Master thesis study

Use of High Strength Steel Grades for Economical Bridge Design

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Summary

Bridges offer great potential for the use of high strength steel grades (HSS). The main advantages are generally a result of reduced weight and cross-sectional dimensions. Design stresses can be increased and plate thickness may be reduced, resulting in significant weight savings. Reduced plate thickness can also save on welding costs as well as on fabrication, erection and transportation costs. Simplified structural components and construction techniques are often possible, particularly for large structures, and foundation costs may also be reduced due to lower dead weight.

High strength steels can be delivered as quenched and tempered (Q&T) or as thermo-mechanically controlled processed (TMPC). In the first case, high strengths can be achieved with minimum yield strength up to 1100 MPa, which can lead to considerable weight savings, while in the second case moderate strengths (min yield strength up to 500 MPa) accompanied with excellent weldability are possible.

Especially quenched and tempered high strength steels may offer big weight savings when used for bridges. However, quenching and tempering production method poses limitations to the product length.

The most economical and efficient use of Q&T steels is in members stressed in tension where the high strength can be fully exploited, and in projects where dead load is predominant (e.g. long span bridges). In compression they are most effective in heavily loaded, stocky columns or in stiffened compression elements where buckling is not the controlling criterion.

Furthermore, hybrid steel girders are more economical than homogeneous girders. Hybrid steel girders are welded girders with different steel grades in flanges and web (usually high strength steel for the flanges, e.g. S550 or S690 and mild steel grades for the web, e.g. S355).

Higher steel grades (e.g. S690) are usually applied in steel members and/or in bridge regions with very high static stresses in order to reduce the cross sectional dimensions and plate thicknesses of these members. As a result the overall steel self-weight of the bridge will be reduced leading to a more economical design in comparison to the case where the same (equivalent) design is made out of mild steels (e.g. S355) only.

This study aims to present the potential advantages that high strength steels (HSS) have to offer in case of bridges, but also possible disadvantages. Special attention is being paid to high strength steel grades up to S700 (700 MPa minimum yield strength) in quenched and tempered condition as they are expected to offer maximum weight savings.

This thesis is divided into two main parts (Part 1 and Part 2):

In Part 1, a literature survey is initially performed (Part 1A) based on scientific documentation and relevant sites found on the Internet. Its purpose is to collect information from previous studies, experimental projects and fabricators, utilizing HSS for application in bridges, around the world. Then in Part 1B, a long span (L= 105 m) roadway bridge is chosen as a case study (the ‘Schellingwouderbrug’ in the Netherlands) and preliminary designs for three bridge types are presented (a single box girder bridge, a warren type truss girder bridge and arch girder bridge with vertical hangers). High strength steel S690 with minimum f_y = 690 MPa is applied in members with very high stresses (e.g. chord members in the truss bridge) and S355 everywhere else (hybrid design concept). The design criteria that have been studied are strength, stability and fatigue.

In Part 2, the preliminary design alternatives are compared on a cost basis (based on calculated steel self-weight and required maximum plate thicknesses) and one is chosen and designed in more detail. It is then checked, by estimating total costs, whether the hybrid design with high strength steel grade S690 will lead to a more economical bridge solution in comparison to an equivalent homogeneous (completely out of S355 steel grade) bridge design. European standards have been used throughout the whole design phase.

Comparing costs between the two hybrid alternative designs (for the same bridge type) and their equivalent homogeneous designs, it has been found that the developed hybrid designs (combination of S355 and S690) for the 'Schellingwouderbrug', result in significant weight savings in comparison to their equivalent homogeneous (only S355) bridge designs (even up to 65% in some cases). The high price for S690 (currently ≈70-75% more expensive than S355) leads to higher material costs (up to 4% higher) for the hybrid designs. Nevertheless, the weight reduction in hybrid designs has a positive impact on the reduction of total costs (up to 6% lower) including fabrication, transportation, erection and maintenance costs.

Keywords: Bridges; High strength steel; Hybrid design; Economy

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Amsterdam, 08. 04. 2012

Eleni Gogou

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Abbreviations and notations

HSS High strength steel (460-700 MPa minimum yield strength)- term adopted for use throughout this thesis project

VHSS Very high strength steel (above 700 MPa minimum yield strength)

HSLA High strength low alloy steel or micro- alloyed steel

HPS High performance (weathering) steel- steel grades with high yield strength developed in USA

BHS Bridge high performance (weathering) steel- steel grades with high yield strength developed in Japan especially for bridges

Q&T Quenched and tempered –delivery condition of steel material according to production process

TMCP Thermomechanically controlled processed- delivery condition of steel material according to production process

PWHT Post weld heat treatment

Hybrid design
Combination of high strength steel and mild steel grades for the design of a steel structure or a steel member/component.

Connection
Location at which two or more elements meet. For design purposes it is the assembly of basic components required to represent the behavior during the transfer of the relevant internal forces and moments in the connection.

Joint
Zone where two or more members are interconnected. For design purposes it is the assembly of basic components required to represent the behavior during the transfer of the relevant internal forces and moments between the connected members.

Critical members/joints/locations
Most heavily loaded and/or fatigue sensitive details in the bridge design. This may refers to specific joints or member connections with respect to a particular member (chord, brace, etc.).

Material costs
Costs calculated for the dead weight of the main steel structure- self weight of steel members (i.e. braces, chords, cross beams) plus an extra 15% for connections and additional steel-.

Total costs
Costs calculated taking into account material, fabrication, transportation, erection and maintenance costs.

Introduction

Today, steel grades S355 up to S460 are been widely used in bridge design and construction, worldwide. Moreover, higher steel grades (e.g. S690Q), with excellent forming and welding properties, are also available for more than 3 decades now. In Europe, however, their use is still, generally, limited mainly due to lack of design rules and long term experience.

Therefore, the market demand is still limited, keeping the price of HSS at quite high levels in comparison to S355 (e.g. in the Netherlands S460 and S690 is about 40% and 70-80% respectively, more expensive than S355 [62]). On the contrary, the U.S. and Japanese bridge markets show a significant market share for these higher steel grades for many decades already.

Bridges offer great potential for the use of high strength steels (hybrid bridge designs) when strength is the governing criterion. The advantages of using HSS are generally a result of reduced weight and dimensions. Design stresses can be increased and plate thickness may be reduced, resulting in significant weight savings. Reduced plate thickness can also save on welding costs as well as on fabrication, erection and transportation costs. Simplified structural components and construction techniques are often possible, particularly for large structures, and foundation costs may also be reduced due to lower dead weight.

Especially high strength steels (in Q&T quality) can reach minimum yield strength of 1100 MPa and thus can offer big weight savings when used for bridges. The most economical and efficient use of Q&T steels is in members stressed in tension and where dead load is the predominant load.

Also, using hybrid steel girders (i.e. welded girders with combination of steel grades, usually HSS in the flanges and ordinary steel grades in the web) instead of homogeneous steel girders offers a more economical solution. In compression they are most effective in heavily loaded, stocky columns or in stiffened compression elements where buckling is not the controlling criterion [1].

When fatigue is the decisive factor in the design of bridges (e.g. arch bridges) the higher yield strength does not seem to offer additional economic advantages, because the static design stresses are limited and the higher grade cannot be effectively utilized.

However, in case fatigue problems are only localized (e.g. in a number of joints/connections) improvements at fatigue sensitive locations can be achieved by altering the design at the specific location (e.g. use cast joints instead of direct welded connections in truss bridges, use locally thicker steel plates etc.) and/or by post weld treatments. Therefore, economic benefits from the hybrid construction (combination of high strength and mild steel grades) can still be gained from the overall bridge steel dead weight reduction.

Post heat treatment is, generally, not recommended for quenched and tempered high strength steels and should be PWHT only when this is specified in the design rules of the steel construction [29].

Quenched and tempered (Q&T) steels have the PWHT temperature limited to below the original tempering temperature of the steel (usually around 580°C), as higher temperatures can change the microstructure of the base material from what was expected or required [61].

Moreover, using high strength steel (HSS) enhances economy in the first place but also contributes in saving resources. A structure in HSS uses less steel for a certain application than one in mild steel.

This study aims to present the potential advantages that high strength steels (HSS) have to offer in case of bridges, but also possible disadvantages. Special attention is paid to high strength steel grades up to S700 (700 MPa minimum yield strength) in quenched and tempered condition (Q&T).

This thesis is divided into two main parts (Part 1 and Part 2):

In Part 1, a literature survey is initially performed (Part 1A) based on scientific documentation and relevant sites found on the Internet. Its purpose is to collect information from previous studies, experimental projects and fabricators, utilizing HSS for application in bridges, around the world. Then in Part 1B, a long span (L= 105 m) roadway bridge is chosen as a case study (the 'Schellingwouderbrug' in the Netherlands) and preliminary designs for three bridge types are presented (a single box girder bridge, a warren type truss girder bridge and arch girder bridge with vertical hangers). High strength steel S690 with minimum f_y = 690 MPa is applied in members with very high stresses (e.g. chord members in the truss bridge) and S355 everywhere else (hybrid design concept). The design criteria that have been studied are strength, stability and fatigue.

In Part 2, the preliminary design alternatives are compared on a cost basis (based on calculated steel self-weight and required maximum plate thicknesses) and one is chosen and designed in more detail. It is then checked, by estimating total costs, whether the hybrid design with high strength steel grade S690 will lead to a more economical bridge solution in comparison to an equivalent homogeneous (completely out of S355 steel grade) bridge design. European standards have been used throughout the whole design phase.

Comparing costs between the two hybrid alternative designs (for the same bridge type) and their equivalent homogeneous designs, it has been found that the developed hybrid designs (combination of S355 and S690) for the 'Schellingwouderbrug', result in significant weight savings in comparison to their equivalent homogeneous (only S355) bridge designs (even up to 65% in some cases).

The high price for S690 (currently ≈70-75% more expensive than S355) leads to higher material costs (up to 4% higher) for the hybrid designs. Nevertheless, the weight reduction in hybrid designs has a positive impact on the reduction of total costs (up to 6% lower) including fabrication, transportation, erection and maintenance costs.

Part 1

Literature and preliminary bridge design

This part consists of two sub parts, Part 1A and Part1B.

In Part 1A, a literature survey is performed based on scientific documents, previous studies, fabricators' sites and other relevant sites on the Internet. The aim of this review is to collect information on the material itself (material properties) and on its use in structural applications.

In Part 1B, a long, single span (L=105 m) roadway bridge crossing Amsterdam-Rijnkanaal in the Netherlands, the 'Schellingwouderbrug', is chosen as a reference bridge to be re-designed by implemented HSS S690 in combination to mild steel grade S355 (hybrid design). Up to now S355 (and in limited cases S460) steel grade is customary used for bridge design in the Netherlands.

Preliminary designs for three bridge types (i.e. a single box girder bridge, a warren type truss girder bridge and arch girder bridge with vertical hangers) are presented using high strength steel S690 (minimum f_y = 690 MPa) in members of very high stresses (e.g. chord members in the truss bridge) and mainly S355 elsewhere (hybrid design concept). The design criteria that have been studied are strength, stability and fatigue. Reference is also made to Appendix A for members cross sectional dimensions, description of design procedure step by step and numerical results.

Part 1A
Literature survey

In Part 1A (chapters 1, 2, 3, and 4), a literature survey is performed based on scientific documents, previous studies, fabricators' sites and other relevant sites on the Internet. The aim of this review is to collect information on the material itself (material properties) and also on its use in structural applications. The most interesting points from this review are summarized in chapter 4.

1 Material

1.1 High Strength Steel (HSS)

1.1.1 General

High strength steel (HSS) is a new generation of steel material exhibiting improved properties over conventional steel grades (e.g. S235, S355, etc.). HSS is available, for more than three decades now, for structural applications such as bridges, buildings, offshore, cranes etc. Figure 1.1 shows the historical development of steel grades available in Europe for rolled products and their delivery condition [2].

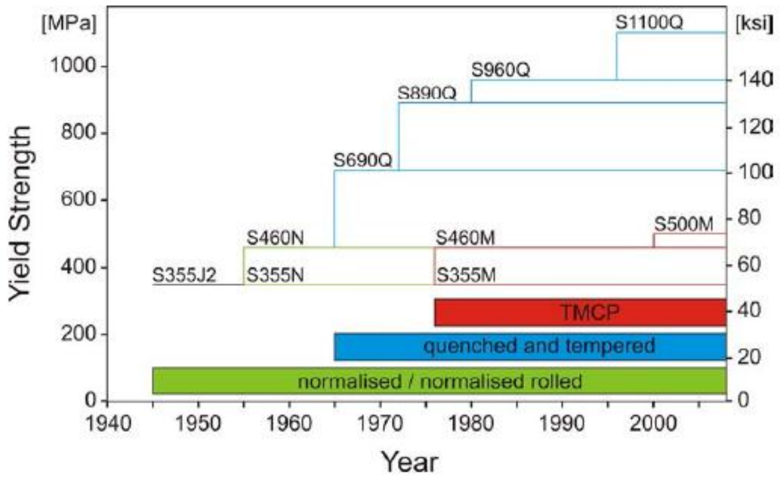


Figure 1.1 Historical development of grades and production processes for rolled steel products [2]

Weight savings thus reduced fabrication, transportation and erection costs are the main reasons using higher strength steel grades in (bridge) construction. As an indication, a weight reduction over 60% can be achieved with S690 steel grades (Figure 1.2).

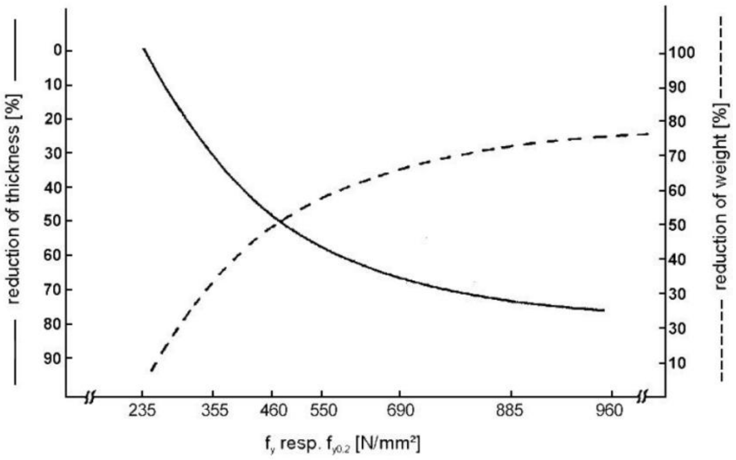


Figure 1.2 Weight and wall thickness reduction with increasing steel strength [3].

High strength steels (HSS: S460-S700) or very high strength steels (VHSS: up to S1100, and even higher for cables) are available for structural applications, as in bridges, buildings, offshore applications etc., all around the world.

These steels must exhibit good toughness and ductility, to avoid brittle failures and at the same time very good weldability and high strength. The combination of these overall requirements is often difficult to be achieved, since the increase of one of these properties may lead to a decrease in others (e.g. increasing the amount of carbon content during steel production, increases strength on one hand, but at the same time reduces weldability). Therefore, a variety of structural HSS grades exists today, which allows for different values of these properties. It is possible, for example, to develop many different steels with minimum yield strength of 690 MPa just by altering their chemical composition or by changing the production process.

The choice of the "right" high strength steel for a particular structural application, however, depends strongly on the material requirements (toughness, strength, weldability etc.) for that application. Applying HSS such that the full properties of the material can be utilized (e.g. using steel exhibiting very high yield strength in regions where high tensile stresses occur), would be an efficient and competitive way of using higher steel grades.

Currently, one of the main limitations is that material costs for HSS are still higher than conventional grades (especially in Europe). Nevertheless, consistent testing and research will promote the material and help to establish new detailed design codes. Thus, it is expected that its demand will be increased and consequently its price will be reduced in the future.

According to European standards, high strength steel can be delivered mainly as quenched and tempered (Q&T) or as thermo-mechanically controlled processed (TMPC). In the first case high strengths can be achieved with minimum yield strength up to 1100 MPa, which can lead to considerable weight savings, while in the second case moderate strengths (min yield strength up to 500 MPa) accompanied with excellent weldability are possible.

Quenched and tempered steel grades with yield strength grades up to 960 MPa are standardized in EN 10025- part 6 "Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition" but constructional steelwork in Europe is still limited to steel grades with minimum yield strength 690 MPa. Higher steel grades are still the domain of the construction equipment industry [4].

High strength steels with minimum yield strength between 460-690 MPa, in the "Quenched and Tempered" condition (Q&T), are suitable, among others, for application in bridges. These grades provide generally, high strength combined with high toughness, good ductility and improved weldability compared to conventional (mild) steel grades.

The need for preheating is determined by the general instructions of EN 1011-2:2001 "Welding Recommendations for welding of metallic materials. Arc welding of ferritic steels" and depends mainly on the chemical composition of the steel and the filler metals (i.e. their hardenability) [29]. Preheating is generally not required for plate thickness up to 30mm.

Q&T steels offer substantial weight savings over traditional steel grades, and designers are increasingly using this advantage in "hybrid steel girder" or "hybrid bridge" construction (i.e. combination of mild and high strength steel grades). Common examples of this practice include beams with high strength flanges and standard strength web, and steel tanks with higher strength steel for the more heavily loaded lower sections, thereby maintaining a constant wall thickness for simplified fabrication. This hybrid approach gives high strength steel a crucial cost advantage [1].

In Europe, a variety of HSS with yield strengths from 460 to 690 MPa are available for bridge applications, although still not widely used. The two main reasons for this drawback are the lack of detailed design codes, especially for grades between S700 and S1100, and also the higher material costs compared to conventional steel grades. So far, European design Standards have developed additional design rules and specifications to extend existing design rules covering steel grades up to S700 only (Eurocode 3 - Design of steel structures - Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S700 (2007)).

Ongoing testing and research contributes in gaining more experience on the structural behaviour of bridge components made with these steel grades, and extend further their use for bridge design.

On the contrary, bridges using HSS in U.S and Japan exist for several decades already.

Use of HSS for bridge construction, in Japan, dates back to 1960 (Miki and al. 2002). Several hundred bridges have been constructed using 500 MPa and 600 MPa yield strength steel, and steel with nominal yield strength of 800 MPa has also been used in several projects. These steels typically require preheating between 100-120 ºC before welding and sometimes post-weld treatment to avoid hydrogen assisted cracking of the weld (cold cracking). In 1992, a new steel grade (f_y=800 MPa) was developed that requires preheating at 50 ºC (Miki and al. 2002) [5].

In 1992, in U.S. a new type of steel, known as high-performance steel (HPS) was developed. High Performance (Weathering) Steel, with yield strength between HPS 70W (485MPa) and HPS 120W (827 MPa), has been developed in the USA over the last decade. They provide high strength, high toughness, good weldability and improved fatigue and corrosion resistance [6].

1.1.2 High strength steel types

Depending on their structural properties, chemical composition or delivery condition, many different types/categories exist, which usually referred as high strength steels (HSS) or high performance steels (HPS).

All these different steel types, however, have more or less similar properties, in the sense that, they refer to high strength steels with better toughness, improved weldability, higher strengths and/or improved corrosion resistance (in case of high performance weathering steels).

Generally, their chemical composition and quality depends strongly on the production process, controlled by the manufacturer, and also on the processes in the fabrication shop (cutting, drilling, welding etc.) to obtain the final product. In any case, it must be ensured, that they all comply with (or are superior of) the specifications provided by the relative international quality standard (American (ASTM), European (EN), Japanese (JIS), etc.).

Focused mainly on the latest developments in steels for design of bridges [7], several steel types/categories are briefly described in this study. All these types of high strength steel, and many others, are available nowadays, to produce stronger, lighter and more slender bridges.

1.1.2.1 High Strength Low Alloy Steel (HSLA) or Microalloyed steels (MA)

Microalloyed (MA) or High Strength Low Alloy (HSLA) steels ([8], [9], and [10]) constitute an important category of steels estimated to be around 12% of total world steel production [8]. High Strength Low Alloy steels contain a low percentage of microalloying elements (below 0.15% in total) and vary from other steels in that, they are not made to meet a specific chemical composition, but rather to specific mechanical properties. They typically contain 0.07 to 0.12% carbon, up to 2% manganese and small additions of niobium, vanadium and titanium (usually max. 0.1%) in various combinations. High-strength low-alloy (HSLA) steels, or microalloyed steels, are designed to provide better mechanical properties and/or greater resistance to atmospheric corrosion than conventional carbon steels [8]. The material is preferably produced by a thermomechanical rolling process, which maximizes grain refinement as a basis for improved mechanical properties. Grain refinement and precipitation strengthening are the primary mechanisms to increase yield strength of microalloyed steels, while maintaining desired levels of ductility and weldability. Furthermore, due to their higher strength and toughness HSLA steels usually require 25 to 30% more power to form, as compared to carbon steels.

A special type of HSLA steels is HSLA-V [11]. This low alloy steel is intended to represent those steel grades where a small addition of vanadium (less than 0.12%) provides enhanced strength over standard low C-Mn steels, while meeting or even exceeding all requirements for ductility, weldability and toughness. They are usually supplied in the as-rolled or as-forged condition, eliminating the need for subsequent heat treatments. This negates the need for higher alloy contents of Cr, Ni and Mo (hence “Low Alloy”) and also provides significant energy savings. It has many applications in structural engineering and especially for bridges has already been used in different types (long span truss, non-standard fixed bridge, deployable bridge, suspension components). Finally, steel manufacturers producing HSLA-V steel, experience lower operating costs compared to C-Mn steels, due to the unique metallurgical characteristics of vanadium in the microstructure and metalworking technology.

1.1.2.2 HIGH Performance Steel (HPS)

HPS developed in U.S., Europe and Japan, have nominal yield strengths between 485-900 MPa and exhibit excellent ductility, toughness and corrosion resistance. HPS can be welded with greater ease than many steels developed in the past [5].

In 1992, AISI partnered with the Carderock Division, Naval Surface Warfare Centre and the Federal Highway Administration (FHWA) to develop new and improved steel alternatives for bridges. The result was a new type of steel, known as high-performance steel (HPS), which provided up to 18% cost savings and up to 28% weight savings when compared with traditional steel bridge design materials. They also have improved fatigue and corrosion-resistance properties [12].

HPS 70W (485MPa) and HPS 120W (827MPa), has also been developed in the USA over the last decade. The key features for this steel is high strength, high toughness, good weldability and ease of fabrication (due to a low carbon equivalent), adequate elongation and yield to tensile strength ratio for ductility and enhanced durability (corrosion resistance is superior to weathering steels currently used) [13]. When produced by quenching and tempering (Q&T), that poses limitations to the product length. However, production by thermo-mechanical controlled processing (TMCP) is also possible.

1.1.2.3 High strength weathering Steel (W)

High strength weathering steels are high strength low alloy steels, which under certain atmospheric conditions (humidity and oxygen should always be present) give an enhanced resistance to rusting compared to that of ordinary carbon manganese steels by forming a protective layer on the outer surface. They are of particular interest to the artists and the designers. The best known of these steels is COR-TEN® an alloy developed by the American USX Corporation. Weathering steel bridges do not require painting. Periodic inspection and cleaning should be the only maintenance required to ensure the bridge continues to perform satisfactorily. Hence, weathering steel bridges are ideal where access is difficult or dangerous, and where future disruption needs to be minimized.

Cost savings from the elimination of the protective paint system outweigh the additional material costs. Typically, the initial costs of weathering steel bridges are approximately 5% lower than conventional painted steel alternatives [7]. In addition, limited maintenance requirements of weathering steel bridges, greatly reduces both the direct costs of the maintenance operations, and the indirect costs of traffic delays or rail possessions.

1.1.2.4 Constant yield point steel

In Japan, a new type of steel has been developed that offers constant yield strength through the range of 16-100mm [7]. With ordinary steels, the yield strength reduces as the plate thickness increases, and this is reflected in the material standards. With these steels designers are able to utilise a higher yield stress for thicker plates, but also design more efficient steel bridges by reducing the flange thickness.

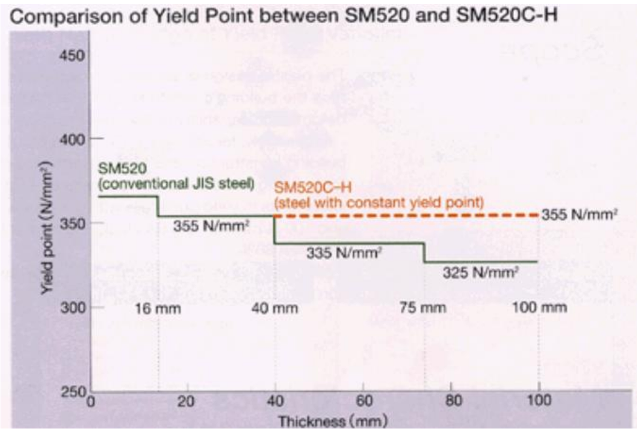


Figure 1.3 Comparison of yield point between conventional JIS steel SM520 and constant yield point steel SM520C-H, [Worldsteel Association].

1.1.2.5 High Toughness Steel

Generally, the toughness of steel products decreases at low temperature and the steel becomes susceptible to brittle fracture. However, the use of high toughness steel plates allows the use of steel for bridges in very cold regions. This steel provides two main benefits: Firstly, cold forming becomes possible with smaller bending radius, and secondly, the steel products can be used in cold regions (toughness is not reduced even at very low temperatures avoiding brittle fracture), Figure 1.4

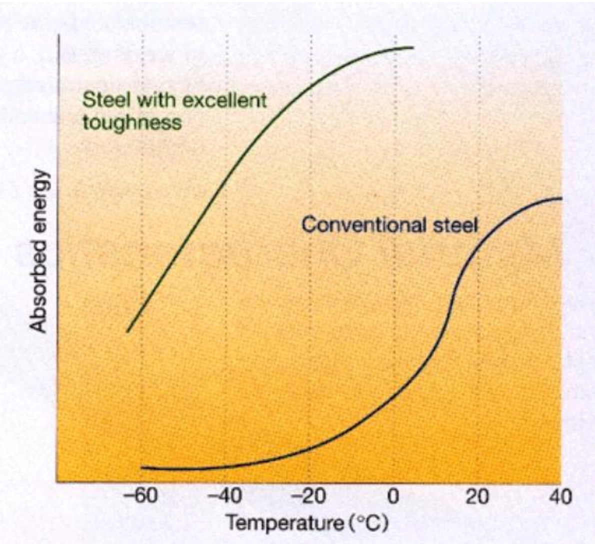


Figure 1.4 Comparison of toughness performance of high toughness steel and conventional steel [Worldsteel Association].

1.1.2.6 Bridge High performance steels (BHS)

High performance steel for bridges (BHS) was recently developed in Japan. BHS is defined as a steel material superior to conventional steel materials for bridge structures in terms of strength, fracture toughness, weldability, workability and corrosion resistance which are required for bridges and has its properties optimized for application to bridges [16]. Honshu-Shikoku bridge project is a good example of effectiveness of a BHS with tensile strength of 780 MPa (680 MPa yield strength).
The Society for the study of High-Performance Steel Application- established at the Creative Project Research group in the Tokyo Institute of Technology-, has discussed the performance requirements of steel bridges and the specifications of steel materials for steel bridges as part of an industrial academic project involving steelmakers and bridge fabricators [33].

For plate girder bridges it was found that increasing the yield strength the weight decreases but exceeding yield strength of 500 MPa will not always be effectively used in design. Therefore, it was proposed that yield strength of 500 MPa is approximately, the upper limit that can effectively be used in girder bridge design and should be adopted as the basic yield strength for BHS. For suspension and cable stayed bridges (bridge types in which reducing the dead load of the superstructure has a significant effect on bridge economics), the same study, proposed a yield strength of 700 MPa as the upper limit for BHS.

