large enough the moment of inertia about the weak axis would be larger than the moment of inertia about the strong axis which would essentially make the weak axis the strong axis. To avoid confusion the terms strong and weak axis are therefore used to describe the axis of the section alone. When doing so the direction of the “strong” axis is always the same even if in reality it is the weak axis.

With the described formulas the cross sectional properties of all box sections were calculated. The properties of all sections is shown in Table 8-5 and Table 8-6. In these tables the maximum buckling length is also shown for “long” (Table 8-5) and “short” (Table 8-6) columns.

Section	A [mm^2]	Iy [mm^4] ($x10^4$)	Iz [mm^4] ($x10^4$)	Buckling length [mm]	Weight [kg/m]
SHS600x100	200000	866667	789867	10460	1570,0
UC356x406x634 + 2x100PL	167750	411988	695333	10460	1316,8
UC356x406x634 + 2x75PL	146000	377691	504313	10460	1146,1
UC356x406x634 + 2x60PL	132950	357113	403834	10460	1043,7
UC356x406x634 + 2x50PL	124250	343394	342465	10460	975,4
UC356x406x634 + 2x40PL	115550	329675	285438	10460	907,1
UC356x406x634	80750	274800	98130	9820	633,9
UC356x406x551	70109	226900	82670	9820	550,4
UC356x406x393 + 2x50PL	88060	192327	253777	10460	691,3
UC356x406x393 + 2x40PL	80460	183181	207225	9820	631,6
UC356x406x340	43300	122500	46850	9820	339,9

Table 8-5: Properties of “long” columns

Section	A [mm²]	Iy [mm⁴] (x10⁴)	Iz [mm⁴] (x10⁴)	Buckling length [mm]	Weight [kg/m]
UC356x406x634 + 2x100PL	167750	411988	695333	5380	1316,8
UC356x406x634 + 2x75PL	146000	377691	504313	5760	1146,1
UC356x406x634 + 2x60PL	132950	357113	403834	5380	1043,7
UC356x406x634 + 2x50PL	124250	343394	342465	5380	975,4
UC356x406x634 + 2x40PL	115550	329675	285438	5380	907,1
UC356x406x634 + 2x30PL	106850	315956	232621	5380	838,8
UC356x406x634 + 2x20PL	98150	302238	183884	4350	770,5
UC356x406x634	80750	274800	98130	5380	633,9
UC356x406x551	70109	226900	82670	5410	550,4
UC356x406x467	59490	183000	67830	7000	467,0
UC356x406x393 + 2x50PL	88060	192327	253777	5380	691,3
UC356x406x393 + 2x40PL	80460	183181	207225	5410	631,6
UC356x406x393 + 2x30PL	72860	174036	164222	3950	572,0
UC356x406x393	50060	146600	55370	7000	393,0
UC356x406x340	43300	122500	46850	7000	339,9
UC356x406x287	36570	99880	38680	7000	287,1
UC356x406x235	29900	79080	30990	7000	234,7
UC356x368x202	25720	66260	23690	6890	201,9
UC305x305x240	30580	64200	20310	6890	240,1
UC305x305x198	25240	50900	16300	7000	198,1
UC305x305x158	20140	38750	12570	6890	158,1
UC254x254x167	21290	30000	9870	6890	167,1
UC254x254x132	16810	22530	7531	6890	132,0
UC203x203x86	10960	9449	3127	6890	86,0

Table 8-6: Properties of "short" columns

With the use of these properties the buckling resistance can be calculated (Table 8-7 and Table 8-8). The plastic capacity of the sections can be calculated with:

$N_{pl,Rd} = A x f_y$

The yield strength is dependent on the thickness of the member:

- For $t \leq 40$ mm $f_y = 355$ N/mm^2
- For $40 < t \leq 80$ mm $f_y = 335$ N/mm^2
- For $80 < t \leq 100$ mm $f_y = 315$ N/mm^2

The buckling resistance can be calculated with:

- $N_{cr} = \pi^2 x E x I / L_k^2$
- $\lambda_{rel} = \sqrt{(N_{pl,Rd}/N_{cr})}$
- $\phi = 0,5x(1+\alpha x(\lambda_{rel}-0,2)+\lambda_{rel}^2)$ — α is the imperfection factor for curve a0, a, b, c or d
- $\chi = 1/(\phi+\sqrt{(\phi^2-\lambda_{rel}^2)})$
- $N_{b,rd} = \chi x N_{pl,Rd}/\gamma_{M1}$ — $\gamma_{M1} = 1$

Section	Curve	λ	ϕ	χ	Nb,Rd [kN]
UC356x406x634 + 2x100PL	b	0,423	0,628	0,917	48442
UC356x406x634 + 2x75PL	c	0,393	0,624	0,901	44073
UC356x406x634 + 2x60PL	c	0,392	0,624	0,902	40160
UC356x406x634 + 2x50PL	c	0,411	0,636	0,891	37097
UC356x406x634 + 2x40PL	c	0,435	0,652	0,879	34015
UC356x406x634 + 2x30PL	c	0,463	0,672	0,863	30900
UC356x406x634 + 2x20PL	c	0,404	0,632	0,895	29435
UC356x406x634	c	0,620	0,796	0,773	20916
UC356x406x551	c	0,633	0,807	0,765	17977
UC356x406x467	c	0,833	1,002	0,641	12778
UC356x406x393 + 2x50PL	c	0,402	0,630	0,896	26435
UC356x406x393 + 2x40PL	c	0,428	0,648	0,882	23782
UC356x406x393 + 2x30PL	c	0,334	0,589	0,932	22738
UC356x406x393	c	0,846	1,016	0,633	10619
UC356x406x340	c	0,856	1,027	0,627	9100
UC356x406x287	c	0,891	1,066	0,606	7861
UC356x406x235	c	0,900	1,076	0,600	6368
UC356x368x202	c	0,940	1,123	0,576	5257
UC305x305x240	c	1,106	1,334	0,481	5220
UC305x305x198	c	1,140	1,380	0,463	4152
UC305x305x158	c	1,141	1,382	0,463	3308
UC254x254x167	c	1,324	1,652	0,379	2862
UC254x254x132	c	1,347	1,689	0,369	2205
UC203x203x86	c	1,688	2,290	0,261	1014

Table 8-7: Buckling resistance of "short" columns

Section	Curve	λ	ϕ	χ	Nb,Rd [kN]
SHS600x100	b	0,649	0,787	0,812	51142
UC356x406x634 + 2x100PL	b	0,823	0,944	0,710	37531
UC356x406x634 + 2x75PL	b	0,827	0,948	0,708	34617
UC356x406x634 + 2x60PL	c	0,762	0,928	0,686	30567
UC356x406x634 + 2x50PL	c	0,800	0,967	0,662	27563
UC356x406x634 + 2x40PL	c	0,845	1,015	0,634	24529
UC356x406x634	c	1,133	1,370	0,467	12639
UC356x406x551	c	1,150	1,394	0,458	10768
UC356x406x393 + 2x50PL	b	0,900	1,024	0,661	19508
UC356x406x393 + 2x40PL	c	0,777	0,943	0,676	18234
UC356x406x340	c	1,200	1,465	0,434	6290

Table 8-8: Buckling resistance of "long" columns

It is visible that the resistance of the "long" columns have significantly been reduced. The same applies for the "short" columns with a buckling length of about 7 m. In this case the use of very small columns leads to a very small effectiveness of the columns (UC203x203x86).

The sections used in the new design and their properties are shown in Table 8-9 and Table 8-10.

Section	A [mm²]	Iy [mm⁴] (x10⁴)	Iz [mm⁴] (x10⁴)	Buckling length [mm]	Weight [kg/m]
HD400x677 + 2x40PL	121541	356259	300181	5380	954,1
HD400x677 + 2x30PL	112741	342062	245613	5760	885,0
HD400x677 + 2x20PL	103941	327865	195339	5380	815,9
HD400x818	104300	392191	135529	5380	818,8
HD400x744	94810	342121	119932	5380	744,3
HD400x634	80800	274171	98251	5380	634,3
HD400x592	75490	250161	90172	4350	592,6
HD400x463	58950	180162	67035	5380	462,8
HD400x382	48710	141317	53616	5410	382,4
HD400x347	44200	124945	48085	7000	347,0
HD400x551	70140	226110	82495	5380	550,6
HD400x509	64900	204528	75401	5410	509,5
HD400x463	58950	180162	67035	3950	462,8
HD400x287	36630	99706	38783	7000	287,5
HD400x262	33460	89406	35019	7000	262,7
HD400x216	27550	71138	28254	7000	216,3
HD400x187	23760	60183	23922	7000	186,5
HD360x162	20630	51539	18562	6890	161,9
HD360x147	18790	46289	16722	6890	147,5
HD360x134	17060	41509	15078	7000	133,9
HD360x134	17060	41509	15078	6890	133,9
HD320x127	16130	30824	9239	6890	126,6
HD320x97,6	12440	22929	6985	6890	97,7
HD260x68,2	8682	10455	3668	6890	68,2

Table 8-9: Properties of "short" columns

Section	A [mm²]	Iy [mm⁴] (x10⁴)	Iz [mm⁴] (x10⁴)	Buckling length [mm]	Weight [kg/m]
SHS600x80	160000	690453	629973	10460	1256,0
HD400x677 + 2x55PL	134741	377555	390471	10460	1057,7
HD400x677 + 2x45PL	125941	363358	329131	10460	988,6
HD400x677 + 2x40PL	121541	356259	300181	10460	954,1
HD400x677 + 2x30PL	112741	342062	245613	10460	885,0
HD400x677 + 2x20PL	103941	327865	195339	10460	815,9
HD400x509	64900	204528	75401	9820	509,5
HD400x463	58950	180162	67035	9820	462,8
HD400x382 +2x40PL	79105	177898	205197	10460	621,0
HD400x382 +2x25PL	67705	164180	141951	9820	531,5
HD400x287	36630	99706	38783	9820	287,5

Table 8-10: Properties "long" columns

The buckling resistance is again calculated. The yield strength is also in this case dependent on the thickness. If the section thickness is larger than 82 mm a yield strength of 450 N/mm^2 is used. If the welded plates are thicker than 40 mm a yield strength of 430 N/mm^2 is used, otherwise 460 N/mm^2 is used. The resistances for “short” and “long” columns are shown in Table 8-11 and Table 8-12.