1.1.3 Chemical composition of structural HSS

Depending on the material properties required for a specific application, the amount and types of alloying elements vary in chemical composition of HSS. These variations in chemical composition accompanied with high quality in fabrication process, determine the final properties of high strength steel grades. It is not the aim of this study to refer extensively in the chemical composition of all different types/categories of high strength steels mentioned in the previous section, of course. Some general information has already been provided for HSLA steels, anyway.

Special attention, however, is paid on quenched and tempered structural steels (Q&T). That is because; they provide high strength, improved toughness properties at low temperatures, very good weldability and sufficient ductility to be used for bridge design. They also offer substantial weight, thus cost, savings and are covered by the European standards. Quality standard EN 10025-6 cover these steels up to grades S960 but design standard EN 1993-1-12 gives additional design rules only up to S700 steel grades. Therefore, for practical reasons this study focuses on the range of S500-S700 (Q&T), which are covered by the Eurocodes.

1.1.3.1 Chemical composition of HSS in Q&T condition

Q&T steels offer many advantages which, in the right circumstances, can generate significant cost savings. Especially for bridges, key design benefits include longer or lighter spans and greater load carrying capacity. They provide high strength to weight ratios, very good weldability, improved toughness and sufficient deformation capacity (especially where overmatched welds are used). Financial benefits can also be realized through reduced transportation and lifting costs (reduced weight), material savings (smaller/lighter sections) and reduced weld volumes (thinner plates).

Some typical quenched and tempered steel grades for structural applications are the steel grades S500, S550, S620, S690, S890 and S960 [EN10025-6]. Quenched and tempered high strength structural steels (usually up to S690) are ideal for applications with heavy sections and heavy live loads (e.g. long span bridges), where weight reduction is important.

Generally, the alloying composition of Q&T steels increases with increasing plate thickness in order to ensure sufficient hardening of the plate in the core region. So, The CEV of a Q&T plate increases with increasing thickness. Most high strength Q&T structural steels are produced with a carbon content of 0.12-0.18 % [18]. A typical chemical composition for these steels is shown in Table 1.1.

Carbon	0.15%
Manganese	0.75%
Phosphorus	0.026%
Sulphur	0.03%
Silicon	0.24%

Nickel	0.85%
Chromium	0.5%
Molybdenum	0.45%
Vanadium	0.05%
Copper	0.31%

Table 1.1 Typical chemical composition for quenched and tempered steels [SteelTalk.com, [14]].

1.1.3.1.1 S690 Q&T high strength structural steel grade

Generally, S690 steel grades can be produced as "Quenched and Tempered" but also as "Thermomechanically rolled" [18]. Among these, S690 quenched and tempered high strength structural steel is of increasing interest for bridge design and construction.

S690 Q&T structural steel plate is a high strength, fine-grained structural steel, especially suitable for heavy structural applications, where weight savings are important. The material is heat treated using the "quench and temper" process and has good bending and welding properties [19]. Their chemical composition, depending on required toughness and plate thickness, is presented in Table 1.2 according to EN 10025-6 (2004).

Grade	Thickness (mm)	C	Mn	P	S	Si	Cr	Ni	Mo	V	Cu	Ti	Nb	N	B	Zr
S690QL	<150	.20 max	1.70 max	.20 max	.010 max	.80 max	1.50 max	2.0 max	.70 max	.12 max	.50 max	.05 max	.06 max	.015 max	.0050 max	.15 max
S690Q	<150	.20 max	1.70 max	.25 max	.015 max	.80 max	1.50 max	2.0 max	.70 max	.12 max	.50 max	.05 max	.06 max	.015 max	.0050 max	.15 max
S690QL1	<150	.20 max	1.70 max	.20 max	.010 max	.80 max	1.50 max	2.0 max	.70 max	.12 max	.50 max	.05 max	.06 max	.015 max	.0050 max	.15 max

Table 1.2 Chemical composition of S690 steel grade according to EN 10025-6:2004 quality standard [Leeco Steel [20]].

Designation symbols Q, QL and QL1 give information on the toughness properties. More specifically:

Q: Longitudinal Charpy V-notch impacts at T_{min}= - 20ºC
QL: Longitudinal Charpy V-notch impacts at T_{min}= - 40ºC
QL1: Longitudinal Charpy V-notch impacts at T_{min}= - 60ºC

1.1.4 Properties of High Strength Steels

1.1.4.1 MECHANICAL PROPERTIES

1.1.4.1.1 Tensile properties

The primary tensile properties of high strength steels are the yield stress, the (ultimate) tensile strength, the strains at rupture and strain hardening, the reduction of area, and the yield-to-tensile strength ratio. Their nominal strength values are given in Tables 1 and 2 of EN 1993-1-12 (2007), (Figure 1.5).

Table 1 — Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

EN10025-6 Steel grade and qualities	Nominal thickness of the element t mm					
	$t \leq 50$ mm		50 mm$<t\leq$100 mm		100 mm$<t\leq$150 mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
S 500Q/QL/QL1	500	590	480	590	440	540
S 550Q/QL/QL1	550	640	530	640	490	590
S 620Q/QL/QL1	620	700	580	700	560	650
S 690Q/QL/QL1	690	770	650	760	630	710

Table 2 — Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled flat products.

EN 10149-2a)	1,5 mm $\leq t \leq$ 8 mm		8 mm $< t \leq$ 16 mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
S 500MC	500	550	500	550
S 550MC	550	600	550	600
S 600MC	600	650	600	650
S 650MC	650	700	630	700
S 700MC	700	750	680	750

a) Verification of the impact energy in accordance with EN 10149-1 Clause 11, Option 5 should be specified.

Figure 1.5 Nominal values of yield and tensile strength for high strength hot rolled structural steel and flat products, according to EN 1993-1-12: 2007.

The stress-strain curve for steel differs significantly between mild (carbon-manganese) and high strength steels (HSS) [21]. Figure 1.6 and Figure 1.7 give representative examples of such curves. The curve will also reflect any pre-test treatment in the form of heat or plastic deformation.

Specifically, the upper yield point is contrasted to the yield level (plateau) for mild steel (Figure 1.6); for high strength steel (Figure 1.7) the yield strength, is defined by the 0.2 percent offset (permanent deformation) or 0.5 percent total deformations. The yield strength (Figure 1.7) differs from the yield stress level (Figure 1.6), on the basis of different characteristics, as illustrated by the two figures. The 0.2 percent offset value is used for steels with no clearly defined yield plateau.

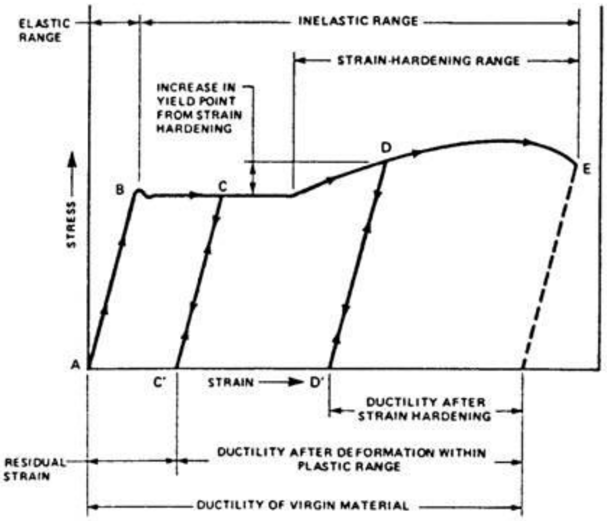


Figure 1.6 Typical stress-strain curve for mild steels (Geschwindner et al., 1994)

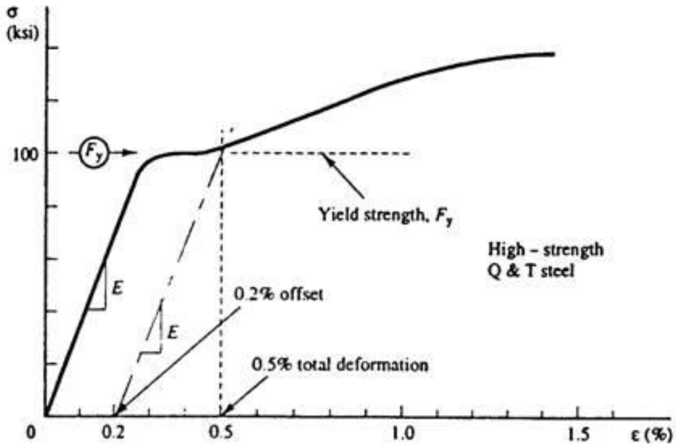


Figure 1.7 Stress strain curve for high strength steels (Geschwindner et al., 1994)

1.1.4.1.2 Toughness

High strength steels show much improved toughness properties compared to conventional steel grades even at very low temperatures (transition to brittle fracture occurs at lower temperatures than conventional steel grades).

The Charpy-V notch impact test is used as a measure of toughness of structural steel where the test temperature and the minimum absorbed energy are specified, although Charpy results cannot be considered to be directly relevant to structural behaviour [15]. Table 1.3 specifies maximum element thickness depending on the steel grade and the minimum Charpy-V energy values. Grades taken from this table and satisfying the conditions given in EN1993-1-10 for the lowest temperature, are assumed to have sufficient toughness and no further testing is needed against brittle fracture [EN 1993-1-1 (2006)].

Table 4 — Maximum permissible values of element thickness t in mm

Steel grade	Subgrade	Charpy energy CVN		Reference temperature T_{Ed} [°C]																				
				10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
		at T [°C]	J_{min}	$\sigma_{Ed} = 0{,}75\ f_y(t)$							$\sigma_{Ed} = 0{,}50\ f_y(t)$							$\sigma_{Ed} = 0{,}25\ f_y(t)$						
EN 10025-6																								
S500	Q	0	40	55	45	35	30	20	15	15	85	70	60	50	40	35	25	145	125	105	90	80	65	55
	Q	-20	30	65	55	45	35	30	20	15	105	85	70	60	50	40	35	170	145	125	105	90	80	65
	QL	-20	40	80	65	55	45	35	30	20	125	105	85	70	60	50	40	195	170	145	125	105	90	80
	QL	-40	30	100	80	65	55	45	35	30	145	125	105	85	70	60	50	200	195	170	145	125	105	90
	QL1	-40	40	120	100	80	65	55	45	35	170	145	125	105	85	70	60	200	200	195	170	145	125	105
	QL1	-60	30	140	120	100	80	65	55	45	200	170	145	125	105	85	70	205	200	200	195	170	145	125
S550	Q	0	40	50	40	30	25	20	15	10	80	65	55	45	35	30	25	140	120	100	85	75	60	50
	Q	-20	30	60	50	40	30	25	20	15	95	80	65	55	45	35	30	160	140	120	100	85	75	60
	QL	-20	40	75	60	50	40	30	25	20	115	95	80	65	55	45	35	185	160	140	120	100	85	75
	QL	-40	30	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	185	160	140	120	100	85
	QL1	-40	40	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	185	160	140	120	100
	QL1	-60	30	130	110	90	75	60	50	40	185	160	135	115	95	80	65	200	200	200	185	160	140	120
S620	Q	0	40	45	35	25	20	15	15	10	70	60	50	40	30	25	20	130	110	95	80	65	55	45
	Q	-20	30	55	45	35	25	20	15	15	85	70	60	50	40	30	25	150	130	110	95	80	65	55
	QL	-20	40	65	55	45	35	25	20	15	105	85	70	60	50	40	30	175	150	130	110	95	80	65
	QL	-40	30	80	65	55	45	35	25	20	125	105	85	70	60	50	40	200	175	150	130	110	95	80
	QL1	-40	40	100	80	65	55	45	35	25	145	125	105	85	70	60	50	200	200	175	150	130	110	95
	QL1	-60	30	120	100	80	65	55	45	35	170	145	125	105	85	70	60	200	200	200	175	150	130	110
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100
EN 10149-2																								
S500	MC	-20	40	80	65	55	45	35	30	20	125	105	85	70	60	50	40	195	170	145	125	105	90	80
S550	MC	-20	40	75	60	50	40	30	25	20	115	95	80	65	55	45	35	185	160	140	120	100	85	75
S600	MC	-20	40	70	55	45	35	30	20	15	105	90	75	60	50	40	35	180	155	130	110	95	80	70
S650	MC	-20	40	65	50	40	30	25	20	15	100	85	70	55	45	35	30	170	145	125	105	90	75	65
S700	MC	-20	40	60	45	35	30	25	20	15	95	80	65	50	45	35	30	165	140	120	100	85	70	60

Table 1.3 Toughness requirements (EN 10025-6:2004, EN 10149-2:1996): Charpy-V values and maximum permissible element thickness [EN1993-1-12 (2007)].

Examples of Charpy V-notch temperature transition curves for some high strength steels with strengths 460 MPa and 690 MPa compared with S355 steel, are shown in Figures 1.8 and 1.9. Furthermore, Figure 1.9 shows typical transition curves for the Charpy-V energy against the temperature for S460ML and S690QL compared to S355J2, as an example. It is obvious that the high strength steels S460ML and S690QL show significantly higher Charpy-V values at the testing temperatures than given in the standard (27J at- 50 ºC and 30J at -40 ºC respectively). Even at room temperature the toughness behaviour is better than for conventional S355J2. These high toughness values also result in good welding properties [4].

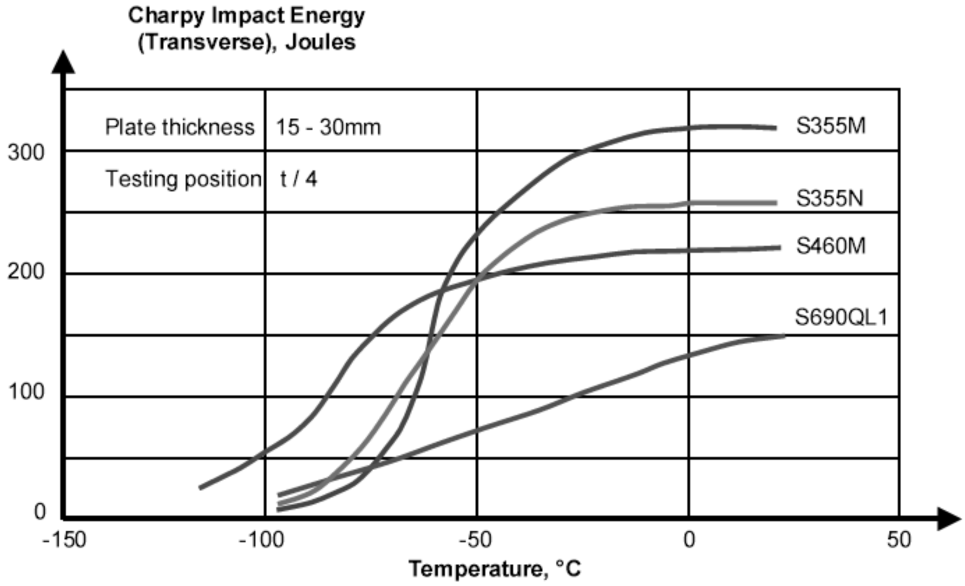


Figure 1.8 Charpy V-notch temperature transition curves for some HSS, [22]

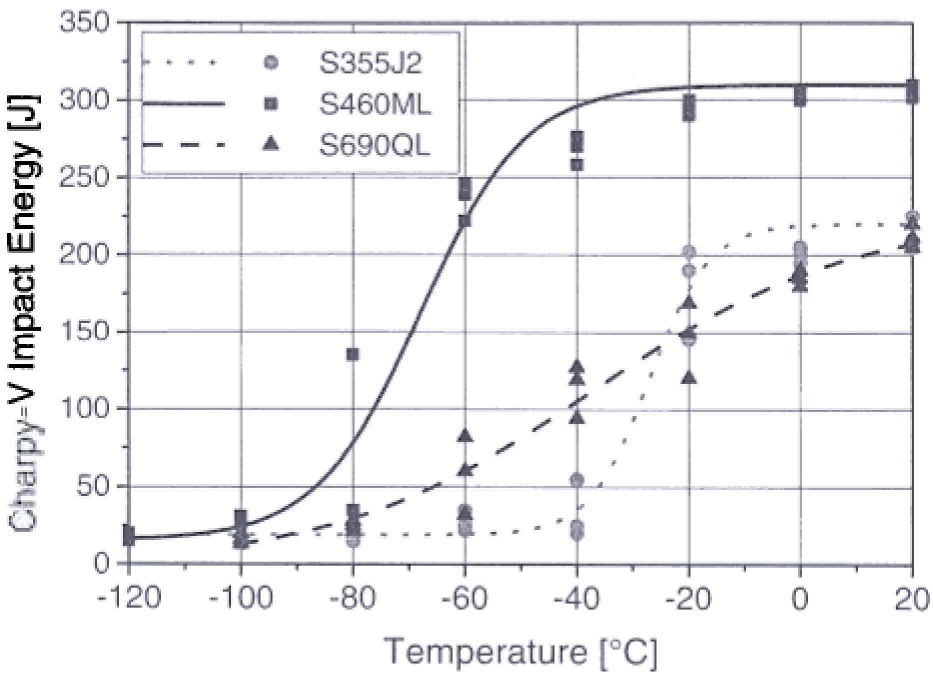


Figure 1.9 Charpy-V temperature transition curves for S460M and, S690QL compared to S355J2 [4].

1.1.4.1.3 Ductility

The requirements for ductility ensure that brittle failures are avoided (i.e. inelastic deformations must be sufficiently large). The carbon content plays an important role here. Increasing the carbon content produces a material with higher strength but lower ductility. Therefore, the carbon content should be kept between 0.15-0.30 % for all structural steels [23]. In case of HSS is possible to keep the carbon content at very low levels usually around 0.15% ensuring high strengths at the same time. The quality of HSS meets similar standards to conventional steel grades. That is also verified by experimental research.

Uni-axial tension tests on HSS coupons have shown that these steels can achieve elongation of up to 20%, which is considered excellent [24]. Tests on beam to column bolted connections (S355) with end plates (S690) have shown that the rotation capacity of specimens using HSS satisfy high deformation demands. Furthermore, rotation capacities of 40 mrad (it is generally accepted that a minimum of 35-40 mrad ensures sufficient rotation capacity of a bolted joint in a partial-strength scenario) and above, were achieved with thinner end plates. However, simple beam analysis with actual joint behaviour has shown that the efficiency of HSS moment connections has no correspondence to the improvement in quality, and the deformation demands of these connections are higher than for mild steel grades.

High strength steels with a tensile to yield strength ratio of 1.05 are considered less ductile than mild structural steel. Therefore it is believed they are suitable only for elastic analysis. Figure 1.10 (b) shows why HSS are considered to be more sensitive to local ductility demands than ordinary steel grades. However, extensive experimental research on plates with holes and bolted connections made of steel grade S690 confirmed that a low f_u/f_y ratio does not affect local ductility significantly.

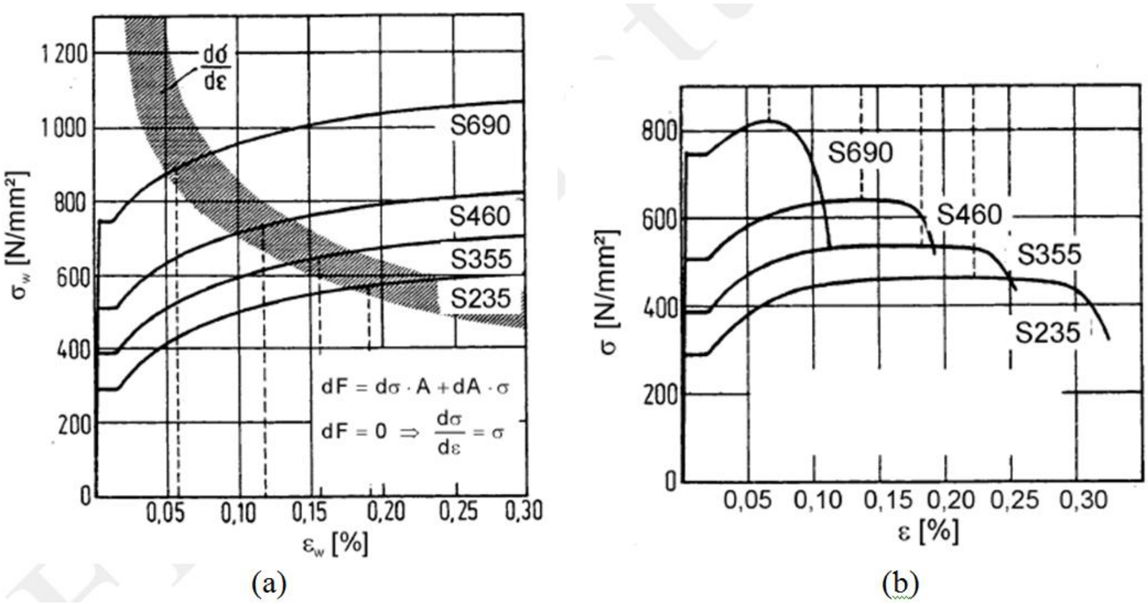


Figure 1.10 (a) Stress strain curves for different steel grades; (b) Load-deflection curves for different steel grades [3].

Tables 1 and 2 of EN 1993-1-12:2007 (Figure 1.5) specify the nominal values of the yield strength (fy) and the ultimate tensile strength (fu) for hot rolled high strength structural steel. It is stated that steels included in these tables can be assumed to satisfy the ductility requirements. Generally the same rules for ductility, as for normal structural steel grades, hold for HSS with the limits of the two first requirements somewhat relaxed. More specifically: $fu/fy \geq 1.05$ and elongation at failure not less than 10% [EN 1993-1-12 (2007)].

The critical part of the steel manufacturing is to control the processing parameters so that the microstructure and, hence, the strength-elongation balance could be optimized. Figure 1.11gives an indication on ductility properties (total elongation %) based on the tensile strength of different structural steel grades.

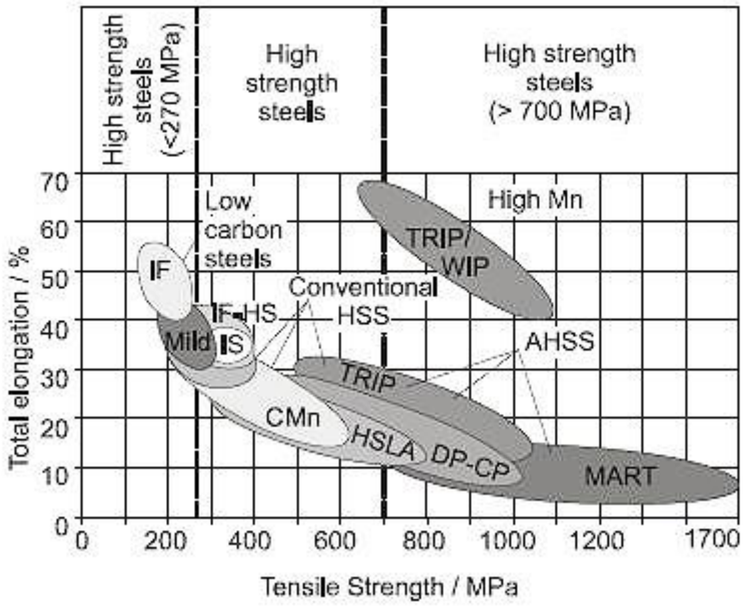


Figure 1.11 Ductility of structural steels compared to their tensile strength [25].

1.1.4.2 Technological Properties

Technological properties of HSS include weldability and formability. Once more, high strength steels (S460-S700) in quenched and tempered condition are mainly of interest in this study.

1.1.4.2.1 Weldability

High strength steels (HSS) show generally improved weldability compared to conventional structural steels and they are suitable for all current welding methods. Generally, no preheating is required for plate thickness up to 30 mm. However, the temperature of the material should be at least RT for welding.

The need for preheating, however, is determined by the general instructions of EN 1011-2 and depends mainly on the chemical composition of the steel and the filler metals (i.e. their hardenability) [29].

Below preheating recommended temperatures for S690 QL1 steel grade (source: steel supplier AJ Marshall, UK) are given as an example:

20mm – 40mm:	75°C
40-60 mm:	100°C
>60mm	150°C

In general, as the parent metal strength increases, greater precautions are needed to ensure that welding procedure is satisfactory, see also Figure 1.12.

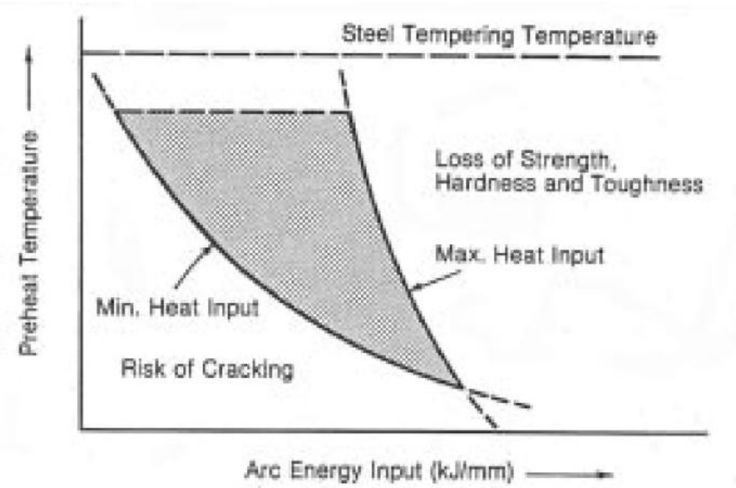


Figure 1.12 Total weld heat input for welding Q&T steels [60]

The shaded area in Figure 1.12 denotes a permissible heat input 'window' limited on the low energy side by the risk of excessive energy and cold cracking. On the other side the heat input is limited by the loss of strength and hardness. This 'window' for the arc energy input is getting smaller as the strength of base material increases [60].

Variations in the welding process such as steel dimensions, weld geometry, heat input and steel chemical composition all influence the resulting microstructure. Nomograms involving thermal severity- joint thickness (mm), heat input of the weld (KJ/mm) and weld preheat required (ºC) are often used to indicate the necessary welding procedure to be followed to produce a sound crack free joint in relation to the particular composition of the steel used which is usually related to carbon equivalent value [15].

Carbon equivalent (CEV or CE) is the most common measure for weldability, which is used to assess the combined effect of carbon and the other chemical elements on the cracking susceptibility of the material. Generally, low values for CE are important for good weldability. Various CE-formulas are available, but for structural steel is usually described by the following equation, which is the formula, proposed by the International Institute of Welding (IIW):

$$CE = C + Mn/6 + (Cr + Mo + V)/5 + (Ni + Cu)/15$$

As an alternative approach adopted from some countries is the Graville diagram shown in Figure 1.13 which separates the steels into three zones rated by their ease of weldability-zone I easily weldable, zone II weldable with care, and zone III difficult to weld [15]. From this diagram can be seen that with increasing carbon equivalent the weldability decreases but it also emphasises the extremely important effect of carbon content on weldability. Reducing the carbon content of steel is the most effective way to improve its weldability [15].

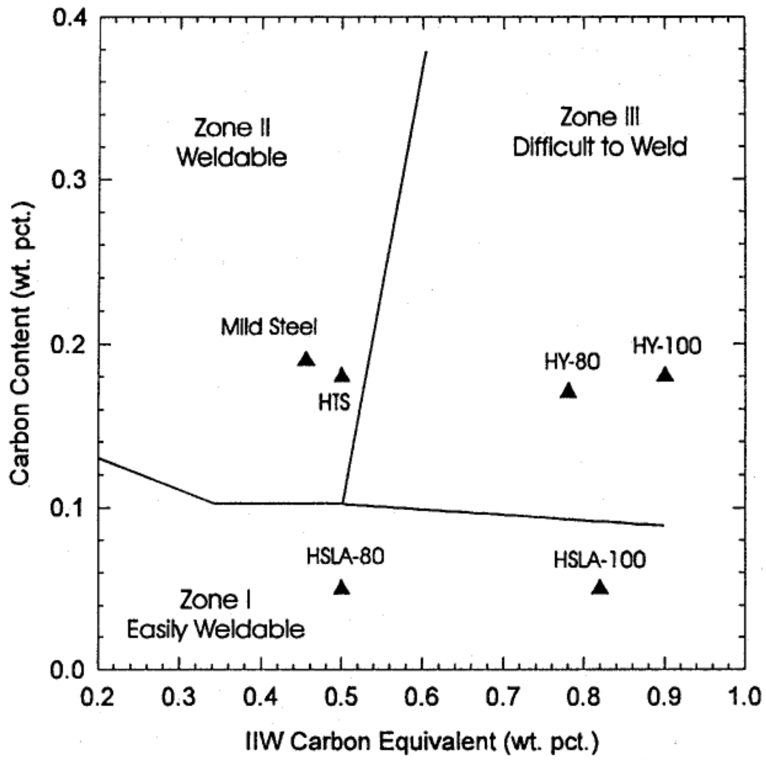


Figure 1.13 Weldability criteria, cracking susceptibility [15]

A very high carbon equivalent value indicates poor weldability and these steels are not suitable for structural applications, where welding is very important to assure structural safety. The CEV is also utilized to assess preheat requirements for a welded joint or assembly, and to take into account the influence of hydrogen and joint restraint.

Finally, HSS are suitable for all current welding methods and they generally require little or no preheat [CSEC Group]. Also post weld heat treatment (PWHT) is not recommended in case of Q&T high strength steels [29].

1.1.4.2.2 Formability

The mechanical properties of the particular steel grade being formed, dictates the loads required (higher grades require higher loads) for forming and the care that should be taken during the process. New developments and production of fine grain structural steel (S690), have allowed for HSS and VHSS to combine strength and weldability with excellent formability. Strength and formability of these steel grades however, extend scope for fabrication. For example, products may be manufactured by press forming rather than welding. A major drawback fabricating HSS and VHSS is they tend to demonstrate a higher “spring-back” during the process, which restricts workability.

To overcome similar limitations new innovative processes like hot forming (at elevated temperatures on a hydraulic press and air cooling) or hydro-forming (at room temperature with force of water or hydraulic fluids) are required. Quenched and tempered high strength structural steels are suitable for both cold (at ambient temperatures) and hot forming.

Another important consideration is that HSS material requires more energy. Two main factors need to be considered. Firstly, increased forming tonnage in combination with increased draw pad pressure requires higher energy; secondly, the higher-strength steel does not have the same draw qualities as the previously used lower-grade steels. This could lead to cracking at the forming radii and unacceptable thinning of the material as it is drawn. This requires the operator to decrease the draw speed by slowing the press.

1.1.5 Production of HSS

Weldable structural steels can be delivered as “normalized (N)”, “quenched and tempered (Q&T)”, or “thermo-mechanically controlled rolled (TM or TMPC)”. All heating, cooling and processing methods affect the microstructure of steel. With classical hot rolling and normalizing of the steel we can achieve moderate values of strength (up to 460 MPa yield strength) and toughness. By quenching and tempering however, we can achieve yield strengths up to 1100 MPa. TM rolled plates are available with minimum yield strength of 500 MPa. Higher steel grades (e.g. S690) are also possible by the TM-process but in a more limited thickness.

The steel manufacturing process can be Basic-Oxygen-Furnace, Electric-Furnace, etc., and is generally the option of the manufacturer. If production process is carefully controlled, the properties such as hardness, ductility and tensile strength can be predetermined to fulfill a variety of uses. The metal is finally shaped under temperature controlled conditions which can alter its characteristics and strength.

In general, the strength of steel is controlled by its microstructure which varies according to its chemical composition, its thermal history and the deformation process it undergoes during its production process. The strength can be enhanced mainly in two ways: by grain refinement (and precipitation hardening) (Figure 1.14) or by increasing the carbon content (CE)-or carbon equivalent (CEV)-, (Figure 1.15). In case of grain refinement, the material obtains high strength, accompanied with good toughness and excellent weldability, while increasing the carbon content, makes the material more brittle.

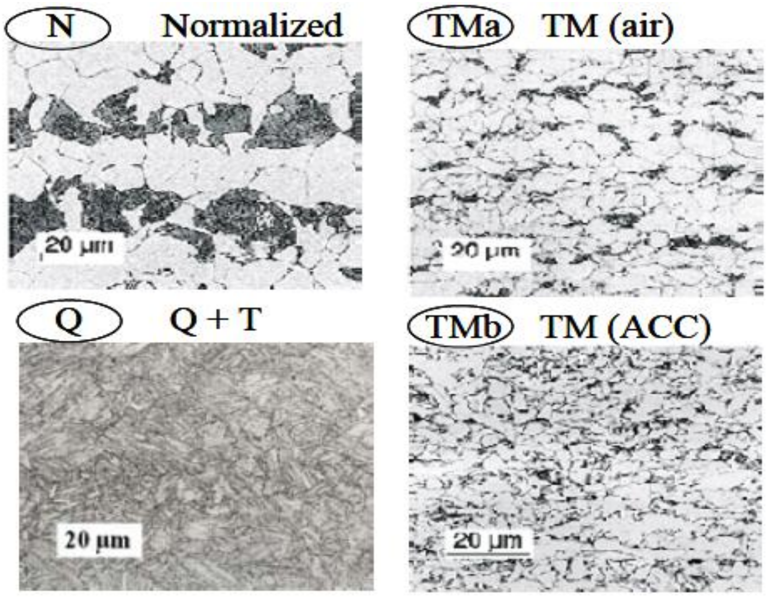


Figure 1.14 Microstructure of conventional normalized steel compared to TM, TM+ACC and Q&T steels [26].

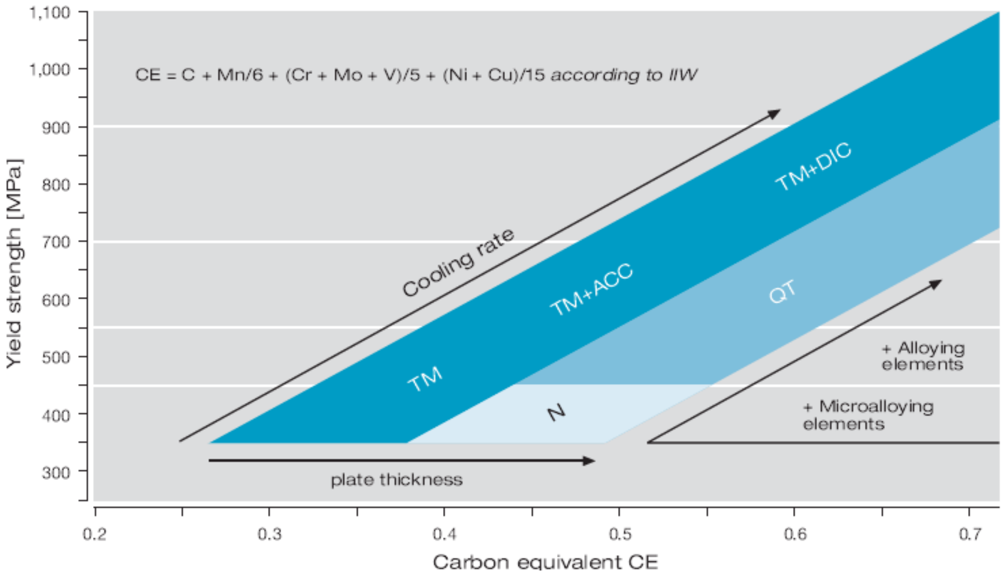


Figure 1.15 Effect of CE and steel processing route on plate strength.

Where,
N: Normalized
QT: Quenched and tempered
TM: Thermomechanically rolled
ACC: Accelerated cooled
DIC: Direct intensive cooled

In Figure 1.15 the relation between the carbon equivalent and the yield strength is plotted. While raising the carbon-equivalent increases strength it also drastically reduces other engineering properties (e.g. weldability). Figure 1.15 shows also that, the same yield strength level is possible on different levels of carbon equivalent depending on the delivery conditions of steel material. However, since welding is irreplaceable as a method of fabrication, the carbon-equivalent mechanism of steel strengthening cannot be used in many applications requiring good weldability (e.g. bridges).

On the other hand, grain refinement and precipitation hardening, increase strength and also improve toughness. Because these two dominant strengthening mechanisms operate in micro-alloyed steels, their carbon content may be very low. This low-carbon content contributes to excellent weldability.

In practice, grain refinement can be achieved during hot rolling by the interaction between micro-alloying elements (niobium, vanadium, or titanium) and hot deformation. Grain refinement may be further enhanced by accelerating cooling after the completion of hot rolling.

Hot and Cold Rolling

There are various methods of forming steel into finished products, including hot forging, hot and cold rolling, seamless tube making and welded tube making. The most widely used process is hot rolling, which accounts for over 90% of all steel production. There are many types of rolling processes, including flat rolling, foil rolling, ring rolling, roll bending, roll forming, profile rolling, and controlled rolling.

Rolling is a metal forming process in which metal stock is passed through a pair of rolls. Rolling is classified according to the temperature of the metal rolled. If the temperature of the metal is above its re-crystallization temperature, then the process is termed as hot rolling. If the temperature of the metal is below its re-crystallization temperature, the process is termed as cold rolling. In terms of usage, hot rolling processes more tonnage than any other manufacturing process and cold rolling processes the most tonnage out of all cold working processes.

High strength steels produced in the Q&T method suitable for hot working only at temperatures below 550ºC since very high temperature affects their mechanical properties [29].

1.1.5.1 Production of Q&T HSS

By quenching and tempering, structural steels can reach minimum yield strength of 1100 MPa. This heat treatment applied subsequent to hot rolling, consists of an austenitisation, followed by quenching and finally tempering [26].

Quenched and tempered steels are used for components subjected to high stresses where the combination of high strength, wear-resistance and toughness are particularly important. Quenching and tempering gives the materials their special properties. Temperature control during quenching and tempering is essential to achieve the desired component properties; however, it must be matched to the respective application.

Generally, the alloying composition of Q&T steels increases with increasing plate thickness in order to ensure sufficient hardening of the plate in the core region. So, The CEV of a Q&T plate increases with increasing thickness. Most high strength Q&T structural steels are produced with a carbon content of 0.12-0.18 %. Figure 1.16 can explain why this is the most favorable range. The martensite hardness increases linear with the carbon content.

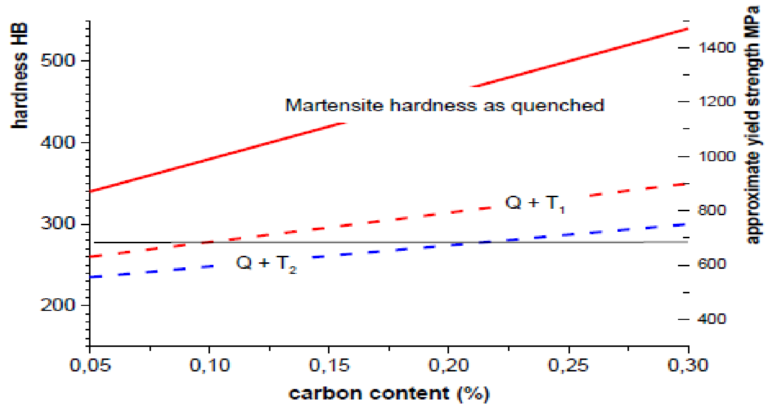


Figure 1.16 Influence of carbon content on hardness and yield strength of Martensite as quenched and after tempered at two temperatures (T2>T1). Relation hardness and yield strength is not perfectly linear and so the right y- axis serves for an estimation only.(Dillinger Hütte, [18])

Quenching

Heat treatment process employed to produce high strength steels involves quickly cooling austenized steel in a quenching medium like air, water or oil. Heat treatment results in the formation of martensitic microstructure, which makes the steel not only extremely hard but also brittle. However, by subjecting steels to heating or tempering (Q&T) after quenching, an optimum combination of high strength and ductility can be achieved.

Tempering

Tempering is heat-treating of metal alloys, particularly steel, to reduce brittleness and restore ductility. In tempering, steel is slowly heated to a temperature between 150 °C and 700 °C (for appropriate toughness, tempering is performed at least at 550 °C), depending on desired properties, in an oil or salt bath and held for about two hours and then allowed to air cool. As steel is physically worked (e.g., rolling, wiredrawing, hammering), hardening takes place, and it grows progressively more brittle.

Heating and quenching also increase hardness. Combined quench-and-temper heat-treating is applied at many different cooling rates, holding times, and temperatures and is a very important means of controlling the properties of steel. Strength and hardness, generally, decrease with increasing tempering temperature and holding time. On the other hand, results from tensile tests show that, elongation and area reduction increase as the tempering temperature and holding time are increased, Figure 1.17 [27].

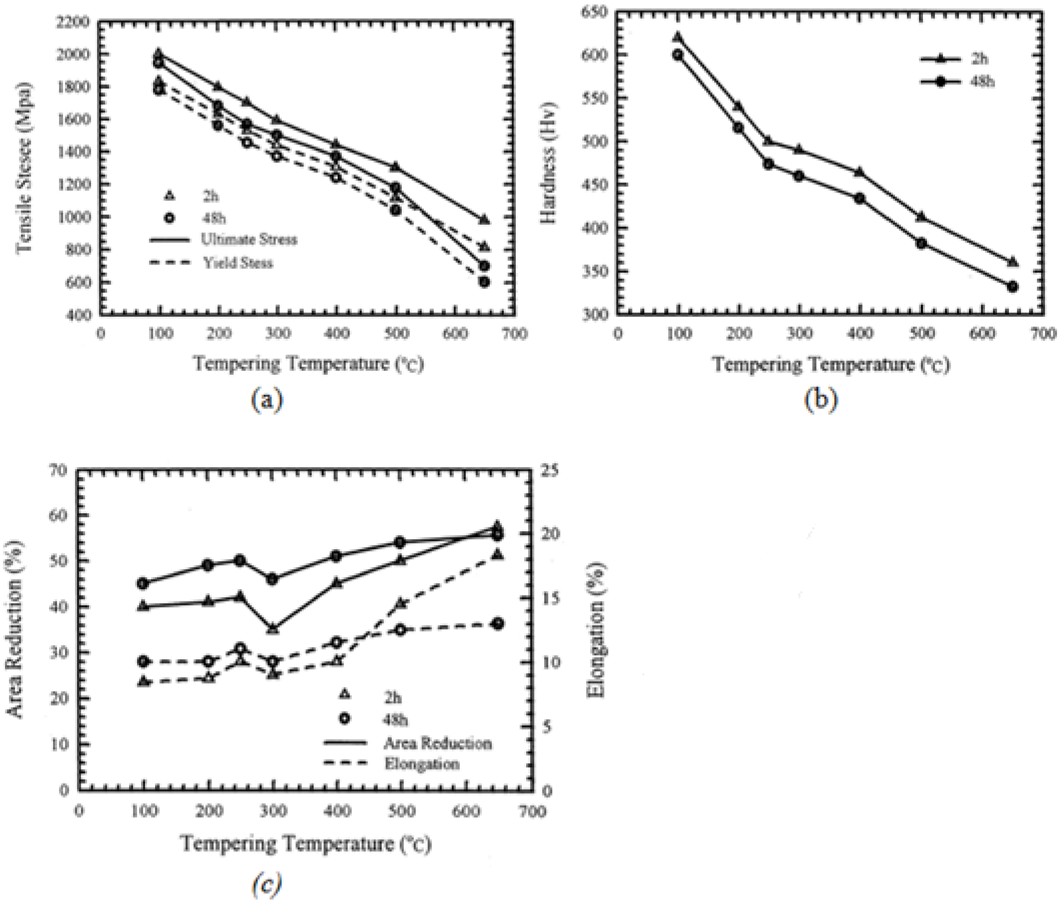


Figure 1.17 Effect of tempering temperature and holding time on (a) tensile strength, (b) hardness, (c) elongation and area reduction (ductility) of Q&T high strength steels [27].

In Figures 1.18 and 1.19 an example of 60 mm thick “Dillimax 890” (Q&T fine grained structural steel with minimum yield strength of 890 MPa produced by Dillinger Hütte GTS) steel is presented with respect to the effect of tempering temperatures on the strength and toughness properties of the material.

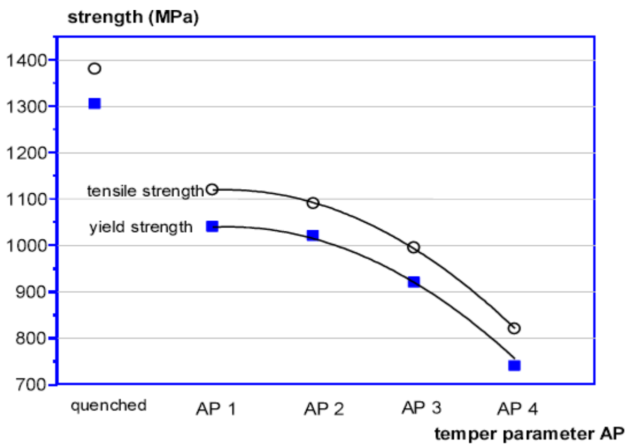


Figure 1.18 Influence of increasing tempering temperatures on tensile properties of Dillimax 890 in 60 mm thickness [18]

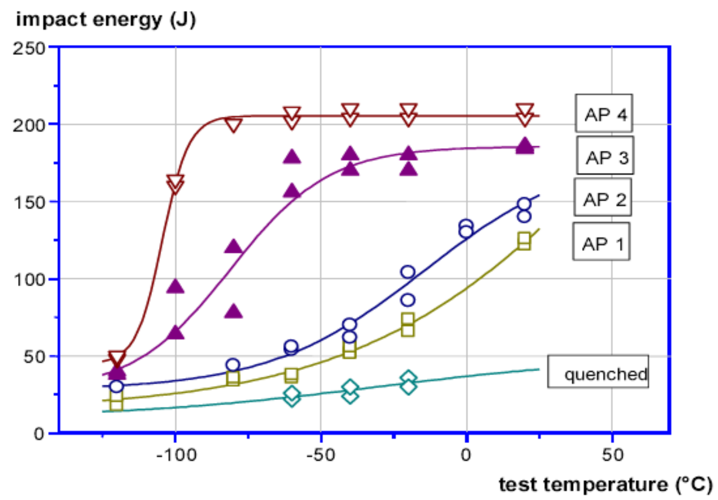


Figure 1.19 Influence of increasing tempering temperatures on the Charpy impact transition of Dillimax 890 in 60 mm thickness [18]

1.1.6 Fabrication of HSS

Fabrication consists of cutting pieces of steel and connecting them together. The material is generally obtained from rolling mills or stocks in the form of I-sections, channels, hollow sections, angles or plates. The steel specified should be rationalized to use relatively few section sizes and a common grade. The sections are being cut to length, drilled and welded as necessary ready for assembly, and in most cases some protective treatment is applied against corrosion.

Operations can include cleaning, sawing, shearing, punching, grinding, bending, drilling, welding and the finishing of the steel. These involve extensive use of numerically controlled processes which improve productivity and quality. Cranes are always involved for moving material within the factory, but the use of mechanical conveyors is more efficient. There are saws and guillotines, drills and punches, and facilities for flame cutting and welding, both by hand and by machine.

Fabricator's shops vary both in the size of the facility and weight of material that they can handle. The sophistication of the available equipment also varies. Increasingly, where investment is available, operations are being automated and computer-controlled. This is necessary, particularly, for fabricating higher strength materials.

In Europe only a limited number of fabricators can process and fabricate HSS mainly because of the required investment. However, in a few years, as the market demand for the new steel material will increase it will be essential that all fabricators invest in new equipment.

1.1.7 Machinability of HSS

Generally, for high strength structural steels with yield strengths below 850 MPa, machinability is considered to be good. In order to achieve good productivity and tooling economy is essential that the right tool is chosen for the right application. The yield stress of the steel plate has to be exceeded during forming and the shear rupture strength has to be exceeded during cutting. This means that in forming and cutting (harder) HSS the forces needed to perform the operation are higher than the lower (softer) steel grades of the same thickness. In the same way, the demands on wear resistance and mechanical strength of the tool material increase. HSS, in the range of S690 QL steels, is usually available from 3-120mm thick plates and can be machined and drilled using high speed steel or cemented carbide tools [28]. It can also sheared, flame cut, cold bended or hot formed.

The tooling environment becomes more complex and demanding with the new HSS material. Rapid deterioration and/or rapture of the tool will more likely be the result of inadequate selection of tool material. This means that the selection of tool steel and coating processes for forming and cutting operations in HSS should not be based on what was done with softer mild steels. Instead, the latest technical innovations should be used to optimize the production economy [28].

For the fabricators higher quality tool steel will lead to a small increase in the cost of tooling but usually give a large return on the investment (Figure 1.20).

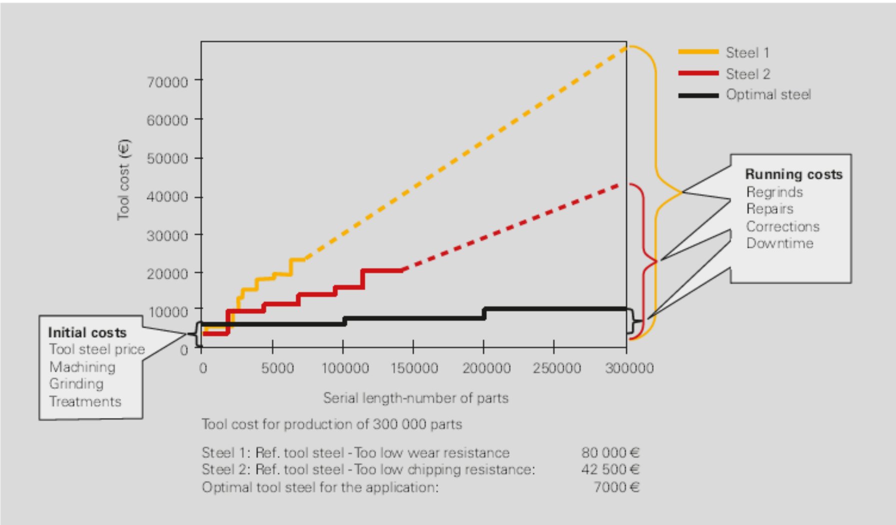


Figure 1.20 Total cost considerations. Steps in lines indicate cost for refurbishment. (Source: SSAB Swedish steel- Uddeholm Tooling [28])

1.1.7.1 Mechanical cutting for HSS

[61] Cutting is an important operation in producing sheets to size, removing waste material, making weld joint preparations and removing defects. Cutting can involve a number of techniques, including:

- Mechanical (such as machining, sawing, shearing, punching, drilling)
- Thermal (for example, oxygen cutting, spark erosion, laser cutting, plasma)

- Hydraulic (water and abrasive water jet cutting)

Regardless of the strength or hardness of the steel plate, successful mechanical cutting requires that the plate is allowed to warm up throughout to room temperature +20 ºC. It is recommended that high strength steels are mechanically cut with guillotine shears.

The most important factors are clearance and cutting angle. The hardness of the cutting blade, also, has substantial effect on the cut. The clearance influences the service life of the cutter blades and therefore the cutting costs. A suitable clearance reduces the stresses subject to the cutter frame. This extends the usable life of shears and allows cutting thicker plates. When cutting HSS the clearance must be increased [29].

If however, plate thicknesses are kept small (about 30mm maximum), no special machinery and techniques are necessary for mechanical cutting of Q&T high strength steels [62]. As the thickness of HSS and the hardness increases, however, thermal cutting is more appropriate.

1.1.8 Costs

1.1.8.1 Material Costs

The cost of steel is typically driven by a number of factors, including the price of the raw material, the price of energy, and the supply-demand relationship for that specific type of steel. It is therefore, important to know the price of the entire steel package and not simply the cost of the material.

Typically, the cost of materials represents only 25-30 % of the total structural steel package. The remaining 70-75 % of the cost is fabrication and erection. Even a 20 % increase in material costs would only result in a 5 % increase in the cost of the steel package [30]. Moreover, the material costs are determined in terms of weight and depend on the manufacturer. Under these considerations HSS can be proved beneficial since the total amount (weight) of material would be smaller and therefore costs for fabrication and erection would be reduced.

Material prices for HSS are currently much higher than mild structural steel grades. As an indication, in the Netherlands S690 is about 70-75 % more expensive than S355 per kg of steel at the moment [source: Mercon Steel]. However, the price is expected to decrease in the next few years as the market demands for high strength steel grades will increase.

1.1.8.2 Manufacturing costs

Flame cutting, drilling and punching holes usually entail virtually the same costs as for regular steel grades. However welding is much cheaper because, thinner plates mean less welding volumes and also preheating to avoid cold cracking may be omitted for HSS for plate thicknesses below 30 mm, (provided that the correct choice is made as to steel quality, consumables and welding process) [2].

2 Design with High Strength Steel

2.1 Codes and Standards

In Europe, the design of higher steel grades, (with a yield strength above 500MPa), is currently, inadequately covered in Eurocodes. EN 1993-1-12:2007 has recently been developed; concerning the design of HSS grades with minimum yield strengths between S460 MPa and S700 MPa, and provides additional rules to the extension of EN 1993 for the design of steel structures. Quality standard EN 10025:2004 is also the new European standard with rules for the delivery conditions of structural steel. It contains six parts and especially EN10025-6 deals with the technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition which are widely used in structural applications.

The bigger restriction of using HSS is lack of detailed design codes and rules. This fact in combination with a higher price for HSS grades (due to their improved properties) are the two major drawbacks when material is to be chosen for structural design. In addition, applying the current design codes, may occasionally lead to conservative and non-cost effective solutions, thus reduce the competiveness of the material.

However, in a design tailored to meet certain specifications (i.e. where the properties of HSS can be fully utilized), the use of HSS steel offer many advantages (see chapter 1) over mild steel grades. Testing and experimental research is the main resources of gaining knowledge on the behaviour and structural performance of these steels. These studies, many times, aim to propose new design rules using the new material. However, further research is necessary which will help to establish new design rules for these grades, to fully utilize their benefits in steel construction.

Nevertheless, several experimental studies and testing have already been performed, from fabricators and technical institutes/universities, worldwide, which have shown positive results in the structural performance of these steel grades. For example, in 2006, TU-Delft started the research project “Very High Strength Steel for Structural Applications”, to investigate the possibilities for using these grades in structural applications in the future. However, further research is necessary which will help to establish new design rules for these grades, to fully utilize their benefits in steel bridge construction.

Finally, HSS and VHSS is been used for many years already in other industries such as crane structures (very thick plates) automobile (very thin plates) or offshore construction and many valuable information from their experience can be gained to enhance a broader use of the material in bridge construction also.

2.2 Bridge design in high strength steel

2.2.1 General

High strength steels suitable for bridges (e.g. quenched and tempered structural steels like S690QL) combine properties like weldability, toughness, ductility and corrosion resistance (weathering high strength steels) in an optimized and balanced way while remaining cost effective. In Europe, their use is still quite limited, while in USA and in Japan are more widely used (460-690 MPa minimum yield strengths).

The main benefit gained, where strength governs, lies in creating more slender and lightweight structures, thus reducing overall costs, (material, welding volume, transportation etc.). However, in case of bridge design, the governing factors are usually stiffness and fatigue. In this respect, it is more often assumed, that a material offering higher tensile strength will not be beneficial for these structures. Reduction of the cross sectional dimensions of the main girders, for example, is limited as this will more likely create stiffness and/or fatigue problems locally or globally.

In addition, according to “design- build” concept, the bridge design should always be developed to optimize the construction methodology which is a primary component of the costs [31]. It is essential therefore, that the whole perspective of designing a bridge should change for acquiring the full benefits of higher steel grades.