Section	Curve	λ	ϕ	χ	Nb,Rd [kN]
HD400x677 + 2x40PL	b	0,510	0,683	0,880	49186
HD400x677 + 2x30PL	b	0,581	0,734	0,846	43890
HD400x677 + 2x20PL	b	0,585	0,736	0,845	40387
HD400x818	a	0,695	0,794	0,850	39889
HD400x744	a	0,705	0,801	0,845	36071
HD400x634	a	0,727	0,819	0,835	31027
HD400x592	a	0,593	0,717	0,893	30998
HD400x463	a	0,752	0,840	0,822	22295
HD400x382	a	0,768	0,855	0,813	18225
HD400x347	a	1,000	1,084	0,666	13536
HD400x551	a	0,739	0,830	0,829	26736
HD400x509	a	0,748	0,837	0,824	24605
HD400x463	a	0,552	0,689	0,907	24605
HD400x287	a	1,013	1,099	0,656	11056
HD400x262	a	1,019	1,106	0,652	10036
HD400x216	a	1,030	1,117	0,645	8171
HD400x187	a	1,039	1,128	0,638	6974
HD360x162	a	1,082	1,178	0,608	5773
HD360x147	a	1,088	1,185	0,604	5222
HD360x134	a	1,109	1,211	0,590	4628
HD360x134	a	1,092	1,190	0,602	4721
HD320x127	a	1,356	1,541	0,440	3264
HD320x97,6	a	1,370	1,561	0,433	2478
HD260x68,2	a	1,579	1,892	0,341	1362

Table 8-11: Buckling resistance of “short” columns

In this case the heaviest HD section used is the HD400x818. The thickness is smaller than 100 mm so buckling curve a is used for all H-sections. The largest box section consist of an HD400x677 section. The buckling curve b is in this case always used.

Section	Curve	λ	ϕ	χ	Nb,Rd [kN]
SHS600x80	b	0,759	0,883	0,749	51546
HD400x677 + 2x55PL	b	0,900	1,024	0,661	38306
HD400x677 + 2x45PL	b	0,932	1,059	0,641	34691
HD400x677 + 2x40PL	b	0,992	1,126	0,602	33679
HD400x677 + 2x30PL	b	1,056	1,203	0,562	29151
HD400x677 + 2x20PL	b	1,137	1,305	0,514	24558
HD400x509	a	1,357	1,543	0,439	13119
HD400x463	a	1,372	1,564	0,432	11712
HD400x382 +2x40PL	b	1,039	1,183	0,572	20829
HD400x382 +2x25PL	b	1,010	1,148	0,590	18390
HD400x287	a	1,422	1,639	0,407	6865

Table 8-12: Buckling resistance "long" columns

The required welds to connect the plates to the flanges are based on the design buckling resistance of the members. The weld sizes are based on an initial deflection of the column. For all columns buckling curve b is valid. For this buckling curve the initial deflection is L/200. According to NEN-EN 1993-1-1 5.3.2 (7) this initial deflection can be replaced by an equivalent distributed load of $8xN_{Ed}xe_0/L^2$ which results in a maximum shear force of $V = 4xN_{Ed}xe_0/L$. N_{Ed} cannot be larger than the buckling resistance and $e_0/L=1/200$ so the maximum shear force in the column $V = 4xN_{b,Rd}/200$. This value should be multiplied by the factor n/(n-1) where $n = F_{cr}/N_{b,Rd}$. When the maximum shear force is known the shear flow (s) can be calculated with the use of s = VxS/I. Where S is the area-moment of the plate and I is the moment of inertia. With the use of the maximum allowable stress in the weld ($f_{w,u,d}$) the effective thickness of the partial penetration welds can be calculated with $v_{eff} = s/(2xf_{w,u,d})$. The total thickness is the effective thickness + 2 mm. The minimum weld sizes based on strength are shown in Table 8-13.

	Nb,Rd	Fcr	e*/L	n	n/n-1	V	S	I	fwud	veff	v
	[kN]	[kN]	[-]	[-]	[-]	[kN]	[mm³]	[mm⁴]	[N/mm²]	[mm]	[mm]
SHS600x80	**51546**	130795	1/200	2,54	1,650	1701	12480000	6904533333	244,8	6,3	9
HD400x677 + 2x55PL	**38306**	73968	1/200	1,93	2,074	1589	5844300	3904707733	244,8	4,9	7
HD400x677 + 2x45PL	**34691**	62348	1/200	1,80	2,254	1564	4682700	3291309600	244,8	4,5	7
HD400x677 + 2x40PL	**33679**	56864	1/200	1,69	2,453	1652	4118400	3001814533	249,4	4,5	7
HD400x677 + 2x30PL	**29151**	46527	1/200	1,60	2,678	1561	3022800	2456132400	249,4	3,9	6
HD400x677 + 2x20PL	**24558**	37004	1/200	1,51	2,973	1460	1971200	1953394267	249,4	3,0	5
HD400x382 +2x40PL	**20829**	38871	1/200	1,87	2,154	898	3389600	2051974933	249,4	3,0	5
HD400x382 +2x25PL	**18390**	30510	1/200	1,66	2,517	926	2047250	1419514333	249,4	2,7	5

Table 8-13: Minimum size of the welds connecting the additional plates to the flanges

On the 2 following pages the used sections in the current design are shown. Marked columns indicate that the column is not supported at the intermediate floor ("long" columns). The columns are 2 stories high so every 2 stories a column splice is required. This is indicated by the lines between the used columns. The splice is located 900 mm above the top of the steel beam used to support the floor.

The influence of possible bending moments is checked with the use of NEN-EN 1993-1-1 and the national annex. For Lateral torsional buckling the critical bending moment (M_{cr}) is required in order to be able to calculate the reduction factor. $M_{cr} = k_{red} x C/L_g x \sqrt{(E x I_z x G x I_t)}$. For all of the used sections the value for k_{red} is 1,0. C depends on the support conditions and the type of loading. L_g is the distance between 2 lateral restraints (in this case equal to the length of the column). The factor C is calculated with the following formula:

$C = \pi x C_1 x L_g/L_{LTB}$ x $(\sqrt{(1+(\pi^2 x S^2/L_{LTB}^2 \text{ x } (C_2^2+1)))}+\pi x C_2 x S/L_{LTB})$ ($L_{LTB} = L_g$ in this case)

$S = \sqrt{(E x I_w/(G x I_t))}$ with $I_w = I_z x h^2/4$ for H-sections

C_1 and C_2 are dependent on the load. For a simply supported beam with an end moment at one side only (Figure 5-7) C_1 becomes 1,75 and C_2 is 0. All factors are now known and the critical bending moment can be rewritten to a formula only dependent on the section properties and the column length:

$C = 1{,}75 x \pi x \sqrt{(1+\pi^2 x S^2/L^2)}$

$M_{cr} = 1{,}75 x \pi x \sqrt{(1+\pi^2 x S^2/L^2)}\ /L x \sqrt{(E x I_z x G x I_t)}$

Only bending moments about the strong axis of the sections are considered because bending moments about the weak axis are very small (only small differences in beam reaction forces occur at the columns for bending about the weak axis). The differences between the beam reaction forces at the strong axis of the columns are much larger. These differences are at most 500 kN for internal columns and 800 kN for perimeter columns. The eccentricity of the connection depends on the section used. In all cases the eccentricity is assumed to be the height of the section divided by 2 + 120 mm. Because the difference in force is dependent on the location of the column and the relevant floor the bending moments are different for all sections. For all sections the eccentricities and the maximum force and bending moment are shown in Table 8-14 and Table 8-15.

Section	e*	Force	Moment	Section	e*	Force	Moment
	[mm]	[kN]	[kNm]		[mm]	[kN]	[kNm]
SHS600x100	420	1000	420	SHS600x80	420	1000	420
UC356x406x634 + 2x100PL	357	500	179	HD400x677 + 2x55PL	362	500	181
UC356x406x634 + 2x75PL	357	315	112	HD400x677 + 2x45PL	362	315	114
UC356x406x634 + 2x60PL	357	300	107	HD400x677 + 2x40PL	362	300	109
UC356x406x634 + 2x50PL	357	270	96	HD400x677 + 2x30PL	362	270	98
UC356x406x634 + 2x40PL	357	200	71	HD400x677 + 2x20PL	362	200	72
UC356x406x634	357	300	107	HD400x509	343	300	103
UC356x406x551	348	700	244	HD400x463	338	700	237
UC356x406x393 + 2x50PL	330	220	73	HD400x382 +2x40PL	328	220	72
UC356x406x393 + 2x40PL	330	700	231	HD400x382 +2x25PL	328	700	230
UC356x406x340	323	300	97	HD400x287	317	300	95

Table 8-14: Additional bending moments current (left) and new design (right) "long" columns

Section	e*	Force	Moment
	[mm]	[kN]	[kNm]
UC356x406x634 + 2x100PL	357	930	332
UC356x406x634 + 2x75PL	357	885	316
UC356x406x634 + 2x60PL	357	430	154
UC356x406x634 + 2x50PL	357	500	179
UC356x406x634 + 2x40PL	357	220	79
UC356x406x634 + 2x30PL	357	300	107
UC356x406x634 + 2x20PL	357	700	250
UC356x406x634	357	200	71
UC356x406x551	348	900	313
UC356x406x467	338	900	304
UC356x406x393 + 2x50PL	330	300	99
UC356x406x393 + 2x40PL	330	700	231
UC356x406x393 + 2x30PL	330	400	132
UC356x406x393	330	400	132
UC356x406x340	323	500	162
UC356x406x287	317	1400	444
UC356x406x235	310	700	217
UC356x368x202	307	100	31
UC305x305x240	296	800	237
UC305x305x198	290	700	203
UC305x305x158	284	400	114
UC254x254x167	265	700	186
UC254x254x132	258	600	155
UC203x203x86	231	650	150

Section	e*	Force	Moment
	[mm]	[kN]	[kNm]
HD400x677 + 2x40PL	362	930	337
HD400x677 + 2x30PL	362	885	320
HD400x677 + 2x20PL	362	430	156
HD400x818	377	500	189
HD400x744	369	220	81
HD400x634	357	300	107
HD400x592	353	700	247
HD400x463	338	200	68
HD400x382	328	900	295
HD400x347	324	900	292
HD400x551	348	300	104
HD400x509	343	700	240
HD400x463	338	400	135
HD400x287	317	400	127
HD400x262	323	500	162
HD400x216	317	1400	444
HD400x187	314	700	220
HD360x162	302	100	30
HD360x147	300	800	240
HD360x134	298	700	209
HD360x134	298	400	119
HD320x127	280	700	196
HD320x97,6	275	600	165
HD260x68,2	245	650	159

Table 8-15: Additional bending moments current (left) and new design (right) "short" columns

For all H-sections reduction factors are required to determine the bending moment resistance of the columns. The calculation of the reduction factor is similar to the one for buckling. For all H-sections used in the current and new design buckling curve a is used. Stronger steel grades in this case do not give the advantage of using a smaller imperfection factor. The bending moment resistance can be calculated with:

- $M_{cr} = 1{,}75 x \pi x \sqrt{(1+\pi^2 x S^2/L^2)} / L x \sqrt{(E x I_z x G x I_t)}$
- $\lambda_{rel} = \sqrt{(M_{pl,Rd}/M_{cr})}$ — $M_{el,Rd}$ is used for class 3 sections
- $\phi = 0{,}5x(1+\alpha x(\lambda_{rel}-0{,}2)+\lambda_{rel}^2)$ — α is the imperfection factor for curve a (0,21)
- $\chi_{LTB} = 1/(\phi+\sqrt{(\phi^2-\lambda_{rel}^2)})$
- $M_{b,rd} = \chi_{LTB} x M_{pl,Rd}/\gamma_{M1}$ — $\gamma_{M1} = 1$

For all sections the reduction factors and the reduced bending moment resistances are shown in Table 8-16 for "H-sections in the current design and in Table 8-17 for H-sections in the new design.

Section	Mpl,Rd	Length	Mcr	λ	ϕ	χ	Mb,Rd
	[kNm]	[mm]	[kNm]				[kNm]
UC356x406x634	4770	9820	28437	0,410	0,606	0,950	4533
UC356x406x634	4770	5380	57359	0,288	0,551	0,980	4676
UC356x406x551	4047	9820	21554	0,433	0,618	0,944	3819
UC356x406x551	4047	5410	43704	0,304	0,557	0,976	3952
UC356x406x467	3350	7000	23233	0,380	0,591	0,958	3209
UC356x406x393	2754	7000	16805	0,405	0,603	0,952	2621
UC356x406x340	2345	9820	8486	0,526	0,672	0,916	2148
UC356x406x340	2345	7000	12890	0,426	0,615	0,946	2217
UC356x406x287	2063	7000	9529	0,465	0,636	0,935	1929
UC356x406x235	1664	7000	6759	0,496	0,654	0,925	1540
UC356x368x202	1410	6890	5089	0,526	0,673	0,916	1291
UC305x305x240	1508	6890	5982	0,502	0,658	0,924	1393
UC305x305x198	1221	7000	4109	0,545	0,685	0,910	1111
UC305x305x158	951	6890	2768	0,586	0,712	0,895	852
UC254x254x167	861	6890	2822	0,552	0,689	0,907	781
UC254x254x132	663	6890	1804	0,606	0,727	0,888	589
UC203x203x86	347	6890	734	0,687	0,787	0,854	296

Table 8-16: Reduced bending moment resistance current design

Section	Mpl,Rd	Length	Mcr	λ	ϕ	χ	Mb,Rd
	[kNm]	[mm]	[kNm]				[kNm]
HD400x818	8667	5380	93629	0,304	0,557	0,976	8463
HD400x744	7727	5380	77789	0,315	0,562	0,974	7525
HD400x634	6541	5380	57430	0,337	0,571	0,969	6336
HD400x592	6044	4350	66686	0,301	0,556	0,977	5907
HD400x551	5543	5380	43985	0,355	0,579	0,964	5345
HD400x509	5074	9820	18494	0,524	0,671	0,917	4651
HD400x509	5074	5410	37806	0,366	0,585	0,961	4878
HD400x463	4544	9820	15332	0,544	0,684	0,910	4134
HD400x463	4544	5380	31887	0,377	0,590	0,959	4356
HD400x463	4544	3950	49152	0,304	0,557	0,977	4437
HD400x382	3664	5410	22457	0,404	0,603	0,952	3487
HD400x347	3284	7000	13384	0,495	0,654	0,926	3040
HD400x287	2674	9820	6204	0,657	0,763	0,867	2319
HD400x287	2674	7000	9560	0,529	0,674	0,915	2447
HD400x262	2420	7000	8134	0,545	0,685	0,910	2201
HD400x216	1961	7000	5854	0,579	0,707	0,898	1760
HD400x187	1505	7000	4585	0,573	0,703	0,900	1354
HD360x162	1303	6890	3550	0,606	0,726	0,888	1157
HD360x147	1183	6890	3056	0,622	0,738	0,881	1043
HD360x134	1073	7000	2573	0,646	0,755	0,872	935
HD360x134	1073	6890	2639	0,638	0,749	0,875	939
HD320x127	989	6890	1878	0,725	0,818	0,835	826
HD320x97,6	680	6890	1225	0,745	0,835	0,826	562
HD260x68,2	385	6890	575	0,818	0,899	0,785	302

Table 8-17: Reduced bending moment resistance new design

The combination of compression and bending is checked with the use of NEN-EN 1993-1-1 6.3.3 which describes the following formulas:

$N_{Ed}/(\chi_y x N_{Rk}) + k_{yy} x M_{Ed}/(\chi_{LT} x M_{Rk}) \leq 1$ $N_{Ed}/(\chi_z x N_{Rk}) + k_{zy} x M_{Ed}/(\chi_{LT} x M_{Rk}) \leq 1$

The factors k_{yy} and k_{zy} are dependent on the shape of the bending moment diagram, the slenderness of the member and the unity check for buckling. For box sections the factor k_{zy} = 0 and the factor k_{yy} is calculated with: $k_{yy} = C_{my} x(1+(\lambda_y-0{,}2)x\ N_{Ed}/(\chi_y x N_{Rk})) \leq C_{my} x(1+0{,}8x\ N_{Ed}/(\chi_y x N_{Rk}))$

For class 1 and 2 H-sections: $k_{yy} = C_{my} x(1+(\lambda_y-0{,}2)x\ N_{Ed}/(\chi_y x N_{Rk})) \leq C_{my} x(1+0{,}8x\ N_{Ed}/(\chi_y x N_{Rk}))$

$k_{zy} = 1-0{,}1x\lambda_z/(C_{mLT}-0{,}25)x\ N_{Ed}/(\chi_z x N_{Rk}) \geq 1-0{,}1/(C_{mLT}-0{,}25)x\ N_{Ed}/(\chi_z x N_{Rk})$

if $\lambda_z < 0{,}4$ $k_{zy} = 0{,}6+\lambda_z \leq 1-0{,}1x\lambda_z/(C_{mLT}-0{,}25)x\ N_{Ed}/(\chi_z x N_{Rk})$

For class 3 and 4 H-sections: $k_{yy} = C_{my} x(1+0{,}6x\lambda_y x\ N_{Ed}/(\chi_y x N_{Rk})) \leq C_{my} x(1+0{,}6x\ N_{Ed}/(\chi_y x N_{Rk}))$

$k_{zy}=1-0{,}05x\lambda_z/(C_{mLT}-0{,}25)x\ N_{Ed}/(\chi_z x N_{Rk})\geq 1-0{,}05/(C_{mLT}-0{,}25)x\ N_{Ed}/(\chi_z x N_{Rk})$

In all cases the factors C_{my} and C_{mLT} are equal to 0,6 which corresponds to a beam loaded by a bending moment at one end only.

All sections used in the current design are class 1 sections. This is however not the case for the sections used in the new design. The use of higher strength steel results in smaller allowable width/thickness ratios for all classes. Because of these reduced ratios the amount of class 1 and 2 sections is reduced for S460 as opposed to S355. The sections mentioned below are class 3 sections (used in the new design only). All other sections are class 1 sections.

- HD400x187
- HD360x162
- HD360x147
- HD360x134
- HD320x97,6
- HD260x68,2

Because the occurring bending moments, the bending moment resistance and all buckling parameters are known all the relevant factors (k_{yy} and k_{zy}) can be calculated. With the use of these factors the maximum allowable compressive force is determined. This has been done with a number of iterations because all the parameters depend on this compressive force.

For all sections used in the current design the maximum allowable forces are shown in Table 8-18 for “short” columns and in Table 8-19 for “long” columns.

For the sections used in the new design these forces are shown in Table 8-20 for “short” columns and in Table 8-21 for “long” columns.