2.2.2 Bridge design aspects and experimental research review

Once again, the advantages of using high strength steels are generally a result of reduced weight and dimensions. Design stresses can be increased and plate thickness may be reduced, resulting in significant weight savings. Reduced plate thickness can also save on welding costs. Simplified structural components and construction techniques are often possible, particularly for large structures, and foundation costs may also be reduced due to lower dead weight.

Especially for applications in bridges, quenched and tempered high strength steels, in the range S460 (460 MPa minimum yield strength) to S690 (690 MPa minimum yield strength), are to be used, since they combine very high strength, with high toughness (even at low temperatures) and very good weldability. Higher steel grades will more likely not be beneficial due to stiffness and fatigue limitations.

The most economical and efficient use of Q&T steels is in members stressed in tension where the high strength can be fully exploited, and in projects where the dead weight of the steel is the predominant load, such as in long span bridges. In compression they are most effective in heavily loaded, stocky columns or in stiffened compression elements where buckling is not the controlling criterion.

One of the oldest bridge examples, the 980 m lattice girder bridge, Minato Ohashi Bridge in Osaka, Japan, uses large tonnage of TS780 MPa steel (min f_y = 690 MPa) in its fabricated box sections. The bridge was constructed in 1974 [Japan Welding Engineering Society, JWES]. When the cost of this design was calculated for different grades of steel, it became clear that, although high strength steel cost more, this was more than offset by reduced fabrication and erection costs. Such examples are not uncommon when the application is well suited to the specification of Q&T steels [1].

Figure 2.1 ***Minato Ohashi Bridge (1974), Osaka, Japan. [JWES]***

The bridge design should always be developed to optimize the construction methodology which is a primary component of the costs.

In the USA, for example, the steel industry had a vision, several decades ago, for expanding the scope of the current specifications for composite structures from simple I-girder and Box-girder to include a variety of steel bridge configuration to encourage the use of other forms where feasible and to provide the necessary guidance to designers.

Thus, in order to make effective use of high performance steel grades some changes to the standard I and box sections were proposed, including unique shapes and methods of fabrication [31]. These potential concepts are shown in the next page, Figure 2.2.

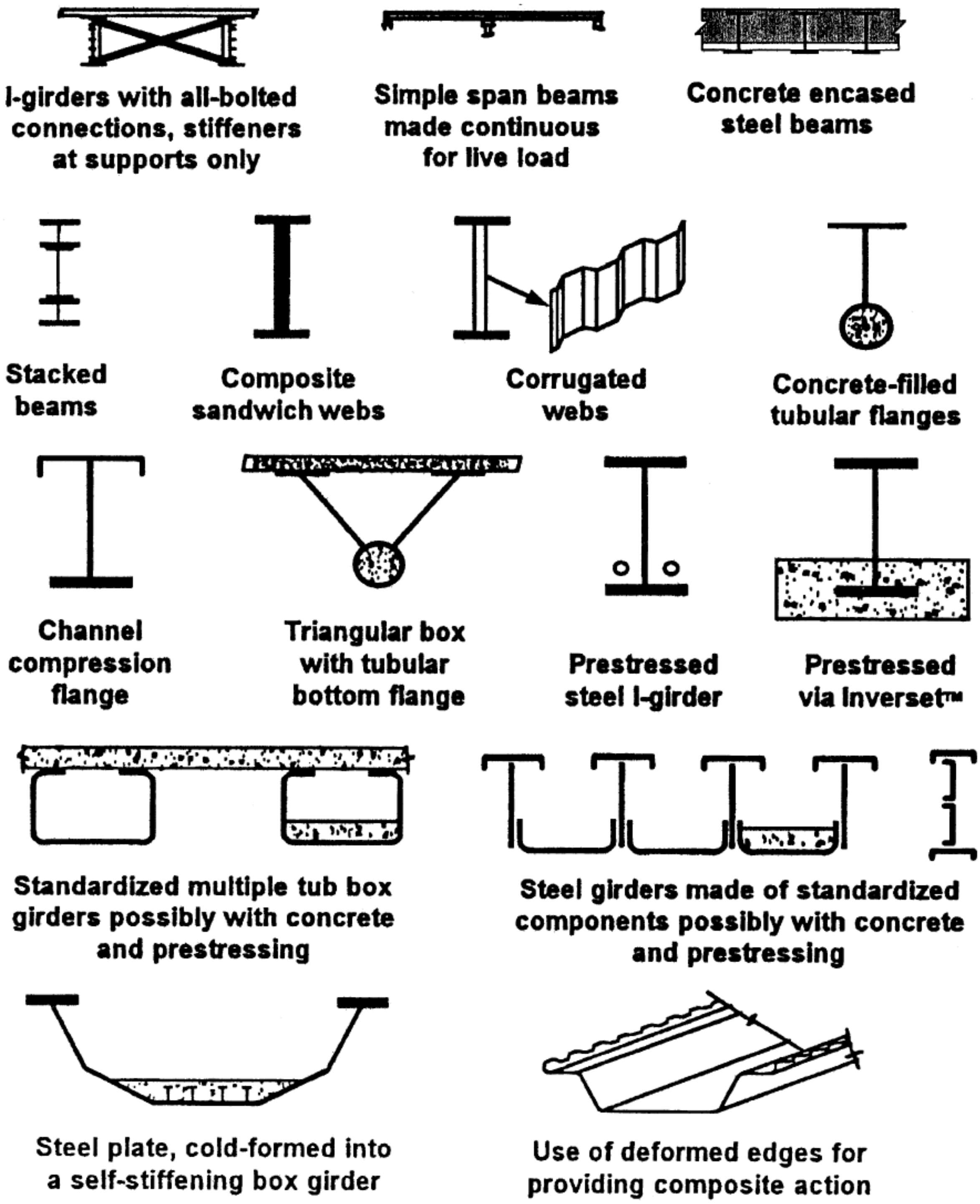


Figure 2.2 Possible concepts designing with HPS, [U.S. FHWA, 1989 [31]]

Furthermore, in order to minimize construction costs, bridge structural systems in Japan [32] are shifting to the simple structures shown in Figures 2.3 for I-girders and 2.4 for box girders.

The basic concept is to reduce the number of main girders to a minimum and also minimize the use of stiffening members, thus significantly simplify fabrication. In addition only small sized cross beams are arranged between the main girders and the use of lower lateral bracing is eliminated.

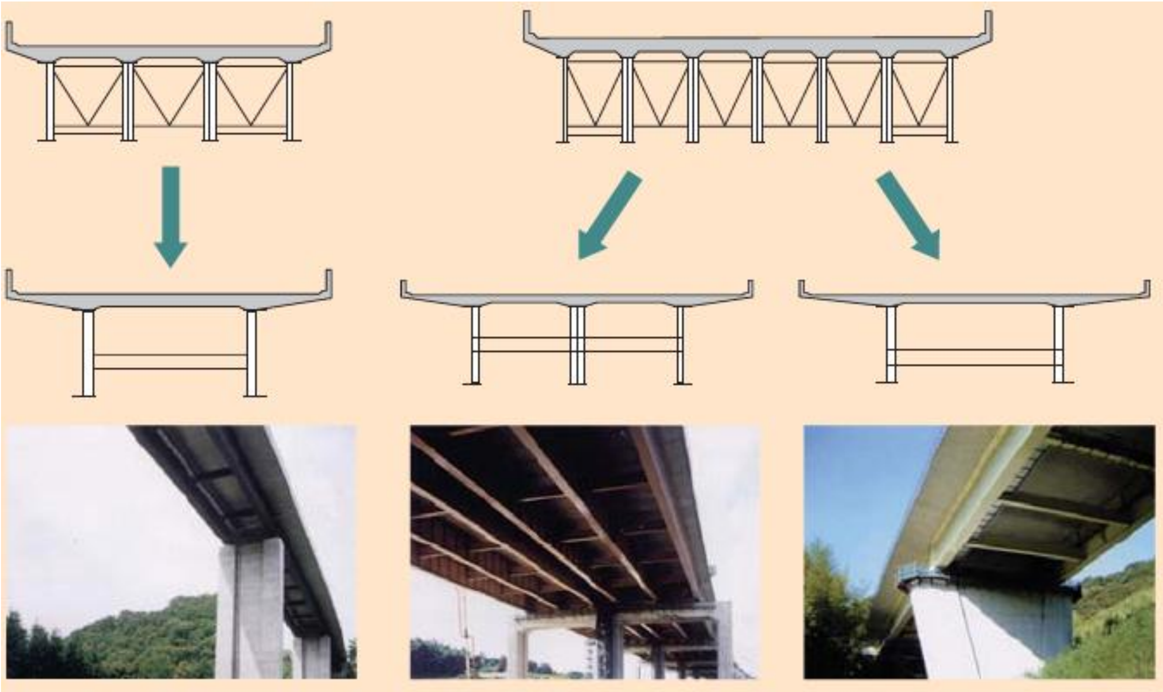


Figure 2.3 Structural innovations for I-girders (Dr. M. Nagai, 2006) [32].

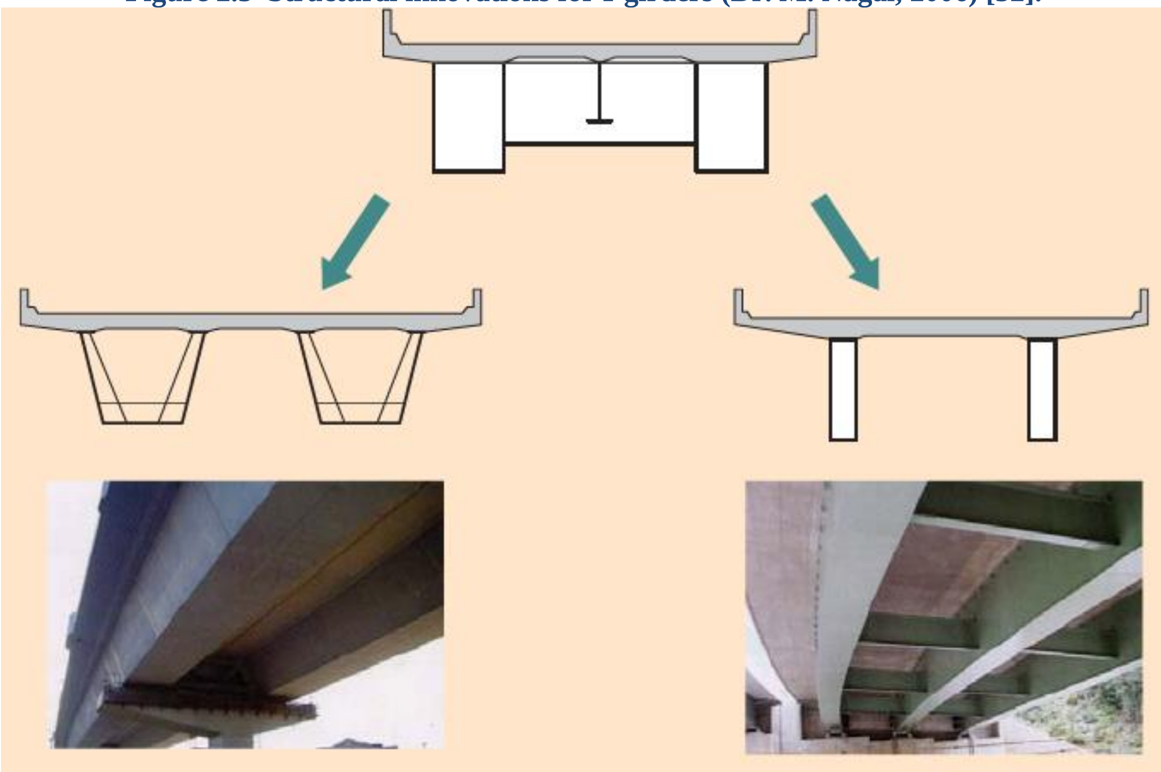
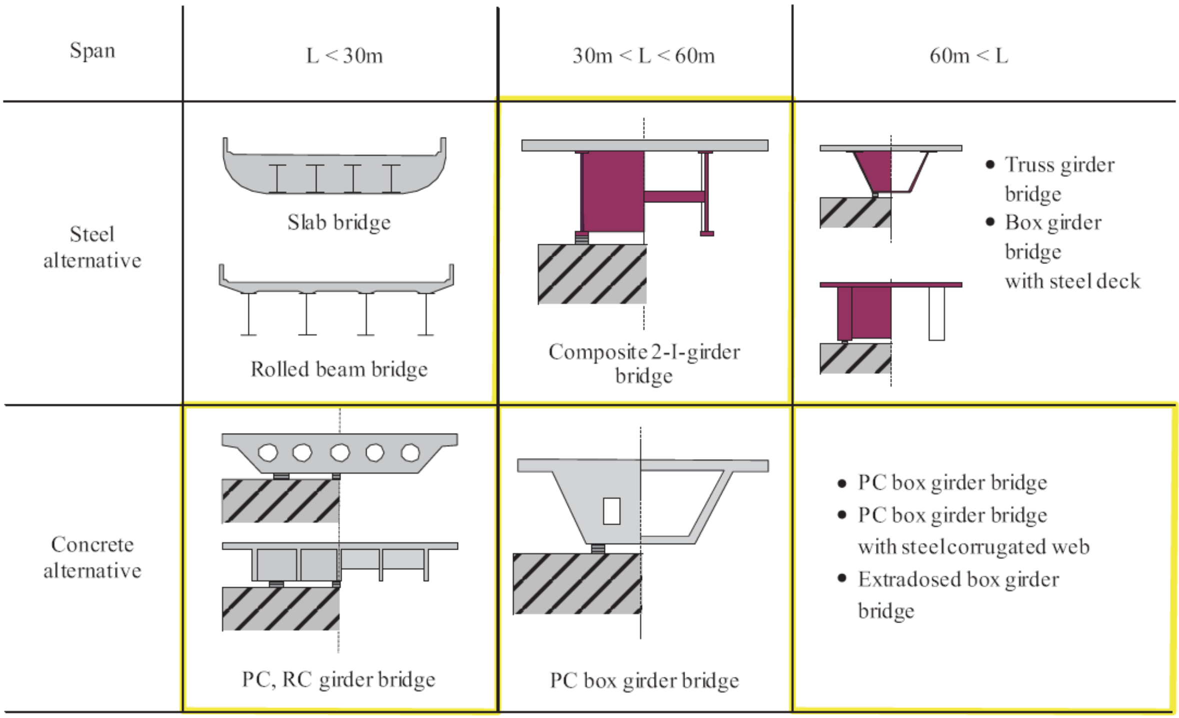


Figure 2.4 Structural innovations for Box girders (Dr. M. Nagai, 2006) [32].

These types of construction are economical for span ranges between 30-60 m as shown in table 2.1.

For small spans (<30 m) and for spans longer than 60-70 m concrete bridges are more economical. A steel alternative expected to be very competitive in the range of 70-120 m, is the double composite I-girder bridge {concrete deck between two main girders). This system is expected to prevent the buckling of thin steel plates subjected to compression and to improve bending strength and torsional rigidity.



Inside of a yellow frame: Competitive (economical)

Table 2.1 Economical steel-bridge and concrete bridge alternatives according to span length (Dr. M. Nagai, 2006) [32].

Furthermore, competitive alternatives can be expected for I-girder bridges in “hybrid” construction concept. In this case, HSS members are adopted for the flanges and relatively low strength steel members for the webs. When wider width is required, twin box girder bridges composed of un-stiffened steel plates can be proposed as a competitive alternative [32].

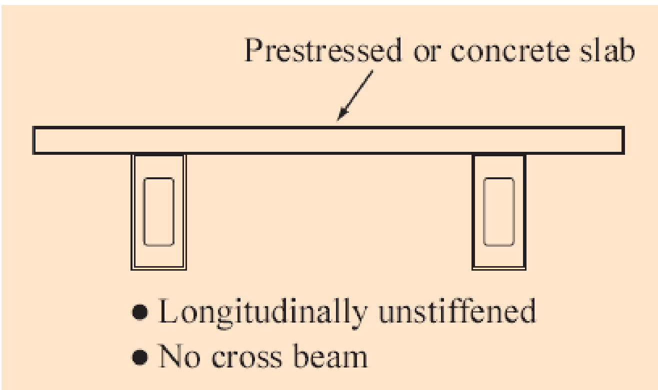


Figure 2.5 Double un-stiffened box-girder with prestressed or concrete slab, [32].

In 1997 a review was performed from TU-Delft in cooperation with TNO, with the scope to explore the possibilities for cost-effective applications of HSS (up to S460) in bridges, buildings, cranes etc. [34]. According to this review HSS is especially cost effective in situations where the weight of the structure forms an important part of the load e.g. long span bridges, high rise buildings and cranes.

HSS may be more expensive to purchase, however, in the fabrication welding is the most costly. HSS have good ductility and weldability which has a favourable effect on mechanical behaviour of the structures and will reduce the fabrication costs significantly. Furthermore, it was concluded that new design rules help design more economically in high strength steel. Today EN 1993-1-12 serves this scope so far in addition to testing and experimental research.

Table 1 Example of cost reduction for a plated structure (submerged arc welding, 4 m long x-joints). Data from Dillinger Hütte[24b]

	Steel grade					
	S355	**S460**	**S550**	**S690**	**S890**	**S960**
Plate thickness (mm)	65	50	40	33	25	23
Base metal costs (%)	65	58	60	48	50	48
Welding costs (%)	35	25	17	14	8	5
Total costs (%)	100	83	77	62	58	53

Table 2.2 Example of plated structure - Data cost savings, [Dillinger Hütte].

In addition, some general guidelines were given depending on the type of the loading that are summarized here:

Members in tension: If tension governs the design, a choice of S460 instead of S355 gives a material saving of 30%. If for example, S460 is considered 20% more expensive, then the overall cost savings will be 10% but on top of that the savings in fabrication costs are much greater.

Stiffness: Where deflection criteria govern (SLS), HSS is not favourable with the current design rules. This is so because higher stresses with the same E-modulus will cause bigger deflections. It could be possible, however, that SLS limits for deflections could be somewhat relaxed in some cases to allow for a more effective use of HSS. New design rules need to be developed for that.

Members in bending: Application of HSS may have a big influence on the design choice. A cost effective choice in that case is the hybrid construction, where the webs on the welded girders are made from lower steel grades (e.g. S355) and the flanges of HSS (e.g. S690). This is so, because the web plates usually, have to be design to satisfy instability criteria. In that case working stresses are lower due to buckling and a higher grade is not effective.

Members in Compression: Here buckling phenomena govern the design which is based on E-modulus. This is the same for all steel grades, but if slenderness is low $\lambda < 60$-80, then the weight savings will still be obtained. Furthermore, sections in HSS have relatively lower residual stresses than sections in S355 or in S235. Therefore, design codes (EN 1993 National Annex D) give for example, a better buckling curve for steels S420-S460. Generally, Table 6.2 for choosing the buckling curve in EN 1993-1-1:2005, holds also for steel grades up to S700 (EN 1993-1-12:2007).

2.2.3 Buckling

Numerous studies have examined, furthermore, the buckling behaviour of HSS (e.g. [35]). In general, they conclude that HSS performs better or at least not worse than ordinary steel grades (for both flexural and local buckling). It is therefore possible that, the normal design rules can be used as a conservative solution [36]. The main reason why HSS behaves better is a smaller influence of geometric imperfections.

The next two graphs show results from the study Güven Kiymaz performed, on high performance steel plates under uniform edge compression [35]. The first graph, Figure 2.6, shows the influence of various imperfections on the behaviour of S690 steel grades for intermediate values of slenderness (relative slenderness λp=1).

The second graph, figure 2.7 shows relation of non-dimensional ultimate strengths of uni-axially compressed imperfect plates (geometric imperfections and residual stresses) and slenderness. In the same figure the elastic buckling curve, Von-Karman's post buckling curve, Eurocode 3 Part. 1.1 (1993) and BS 5400 (1980) plate strength curves and the curve proposed by Chou (1997) are also presented.

It is shown that the influence of yield strength is small for intermediate values of plate slenderness while it is gradually increasing with increasing plate slenderness. Also in the stocky range, high-strength steel plates seem to be stronger than normal grade steel plates when compared in a non-dimensional basis. [36]

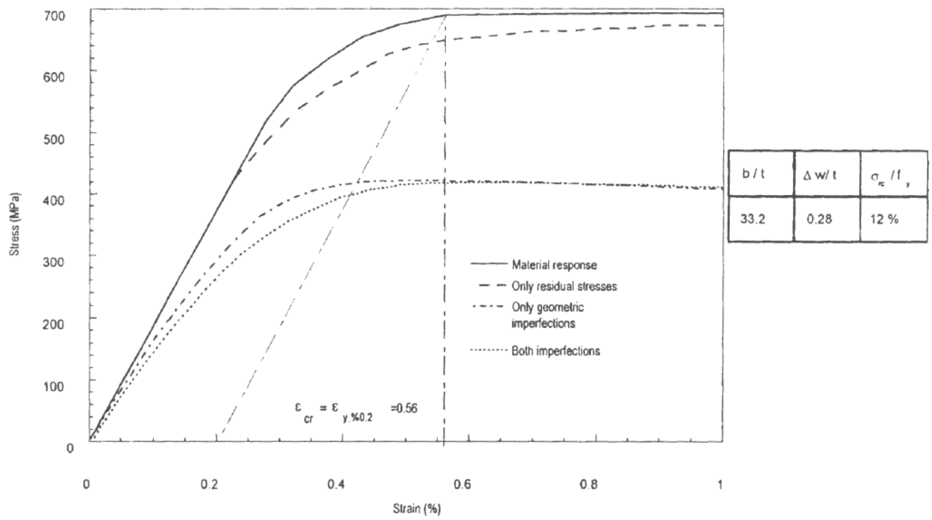


Figure 2.6 ***Stress-Strain behaviour of a uni-axially compressed plate for various imperfection cases, fy= 690 MPa and the relative slenderness λp=1. (Güven Kiymaz,2003 [36])***

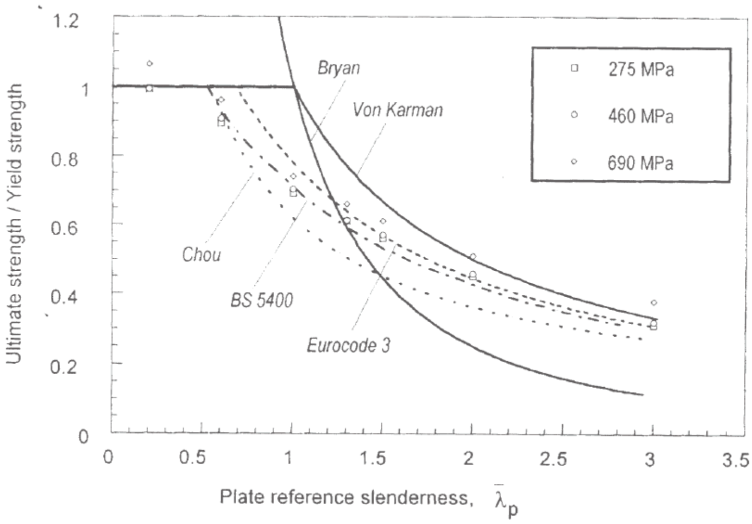


Figure 2.7 Non-dimensional strength of uni-axially compressed imperfect plates with various yield strengths over a range of plate slenderness. (Güven Kiymaz, 2003 [36])

2.2.4 Fatigue

The graph in Figure 2.8 shows that fatigue strength increases with tensile strength for plane materials, but in welded and notched details fatigue strength is independent of increasing yield strength. This is why the application of high strength steel in fatigue loaded structures with a high number of stress cycles during their lifetime, such as bridges, is still questionable.

Experimental research though, based on fatigue data for parent and welded high strength steels, indicates that the general performance of high strength steels is as good as the medium strength steels.

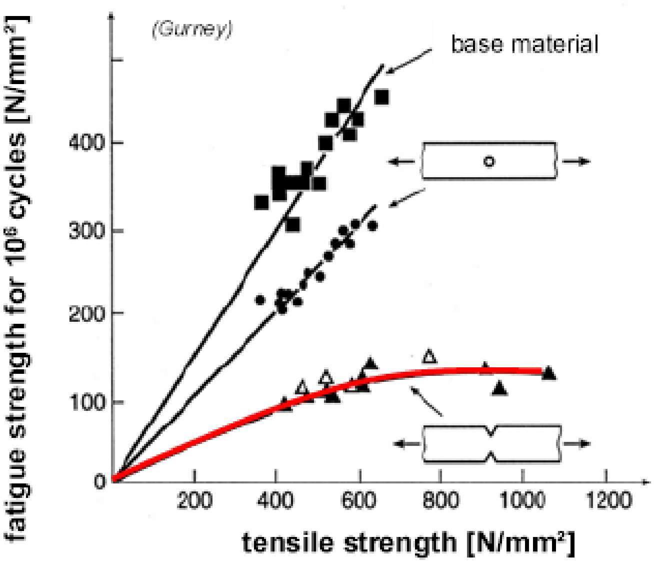


Figure 2.8 Fatigue strength in relation to material yield strength [37].

Gurney (1979) concluded that the higher the yield strength of base materials the more sensitive the fatigue strength of the material becomes to both the presence of notches and to the surface condition. In case of low notch values notch sensitivity of fatigue strength is minimized. Therefore the use of high strength steel in welded connections requires high fabrication quality and avoidance of large stress concentrations in joints [38]

2.2.4.1 Fatigue of welded details

It is well known that welded details determine the fatigue behaviour in a welded structure. This was also shown in Figure 2.8. Therefore, several conditions need to be fulfilled for making use of the HSS in fatigue loaded structures in an effective way. Welded details should be at the least severe loaded locations in the structure, wherever possible.

It is also important to ensure high quality details with good surface finishing minimizing defects, so that large stress concentrations at these locations are avoided. Removing overfills in welded joints, for example, can improve the fatigue behaviour of the joint, although it will remain lower than that of the parent metal [39]. However, in cases of high overloads high strength steels can be more beneficial than regular grades, since it allows for higher stress levels.

Puthli et al. (2006) examined the fatigue strength of longitudinal attachments welded to plates and tubes made of high strength steels S460, S690, S960 and S1100. They showed that high strength steels do not exhibit any disadvantage in fatigue resistance compared to mild steels.
EN 1993-1-9 (2005) indicates that the fatigue strength of the steel structure depends on the applied detail (notch factor), plate thickness and machining condition. This standard is generally applicable also to high strength steels covered by EN 1993-1-12 (2007).

Furthermore, fatigue performance of welded details is strongly dependent on the initiation of a crack, usually located at the weld toe or at the weld root, followed by crack propagation. The number of cycles required to initiate a crack depends on applied stress, weld geometry and material properties. Crack nucleation at the weld joint is related to the fatigue strength of the base material because the presence of the welded joint acts as a notch [40].

Since fatigue crack propagation characteristics are the same for low and high strength structural steels an effective way to improve the fatigue resistance of welded details in HSS, would be to introduce a longer initiation period [41]. This can be achieved in two ways. An improved weld procedure which will give smaller global and local stress concentrations and a global post weld improvement technique (e.g. post weld shot peening).

The strength of the weld metal in relation to the strength of the base metal is particularly important for the behaviour of a welded joint. In normal steel grades the weld metal is always stronger than the base metal (over matched welds) in order to achieve a more ductile behaviour of the welded connection and therefore enhance the safety of the structure.

When HSS (above S690) is used, however, the weld metal will be weaker than the base metal (under matched welds). When such joints are loaded up to failure, large strains will occur in the welds and not in the adjacent parent metal. Due to small dimensions of the weld the deformation capacity of these connections can be limited and potential stress redistribution will also be limited (the weld will fail before the load is redistributed). So special attention should be paid when under matched welds are used with respect to deformation capacity and the SCFs (stress concentration factor resulting from unequal stress/strain distribution, which in case of under- matched welds will be concentrated In the weld itself).