The percentage shown next to the maximum allowable force displays the percentage of buckling resistance that remains when the bending moment is applied.

Section	kyy	kzz	Nb,Rd	Nmax	Relative
			[kN]	[kN]	[%]
UC356x406x634 + 2x100PL	0,730	0,000	48442	46870	96,8
UC356x406x634 + 2x75PL	0,748	0,000	44073	42728	96,9
UC356x406x634 + 2x60PL	0,728	0,000	40160	40160	100,0
UC356x406x634 + 2x50PL	0,723	0,000	37097	37097	100,0
UC356x406x634 + 2x40PL	0,717	0,000	34015	34015	100,0
UC356x406x634 + 2x30PL	0,710	0,000	30900	30899	100,0
UC356x406x634 + 2x20PL	0,665	0,000	29435	29434	100,0
UC356x406x634	0,683	0,825	20916	20652	98,7
UC356x406x551	0,684	0,831	17977	16793	93,4
UC356x406x467	0,724	0,779	12778	11833	92,6
UC356x406x393 + 2x50PL	0,755	0,000	26435	26059	98,6
UC356x406x393 + 2x40PL	0,746	0,000	23782	23219	97,6
UC356x406x393 + 2x30PL	0,673	0,000	22738	22712	99,9
UC356x406x393	0,734	0,768	10619	10208	96,1
UC356x406x340	0,734	0,769	9100	8590	94,4
UC356x406x287	0,722	0,792	7861	6428	81,8
UC356x406x235	0,736	0,771	6368	5676	89,1
UC356x368x202	0,743	0,736	5257	5164	98,2
UC305x305x240	0,729	0,751	5220	4553	87,2
UC305x305x198	0,731	0,754	4152	3580	86,2
UC305x305x158	0,739	0,743	3308	2980	90,1
UC254x254x167	0,739	0,766	2862	2341	81,8
UC254x254x132	0,739	0,772	2205	1757	79,7
UC203x203x86	0,713	0,835	1014	584	57,6

Table 8-18: Maximum allowable compressive forces “short” columns current design

Section	kyy	kzz	Nb,Rd	Nmax	Relative
			[kN]	[kN]	[%]
SHS600x100	0,844	0,000	51142	50568	98,9
UC356x406x634 + 2x100PL	0,965	0,000	37531	36664	97,7
UC356x406x634 + 2x75PL	0,970	0,000	34617	34088	98,5
UC356x406x634 + 2x60PL	0,951	0,000	30567	30566	100,0
UC356x406x634 + 2x50PL	0,929	0,000	27563	27562	100,0
UC356x406x634 + 2x40PL	0,905	0,000	24529	24529	100,0
UC356x406x634	0,765	0,719	12639	12424	98,3
UC356x406x551	0,765	0,728	10768	10268	95,4
UC356x406x393 + 2x50PL	1,012	0,000	19508	19146	98,1
UC356x406x393 + 2x40PL	0,954	0,000	18234	17937	98,4
UC356x406x340	0,780	0,724	6290	6085	96,7

Table 8-19: Maximum allowable compressive forces “long” columns current design

Section	kyy	kzz	Nb,Rd	Nmax	Relative
			[kN]	[kN]	[%]
HD400x677 + 2x40PL	0,756	0,000	49186	48763	99,1
HD400x677 + 2x30PL	0,767	0,000	43890	43889	100,0
HD400x677 + 2x20PL	0,741	0,000	40387	40386	100,0
HD400x818	0,710	0,805	39889	39172	98,2
HD400x744	0,715	0,800	36071	35758	99,1
HD400x634	0,723	0,795	31027	30609	98,7
HD400x592	0,684	0,837	30998	29912	96,5
HD400x463	0,734	0,788	22295	22022	98,8
HD400x382	0,733	0,795	18225	16998	93,3
HD400x347	0,777	0,734	13536	12582	93,0
HD400x551	0,728	0,792	26736	26321	98,4
HD400x509	0,729	0,795	24605	23642	96,1
HD400x463	0,675	0,846	24605	23970	97,4
HD400x287	0,787	0,725	11056	10640	96,2
HD400x262	0,785	0,730	10036	9499	94,6
HD400x216	0,761	0,770	8171	6584	80,6
HD400x187	0,749	0,878	6974	5980	85,7
HD360x162	0,760	0,860	5773	5642	97,7
HD360x147	0,730	0,886	5222	4156	79,6
HD360x134	0,732	0,885	4628	3714	80,3
HD360x134	0,746	0,873	4721	4198	88,9
HD320x127	0,742	0,766	3264	2670	81,8
HD320x97,6	0,706	0,895	2478	1826	73,7
HD260x68,2	0,683	0,927	1362	695	51,0

Table 8-20: Maximum allowable compressive forces “short” columns new design

Section	kyy	kzz	Nb,Rd	Nmax	Relative
			[kN]	[kN]	[%]
SHS600x80	0,906	0,000	51546	51383	99,7
HD400x677 + 2x55PL	1,011	0,000	38306	37517	97,9
HD400x677 + 2x45PL	0,994	0,000	34691	34691	100,0
HD400x677 + 2x40PL	0,992	0,000	33679	33678	100,0
HD400x677 + 2x30PL	0,952	0,000	29151	29150	100,0
HD400x677 + 2x20PL	0,909	0,000	24558	24558	100,0
HD400x509	0,807	0,719	13119	12909	98,4
HD400x463	0,804	0,726	11712	11225	95,8
HD400x382 +2x40PL	1,073	0,000	20829	20506	98,4
HD400x382 +2x25PL	1,012	0,000	18390	18389	100,0
HD400x287	0,819	0,723	6865	6661	97,0

Table 8-21: Maximum allowable compressive forces “long” columns new design

<table>
<tr><td>51</td><td colspan="2">UC356x406x634 + 2x60PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
<tr><td>50</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393</td></tr>
<tr><td>49</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x340</td><td colspan="2">UC356x406x340</td></tr>
<tr><td>48</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x551</td></tr>
<tr><td>47</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x551</td></tr>
<tr><td>46</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x634</td></tr>
<tr><td>45</td><td colspan="2">UC356x406x634 + 2x20PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x467</td><td colspan="2">UC356x406x467</td></tr>
<tr><td>44</td><td colspan="2">UC356x406x634 + 2x20PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td></tr>
<tr><td>43</td><td colspan="2">UC356x406x634 + 2x20PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
<tr><td>42</td><td colspan="2">UC356x406x634 + 2x20PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
<tr><td>41</td><td colspan="2">UC356x406x634 + 2x20PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x393</td><td colspan="2">UC356x406x551</td></tr>
<tr><td>40</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x634 + 2x30PL</td><td colspan="2">UC356x406x393 + 2x30PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x393 + 2x50PL</td></tr>
<tr><td>38</td><td colspan="2">UC356x406x634 + 2x60PL</td><td colspan="2">UC356x406x634 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x393 + 2x50PL</td></tr>
<tr><td>37</td><td colspan="2">UC356x406x634 + 2x75PL</td><td colspan="2">UC356x406x634 + 2x30PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td></tr>
<tr><td>28</td><td colspan="2">UC356x406x634 + 2x60PL</td><td colspan="2">UC356x406x634 + 2x30PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td></tr>
<tr><td>27</td><td colspan="2">UC356x406x634 + 2x30PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
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<tr><td>18</td><td colspan="2">UC356x406x634 + 2x100PL</td><td colspan="2">UC356x406x634 + 2x50PL</td><td colspan="2">UC356x406x634 + 2x50PL</td><td colspan="2">UC356x406x634 + 2x30PL</td><td colspan="2">UC356x406x393 + 2x30PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
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<tr><td>13</td><td colspan="2">UC356x406x634 + 2x75PL</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x634</td></tr>
<tr><td>11</td><td colspan="2">SHS600x100</td><td colspan="2">UC356x406x634 + 2x60PL</td><td colspan="2">UC356x406x634 + 2x60PL</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x634 + 2x30PL</td><td colspan="2">UC356x406x634 + 2x50PL</td></tr>
<tr><td>9</td><td colspan="2">UC356x406x634 + 2x60PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x467</td></tr>
<tr><td>7</td><td colspan="2">UC356x406x634 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x634</td><td colspan="2">UC356x406x551</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
<tr><td>4</td><td colspan="2">UC356x406x634 + 2x100PL</td><td colspan="2">UC356x406x634 + 2x50PL</td><td colspan="2">UC356x406x634 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x50PL</td><td colspan="2">UC356x406x393 + 2x40PL</td><td colspan="2">UC356x406x393 + 2x40PL</td></tr>
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<tr><td>Column</td><td>Ground</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td>6</td><td>7</td><td>8</td><td>9</td><td>10</td><td>11</td><td>12</td><td>12M</td></tr>
<tr><td>Height</td><td>5,760</td><td>5,700</td><td>5,020</td><td>5,380</td><td>5,200</td><td>3,950</td><td>3,950</td><td>3,950</td><td>3,950</td><td>3,950</td><td>3,950</td><td>4,350</td><td>5,410</td><td>4,410</td></tr>
</table>