Full penetration butt welds have the lowest SCF factor compared to fillet welds and/or partly penetration butt welds. Material thickness is also an important parameter, especially in the high cycle fatigue strength of welded details. Generally, the thicker the plates the lower the fatigue strength under the same stress range [42].

2.2.4.2 IMPROVEMENTS FOR BETTER FATIGUE PERFORMANCE

2.2.4.2.1 Post weld treatments

The graph in Figure 2.9 is based on a parametric study [37] on medium span (20-70 m) composite bridges (with steel grades S355, S460 and S690), as an example showing the necessity for the improvement of fatigue resistance in case of high strength steels.

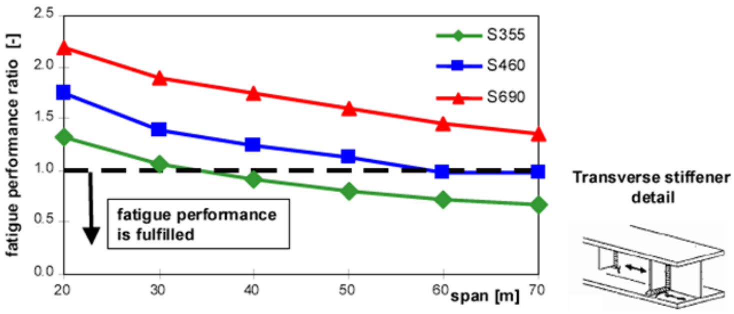


Figure 2.9 Fatigue performance of a highway composite bridge and transverse stiffener detail [37].

Improvement can be achieved by post weld treatments. Generally, post weld methods can significantly improve the fatigue strength of welded structures especially under constant amplitude loading, Figure 2.10. Processes providing best fatigue resistance are those which change weld toe geometry and introduce compressive residual stresses at the weld toe [43]. This is consistent with fatigue tests which exhibit weld toe fatigue crack initiation systematically.

Significant improvement is obtained by GTAW remelting of the weld toe and shot peening [43]. Shot peening can significantly improve the fatigue performance and enhance further the use of HSS in bridge construction. It has the advantage that is a “global treatment” and not only localized at the weld toe.

The results of shot peening on fatigue lives of offshore welded connections are remarkable, especially in case of high cycle fatigue [41]. Furthermore, this method is easy to implement in the industrial process at lower costs compared to other methods such as GTAW dressing, toe grinding, or hammer post welded treatments.

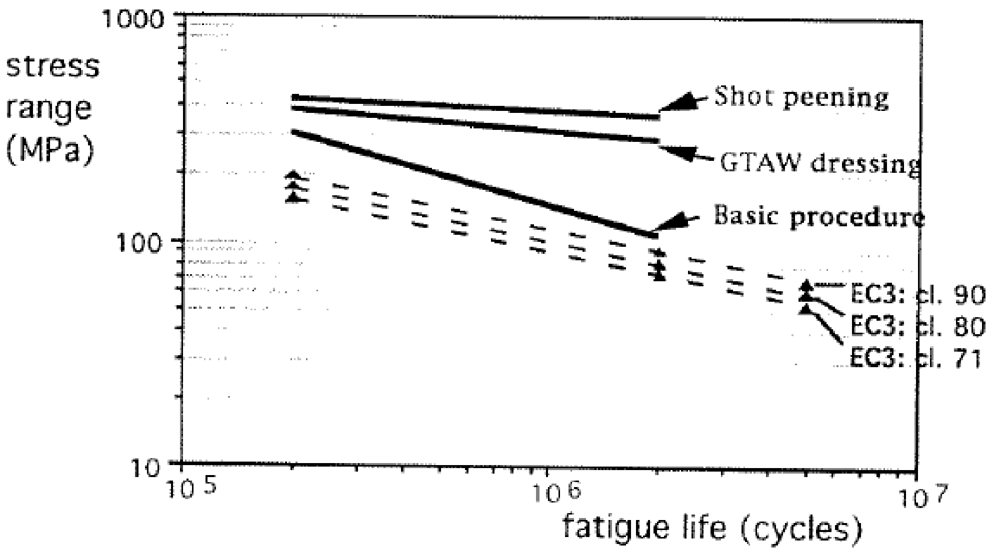


Figure 2.10 Improvement techniques for fatigue performance [43].

The international institute of welding (IIW) gives recommendations with regard to post weld improvements of arc welded steel structures [63]. Four methods are described namely, burr grinding, tig dressing, hammer peening and needle peening. The benefits from all these methods are higher for steels with specified yield stress above 350 MPa in comparison to lower steel grades (i.e. for steels with specified yield stress above 350 MPa a factor of 1.5 on allowable stress range can be considered, limited to an increase to FAT 100). The higher fatigue category that can benefit from these treatments is FAT 90 (fatigue strength at 2 million cycles).

2.2.4.2.2 Cast joints

A very efficient way of reducing stress concentrations on welded details is to position them away from the critical, highly stressed, locations on the structure. Cast joints, if designed properly, successfully provide this benefit.

Cast steel is available up to 1100 MPa yield strength and is more and more applied in fatigue loaded bridge and offshore structures. A hybrid connection of cast steel welded to a rolled steel member could make the use of high strength steels relatively more efficient.

A very efficient and promising application of HSS is in stiff truss like structures, typically made of circular hollow sections (CHS). Use of castings in combination with CHS could be promising for the design of highly resistant fatigue joints [38].

In 2009 an experimental program was performed in TU-Delft University of Technology, in order to investigate the fatigue strength of hybrid connections made of rolled and cast steel with high yield strength [38]. The experimental program included two parts: Large scale test series on trusses made of welded CHS and K-joint cast members (Figure 2.11) and small scale test series on V-welded plate connections made of rolled and cast steel (Figure 2.12). Strengths for rolled and cast steels were 690 MPa and 890 MPa, while for the small scale tests also S460 steel was tested.

Results show, that fatigue class 71 in the Eurocodes, is rather conservative. Most cracks seem to initiate at the HAZ next to the weld locations, while the use of ceramic backing is found not to improve the fatigue performance. Instead, the steeper weld root angle of the specimens welded with use of ceramic backing resulted in lower fatigue strength. Furthermore the results on steels S690 and S890, showed slightly higher fatigue strength (although, the number of test specimens was quite low to evaluate the influence of the yield strength).

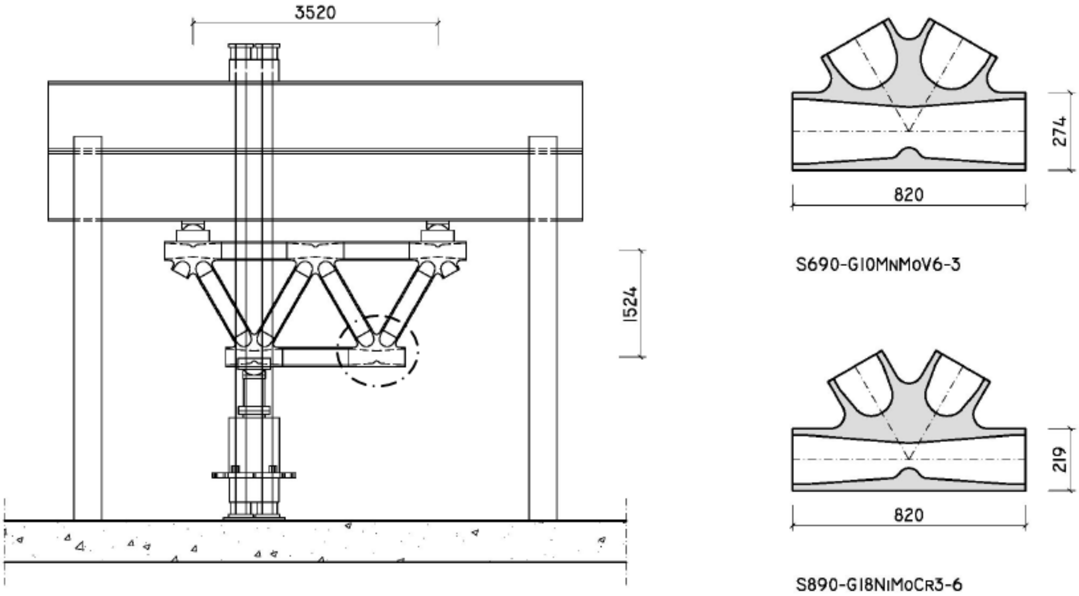


Figure 2.11 Conceptual drawing of large scale setup for CHS trusses with VHSS K-joint cast steel members [38].

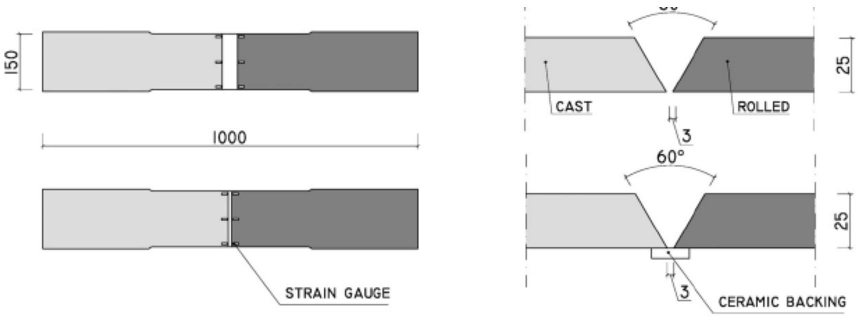


Figure 2.12 Geometry of small scale V-welded plates from rolled (S460, S690 and S890) and cast (G20Mn5, G10MNMoV6-3 and G18NiMoCr3-6) steel [38].

Finally, economic efficiency of cast steel joints depends largely on the manufacturing of the formwork. High cost effectiveness can be achieved if the bridge design allows a large number of equal joints.

2.2.4.2.3 Transverse Butt welds

Fatigue performance of welded structures can be improved significantly with optimized design of welded details and high quality welding techniques. In the codes, several welded details are classified in categories according to their fatigue strength (Δσ value, usually at 2 million cycles). From these, the transverse butt weld detail appears to have low stress concentration leading to a relatively high detail classification. Therefore, it should be preferred, when possible, over fillet welds with inherent poor fatigue performance.

Axial fatigue tests on 10 mm butt welds made with S355 up to S960 (Demofonti et al.,2005) showed that advantages for S960 were observed in cases of variable amplitude loading. Furthermore, machining of welds gives lower notch factor which is found to be favourable for HSS. However this might be time consuming in practice.

Other tests performed on various 6-8 mm plate butt weld connections made of S690, S960 and S1100 (ECSC, 2005), have result in a slope m=5 for the S-N curves, which is higher than the m=3 slope in the design codes for regular steel grades. The slope m = 5 results in improved fatigue behaviour in the high cycle fatigue region (important for bridges) but reduces the benefits in the low cycle fatigue region (see also Figure 2.13). For all steel grades however, characteristic values of fatigue strength for these connections are above the values of EN 1993-1-9 (2005).

In 2007, TU-Delft in cooperation with Netherlands Institute for Metal research, performed fatigue tests on base plate material [44] and transverse butt welded joints [44, 45] made of S690 and S1100. Fatigue results were clearly quite different for the base materials compared to the welded joints.

For both, the base materials and the transverse butt welds, fatigue strengths lied above the characteristic values presented in the EC, in the high cycle regime. This was mainly because of the higher slope values in S-N curves. Transverse butt welds made of S690 gave better fatigue strength than higher grades (S1100), while for the base plate materials the differences between the grades were not that obvious, Figure 2.13.

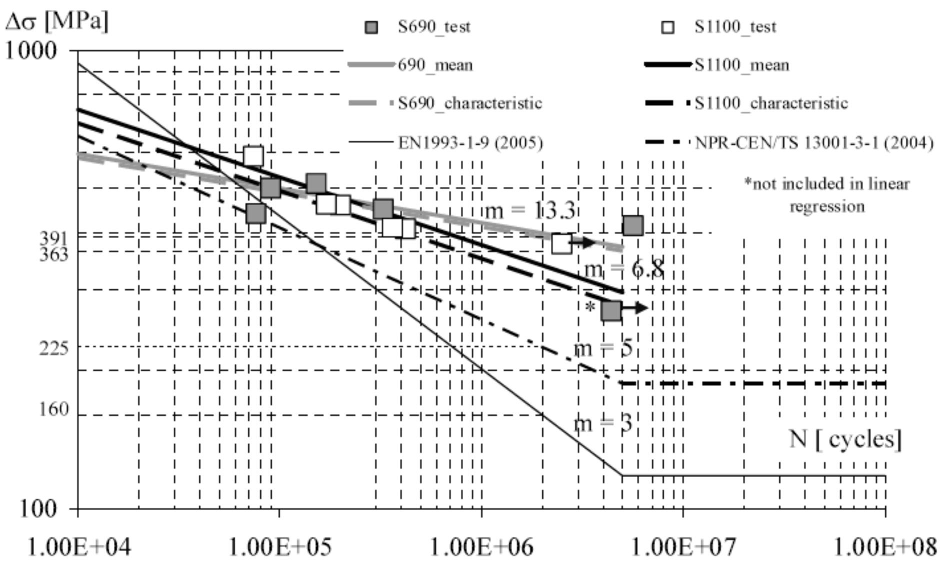


Figure 2.13 ***Base material fatigue strength [44]***

In Figure 2.13 both S690 and S1100 test results for base material are above class 160 of EN 1993-1-9 (2005) and class 255 of the NPR-CEN/TS 13001-3-1 (2004) in the high cycle fatigue region. Slope m=5 presented in the crane code seems to be more accurate than m=3 presented in the Eurocode. The calculated strength for S690 is 391 MPa (Δσ value at

2 million cycles), much higher than the 339 MPa value for S1100. Thus it is obvious that higher yield strength leads to higher fatigue strength also for base material.

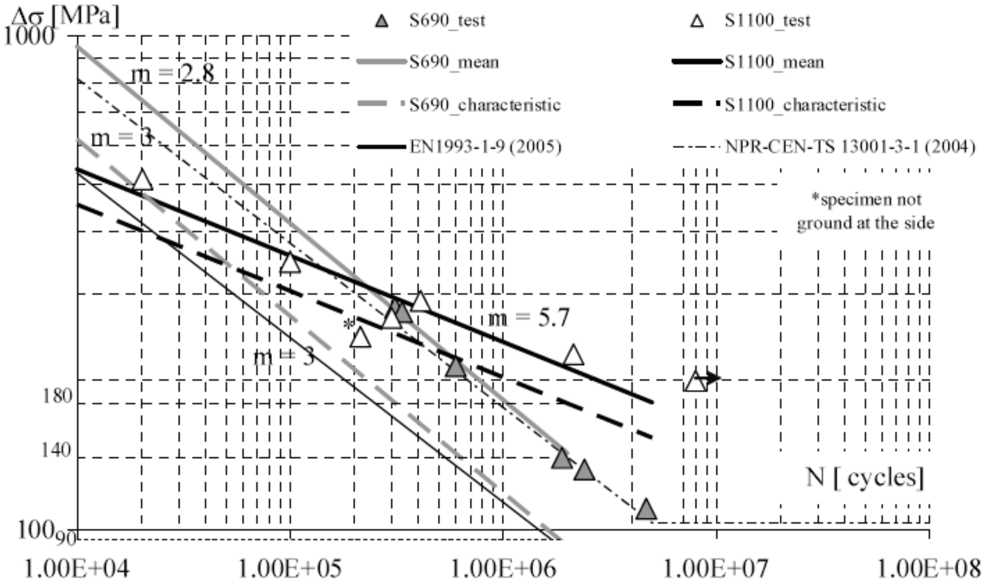


Figure 2.14 ***Transverse butt welds fatigue results [44]***

The behaviour of transverse butt welds however is different as shown in Figure 2.14. The calculated strength for S690 was 92 MPa, which is in good correspondence with a high quality butt weld of regular steel (class 90 of EN 1993-1-9 (2005)), but lower than class 140 of NPR CEN/TS 13001-3-1 (2004). Plus, the slope of S-N curves (m=2.8) is lower than the slope value m=3 according to both codes.

On the other hand S1100 butt welds show a complete different behaviour than S690. The slope is higher m=5.7 and the strength is 180 MPa which is a higher value compared to both codes. The results are summarized in Table 2.3.

	Base material tests		**Transverse butt weld tests**	
	S690	**S1100**	**S690**	**S1100**
$\Delta\sigma_{D,mean}$	399	363	143	212
$\Delta\sigma_c$	391	339	92.5	180
m	13.3	6.8	2.8	5.7
#specimens	5	6	6	6

Table 2.3 ***Fatigue strength comparison (N= 2*10^6 cycles) [44]***

Additionally, as a part of the same PhD programme at TU-Delft, for the applicability of VHSS in civil engineering structures, transverse butt welds (S690 and S1100) were tested with respect to fatigue [45]. Results from tensile tests on these specimens showed a brittle failure since the yield to tensile strength ratio is low compared to regular steels (especially for S1100).

However, research also showed that the higher tensile strength of S1100 transverse butt welds gives higher fatigue strength and that the fatigue strength of S690 butt welded joints lied well above the design value in EN 1993-1-9 code.

2.2.5 Deformation capacity of welded details

The safety of structures made of high strength steel depends on the deformation capacity of the (welded) joints [46]. Generally, welds should be stronger than base steel material (over matched welds) to avoid brittle failure in the weld itself. In this way a more ductile connection is created, while failure is expected to occur on the base metal first.

However this is only possible in case of steel grades up to S690. For higher grades (e.g. S1100) the weld metal will be weaker than the base metal (under matched weld) leading to yielding in the weld metal first.

Static tests [47], [48] performed from TU-Delft University of Technology for three types (A, B and C) of welded specimens, made of S690 and S1100 in order to investigate the deformation capacity of these grades (Table 2.4). According to this study, the deformation capacity of S690 specimens proved to be very good.

Where in Table 2.4, specimen type:
A: Cross plate connection with fillet welds loaded in shear
B: X-joint with load carrying full penetration welds with low SCF
C: X-Joint with load carrying full penetration welds with high SCF

S690 12 mm	Overmatched			Undermatched		
Specimen Type	Test number	Fmax kN	Failure mode	Test number	Fmax kN	Failure mode
A	7A1	454	Weld	8A1	445	Plate
B	1B1	545	Plate	3B1	580	Plate
C	1C1	560	Plate	3C1	560	Weld

Table 2.4 ***Comparison of test results for OV- versus UM- welding [47]***

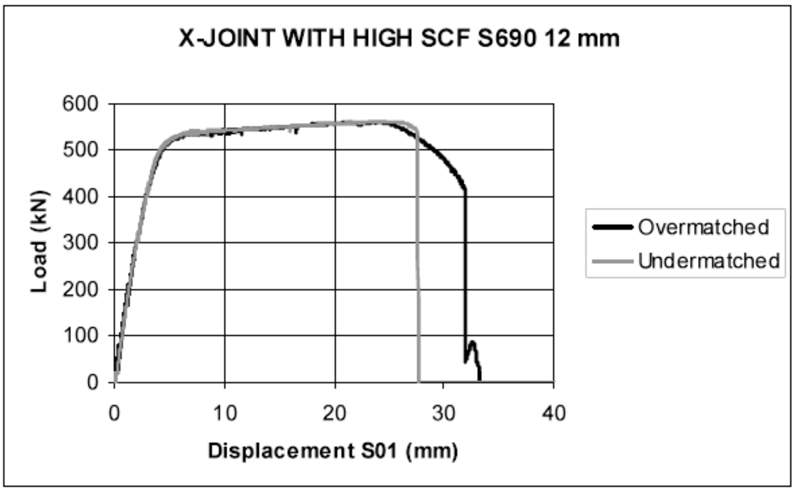


Figure 2.15 Load-Deformation relation 12mm S690 X-Joint (type C), [48]

The main conclusions from this study were that, over matched welded joints in S690, have a good strength and deformation capacity, while under matched S690 welds can be compensated by weld reinforcement (under matched weld metal is compensated by the larger weld). In case of S1100 under matched welds are the only option, therefore, the possibility of weld reinforcement should be considered there also.

Furthermore, numerical modelling of under matched welded high strength steel connections [46] concluded that, in general, a full plastic design will lead to safe and economical HSS structures where over matched welds can be made (with steel grades up to S690) .

Over matched welds will create sufficient deformation capacity to the joint and the adjacent parent metal. For specific joints weld reinforcement can compensate the weak weld metal and make the connection as a whole over matched.

In cases with under matched non-reinforced welds, sufficient deformation capacity cannot be guaranteed and the deformation capacity of the joint should be confirmed and determined by tests and FEM analysis.

At the end, the deformation capacity needed for the structure will determine whether or not this weld is suitable for the specific application. However, to achieve a good deformation capacity the ratio weld area at the vertical plate/weld thickness should be at least inversely proportional to the relative weld strength: $A_{weld} \geq \{R_{m,\ plate} / R_{m,\ weld}\} * A_{plate}$

Finally, it seems that ratcheting loads give nearly the same results as static tests with respect to strength and deformation capacity of the welded joints. The results are comparable with the results of the static tests with respect to the maximum load and the deformation capacity at failure [49], [50].

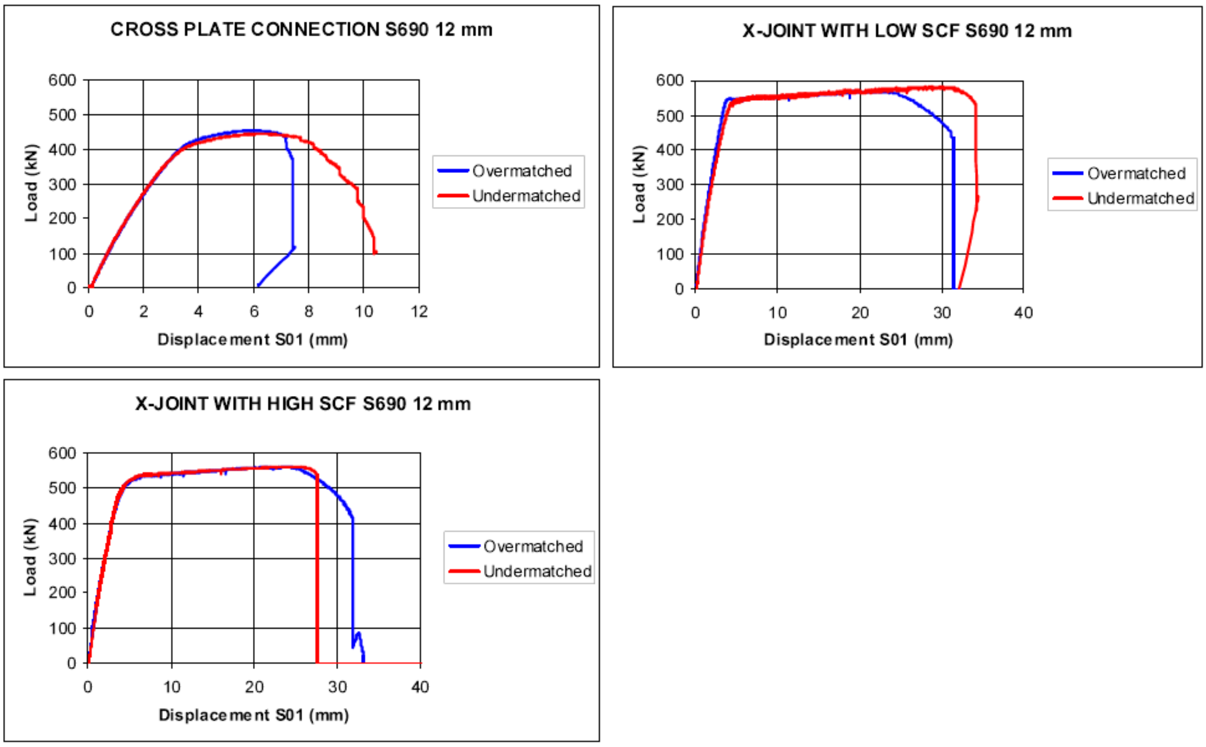


Figure 2.16 Load displacement curves for OM and UM test specimens showing the differences in deformation capacities [47]

2.3 Economic and other benefits of using HSS for bridge design

The advantages of using high strength steels are generally a result of reduced weight and dimensions. Design stresses can be increased and plate thickness may be reduced, resulting in significant weight savings. Reduced plate thickness can also save on welding costs as it leads to smaller weld volumes. Simplified structural components and construction techniques are often possible, particularly for large structures, and foundation costs may also be reduced due to lower dead weight.

The most economical and efficient use of high strength steel for bridge design, is Q&T steels in members stressed in tension where the high strength can be fully exploited, and in projects where the dead weight of the steel is the predominant load, such as in long span bridges. In compression they are most effective in heavily loaded, stocky columns or in stiffened compression elements where buckling is not the controlling criterion.

Usually, the cost of the material increases as the strength increases (Figure 2.17, left). However, when the higher strength can be fully utilized then the relative material cost is reduced (Figure 2.17 right) [36].

In that respect, a very cost-effective solution is "hybrid girders". These are girders with flanges of HSS and web of lower steel grade, which are proved to be more economical than homogeneous girders. It is suggested that the strength of the flanges should not exceed twice that of the web for serviceability reasons.
For economy in fabrication, rules require matching electrodes which this requirement can be met with HSS up to S690 [51]. These grades are commonly used in US and in Japan for many years, while in Europe their use is still limited.

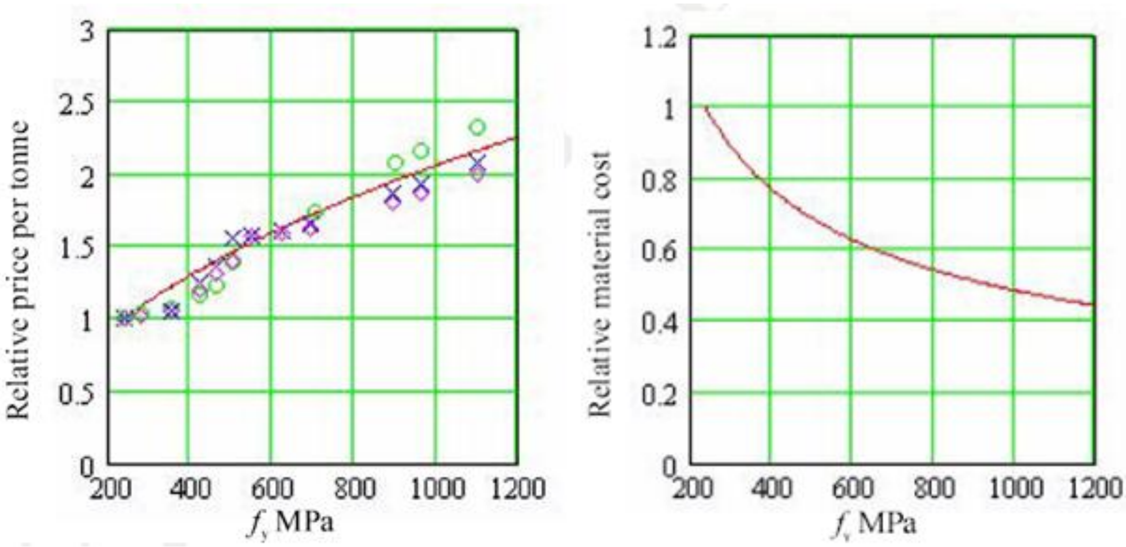


Figure 2.17 Left: Approximate price per tonne of hot rolled steel, normalized with price of S235 as function of yield strength. Right: Approximate material costs normalized with the cost of 235 assuming that the strength can be fully utilized [36].

In U.S. many research activities are focused on exploring the benefits of HSS bridges over concrete bridges and making steel bridge construction more cost-effective. Most of them show that bridges built with high strength steel plates achieve at least 10% overall cost reduction. In addition, weathering steel grades can save up to 18 % of costs [52].

The competitive advantages of micro-alloyed steel compared to hot rolled carbon steel include superior fabricability and weight reduction by at least 25 %.The lower weight more than offsets the slightly higher unit cost of micro-alloyed steels, adding economic value to both steel producers and steel users. The following simplified calculation is used as an example to illustrate the economic benefits gained (for both steelmakers and users) by substituting mild steel with HSS (Figure 2.18) [53].

HIGHER PROFITS FOR STEELMAKERS
(Dollars per Ton of Steel)

	Carbon Steel	Microalloyed Steel
Selling Price	350	415
Microalloying Cost	-0	-15
Other Production Costs	-300	-300
Steelmaker's Profit	50	100

Because of 25% Weight Reduction,
0.75 Ton of Microalloyed Steel
Replaces
1 Ton of Carbon Steel

LOWER COSTS FOR USERS
(Dollars per Ton of Steel)

Carbon Steel (1 Ton x $350)	350
Microalloyed Steel (0.75 Ton x $415)	311
Cost Savings	**39**

Figure 2.18 Example of economic benefits for both the steelmakers and the users by using micro-alloyed steel [53]

It is evident that the substitution is economically attractive for both the producer and user. Additional benefits include ease of fabrication, improved overall properties (e.g., toughness, ductility), and lower transportation costs [53]. The range of cost savings is shown in Figure 2.19.

The weight reduction achievable depends not only on the difference in strength but also on the mode of loading. For straight loading in tension, the weight reduction is proportional to the difference in strength. An increase in yield strength by a factor of two may reduce the weight of steel by two (a situation found in concrete reinforcing bars). For other types of loading (e.g. bending), a two fold increase in strength may contribute to a weight reduction of 34 % or more.

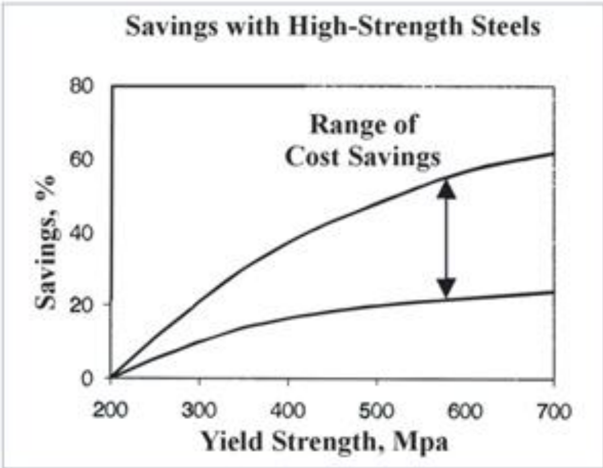


Figure 2.19 Cost savings by the use of HSS [SSAB Sheet Steel Handbook]

However, in case that buckling governs the saving in material costs will be slightly smaller and will depend on the type of buckling and the slenderness of the component.

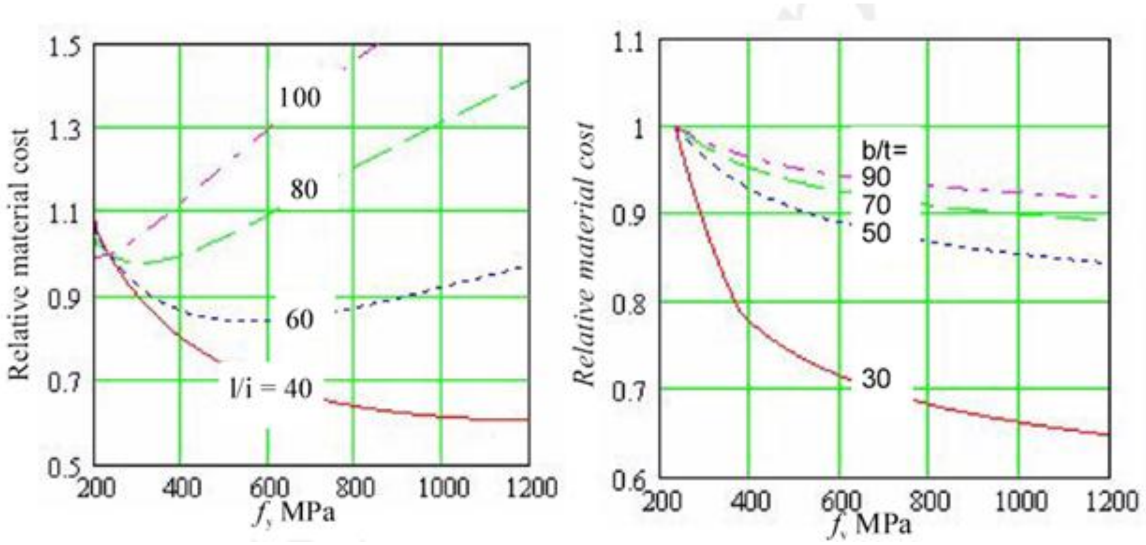


Figure 2.20 Left: Relative material cost for column subject to flexural buckling as a function to yield strength. Reference costs are for S235. Right: relative costs for a plate supported along all four edges, subject to uniform compression as a function of yield strength. Reference costs are for S235 [36].

Figure 2.20 (left) shows that you save money with increasing strength in case of stocky columns (I/i= 40). For I/i= 60 we gain minimum costs for yield strength 600 MPa.

In Figure 2.20 (right) the plate slenderness is represented by the width over thickness ratio (b/t). It is obvious that increasing the yield strength always reduces the material costs but for slender plates the savings are quite small. Therefore the plate slenderness has to be kept low in order to obtain substantial cost savings.

3 Examples of existing bridge applications, case studies and cost based research

3.1 General

More a more experience is gained by using HSS in modern design of large heavy structures. Many projects around the world have already been realized, in the last 3 decades using higher steel grades, as for example S690.

Taking into account that the price of high strength steels remains still at higher levels than conventional steel grades (e.g. S355), and that partly adequate design rules are available, it seems that the most cost-effective way of using them in bridge design is in hybrid girder construction.

Conclusions from earlier research and some examples of existing HSS bridge designs are briefly discussed here; to highlight the benefits they gained by using the new material.

First of all an overview of different bridge types, based on literature study, and their currently most economical spans is shown in tables 2.5 and 2.6.

Bridge type	Span range
Multi-beam/Composite deck	15 m to 100 m
Box girder	45 m to 180 m
Truss	40 m to 500+ m
Arch	30 m to 500 m
Cable-stayed	200 m to 850+ m
Suspension	850+ m

Table 3.1 Types of bridges and economical span length range [54]

Bridge type	Span range
Girder bridges	
Small	10-20 m
Medium	20-100 m
Large	> 100m
Arches	up to 300 m
Suspension	500-1500 m (not economical below 500 m)
Cable stayed	100-500 m

Table 3.2 Typical span range depending on the bridge type [55]

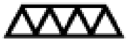
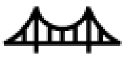
Bridge Type	Schematic	Span Range	Use of Plates
Multi-Beam		25 - 100m	Webs and flanges of main girders and diaphragm beams. Bearing and intermediate stiffeners.
Box-Girder		45 - 180m	Webs and flanges of box girders. Internal stiffeners and diaphragms.
Truss		40 - 500m	Fabricated truss elements, Gusset plates etc.
Arch		Up to 500m	Fabricated arch elements.
Cable-Stay		Up to 850m	Webs, flanges, stiffeners and diaphragms for orthotropic decks. Elements for truss decks.
Suspension		Up to 2000m	Webs, flanges, stiffeners and diaphragms for orthotropic decks. Elements for truss members.

Table 3.3 Availability of steel plates for different bridge types, [56]

3.2 Examples in Europe

3.2.1 The Prince Clause Bridge, the Netherlands

This cable-stayed bridge (fig.19) which opened in June 2003 spans the Amsterdam Rhine canal between the Kanaleneiland and the new Papendorp business quarter. It has a total length of 300m (150 m cable stayed span) and 35m width and carries regional tramway rail routes, road traffic, cyclists and pedestrians. High strength structural steel S460, in thicknesses 20-100mm, was used for the heavily stressed pylon elements [57].

Figure 3.1 The Prince Clause bridge, Utrecht.

3.2.2 Bridge HST over the Hollandsch Diep

[57] This is a new high speed railway bridge, located between Dordrecht and Breda, over Hollansch Diep (1200 m wide). The steel structure consists of hammer-head-shaped elements above the piers with connected box girder sections. Plates of S460N and S355J2G3, 100- 210 mm thick were supplied for this structure. Exceptionally broad plates of 4,550 mm where prefabricated and transferred on site by ship and lifted into position with a 500 t floating crane, permitting minimization of costly and complex on site welding.

Figure 3.2 Bridge HST, Hollandsch Diep 1998-2006

3.2.3 The Ennëus Heerma bridge

[57] It is an unusual shaped bow string bridge of 230 m length, 38 m width and 26 m height (four motor traffic carriageways, two tram lines, two cycle tracks and a pedestrian walkway. It was completed in 2001. TM-rolled fine grain steel of minimum yield strength 460 MPa in thickness up to 100 mm were used for the cross members of the composite carriageway deck and the central arches in order to safely absorb the high stresses occurring at these zones of the bridge.

Figure 3.3 The Ennëus Heerma bridge, The Netherlands [Wikipedia].

3.2.4 Fast Bridge 48 Military Bridge, Sweden [Höglund]

The bridge was designed and constructed only for military purposes. The main features include:
Geometry: span 32-48 m with 4-6.8 m long (truss girder) sections, 4 m wide and 1.6 m deep.
System: 2 truss girder single-span bridge with VHSS deck (S1100, 5mm thick), stiffened by cold formed steel channel sections with web folds. The bottom chord is made of cold formed sections whereas the diagonals in the truss are made of S460 rectangular hollow sections. The coupling plates are made of 50 mm S960 plates.

The fast bridge 48 required steel with 1100 MPa yield strength and impact toughness 40 J in 40 ºC. Due to the fact that these grades were not covered by the national standards extensive testing on buckling and fatigue behaviour was carried out to verify the structural integrity of all components. However there are no limitations on the deflections for Military bridges.

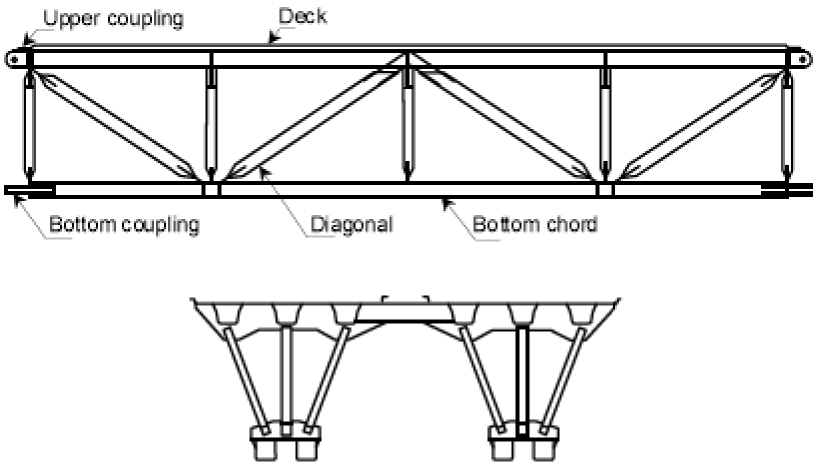


Figure 3.4 Cross section of Fast bridge 48.

As there are no limitations on deflections for military bridges, the strength of very high strength steel could fully utilize. The result is a light-weight bridge competitive to aluminium alloys and polymers. Furthermore, reduced fabrication costs and extended service life added more positive value to the specific design. With further development this design can reach up to 200 m span length, with intermediate supports dropped from the bridge during launching.

Figure 3.5 The Fast Bridge48 loaded by a tank.

3.2.5 Composite bridge near Ingolstadt, Germany [Müller]

This multi-span composite highway bridge having span lengths 24 m, 5x30 m, and 20 m is carrying a 15 m wide concrete slab. This is an integral design (meaning no bearings where used), where the steel girders are directly connected to the columns (composite piers) by flexible steel plates.

S690 steel grade was used for the semi-rigid connection between the piers and the girders. To ensure a semi-rigid connection the steel plates where designed as to satisfy certain requirements:

The plate thickness must be small enough to reduce restraints from translational and rotational movements of the structure at the columns. The plate thickness must also be thick enough to resist the normal forces and the restraining moments from movements safely. These contradictory requirements where solved by using S690 steel plates.

3.2.6 Verrand viaduct, Italy [Miazzon]

Verrand viaduct is part of the Mont Blanc Aosta highway and passes over the valley with the country road and the Dolra Baltea River. The purpose was to create a unique motorway viaduct for all the road ways with width of 20m. It was decided to design an orthotropic steel deck with two main beams and interior bracing, of five spans (97.5 m-135 m-135 m-135 m-97.5 m) with four intermediate piers.

The lattice launch girder, 84m long, was realized by using HSS tubular sections S690.Therefore the weight of the girder was significantly reduced, which allowed launching erection method without changing the cross section dimensions of the steel deck.

3.2.7 Sweden, Hybrid Girder Bridge

In Sweden is a common practice to mix different steel grades for a single cross section, with the stronger grade at the flanges and the weaker in the webs. These types of girders (hybrid girders), are standardized in Swedish codes. It is required that the flange strength may not exceed 50 % of the web strength. Furthermore, when the design is reduced with respect to buckling of normal stresses, the effective web thickness is based on the strength of the flange and not the web itself.

In 1995, a single span bridge was erected in Mttådalen with a span of 20 m and free width 7 m. For the lower flanges S690 steel was used, while for the upper flange and the web S460 steel grade was used. Using S690 also for the upper flange proved not to be cost effective, since it is often submitted to buckling and/or lateral torsional buckling during erection and casting.

Three alternative designs were made with respect to different material combinations: S355 all over the cross section; S460 only at the bottom flange and to the right; S460 in the web and top flange and S690 for the bottom flange. A comparison based n costs of the three designs is presented in the next graph. Clearly the hybrid design is proved to be the most cost effective.

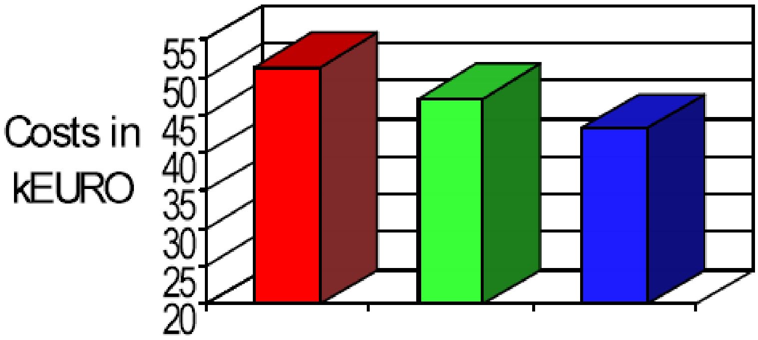


Figure 3.6 S355 all over the cross section; S460 only at the bottom flange and to the right; S460 in he web and top flange and S690 for the bottom flange.

3.2.8 Nesenbachtalbrücke, Germany

This is a 572 m long composite bridge with span lengths of 35.10 m to 89.50 m. The architectural requirement for slender structures was feasible with hollow section lattice structure with small construction depth and members diameter. The columns are integrated in the shape of trees with branches and trunks with high slenderness ratios. The slender dimensions could be achieved by S690QL1 steel.

Figure 3.7 *The* Nesenbachtalbrücke, Stuttgart

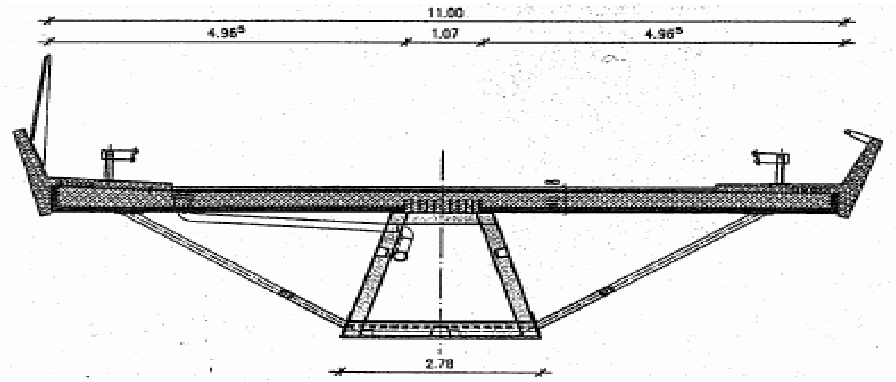


Figure 3.8 The cross section of Nesenbachtalbrücke.

3.2.9 Footbridge over Bayerstraße in Munich, Germany

It is a pedestrian bridge spanning the "Bayerstraße" street, completed in 2005. It has a length of 38.70m and a 4m width. The supporting framework was made out of h two arched girders and one stiffening girder with a surface plate of concrete. High strength steel S690 G5 QL was used for the main girders (tubular steel sections 127x30mm).

Figure 3.9 Pedestrian bridge “Bayerstraße” in Munich, Germany

3.2.10 COMBRI project

COMBRI (Competitive steel and composite bridges by innovative steel plated structures) [58] is a European research project, the goal of which is to make advances in design for the bridges of the future. So far, results of this program show the following:

The use of hybrid girders: the redesign of a S355 steel box girder, proposing a S460 and S690 steel hybrid girder, gives a reduction in cost of material of 10 % in spans and 25 % at the piers. The recommendation of double composite action: using composite action in the bottom flange at the piers, where compression is acting, is a competitive solution despite the greater complexity of design required.

The rationalization of the use of transverse stiffeners on the webs: the cost in terms of labor requires its minimization and, as an alternative, the use of longitudinal stiffeners and simpler constructive details. It is shown that longitudinal stiffeners are not the most competitive solution for web depths of less than 4 m. The design of simpler diaphragms: while not consuming a lot of steel, from an economic viewpoint, it is important to reduce hours of fabrication through eliminating components, possibly transverse stiffeners also.

The design for using the launching technique for bridges: the resistance to a patch load has been studied, improving on the Eurocode EN 1993-1-5 design methodologies and enabling longer loaded lengths and, thereby making the launching of bridges with prefabricated concrete slabs possible.

The methodologies of Eurocode EN 1993-1-5 for the flanges, principally the bottom flange of the box girder cross-section, can lead to unsafe results for low rigidity solutions and, so, minimum rigidities are recommended for stiffeners and the use of large trapezoidal stiffeners provide two stiffened lines for the same effort of welding required for a single open section stiffener and its torsional rigidity provides an increase in critical stress. The results of the COMBRI research are presented in the final report in such a manner that they also serve for enhancing the EN 1993-1-5.

3.3 Examples in Japan

3.3.1 Tokyo Gate Bridge (Japan)

The bridge opened in July 2011. The use of high yield strength steel “SBHS” (Steel for Bridge High Performance Structures), reduced approx. 3 % of the total weight and approx. 12 % of the total construction cost (estimated by MLIT).

Figure 3.10 Tokyo Gate Bridge

3.3.2 Nagata Bridge, Japan

It is suspended over the Tama River, connecting Fussa and Akiruno cities in Tokyo (ordered by the Tokyo Metropolitan Government Bureau of Construction).It is a four span continuous bridge, 250m in length The total weight of the steel used is approximately 600 tons.

The Nagata Bridge is the bridge for which SBHS was first adopted after it was established as JIS steel. SBHS was used because is highly strong and tough, and has excellent weldability and cold formability when compared to conventional rolled steel for welded structures.

The yield strength, which refers to the design strength in the structural design, is 10-20% higher than conventional steel. This allows for economical design that includes weight savings.

Figure 3.11 Nagata bridge, Japan

3.3.3 The Akashi Kaikyo Bridge

The Akashi Kaikyo Bridge forms part of the arterial highway that connects Honshu and Shikoku. This three-span, two-hinge stiffened truss suspension bridge is 3,911 m long with the world's longest center span of 1,991 m. Nippon Steel delivered approx. 4,200 tons of HT690 and HT780 steel for the stiffening girders; this steel can decrease preheating temperatures.

Figure 3.12 The Akashi Kaikyo Bridge, Japan.

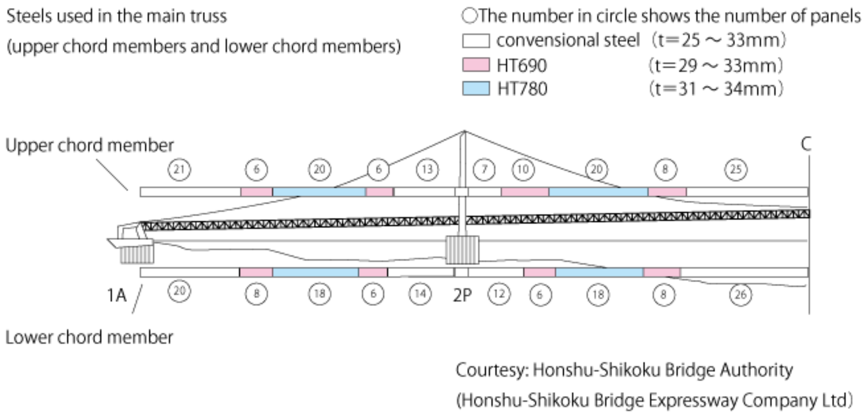


Figure 3.13 Application of HSS in the Akashi Kaikyo Bridge, Japan.

3.4 Bridge examples and case studies in U.S.

Especially in the USA extensive research programs for HPS (high performance steels), in bridges, have been performed. These help gaining experience and learn more about the use of these steels in bridges. Some of them are presented here.

In 1992, AISI partnered with the Carderock Division, Naval Surface Walfare centre and the Federal Highway administration (FHWA), to develop new and improved steel alternatives for bridges. The team brought together a cadre of professionals in steel production, bridge design, bridge fabrication and welding, as well as specialists from the U.S. government and academia.

The result was a new type of steel, known as high-performance steel (HPS), which provided up to 18 % cost savings and up to 28 % weight savings when compared with traditional steel bridge design materials. HPS 100W, HPS 70W (485 MPa yield strength) and HPS 50W (345 MPa yield Strength) produce bridges that are more cost-effective, higher in strength, lighter in weight, and have greater atmospheric resistance than conventional steels. They also have improved fatigue and corrosion-resistance properties.

In the International Conference on Transportation Engineering, 2009 (ASCE) was presented a study on costs of long span bridges with application of HSS. In this study, trial designs of cable- stayed bridges with main span length of 1400 m, steel weight and construction cost of the bridges made of different steels named as Q345, Q420, HPS70W and HPS100W of which their yield points are 345, 420, 485 and 690 MPa, respectively, were investigated. The results indicate that the steel weight and construction cost can be reduced obviously with the increase of steel strength, although the design stresses of some plates of the box girder are not governed by material strength due to the influence of the stability of stiffened plates and the limitation of minimum plate thickness.

[6] According to U.S. Department of transportation, Federal Highway Administration (FHWA), there is the need for a fundamental change in the methods and materials used in bridge construction. A large number of the of the 595,000 highway bridges in U.S. are substandard or deficient in one or more ways, even after 25 years of federal, state and local improvement programs. Fatigue and deterioration govern the service life of these structures.

Therefore, FHWA suggests, that it would be a good solution to solve these problems, by fully implement the use of High Performance Steel for all structural steel bridge elements where the high strength of HPS leads to fewer lines of girders, shallower beams and longer spans. These HPS bridges also have improved fatigue and corrosion resistance.

Benefits from the use of HPS in the USA (according to FHWA):

- Longer span lengths and fewer piers
- Lower foundation and superstructure costs
- Wider beam spacing and fewer beams
- Increased vertical clearance without expensive approach roadway work
- Fewer maintenance requirements and longer service life
- Lower initial and life-cycle costs

A parametric study conducted of FHWA of two span continuous plate girder bridges with span lengths of 30 m and 75 m indicated that the optimum girders used ASTM A709 grade 345W (345 MPa nominal yield strength) steel for all webs and positive moment top flanges and HPS 485 W for the negative moment top flanges and all bottom flanges (AASHTO 2003).

Girder designs consisting entirely of HPS 485W were 13-20 % lighter than those fabricated entirely from grade 345W, but were about 3 % more expensive due to the higher cost of material. However as HPS becomes more common in the market its costs is likely to be reduced.

As a part of a master's thesis project at the University of Missouri-Columbia in the year 2000 [13], six alternative bridge designs (alternating the number of girders and the material used) where developed for a particular site.

The purpose of the paper was to demonstrate the benefits of HPS 485W (yield strength of 485 MPa) girder bridges compared to conventional 345W (345 MPa yield strength) steel girder bridges. Three homogeneous HPS 485W designs where compared to two 345W designs and also a hybrid design 345W/485W was also explored. For design optimization, based on current trends, 345W steel was used for all stiffeners, diaphragms and slice connection plates. The designs followed the AASHTO Standard specifications (16th edition) Load Factor method.

The objective was to compare construction, fabrication, erection and shipping costs and find the most cost effective design. However, the bridge owners were mostly interested in comparing the total costs.

The study concluded that at the time, and for current prices of the material, the design with the 7 hybrid girders was the most cost effective (11 % cost savings at the time of the study) and that lower material costs for HPS will have significant advantages for the use of these steels in the future (14.6 % was the estimation accounting for projected costs). In the table below the six design alternatives are presented.

Table 1: Bridge Design Alternatives Summary

Design Alternative	Girder Lines	Total Diaphragms	Additional Stiffeners	Steel Weight tonnes (tons)		
				45W(50W)	HPS	Total
9 Girder 345W(50W)	9	120	38	326.6 (360.1)	0 (0)	326.6 (360.1)
7 Girder 345W(50W)	7	90	46	310.5 (342.3)	0 (0)	310.5 (342.3)
9 Girder HPS	9	120	4	13.2 (14.6)	264.6 (291.7)	277.8 (306.3)
8 Girder HPS	8	105	2	13.0 (14.3)	259.7 (286.3)	272.7 (300.6)
7 Girder HPS	7	90	2	12.9 (14.2)	257.7 (284.1)	270.6 (298.3)
7 Girder Hybrid	7	90	28	182.7 (201.4)	94.0 (103.6)	276.7 (305.0)

Table 3.4 Bridge design alternatives for cost comparisons, University of Missouri-Columbia 2000 [13]

Another paper, [59] collects experience from the use of HPS in the state of Nebraska and describes its application in four different projects. The study indicated that the best use of High Performance Steels is in hybrid construction, as it is more economical and efficient to use the high strength steel in regions of high tensile stresses (bottom flanges in spans and top flanges at supports and in regions with high tensile stresses). Two of the projects this paper is dealing with, are mentioned briefly here.

3.4.1 Dodge Street Bridge

This is the second high strength steel bridge constructed in Nebraska and it was the first application of HPS 485W (485 MPa yield strength) in hybrid form. The hybrid arrangement used for this bridge, later proved to be the most economical form of using HPS 485W in bridge construction.

3.4.2 Springview South Bridge

Two alternative bridge designs where proposed for this project. One made with high performance concrete and one in steel. In the steel alternative HPS 485 integrated with S345 conventional steel. The steel alternative proved 10 % more economical so awarded the project. The girder cross section in all positive bending moments' regions is entirely composed of S345 steel. In negative moment regions however, a hybrid girder section, consisting of HPS in the flanges and traditional S345 in the webs was realized.

Although HPS was more expensive than conventional S345 at the time, the benefits that were attained by including it in the hybrid girder design proved economical. It was anticipated to be more cost effective to deliver and construct, because each girder could be delivered in 5 segments (30.48 m length weighing less than 89 KN [10 ton] each). In this way, less material, lighter girder sections and increased strength was realized. Therefore, the hybrid steel girder alternative cannot only compete with its traditional S345 steel equivalent, but can also compete successfully with high performance concrete designs.