51	UC356x406x467		UC356x406x467		UC356x406x393		UC356x406x287		UC356x406x235		UC254x254x132	
50	UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x202	
49	UC356x406x235		UC356x406x235		UC356x406x202		UC356x406x202		UC305x305x198		UC356x406x202	
48	UC356x406x287		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x202	
47	UC356x406x287		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x202	
46	UC356x406x287		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x202	
45	UC356x406x340		UC356x406x287		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x202	
44	UC356x406x393		UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x235		UC356x406x202	
43	UC356x406x467		UC356x406x340		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x202	
42	UC356x406x467		UC356x406x393		UC356x406x340		UC356x406x235		UC356x406x235		UC356x406x202	
41	UC356x406x287		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x235		UC356x406x202	
40	UC356x406x551		UC356x406x393		UC356x406x393		UC356x406x340		UC356x406x287		UC305x305x198	
38	UC356x406x467		UC356x406x467		UC356x406x393		UC356x406x287		UC356x406x287		UC305x305x158	
37	UC356x406x393		UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x235		UC356x406x202	
28	UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x202	
27	UC356x406x467		UC356x406x467		UC356x406x340		UC356x406x235		UC356x406x235		UC203x203x86	
25	UC356x406x551		UC356x406x551		UC356x406x551		UC356x406x467		UC356x406x340		UC305x305x158	
23	UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x202	
22	UC356x406x393		UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x235		UC254x254x167	
20	UC356x406x634		UC356x406x634		UC356x406x551		UC356x406x393		UC356x406x287		UC254x254x132	
18	UC356x406x551		UC356x406x393		UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x202	
15	UC356x406x551		UC356x406x551		UC356x406x467		UC356x406x393		UC356x406x393		UC305x305x240	
13	UC356x406x467		UC356x406x467		UC356x406x393		UC356x406x287		UC356x406x235		UC356x406x202	
11	UC356x406x634		UC356x406x634		UC356x406x551		UC356x406x467		UC356x406x340		UC254x254x167	
9	UC356x406x393		UC356x406x393		UC356x406x287		UC356x406x235		UC356x406x235		UC356x406x202	
7	UC356x406x393		UC356x406x393		UC356x406x340		UC356x406x287		UC356x406x235		UC254x254x167	
4	UC356x406x551		UC356x406x551		UC356x406x467		UC356x406x287		UC356x406x287			
2	UC356x406x634		UC356x406x467		UC356x406x467		UC356x406x340		UC356x406x287			
1	UC356x406x393 + 2x50PL		UC356x406x393 + 2x40PL		UC356x406x634		UC356x406x551		UC356x406x467			
Column	13	14	15	16	17	18	19	20	21	22	23	24
Height	4,010	3,950	3,950	3,950	3,950	3,950	3,950	4,350	6,040	7,000	5,890	0,000

8.4 Appendix IV

The resistances of the connections are calculated in this appendix. There are 4 types of connections used:

- Box section – box section (welded connection)
- Box section – H-section (welded connection)
- H-section – H-section (welded and bolted connection) (t_f > 50 mm)
- H-section – H-section (bolted connection)

In all cases the axial force is transferred by bearing. Therefore the connections do not have to be designed with the use of the entire axial force. In all cases the connections should be able to resist the following forces:

- $0{,}25xN_{Ed} + M_{y,Ed} + M_{z,Ed}$ (results in forces or stresses dependent on the type of connection)
- $M = 0{,}25xM_{y,Rd}$
- $M = 0{,}25xM_{z,Rd}$
- $V = 0{,}025xN_{Rd}$

Box section – Box section

Bolted connections are not an option in this case so the splice has to be welded. The strength of the welds depend on the thickness of the thickest plate. Plates with a thickness exceeding 40 mm have a reduced tensile strength and therefore the strength of the welds will also be reduced. The splice consists of 2 box sections welded on site. The size of the weld is dependent on the maximum stress that can occur due to the different forces. The maximum stress is the maximum value of:

- $0{,}25xN_{Ed}/A + M_{y,Ed}/W_{y,el} + M_{z,Ed}/W_{z,el}$
- $0{,}25xM_{y,Rd}/W_{y,el}$
- $0{,}25xM_{z,Rd}/W_{z,el}$
- $0{,}025xN_{Rd}/\min(2xb_fxt_f\ ;\ 2xb_pxt_p)$ (f = flange, p = plate)

The last stress is a shear stress. A reduced strength should be applied for shear stresses. They are however not governing in any of the designs. The cross-sectional properties are based on the flanges and the welded plates only. The web cannot be welded so it is excluded from the calculation of the occurring stresses.

When the maximum stress is known the required weld size can be calculated. The maximum allowable weld stress is in all cases: $\sigma_{max} = 0{,}9xf_u/\gamma_{M2}$ which results in $0{,}72xf_u$. For plates with a thickness up to 40 mm a maximum stress of 388,8 N/mm^2 is allowed. For thicker plates the tensile strength is 530 N/mm^2 and the maximum stress in the weld is 381,6 N/mm^2. The stress in the weld can be calculated by using the thickness of the flange or plate and the size in the weld. The stress in the weld is equal to:

$\sigma_{weld} = tx\sigma/v$

t is the thickness of the flange or plate. σ is the maximum design stress for the welds and v is the effective throat size of the welds.

If full contact between the top and bottom column is not possible an additional plate is welded to the bottom column. The thickness of the plate is such that the stresses are transferred with an angle of 45˚ to the column below.

Box section – H-section

The design of the welds is similar to the method used for the box section – box section connections, however an additional side plate is added to the H-section so the entire force can be transferred through bearing. The force present in the side plates of the box section is transferred entirely through the additional side plate of the H-section. This force is determined with the use of the maximum stress in the section (without reducing the compressive force):

- $N_{Ed}/A + M_{y,Ed}/W_{y,el} + M_{z,Ed}/W_{z,el}$

By multiplying the compressive stress with the area of the side plate ($A = b_p x t_p$) the maximum force in the side plate is obtained. The plate is checked for a shear force and a bending moment. The shear force is equal to F/2 and the bending moment is $Fxb_p/4$. The welds connecting the plates to the flanges are checked for the shear force only.

H-section – H-section (t_f > 50 mm)

For H-sections with flanges thicker than 50 mm it was chosen to weld plates between the flanges parallel to the web and to connect these plates with bolts and splice plates. The plates are welded with the use of half V-welds. For each welded plate 2 splice plates are used. In total 2 welded plates are used per splice. The maximum compressive force in the welded plates is the maximum value of:

- $0{,}25xN_{Ed}/2 + M_{y,Ed}/(2xz_y) + M_{z,Ed}/z_z$
- $0{,}25xM_{y,Rd}/(2xz_y)$
- $0{,}25xM_{z,Rd}/z_z$

z_y is the lever for bending about the strong axis ($z_y = (h-t_f)/2$) and z_z is the lever for bending about the weak axis. z_z is in all cases the center to center distance of the welded plates. The maximum tensile force in the welded plates is the maximum value of these values excluding the one with the design loads.

The shear force is also of importance in this type of connection. The design shear force per weld plate is determined with: $V = 0{,}025\ N_{Rd}/2$.

The bolts are all loaded in 2 shear planes. A reduction factor does not have to be applied because the maximum amount of bolts in a row is 5.

The splice plates are checked for the maximum compressive and tensile force using the total cross-section for the compressive force and the net section for the tensile force. The shear force causes a bending moment: M = Vxe1 at the location of the first bolts. The net section at the bolts is checked for the shear force and the bending moment.

The welded plates are checked at the locations of the welds and the bolts. A combination of shear force and bending moment occurs in both sections. The bending moment M = Fxe2 is divided by ratio to a bending moment at the welds and at the bolts. By using the net section at the bolts the bending moment is checked.

The shear force is multiplied by a factor to take account of the eccentricity. The factor is dependent on the connection length and the end distance: $(L_{connection} + e1)/L_{connection}$. The increased shear force also results in a bending moment equal to: M = Fxe2. The combination of shear force and bending moment is checked at the welds and at the bolts. At the welds only a shear force occurs.

The welds are half V-welds. The stress in the welds depend on combined stress in the weld plates. The stress in the weld is calculated with:

$\sigma_{weld} = t x \sigma/(v x \sqrt{3})$

t is the thickness of the weld plates, σ is the combined stress in the weld plates and v is the effective throat thickness of the welds. The stress should be smaller than $f_{wud} = 249,4$ N/mm^2.

H-section – H-section

When the flanges of the H-sections are not thicker than 50 mm bolted splice plates are used. The use of splice plates outside the section is not possible due to the required free space. The flanges are therefore only connected with the use of splice plates inside the section. The web is connected with the use of 2 splice plates. The maximum force in the splice plates differs for the splice plates connecting the web and the ones connecting the flanges. The splice plates connecting the flanges are assumed to transfer all the bending moments, 70% of the axial force and the shear force in the weak direction. The splice plates connecting the web are assumed to transfer 30% of the axial force and the shear force in the strong direction.

The maximum force in the web and flanges is calculated with the following formulas:

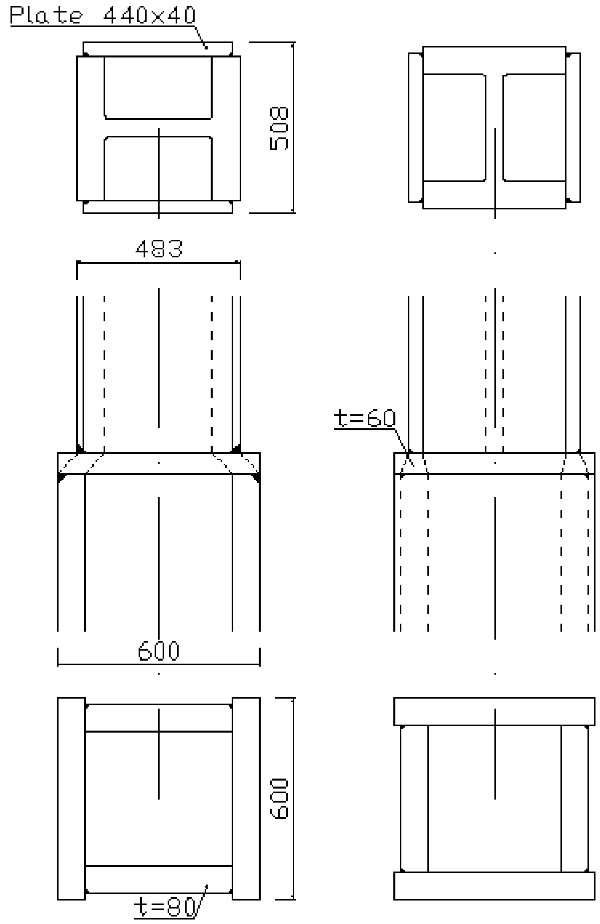
- $0,7x0,25xN_{Ed} + M_{y,Ed}/z_y + M_{z,Ed}/z_z$ (Flanges)
- $0,25xM_{y,Rd}/z_y$ (Flanges)
- $0,25xM_{z,Rd}/z_z$ (Flanges)
- $0,3x0,25xN_{Ed}$ (Web)

The shear force $V = 0,025xN_{Rd}$. $z_y = h-t_f$, $z_z = b/2$.