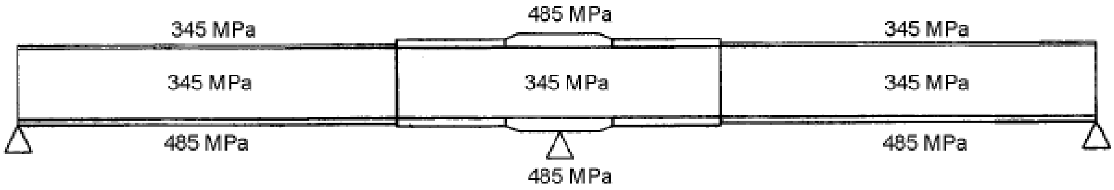


Figure 3.14 Economical use of HPS in bridges, Nebraska [59].

4 Conclusions from literature review

- High strength steels may offer many benefits when used for bridge design. The main advantages are generally a result of reduced weight and cross sectional dimensions. Design stresses can be increased and plate thicknesses may be reduced resulting in significant weight savings. Reduced plate thickness can also save on welding costs as well as on fabrication, erection and transportation costs. Foundation costs may also be reduced due to lower dead weight.

- Quenched and tempered (Q&T) high strength steels offer very high strengths for structural applications (up to 1100 MPa minimum yield strength), thus significant weight savings, accompanied with high toughness values (even at low temperatures), good weldability and sufficient ductility (although they have lower f_u/f_y ratios). These overall properties make this steels the most appropriate choice in case of bridges.

- Their chemical composition can be adjusted according to the specific structural application demands.

- Strength and hardness, generally, decrease with increasing tempering temperature and holding time. On the other hand tensile tests show that elongation and area reduction increase as the tempering temperature and holding time increase. For appropriate toughness values tempering is performed at least at 550 °C.

- Weldability is improved due to grain refinement and low CEV values (carbon content usually around 0.15 %). However, high strength steels seem to be more susceptible to hydrogen cracking. Generally, as the parent metal increases fewer variations in welding procedure are allowed.

- Preheating temperatures depend on the steel grade and chemical composition. It is generally not required for plate thicknesses below 30mm.

- Post welding heat treatment is generally not recommended for Q&T steels. In addition, they are not suitable for hot working above 550 °C since this may change their mechanical properties.

- Design codes available today in Europe, for these steels, are only available as additional rules for steel grades up to S700 (EN 1993-1-12:2007) although quality standards cover steel grades up to 960 MPa. This lack of detailed European design rules, limits the use of these higher grades in the construction industry, in Europe.

- Especially when talking about economical design of bridges using high strength steel, it is important to realize that the whole design consideration should change and development of detailed design codes are essential for gaining the maximum benefits the new material has to offer. This has been done in U.S and in Japan for several years now.
- The price of high strength steel still lies in high levels in comparison to S355, especially in Europe. This is because the market demands for these grades are still quite limited and only a few fabricators are able to work with the new material. However, this will change in the

future as the market demand is expected to increase, resulting in significant reduction of the high strength steel price.

- In spite of the higher material costs, high strength steel bridges can be more economical in terms of total costs when properly designed. Total costs include material, fabrication, erection, transportation and maintenance costs.

- To provide the best (high strength) steel quality and gain the maximum profit, it is important that fabricators will invest in higher quality equipment as a small increase in initial cost may give a large return on the investment. For small plate thicknesses, however, high strength steel can be machined as the regular steel grades with no extra investment.

- The most economical and efficient use of Q&T steels is in members stressed in tension where the high strength can be fully exploited, and in projects where the dead weight of steel is the predominant load (e.g. long span bridges). In compression they are most effective in heavily loaded, stocky columns or in stiffened compression elements where buckling is not the controlling criterion. In any case, must be ensured that fatigue or buckling is not the governing criterion, by proper bridge design.

- Hybrid steel girders are welded steel girders which combine different steel grades. Usually high strength steel is used for the flanges while normal steel grades are used for the web. This combination is proven to offer the highest level of economy in comparison to homogeneous steel girders.

- Fatigue of welded details usually governs the design of steel and composite (steel-concrete) bridges. Material strength in that case plays no role and thus properties of high strength steel cannot be fully utilized since the stress level is limited at critical locations. However, experimental research shows that these steels do not exhibit any disadvantage in fatigue resistance compared to mild steels [Puthli et al. (2006)]. In addition, alternative and more effective design solutions as cast joints, bolted connections where possible, transverse butt welds and plate thickness increase at critical locations, can greatly improve fatigue performance. Post weld treatment can also be incorporated, if necessary, to improve further the fatigue behavior of welded details.

- Buckling behavior of high strength steel can be considered better than mild steel in cases of heavily loaded stocky sections. The main reason for that is that higher steel grades (e.g. S690) are less sensitive to geometric imperfections in comparison to S355.

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Part 1B

Preliminary design

In Part 1B (chapters 5 and 6), a long single span (L=105 m) roadway bridge crossing Amsterdam-Rijnkanaal in the Netherlands, the ‘Schellingwouderbrug’, is chosen as a reference bridge to be re-designed using HSS. Three bridge type alternatives (based on the specific bridge geometry, location and site restrictions) are considered and preliminary designs are performed separately for each alternative. Design criteria are mainly strength, stability and fatigue. Also bridge weight and associated costs influence the design. Finally, the main results are summarized in chapter 6. More information on modeling and calculations are included in Appendix A.

5 Preliminary bridge designs using HSS

5.1 Setting the scene

5.1.1 Material choice

In Europe, quenched and tempered high strength steels are appeared to offer maximum weight savings for heavy constructions. In this thesis, S690 (in Q&T delivery condition) steel grade is chosen to be the upper limit of HSS grades to be used for the bridge design, in order to explore the maximum possible benefits of HSS.

Higher steel grades are not considered beneficial in case of bridges, since it is more likely that at very slender cross sections, stability and fatigue will be the governing factors. In addition, S700 is the upper limit for the yield strength which is covered by the European design codes [EN 1993-1-12:2007].

Hybrid bridge designs are performed using high strength steel S690 (and S460) in combination with mild steel grade S355.

5.1.2 Scope and planning

This thesis aims to examine the possibility of developing a structurally safe but also cost effective design using HSS, in order to enhance the use of higher steel grades for future applications in bridges in the Netherlands and generally in Europe. To accomplish that, the geometry and location of a long span roadway steel bridge (all in grade S355) in the Netherlands will be considered as reference.

The design phase is divided in two main parts:

Firstly, bridge type alternatives are considered, using high strength steel, for the reference bridge (i.e. based on geometry and location) and preliminary designs are performed for each bridge type. The bridge type (truss, arch, etc.) is chosen based on economic span lengths and L/D ratios, found in literature.

In the second part, one of the preliminary (global) designs that satisfy all design criteria (strength, fatigue, stability) is chosen based on minimum steel self-weight. Then, a detailed global design is performed (i.e. design of connections for strength and fatigue). The total costs for this design are then estimated on terms of material (dead weight and weld volume), fabrication, transportation, erection and maintenance costs.

Reference is made to the Appendixes at the end of this study (Appendix A and Appendix B) for detailed information on trial designs.

Due to time limitation it is not possible to perform a complete new global design in S355, also. Nevertheless, estimation on member dimensions, plate thicknesses, structural weight and related costs, for the final design choice, is attempted for comparison.

It is important to mention here that different design philosophy is needed to make effective and economic use of both materials (i.e. mild steel grades and HSS grades). Thus, results from this comparison should be treated with care.

Finally, conclusions for steel bridge design, using high strength steel are drawn. Also, recommendation for further research is given.

5.1.3 The reference bridge

5.1.3.1 'SCHELLINGWOUDERBRUG'

The so called 'Schellingwouderbrug' in Amsterdam (opened in 1957), is a steel arch bridge crossing the Amsterdam-Rijnkanaal in the Netherlands. The renovation of this steel arch bridge is part of the 'KARGO' project for maintenance and renovation of several steel bridges (fixed and movable) in the Netherlands. This project started in September 2011 and expected to finish in June 2012.

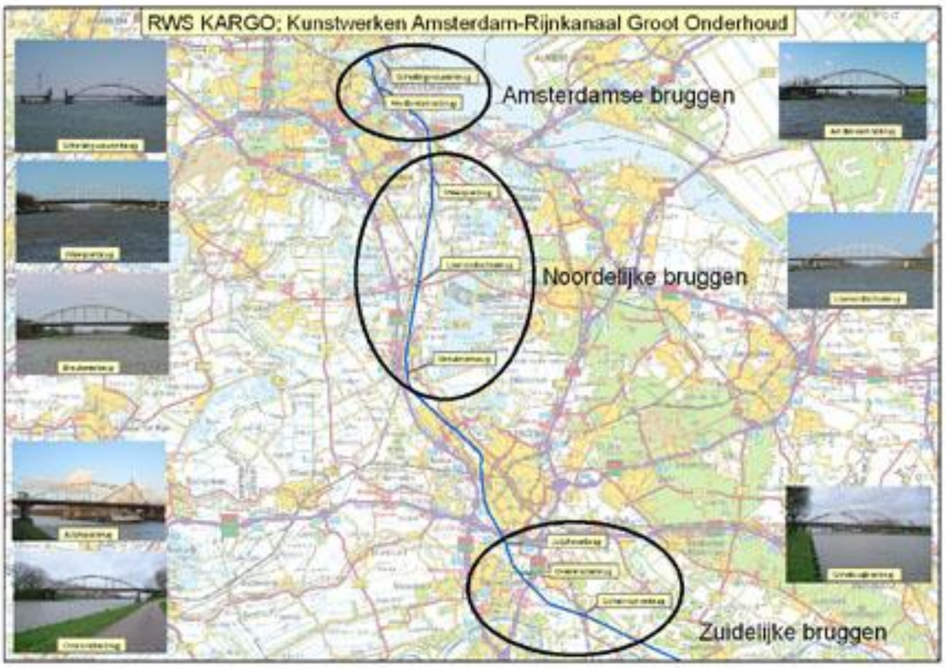


Figure 5.1 Bridges covered by the KARGO (Kunstwerken Amsterdam-Rijnkanaal Groot Onderhoud) project [Rijkswaterstaat].

The bridge is in the range of medium to large span bridges in the Netherlands and thus, it is expected that the benefits of using HSS will be more pronounced than for small spans, especially due to higher self-weight in relation to fatigue loading. In addition, an important role in the choice played also the interest of Iv- Infra on these span lengths for future design projects.

5.1.3.2 Bridge Geometry

The steel arch roadway bridge, 'Schellingwouderbrug' (Figure 5.2), has a single span of 105.30 m which is simply supported at its two end piers on four bearings. Its width, 16.30 m in total, consists of a double lane roadway (one per direction) and two cantilevers, 3.60 m each, for cycle/ foot paths. The construction depth is limited to only 1.65 m below the deck, due to clearance requirements to facilitate river traffic [1].

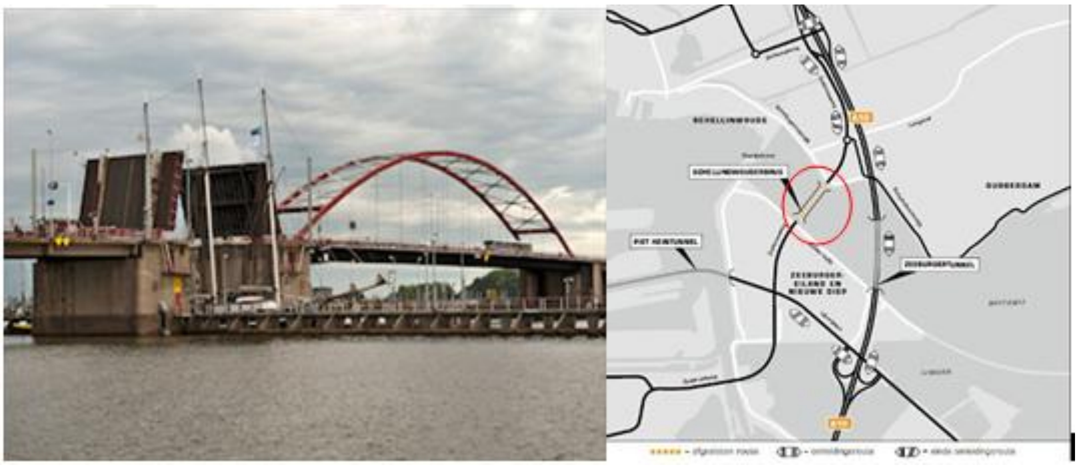


Figure 5.2 Left: The 'Schellingwouderbrug', Amsterdam [Rijkswaterstaat]. Right: the location of the bridge [Gemeente Amsterdam]

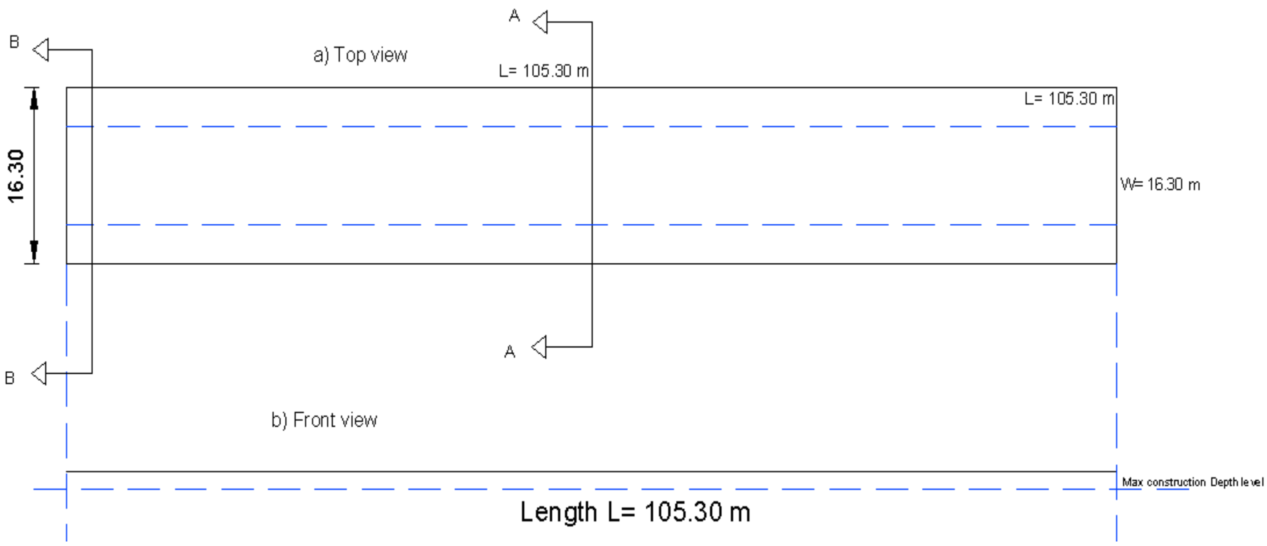


Figure 5.3 Bridge geometry: cross sectional view. Maximum construction depth is limited to 1.65 m below the deck level.

5.2 Basis of Design

5.2.1 Conceptual choice

5.2.1.1 MAIN LOAD CARRYING SUPERSTRUCTURE

Generally, when selecting the "correct" bridge type, it is important to find a structure that will perform its required function and present an acceptable appearance at minimum costs. In that respect different types have to be examined before the final decision.

The initial choice for the bridge type is based on cost effective span lengths and L/D ratios, derived from previous experience.

Thus, given the bridge geometry, general guidelines provided by several scientific documents (e.g. [2], [3], [4], and [5]), for a single span simply supported bridge with length over 100 m; suggest that economic solutions for the longitudinal structural system generally are:

- Trough truss (span to depth ratios: L/D= 10-15)
- Box girders (span to depth ratios: L/D= 20-30)
- Arches
- Cable stayed

Thus, for the span of 105.30 m and taking into account also the location and constructional restrictions for the reference bridge two types for the longitudinal system are the most suitable: a truss and an arch bridge (a cable stayed bridge is not considered due to difficulties of locating the pylons and anchor the cables).

The box girder bridge, although conflicting with the L/D ratios found in literature (should be around 20-30) is chosen to be examined, just for research purposes. The author thought it would be interesting to check, whether it is possible to achieve higher L/D ratios by using S690 for the bottom (tensile) flange of the box. This requires extensive research, however, and cannot be covered explicitly in this thesis study.

For the main structure, combination of different steel grades (hybrid construction), e.g. S355 and S690 (Q&T), will be considered, with respect to the highest level of economy. Therefore, S690 is applied on members and/or regions of high stresses, and lower grades elsewhere. Especially when strength is governing, using S690 for the whole section will reduce the dead weight and plate dimensions, thus overall costs, significantly.

5.2.1.2 DECK

For the deck structural system 2 possibilities can generally be considered.

- The deck system being part of the main superstructure.
- The deck system acting separately.

The first option is generally considered as more cost effective solution when concrete deck acts compositely with the main structure or when the orthotropic deck acts as the top flange of the main girders.

In such case, the deck system is designed continuous along and across the span. This continuity (plate action) ensures that it will participate to the overall structural action of the superstructure. Its principal function is to provide support to local vertical loads (from highway traffic and pedestrians) and transmit these loads to the primary superstructure of the bridge [4].

However, in the case an orthotropic steel deck is being used, fatigue may be critical as many fatigue sensitive details will be present. Furthermore, for new bridges, a design life of 100 years is required and therefore the design requirements with respect to fatigue (and maintenance) will more likely not benefit the use of high strength steel.

Taking into account these problems, it could be then wise to consider the deck as lying on top of the cross beams (maybe in composite action) and not on the main girders, and design it such that it could be possible to remove it and replace it any time (maintenance requirements), independently of the rest of the bridge (second option). In this case the design life for the deck can be 30 years or even less, and thus the requirements for fatigue far more relaxed.

Another advantage that this second option can offer is that makes it possible to use any material of preference (e.g. FRP, timber etc.) for the deck, and not limit possible choices in orthotropic steel deck and concrete.

The cost effectiveness of such an option can be an interesting subject for further investigation.

For the 'Schellingwouderbrug', the deck structure is not considered in detail for global designs. For sake of simplicity and to be benefited from the higher dead weight (i.e. higher static load will make higher steel grade more effective) concrete deck is generally adopted as the deck system in 'Schellingwouderbrug'.

Exception is made in case of the box girder bridge, where orthotropic steel deck is used, and also acts as the top flange of the box girder. This is done so, because, due to the extremely small depth of the box girder in comparison to the bridge length (i.e. very large L/D ratio) the deflections is expected to be quite high, thus a concrete deck would just made the situation even worse (higher dead weight than orthotropic deck).

5.3 Preliminary bridge designs

5.3.1 General

In this stage the bridge configuration and the materials to be used in the design are determined. For analytical description and calculations, reference is made to Appendix A.

For the 'Schellingwouderbrug' three bridge type alternatives are considered and preliminary trial designs are performed for each alternative, namely:

A) Box Girder Bridge

B) Truss bridge
C) Arch bridge

The design criteria are: strength, fatigue, and stability. A check for stiffness (in terms of maximum vertical deflection at midspan) is only performed in case of the box girder due to the very slender box structure and clearance requirements below the bridge.

Total costs (material, fabrication, erection, transportation costs) will also play major role for the final choice between different possible designs.

5.3.2 Global analysis

For the calculation of internal forces, elastic global analysis is used for bridges. This requires that the stresses (from loading) are limited to the yield strength of the material. Linear stress distribution over the cross section and linear stress-strain relation is assumed.

For calculating the resistance, plastic or elastic cross sectional properties may be used depending on the classification of steel members [EN 1993-1-1, EN 1993-2].

5.3.3 Loads

Only vertical loads due to dead load (steel self-weight, deck, asphalt layer, additional dead loads etc.) and traffic loads are considered for global verifications.

Loads and load combinations are determined based on European Standards.

Load combinations are taken from EN 1990:2002-Eurocode: Basis of structural design (Annex A2: Application for bridges).

Dead loads are calculated according to EN 1991-1-1:2002 (Eurocode 1: Actions on structures-Part 1.1: General actions, densities, self-weight, imposed loads on buildings) depending on the material.

Traffic loads on the bridge are calculated using the European standards for actions on roadway bridges EN 1991-2:2003 (Eurocode 1: Actions on structures- Part 2: Traffic loads on bridges. For strength verification group load model gr1a for traffic loads is applied, while for fatigue check, FLM3 (heavy truck) is applied in the preliminary phase to estimate the maximum fatigue stress range caused by traffic.

5.3.4 Material strength

The minimum yield (fy) and ultimate (fu) strength of three steel grades (S355, S460 and S690) are presented in Table 5.1 according to EN 1993-1-1:2003 and EN 1993-1-12: 2007.

	t ≤40 mm		40<t ≤80 mm		
Grade	fy (N/mm²)	fu (N/mm²)	fy (N/mm²)	fu (N/mm²)	ε=√(235/fy)
S355	355	470	355	470	0,81
S460	460	570	440	550	0,71
	t ≤50 mm		50<t≤100 mm		
S690	690	770	650	760	0,58

Table 5.1 Yield and ultimate strengths of steel grades under consideration, based on maximum plate thickness, EN 1993-1-1 (for S355 and S460) and 1993-1-12 (for S690).

5.3.5 Box girder bridge

The first design for the Schellingwouderbrug is a single box girder section with vertical webs (Figure 5.4).

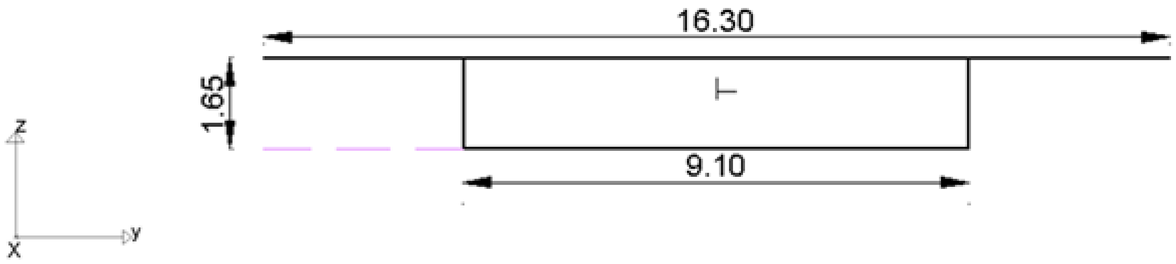


Figure 5.4 Cross section of the box girder bridge

The length of 105.30 m is in the economic span range (70-120 m) for simple supported span box girders with orthotropic steel deck [8]. The economic span to depth ratios for box girders, however, are L/D =20-30. For the "Schellingwouderbrug" the span to depth ratio is L/D = 64, exceeding by far the economic limits.

Despite the fact that it obviously seems like an inappropriate solution for this case, mainly due to depth limitation, it is still interesting to investigate, whether such a slender box section is sufficient to carry the heavy loads of a long span bridge using higher steel grades in regions of high stresses. Furthermore, this could possibly help setting the upper limit of structurally effective L/D ratios for box girder bridges made with HSS grades.

Elastic global analysis is performed that means that the stress level is limited up to the yield strength of the material. The design criteria are strength, stability, fatigue (and stiffness in terms of maximum vertical deflection in midspan, in this case).

It must be mentioned here that check for deflections (stiffness) is not generally necessary at the preliminary stage, since there is no upper limit for deflections defined in the Eurocodes. However, deflections with a value larger than let's say L/300 may cause dynamic problems especially in shallow and slender cross sections. In addition to that, clearance requirements should always be satisfied and therefore, it is strictly forbidden to enter the area available for traffic below the bridge (Figure 5.5).

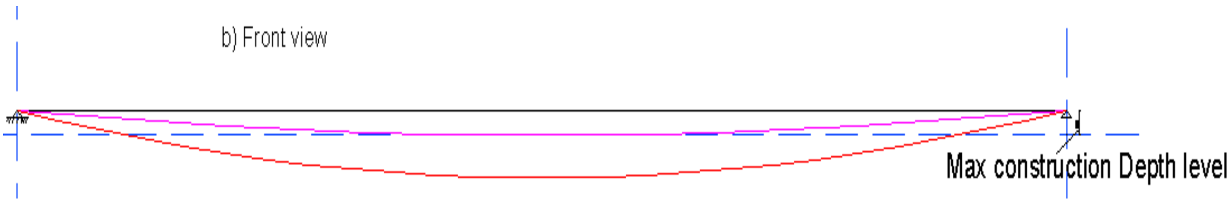


Figure 5.5 Clearance requirements determine the maximum allowed vertical deflection. Deflections entering that area (red line) are forbidden.

Therefore, the allowable deflections for the box girder are limited to a maximum value of L/300= 0.35 m.

High strength steel grade is considered only for the bottom flange which is under high tensile stresses. Relatively large plate thickness (40-50 mm) was necessary to take over the large stresses, though. This is not very beneficial especially in case of HSS, as preheating will be required and welding and machining such thick plates may need special care.

The top plate is an orthotropic steel deck with plate thickness up to 30 mm. As hand calculations are only used for this design, the longitudinal stiffeners are distributed over the width of 16.3 m and added as an equivalent thickness on the top flange, to simplify the calculations procedure.

5.3.5.1 Final conclusions

After several trial box designs (see Appendix A) it is concluded that it is not economical to use a box girder (alone) for the 'Schellingwouderbrug' due to depth limitations (extremely large L/D ratio).

This has been also verified by the literature findings, where it was found that economic L/D ratios for box girders are in the range of 20 - 30. In this case L/D gives an extremely big value (= 64!). Stiffness and local plate buckling govern the design and thus, thicker plates and/or more webs are required which lead to a very uneconomic solution, especially when high strength steel is used. However, in case of no height limitations an optimal L/D ratio for box girders in HSS could be further investigated.

If costs were not the basic criterion, it has been proven that it is possible to consider (at least at the preliminary phase) a box girder so slender using HSS for the bottom flange (region with very high tensile stresses) and with moderate thicknesses (max thickness= 50 mm for the bottom flange).

5.3.6 Truss girder bridge

5.3.6.1 GENERAL

In the case of a truss design there is no limitation for the construction depth since the truss can also be built above the deck and the rules about economic span to depth ratios (L/D) can be used as a starting point.

The length of 105.30 m is in the range of economic span lengths for trusses (60-120 m for roadway bridges [9]). The optimum value of span to depth ratio (L/D) depends on the magnitude of the live load to be carried. Cost effective L/D ratios are about 10-15 [2]. Especially when total costs are considered a ratio nearer 15 will represent optimum value [6].

However, these values are calculated assuming S355 steel grades, thus higher L/D ratios may be more economical for HSS. This may be an interesting subject to be further investigated. However this is outside of the aim of this thesis project.

5.3.6.2 CHOICE OF TRUSS TYPE

It is well known that labor and fabrication costs add the most in the total costs. Furthermore, in a lattice construction almost all the fabrication costs are in the bracings. Hence the most economical solution can generally be achieved by:

- Reducing number of bracings
- Using appropriate joint type relatively easy to fabricate
- Using appropriate type of member section

This is also illustrated as an example in Figure 5.6 below, where three types of trusses with different joint configurations are chosen (N, KT and K joints).

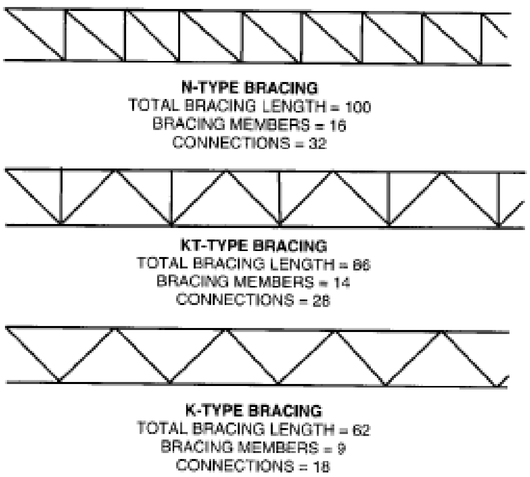


Figure 5.6 Truss girder layout [Cidect Design Guide No.5]

With the above in mind and due to limitations on the construction depth below the deck, through Warren truss configuration, type is considered (Figure 5.7). The depth is selected based on economic L/D ratios and the panel length L_i is chosen to be constant over the full length.