The bolts connecting the web are loaded in 2 shear planes. The bolts connecting the flanges are however loaded in single shear. It is therefore possible to assure that the shear plane is located at the shaft of the bolt instead of the thread. The shear resistance is therefore larger because a larger shear area may be used. The factor α_v is 0,6 for every strength class so the use of 10.9 bolts becomes more favorable in this case. The flanges are therefore connected with the use of 10.9 bolts.

The flange and web plates are checked for the maximum axial force and the shear force requirement. The shear force also results in a bending moment equal to M = Fxe1. The combination of shear force and bending moment is checked for the net section at the first bolts.

DETAIL 1: HD400X677 + 40PL (TOP) – SHS600X80 (BOTTOM)



Forces:

- N_{Ed} = 27350 kN
- $M_{y,Ed}$ = 69 kNm
- $M_{z,Ed}$ = 165 kNm

Capacities:

- N_{Rd} = 55909 kN
- $M_{ypl,Rd}$ = 8840 kNm
- $M_{zpl,Rd}$ = 7322 kNm

Section properties:

- A = 104964 mm^2
- $W_{y,el}$ = 14153365 mm^3
- $W_{z,el}$ = 11799509 mm^3
- t_f = 81,5 mm
- A_{fl} = 69764 mm^2
- A_{pl} = 35200 mm^2
- f_u = 530 N/mm^2
- f_{wud} = 244,8 N/mm^2
- σ_{max} = 381,6 N/mm^2

Maximum tensile stress in section:

- σ_{combi} = 0,25x27350x1000/104964 + (69/14153365 + 165/11799509)x1000000 = 84,0 N/mm^2
- σ_{My} = 0,25x8840x1000000/14153365 = **156,1 N/mm^2**
- σ_{Mz} = 0,25x7322x1000000/11799509 = 155,1 N/mm^2
- σ_V = 0,025x55909x1000/(2x440x40) = 39,7 N/mm^2

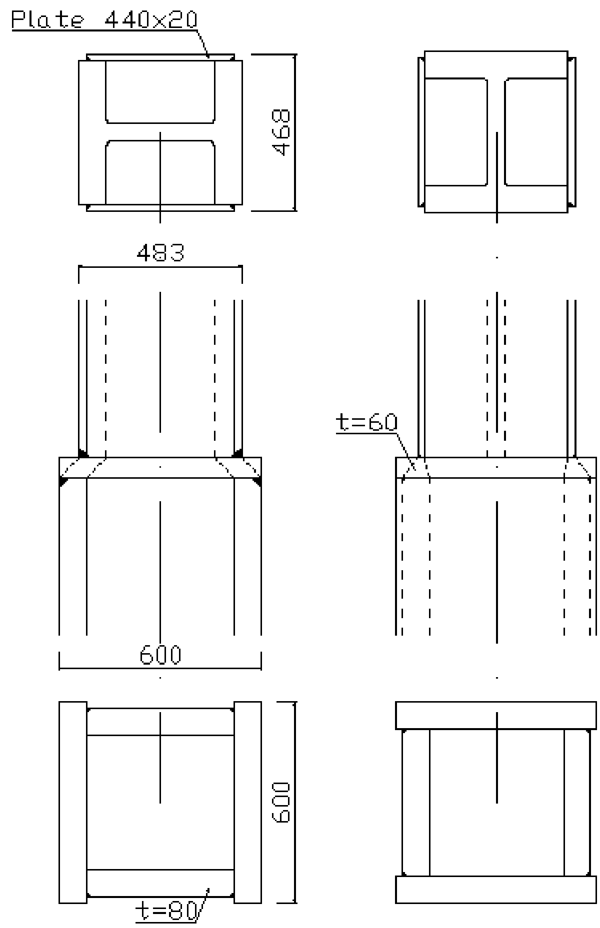
Welds (strong axis) v = 36 mm → 34 mm effective:

- σ_{weld} = 81,5x156,1/34 = 374,3 $N/mm^2 \leq$ 381,6 N/mm^2

Welds (weak axis) v = 19 mm → 17 mm effective:

- σ_{weld} = 40x156,1/17 = 367,4 $N/mm^2 \leq$ 381,6 N/mm^2

DETAIL 2: HD400X677 + 20PL (TOP) – SHS600X80 (BOTTOM)



Forces:

- N_{Ed} = 44682 kN
- $M_{y,Ed}$ = 26 kNm
- $M_{z,Ed}$ = 0 kNm

Capacities:

- N_{Rd} = 47813 kN
- $M_{ypl,Rd}$ = 7950 kNm
- $M_{zpl,Rd}$ = 5346 kNm

Section properties:

- A = 87346 mm^2
- $W_{y,el}$ = 12977602 mm^3
- $W_{z,el}$ = 8327585 mm^3
- t_f = 81,5 mm
- A_{fl} = 69764 mm^2
- A_{pl} = 17600 mm^2
- f_u = 530 N/mm^2
- f_{wud} = 244,8 N/mm^2
- σ_{max} = 381,6 N/mm^2

Maximum tensile stress in section:

- σ_{combi} = 0,25x44682x1000/87346 + (26/12977602 +0/8327585)x1000000 = 129,9 N/mm^2
- σ_{My} = 0,25x7950x1000000/12977602 = 153,1 N/mm^2
- σ_{Mz} = 0,25x5346x1000000/8327585 = **160,5 N/mm^2**
- σ_V = 0,025x44682x1000/(2x440x20) = 67,9 N/mm^2

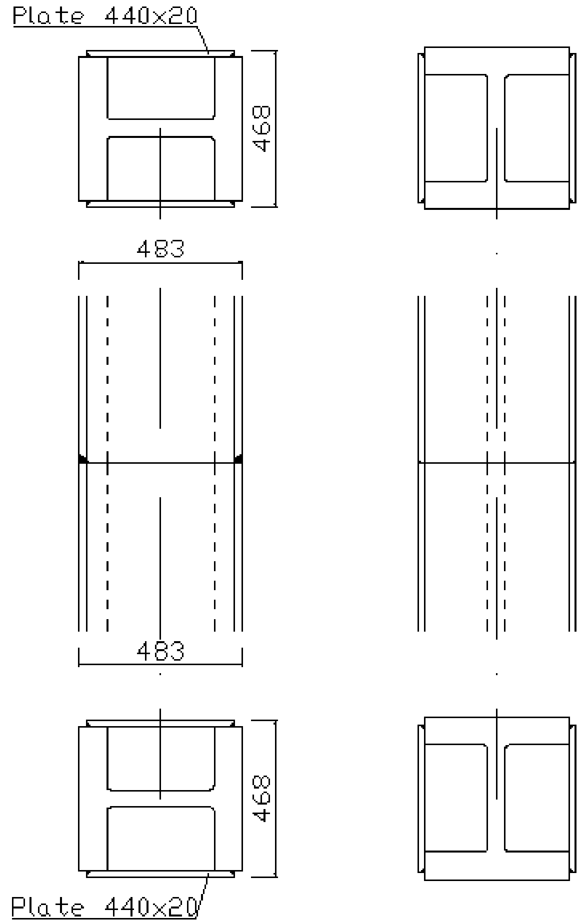
Welds (strong axis) v = 37 mm → 35 mm effective:

- σ_{weld} = 81,5x160,5/35 = 373,7 N/mm^2 ≤ 381,6 N/mm^2

Welds (weak axis) v =11 mm → 9 mm effective:

- σ_{weld} = 20x160,5/9 = 356,6 N/mm^2 ≤ 381,6 N/mm^2

DETAIL 3: HD400X677 + 20PL (TOP) – HD400X677 + 20PL (BOTTOM)



Forces:

- N_{Ed} = 41412 kN
- $M_{y,Ed}$ = 9 kNm
- $M_{z,Ed}$ = 70 kNm

Capacities:

- N_{Rd} = 47813 kN
- $M_{ypl,Rd}$ = 7950 kNm
- $M_{zpl,Rd}$ = 5346 kNm

Section properties:

- A = 87346 mm^2
- $W_{y,el}$ = 12977602 mm^3
- $W_{z,el}$ = 8327585 mm^3
- t_f = 81,5 mm
- A_{fl} = 69764 mm^2
- A_{pl} = 17600 mm^2
- f_u = 530 N/mm^2
- f_{wud} = 244,8 N/mm^2
- σ_{max} = 388,8 N/mm^2

Maximum tensile stress in section:

- σ_{combi} = 0,25x44682x1000/87346 + (9/12977602 +70/8327585)x1000000 = 127,6 N/mm^2
- σ_{My} = 0,25x7950x1000000/12977602 = 153,1 N/mm^2
- σ_{Mz} = 0,25x5346x1000000/8327585 = **160,5 N/mm^2**
- σ_V = 0,025x44682x1000/(2x440x20) = 67,9 N/mm^2

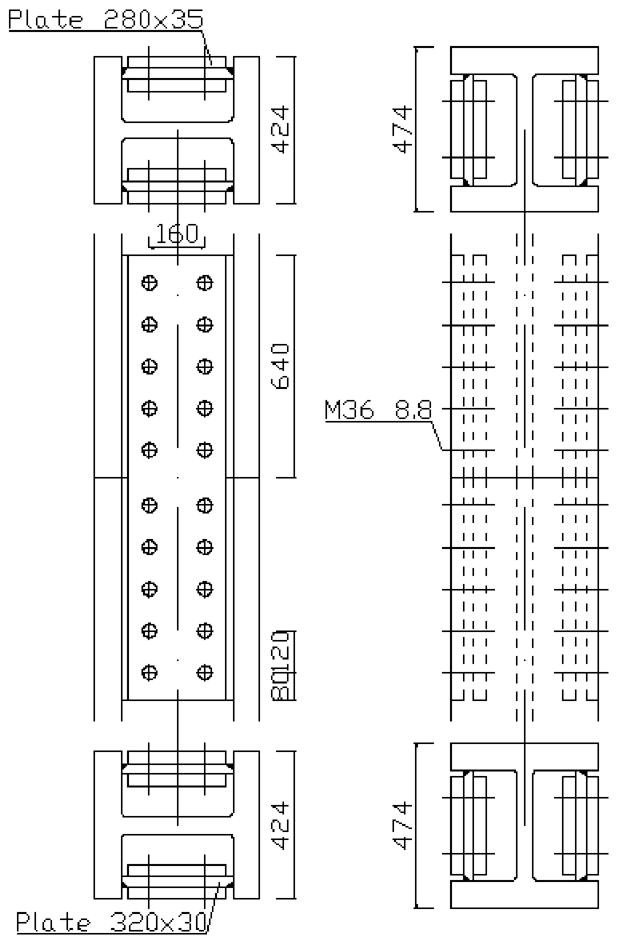
Welds (strong axis) v = 36 mm → 34 mm effective:

- σ_{weld} = 81,5x160,5/34 = 384,7 N/mm^2 ≤ 388,8 N/mm^2

Welds (weak axis) v = 11 mm → 9 mm effective:

- σ_{weld} = 20x160,5/9 = 356,6 N/mm^2 ≤ 388,8 N/mm^2

DETAIL 4: HD400X634 (TOP) – HD400X634 (BOTTOM)



Forces:

- N_{Ed} = 29693 kN
- $M_{y,Ed}$ = 85 kNm
- $M_{z,Ed}$ = 398 kNm

Capacities:

- N_{Rd} = 37168 kN
- $M_{ypl,Rd}$ = 6541 kNm
- $M_{zpl,Rd}$ = 3274 kNm

Max force in weld plates:

- $F_{c,Ed}$ = 5154 kN
- $F_{t,Ed}$ = 4117 kN
- V_{Ed} = 465 kN
- M_{weld} = 154,6 kNm
- M_{bolt} = 51,5 kNm

Bolts:

- $F_{v,u,d}$ = 2x10x313,7 = 6274 kN
- $F_{b,u,d}$ = 10x26,58x30 = 7974 kN

Splice plates:

- $F_{c,Rd}$ = 2x280x30x460/1000 = 7728 kN
- $F_{t,Rd}$ = 0,9x2x(280-2x39)x30x540/(1000x1,25) = 4712 kN
- Shear + bending check: $\sqrt{(3x(465x1000/(2x(280-2x39)x35))^2 + (465x80x1000/(2x(280-2x39)x30^2/6))^2)}$ = 454,1 N/mm^2 ≤ 460 N/mm^2

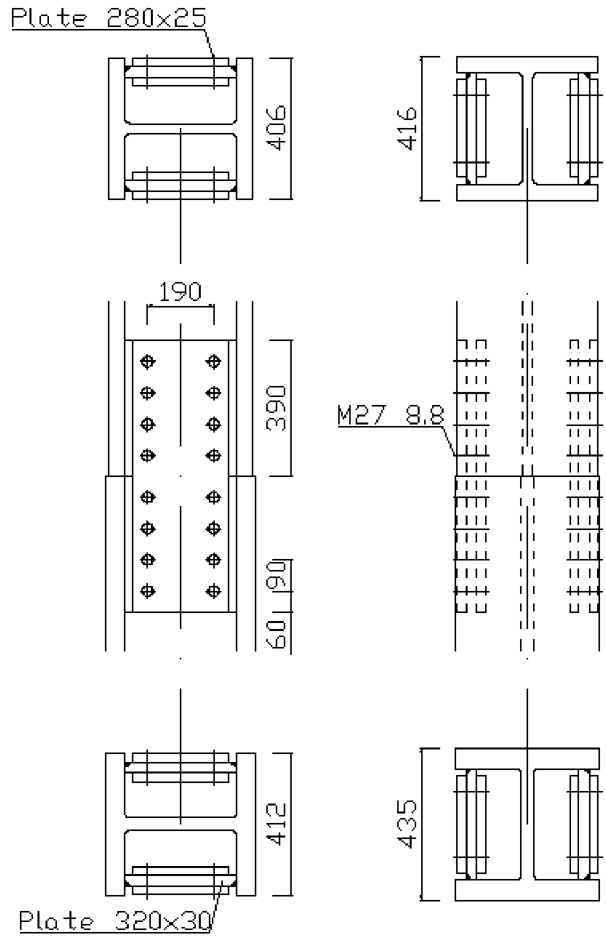
Weld plates:

- $\sigma_{combi,weld}$ = $\sqrt{((154,6x1000000/(30x640^2/6))^2 + 3x(5154x1000/(2x640x30))^2)}$ = 244,4 N/mm^2
- $\sigma_{combi,bolt}$ = $\sqrt{((51,5x1000000/(2x(30x640^3/12-5x30x39^3/12-2x30x39x(240^2+120^2))/640))^2 + 3x(5154x1000/(2x(640-5x39)x30))^2)}$ = **336,1 N/mm^2**
- $\sigma_{combi,weld,shear}$ = √3x464x1000x(480+80)/(2x480x640x30) = 24,4 N/mm^2
- $\sigma_{combi,bolt,shear}$ = $\sqrt{(((464x1000x(480+80)x80/480)/((640-5x39)x30^2/6))^2 + 3x((464x1000x(480+80)/480)/((640-5x39)x30))^2)}$ = 326,6 N/mm^2

Welds, f_{wud} =249,4 N/mm^2, v = 19 mm → 17 mm effective:

- σ_{weld} = 30x244,4/(17x√3) = 249,0 N/mm^2 ≤ 249,4 N/mm^2

DETAIL 5: HD400X382 (TOP) – HD400X463 (BOTTOM)



Forces:

- N_{Ed} = 20493 kN
- $M_{y,Ed}$ = 65 kNm
- $M_{z,Ed}$ = 13 kNm

Capacities:

- N_{Rd} = 22407 kN
- $M_{ypl,Rd}$ = 3664 kNm
- $M_{zpl,Rd}$ = 1854 kNm

Max force in weld plates:

- $F_{c,Ed}$ = 2778 kN
- $F_{t,Ed}$ = 2489 kN
- V_{Ed} = 280 kN
- M_{weld} = 71,9 kNm
- M_{bolt} = 18,3 kNm

Bolts:

- $F_{v,u,d}$ = 2x8x176,3 = 2821 kN
- $F_{b,u,d}$ = 8x19,44x30 = 4666 kN

Splice plates:

- $F_{c,Rd}$ = 2x280x20x460/1000 = 5152 kN
- $F_{t,Rd}$ = 0,9x2x(280-2x30)x20x540/(1000x1,25) = 3421 kN
- Shear + bending check: √(3x(280x1000/(2x(280-2x39)x25))^2 + (280x80x1000/(2x(280-2x39)x30^2/6))^2) = 369,2 N/mm^2 ≤ 460 N/mm^2

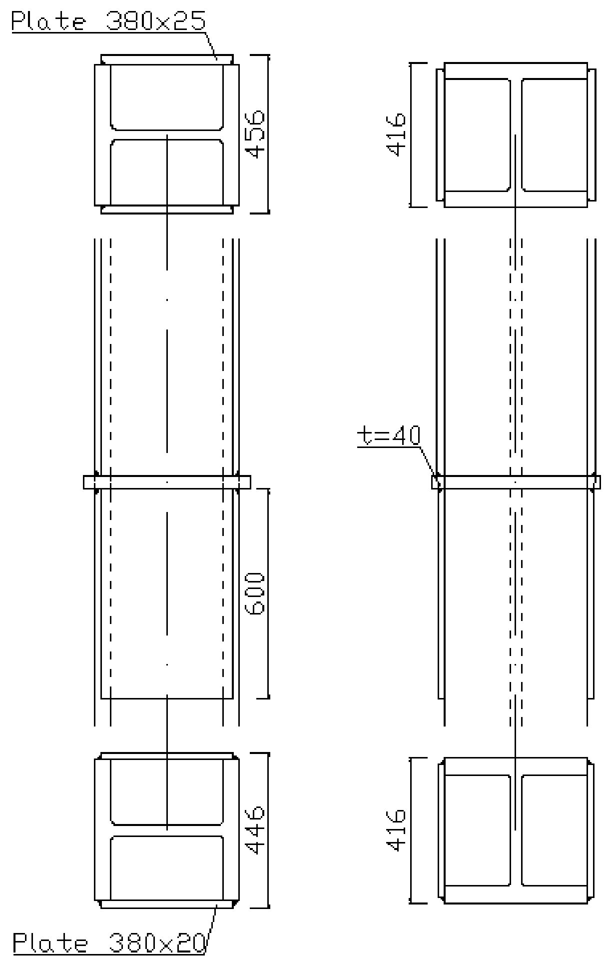
Weld plates:

- $\sigma_{combi,weld}$ = √((71,9x1000000/(30x390^2/6))^2 + 3x(2778x1000/(2x390x30))^2) = 226,4 N/mm^2
- $\sigma_{combi,bolt}$ = √((18,3x1000000/(2x(30x390^3/12-4x30x30^3/12-2x30x30x(135^2+45^2))/390))^2 + 3x(2778x1000/(2x(390-4x30)x30))^2) = **298,7 N/mm^2**
- $\sigma_{combi,weld,shear}$ = √3x280x1000x(270+60)/(2x270x390x30) = 25,3 N/mm^2
- $\sigma_{combi,bolt,shear}$ = √(((280x1000x(270+60)x60/270)/((390-4x30)x30^2/6))^2 + 3x((280x1000x(270+60)/270)/((390-4x30)x30))^2) = 277,0 N/mm^2

Welds, f_{wud} =249,4 N/mm^2, v = 18 mm → 16 mm effective:

- σ_{weld} = 30x226,4/(16x√3) = 245,0 N/mm^2 ≤ 249,4 N/mm^2

DETAIL 6: HD400X382 (TOP) – HD400X382 +25PL (BOTTOM)



Forces:

- N_{Ed} = 17499 kN
- $M_{y,Ed}$ = 195 kNm
- $M_{z,Ed}$ = 13 kNm

Capacities:

- N_{Rd} = 22407 kN
- $M_{ypl,Rd}$ = 3664 kNm
- $M_{zpl,Rd}$ = 1854 kNm

Section properties:

- A = 57976 mm^2
- $W_{y,el}$ = 7479270 mm^3
- $W_{z,el}$ = 6222551 mm^3
- t_f = 48 mm
- A_{fl} = 38976 mm^2
- A_{pl} = 19000 mm^2
- f_u = 540 N/mm^2
- f_{wud} = 249,4 N/mm^2
- σ_{max} = 388,8 N/mm^2

Maximum tensile stress in section:

- σ_{combi} = √((0,25x17499x1000/57976 +(195/7479270+13/6222551)x1000000)^2) = 103,6 N/mm^2
- σ_{My} = 0,25x3664x1000000/7479270 = **122,5 N/mm^2**
- σ_{Mz} = 0,25x1854x1000000/6222551 = 74,6 N/mm^2
- σ_V = 0,025x44682x1000/(2x440x20) = 29,5 N/mm^2
- $\sigma_{combi,comp}$ = √((17499x1000/57976 +(195/7479270+13/6222551)x1000000)^2) = 330,0 N/mm^2

Welds (strong axis) v = 18 mm → 16 mm effective:

- σ_{weld} = 48x122,5/16 = 367,4 N/mm^2 ≤ 388,8 N/mm^2

Welds (weak axis) v = 10 mm → 8 mm effective:

- σ_{weld} = 25x122,5/8 = 382,7 N/mm^2 ≤ 388,8 N/mm^2

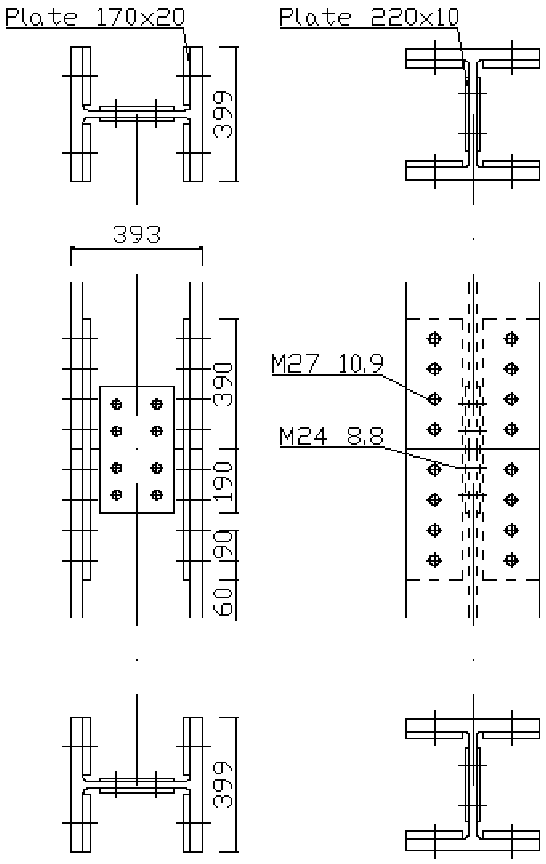
Cover plate F = 330,0x380x25/1000 = 3135 kN:

σ_{combi} = √((3135x1000x380/4/(20x600^2/6))^2 + 3x(3135x1000/(2x20x600))^2) = 335,9 N/mm^2

Welds, a = 11 mm:

σ_{weld} = 3135x1000/(2x600x11) = 237,5 N/mm^2 ≤ 249,4 N/mm^2

DETAIL 7: HD400X287 (TOP) – HD400X287 (BOTTOM)



Forces:

- N_{Ed} = 14367 kN
- $M_{y,Ed}$ = 60 kNm
- $M_{z,Ed}$ = 12 kNm

Capacities:

- N_{Rd} = 16850 kN
- $M_{ypl,Rd}$ = 2674 kNm
- $M_{zpl,Rd}$ = 1360 kNm

Max forces:

- F_{flange} = 1806 kN
- F_{web} = 1078 kN

Bolts:

- $F_{v,u,d,flange}$ = 8x274,8 = 2198 kN
- $F_{b,u,d,flange}$ = 8x19,44x20 = 3110 kN
- $F_{v,u,d,web}$ = 2x4x135,6 = 1085 kN
- $F_{b,u,d,web}$ = 2x4x18,28x10 = 1462 kN

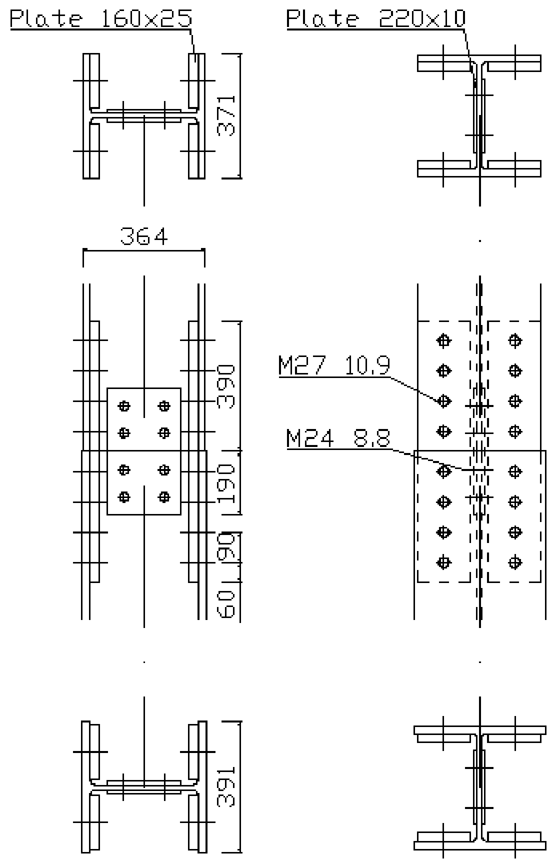
Flange plates (shear force = 0,025x16850/4 = 105 kN/flange plate):

- $N_{t,u,d}$ = 0,9x2x(170-30)x20x540/(1000x1,25) = 2177 kN
- σ_{combi} = $\sqrt{}$((105x1000x60/((20x170^3/12-20x30^3/12)x2/170))^2 + 3x(105x1000/((170-30)x20))^2) = 92,6 N/mm^2

Web plates (shear force = 0,025x16850/2 = 211 kN/flange plate):

- $N_{t,u,d}$ = 0,9x2x(220-2x26)x10x540/(1000x1,25) = 1306 kN
- σ_{combi} = $\sqrt{}$((211x1000x55/((10x220^3/12-2x10x26^3/12-2x10x26x60^2)x2/220))^2 + 3x(211x1000/((220-2x26)x10))^2) = 283,7 N/mm^2

DETAIL 8: HD400X187 (TOP) – HD360X162 (BOTTOM)



Forces:

- N_{Ed} = 3130 kN
- $M_{y,Ed}$ = 442 kNm
- $M_{z,Ed}$ = 127 kNm

Capacities:

- N_{Rd} = 9590 kN
- $M_{ypl,Rd}$ = 1303 kNm
- $M_{zpl,Rd}$ = 460 kNm

Max forces:

- F_{flange} = 2093 kN
- F_{web} = 235 kN

Bolts:

- $F_{v,u,d,flange}$ = 8x274,8 = 2198 kN
- $F_{b,u,d,flange}$ = 8x19,44x20 = 3110 kN
- $F_{v,u,d,web}$ = 2x4x135,6 = 1085 kN
- $F_{b,u,d,web}$ = 2x4x18,28x10 = 1462 kN

Flange plates (shear force = 0,025x9590/4 = 59 kN/flange plate):

- $N_{t,u,d}$ = 0,9x2x(160-30)x25x540/(1000x1,25) = 2527 kN
- $\sigma_{combi} = \sqrt{}((59x1000x60/((25x160^3/12-25x30^3/12)x2/160))^2 + 3x(59x1000/((160-30)x25))^2) =$ 46,1 N/mm^2

Web plates (shear force = 0,025x9590/2 = 119 kN/flange plate):

- $N_{t,u,d}$ = 0,9x2x(220-2x26)x10x540/(1000x1,25) = 1306 kN
- $\sigma_{combi} = \sqrt{}((119x1000x55/((10x220^3/12-2x10x26^3/12-2x10x26x60^2)x2/220))^2 +$ $3x(119x1000/((220-2x26)x10))^2) = 142,1$ N/mm^2