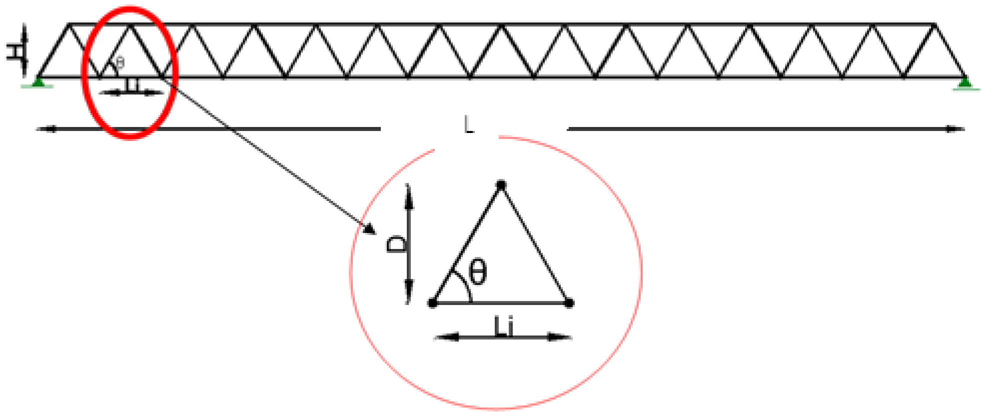


Figure 5.7 Warren truss configuration

Where,
H is the total construction height
D is the depth of the truss regarding L/D ratios (between center lines of chord members).
L_i is the length of the truss panel (field length)
θ is the brace inclination

5.3.6.3 Choice of cross sections

For the truss members subject to high axial forces (chords and braces), bending (chords and braces) and torsion (chords) and also thinking in terms of fabrication costs and overall economy, RHS are initially chosen (and circular-tubular- hollow sections, later on).

RHS members have evolved as a practical alternative for CHS members. This is because,

- No specialized profiling is required allowing easy connections to the flat face which makes them popular for columns and trusses.

- Structures made with RHS members are more economic to fabricate than with CHS members [3], if complete automated equipment is not available, because end cutting for joints with CHS require special profile which is much more expensive (when needs to be made manually or semi-automatically)

- If the deck is laid directly on the chord member, RHS offer superior surfaces to CHS for attaching and supporting the deck.

- Additional aspects need to be considered when choosing between RHS and CHS are the relative ease of fitting weld backing bars to RHS and of handling and stacking RHS. The latter is important because material handling is said to be the highest cost in the shop.
- Literature findings point towards a promising bridge design with HSS when truss designs with CHS members (and moreover with cast joints especially for large span bridges) are chosen, and thus majority of experimental testing and research are concentrated around CHS only.

Therefore, to enhance the knowledge of RHS members in bridge design, RHS members are initially chosen for the 'Schellingwouderbrug'. This is mainly based on the personal preference of the author to examine the possibility and suitability of RHS members, when high strength steel grades are to be used for bridge design.

The cross beams, however, mainly act in bending, so I- section profile (usually welded out of plates) is chosen.

5.3.6.3.1 Advantages of hollow sections over open sections

These sections offer structural advantages especially in case of members subject to compression and/or in torsion. Considering also their efficiency for lateral stability due to significant large torsional stiffness they offer a highly suitable solution for lattice girders [3].

Circular hollow sections (CHS) offer a pleasing shape but specialized profiling is needed when joining circular shapes together. Rectangular hollow sections (RHS) on the other hand offer an alternative allowing easy connections to the flat face and they are very popular for trusses.

5.3.6.3.2 Economy

Fabrication costs (in terms of labor hours required to produce a certain structural component) need not to be more for hollow sections than for open sections. In fact, they can even be less depending on the joint configurations. The efficiency of hollow sections joints is a function of a number of parameters which are defined by the dimensions of the connecting members.

Rectangular hollow sections are not standardized in very large dimensions and especially for HSS, thus they are usually made out of plates welded together (i.e. four longitudinal welds are required to create a RHS section).

On the other hand, for CHS a single longitudinal weld is required. In that respect fabrication costs may be lower in comparison to RHS members. In addition, CHS members are standardized in rather large dimensions (e.g. maximum available is 660x50 for S355 steel grade) and can be ordered from stock, directly, thus costs are reduced significantly.

Handling and erecting costs can be less for hollow section trusses than for alternative trusses. They have greater stiffness and lateral strength and that makes it easier to pick up and more stable to erect. Furthermore trusses made of hollow sections may be lighter than in case of different sections. In addition, for truss members mainly axially loaded, hollow sections represent the most efficient use of a steel cross section in compression.

Protection costs are appreciably lower for hollow sections trusses than for other trusses. Hollow sections trusses may have lower section sizes due to their higher structural efficiency. The absence of re-entrant corners makes the application of paint easier and the durability is longer.

It is false economy to try to attempt to minimize the mass by selecting a multiple of sizes for the braces. It is more preferable to use the same section size for a group of brace members. CHS joints are more expensive to fabricate than RHS due to end cut (i.e. straight bevel cuts for RHS while more expensive profile end cuts required for CHS when the tubes are to be directly welded together).

5.3.6.4 MODEL

The model of the truss bridge is made by using beam elements in Scia Engineer FE program (Figure 5.8). The chords are modeled as continuous beam elements over the whole length. The braces are modeled hinged on both sides for strength and stability checks. For the fatigue check rigid connections between chord and brace members are modeled to include the influence of secondary bending moments in the brace members.

Wind bracing is not included in the model since its presence does not affect the load carrying capacity under vertical loads. However, is theoretically being considered in terms of stability.

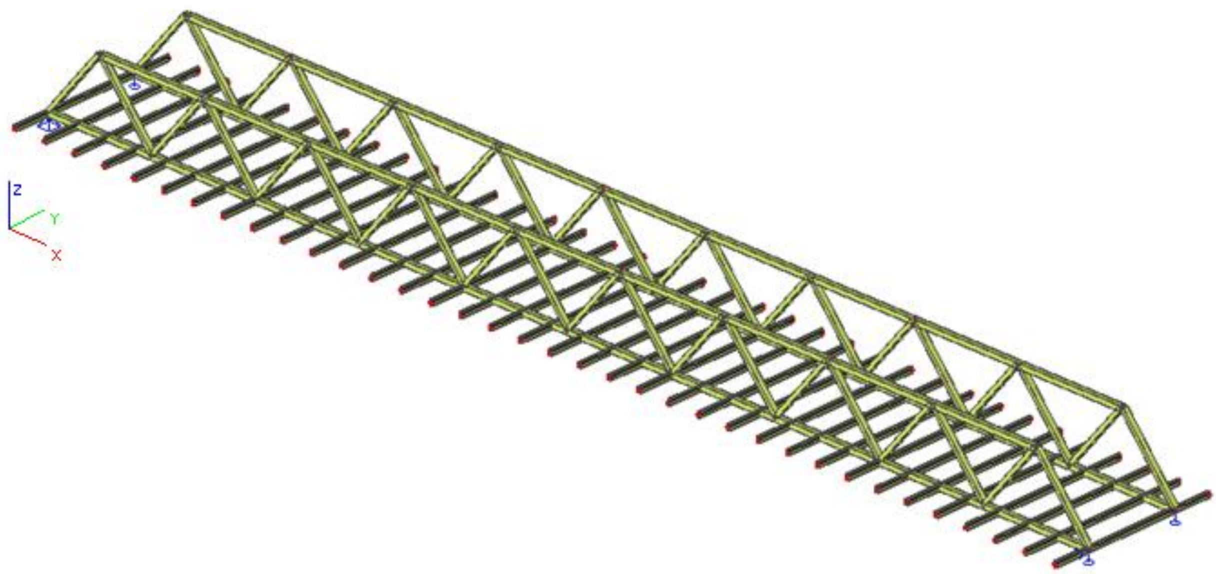


Figure 5.8 3D FEM model in “Scia Engineer”, L/D= 15, L_i = 10.5 m

The superstructure consists of the two vertical truss planes (Warren type) connected with cross beams every 3 m. The concrete deck rests on top of the cross beams (not in composite action). The traffic loads are positioned directly on top of the cross beams.

The bridge is simply supported on its two ends. The supports are chosen such that the bridge members can freely expand or shrink in case thermal fluctuations (Figure 5.9 and 5.10).

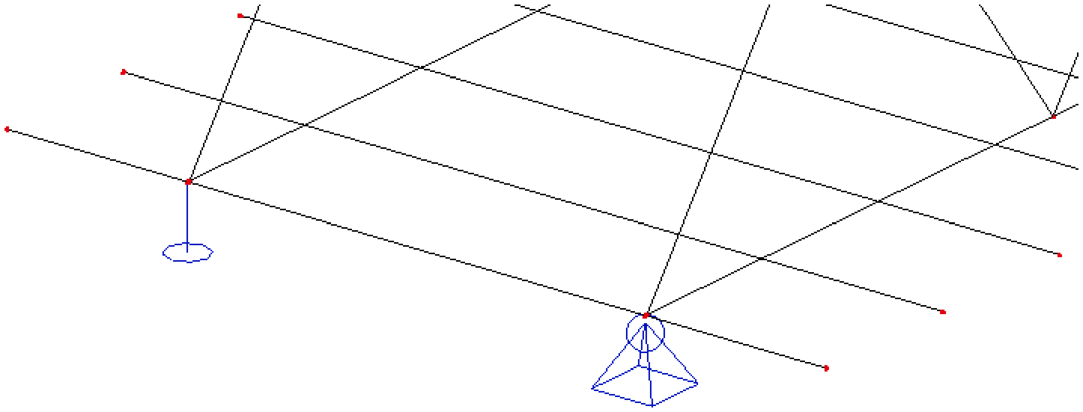


Figure 5.9 Support condition- Left end

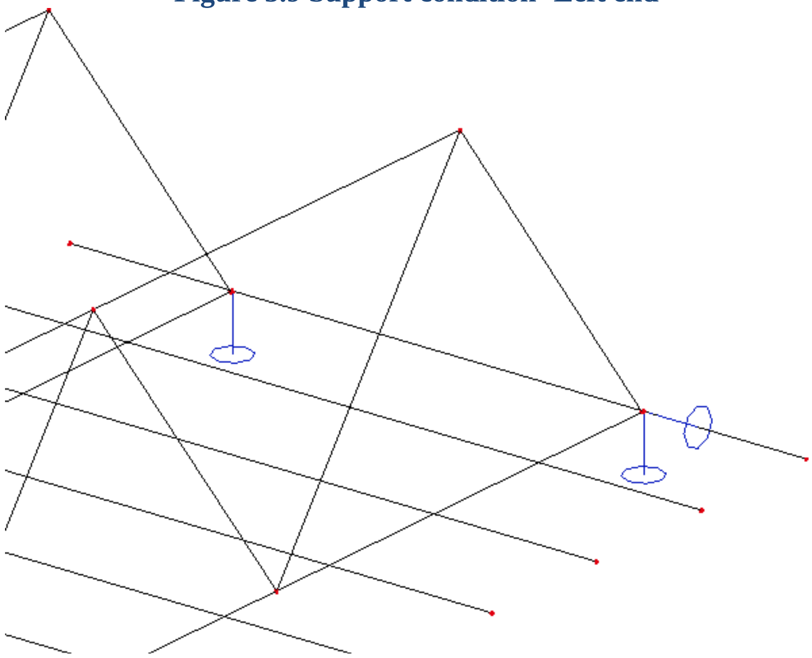


Figure 5.10 Support condition- Right end

5.3.6.5 MATERIAL CHOICE

Depending on the maximum stress level in each member, high strength steel grade, S690, is used for the chord members, S460 is applied usually in the more heavily loaded compression brace members, due to stability requirements (end braces) and to the most heavily loaded tension braces. S355 is used in the rest of the braces and in the cross beams.

5.3.6.6 RESULTS

Two preliminary designs for the truss bridge model in Figure 5.8 (see also Appendix A), have been developed each of which satisfy the main three design criteria strength, stability and fatigue. Fatigue stresses, according to FLM3 are not governing in any truss design.
On the other hand, buckling of compression members found to be critical in some cases. However, the stresses are kept at relatively high levels and the cross sectional dimensions relatively small. A more accurate fatigue calculation is recommended.

Initially, another truss model had also been attempted with larger field length (L_i = 15 m). Stability was governing that design due to extremely large, unsupported, member lengths. This design, however, is also included in appendix A.

Thus, use of high strength steel seems to be beneficial and a more detailed design together with total cost estimations should be considered. Also the use of circular hollow sections with cast joints is interesting to be investigated. Finally, truss optimization can potentially lead to an economic and effective bridge design with high strength steel.

5.3.7 Arch bridge

5.3.7.1 General

Steel arch bridges are generally an economic solution for spans 50-500 m [2]. The type of arch bridge used depends largely on the type of loading (highway or railway). An increase of loading, especially traffic loading, results for optimum design in an increase of arch height ‘f’.
Generally the f/L ratio is the nearly the same for highway and railway bridges and varies between 0.13-0.18 [3].

5.3.7.2 Choice of arch type

In case of an arch bridge design for the ‘Schellingwouderbrug’ a tied arch bridge type is only possible for the specific location. Furthermore, in order to fully utilize high strength steel a tied arch bridge with stiffening girder is preferred. In that case the stiffening girder predominates and is subject to large axial forces and bending moments induced by the arch, while the rather slender arch is mainly loaded in compression.

5.3.7.3 Structural system

The main superstructure is the two vertical (parallel) planes each of which consists of the arch connected to the main girder with vertical hangers. It is assumed that the connections between the hangers and both the arch and girder are pinned, so no bending moments in the hangers occur.

The main girders in both planes are connected with cross beams with c.t.c distance of 3m over the full length of the bridge. For the arch and girder members box shaped members (welded or RHS) are chosen. The concrete deck is resting on top of the cross beams but not in composite action acting separately from the main structure. In the arch model (made in Scia Engineer) is being treated as a separate load case (dead load LC6).

5.3.7.4 Analysis and modeling

The model of the truss bridge is made by using beam elements in Scia Engineer FE program (Figure 5.11). Elastic global analysis, assuming pin joints for the hangers and loading directly on the cross beams, is performed in order to obtain maximum normal forces in all members (arch, girders, and hangers).

Design of hangers is generally slightly different than for other steel members and here is not considered in much detail. Therefore, for fatigue verifications again pinned connections are assumed for the hangers as this will not have an influence on the main structure. However, in reality the diameter calculated for the hangers may not be sufficient for fatigue and needs to be increased.

In addition the main girders are taken to be stiffer and have bigger dimensions than the arch (parabolic) to achieve no bending moments in the arch under full loading.

Wind bracing is again not included in the model since its presence does not affect the load carrying capacity under vertical loads. However, is theoretically being considered in terms of stability.

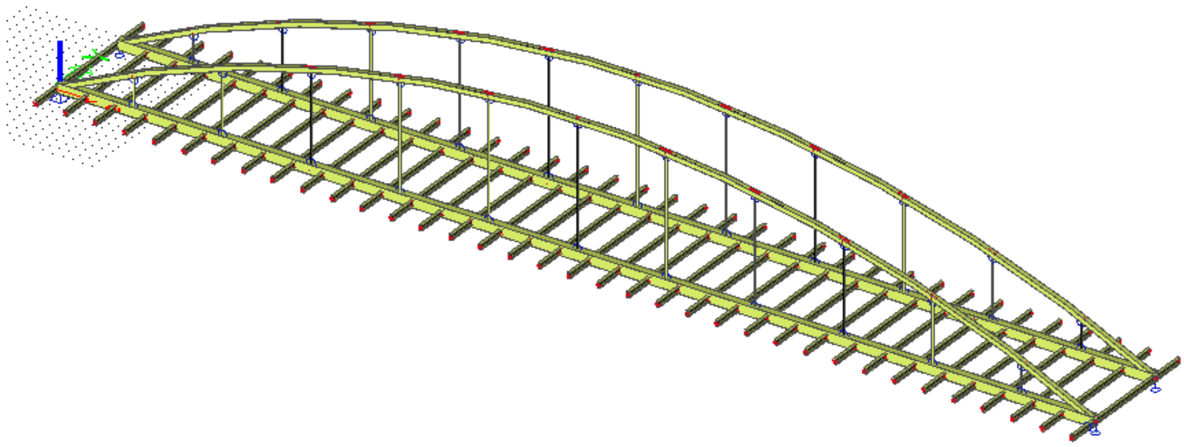


Figure 5.11 Arch bridge configuration

5.3.7.5 Design criteria

The designs criteria are strength, fatigue, and stability.

5.3.7.6 Results

Several trial designs are performed based on this arch configuration (see Appendix A). The results show that arch design is governed by fatigue stresses especially in the main girders. This limits the maximum stress level for static stresses also, thus makes use of HSS ineffective. S690 can only be used for the hangers but this was anyway the case so far for these members.

6 Results

In Part 1B, three preliminary designs for a single, long span bridge in the Netherlands- the Schellingwouderbrug-, using high strength steel grade S690 in the heavily loaded members.

The three designs involve a box girder bridge, a warren type truss girder bridge and a tied arch bridge with vertical hangers. The main design criteria were strength, stability and fatigue. However, only the governing criteria were investigated for each bridge.

For example, buckling was not mentioned in case of an Arch bridge because it was found right almost from the beginning that fatigue stresses were high due to large bending moments in the deck and use of high strength steel was not efficient. Plus the member buckling lengths were smaller than in the case of the truss while the cross sectional areas were much bigger. Stiffness was also considered especially in case of the box girder due to the very slender cross section.

The main results are:

1. It is found that it is generally possible to consider (at least at the preliminary phase) a box girder so slender using S690 or even S500 for the bottom flange (region with very high tensile stresses) and with relatively small thicknesses (min thickness= 50 mm for the bottom flange). However the demands for stiffness and local buckling will probably not lead to the most economical solution.

2. The extremely slender box section, due to clearance restrictions below the bridge deck, cannot satisfy the demands mainly for stiffness. The strength was sufficient if the gross sectional area was effective but the effective area was 60 % reduced due to local buckling effects (class 4 cross section). The plate thicknesses need to be kept relatively small to avoid expensive preheating.

3. It is not possible to use a box girder (alone) for the 'Schellingwouderbrug' due to depth limitations. This is already verified by the literature findings, where it was found that economic L/D ratios for box girders are in the range of 20 to 30. In this case L/D gives an extremely big value (= 64!) and thus, leads to an uneconomic solution.

4. Two preliminary designs for the truss bridge have been developed each of which satisfy the main three design criteria strength, stability and fatigue. Fatigue is not governing in any truss design.

5. On the other hand, buckling found to be critical in some cases. However, the stresses are kept at relatively high levels and the cross sectional dimensions relatively small.

6. Thus, use of high strength steel seems to be beneficial and a more detailed design together with total cost estimations should be considered. Also the use of circular hollow sections with cast joints is interesting to be investigated. Finally, truss optimization can potentially lead to an economic and effective bridge design with high strength steel.

7. Fatigue stresses due to bending are governing the tied arch configuration. The stresses are kept quite small and the member cross sections need to be increased a lot to keep fatigue stresses low. This, however, keeps also static stresses much below 400 MPa and thus it is not proving to be efficient to use high strength steel grades. Explicit detailing (FEM modeling) together with a more accurate fatigue calculation is recommended before we can conclude on the unsuitability of the arch bridge in hybrid construction. However, this was not possible to be included in this study.

8. Preliminary design points towards a truss bridge design for the Schellingwouderbrug. Apart from the design criteria there are other important issues that need to be considered in order to determine the suitability of the truss bridge design for the specific location. Those are erection method, bridge transportation on site and also maintenance. These practical aspects also determine a big percentage of the total costs.

6.1 Extra considerations

The ‘Schellingwouderbrug’ is located above a big canal and thus transportation is also possible from land as from the river. However, erection methods are limited to launching and lifting since temporary supports would interrupt the boat traffic. For example it is possible that the whole bridge could be totally prefabricated in its final state (painted etc.), transferred on site by boat and then lifted up in place with cranes. High strength steels can benefit this method even more due to low dead weight of steelwork they provide.

The truss bridge can therefore be an interesting solution for the use of high strength steel grades in case of the ‘Schellingwouderbrug’ leading to an effective and economical design.

Part 2
Detailed design

In chapter 7 material costs are calculated for several bridge designs based on their self-weight. The designs are compared and one of them is chosen based not only on material costs but on total costs consideration.

The chosen design (i.e. hybrid design of a truss bridge) is designed in more detail, in chapter 8. Initially, rectangular hollow section (RHS) members are considered and design of connections is performed for adequate strength and fatigue. Later on, a hybrid design with circular hollow section (CHS) members is also developed using the same bridge configuration. All the bridge designs are made according to European standards.

In addition estimation of cross sectional dimensions and associated costs is attempted for equivalent designs out of S355 only for both RHS and CHS truss designs.

In chapter 9, material costs are calculated and also total costs are estimated for each hybrid design (S355, S460 and S690) and their homogeneous equivalents (S355 only). The designs are compared on a total costs basis. The comparison aims to check whether the hybrid bridge designs can be more economical than the homogeneous ones.

7 Choice for bridge design

7.1 Comparison

Based on the conclusions drawn in Part 1 (literature survey and 3 preliminary bridge designs for a long span bridge –The Schellingwouderbrug-), it is concluded that, a truss bridge is a potentially good design using higher steel grades.

For the truss bridge with concrete deck (not in composite action), three alternative designs were presented and discussed, with respect to the basic design criteria (i.e. strength, stability, fatigue). Two of them appeared to satisfy all design requirements allowing for sufficiently high stresses and making use of HSS possible.

A rough comparison for the two hybrid truss designs (with RHS members) is shown in Table 8.1. Material costs are calculated based on the self-weight of the steel structure only (steel members), assuming that S460 and S690 is 30% and 75% respectively, more expensive per kg of steel, in comparison to S355. If for example, S355 costs 1.00 €/kg material then S460 costs 1.30 €/kg and S690 1.75 €/kg. It is important to note that in these prices are not included other important costs, like fabrication (e.g. welding costs), handling, transportation costs, etc. which obviously have a great influence on final total costs of the bridge design. However, these all can be assumed to be in favor of HSS (for small thicknesses 20-30 mm), due to lower self-weight and smaller cross sectional dimensions. Furthermore, the self-weight of the structure presented in Table 8.1, should be increased by about 15% to account for additional steel (e.g. local stiffeners, details, etc.).

	Design Truss 2				Design Truss 3			
	A (mm²)	min fy (MPa)	U.C	weight (tn)	A (mm²)	min fy (MPa)	U.C	weight (tn)
Top chord	96640	460	0,91	159	66500	690	0,94	110
Bottom chord	60800	690	0,93	100	47800	690	0,91	79
End Braces	29444	690	0,78	9	26100	460	0,9	7
Rest braces	29444	355	0,61	57	26100	355	0,8	65
Cross beams	26400	355	0,67	122	30640	355	0,54	141
Total steel weight (tn)				**448**				**401**
Material cost estimation (€)				648000				544742

Table 7.1 Comparison of designs Truss 2 and 3 with respect to steel structure weight and subsequent material costs (see appendix A).

According to Table 8.1, design "Truss 3" leads to a lighter (11 % weight reduction) and more economic (about 16 %) design with respect to material use and material costs. This is mainly due to smaller cross sectional dimensions in comparison to "Truss 2". In addition, maximum plate thickness is about 30 mm (top chord) in "Truss 2" and 25 mm in "Truss 3" (top chord). Thus, welding volume as well as fabrication costs can also be reduced. Reduced weight will of course have a favorable effect on transportation and handling costs. Furthermore, smaller cross sectional areas reduce the required painted area, resulting in reduced maintenance and corrosion-protection costs.

For the braces and the cross beams lower steel grades are also possible with the given dimensions (e.g. S420, S550, S275 and even S235). However, in order to facilitate cost estimation, S355 is considered as the minimum steel grade to be used.

"Truss 3" design has smaller field length (10.5 m in comparison to 15 m in "Truss 2"), thus more braces and joints to be welded in comparison to "Truss 2". However, careful design of the joints may allow for repetition in the fabrication shop, and weight reduction can offset additional fabrication costs, due to increased number of joints. The most optimal solution may be somewhere in between (e.g. increase a bit the field length and the cross sectional areas in order to reduce the connections and additional welds). This requires further investigation.

Design "Truss 3" is also compared to design "Arch 2", in the same manner as with "Truss 2" design. In design "Arch 2", fatigue seemed to be the governing factor right from the beginning, and so, static stresses were kept at very low levels (when static stresses exceeded 400 MPa, high fatigue stresses arise in the main girders) making use of HSS ineffective. The comparison (see Table 7.2) is meant to check, if a hybrid construction (using HSS for the heavily loaded carrying members and lower grades for the less heavily loaded members) provides any benefits over a design out of S355 (and also support the choice for a truss over an arch bridge for the 'Schellingwouderbrug'). To facilitate comparison, U.C. ratios (stress/strength) are considered to be relatively the same for both designs.

Maximum plate thickness for the 'truss 3" design is 25 mm (in the top chord) which is smaller than in design "Arch 2" (30 mm in the main girders). This will of course have a positive effect on fabrication costs and welding volumes (i.e. will be reduced).

	Design Arch 2				Design Truss 3			
	A (mm²)	min fy (MPa)	U.C	weight (tn)	A (mm²)	min fy (MPa)	U.C	weight (tn)
Arch/Top chord	67000	355	0,95	110	66500	690	0,94	110
Girder/Bottom chord	128400	355	0,92	212	47800	690	0,91	79
Hangers/End braces	1809	690	0,86	2	26100	460	0,9	7
Rest braces	-	-	-	-	26100	355	0,8	65
Cross beams	23400	355	0,62	108	30640	355	0,54	141
Total steel weight (tn)				**432**				**401**
Material cost estimation (€)				433000				544742

Table 7.2 Comparison of designs Truss 3 and Arch 2 with respect to steel structure weight and subsequent material costs (see Appendix A).

In Table 7.2, it is shown that "Truss 3" hybrid design leads to a lighter construction (7% weight reduction) but as expected, material costs are much higher (by 26%) in comparison to the arch bridge made in S355 (HSS was only used for the hangers as customary). This is, in any case, only a present-day disadvantage, since the material price continuously varies. In addition, total cost estimation depends on many other parameters.

So, from the preliminary phase it seems, it is possible to develop a competitive truss design in HSS, (considering it mainly in a hybrid construction, -combination of steel grades-). In Table 7.1 and 7.2, it is obvious that unity check ratios are very close to unity especially for the chord members, thus optimization is necessary by performing a more in detail design.

Further detailing for the truss design may lead to a more optimal design or even show possible disadvantage (i.e. how can we deal with fatigue?) of HSS, which is the aim of the second part of this thesis study. For example, in the preliminary phase, all the braces are assumed to have the same cross sectional area and S355 is assumed to be used for all the intermediate brace members (and S460 for the end braces) based on the most heavily loaded brace member (i.e. braces 2 and 3). But in fact, only the braces closer to the supports need to withstand the higher normal stresses. Therefore, all intermediate braces close to midspan may be built out of even smaller steel grades. Alternatively, smaller brace cross sections can be used in truss region around midspan. In that way, material as well as total costs may be further reduced.

It must be noted that in these trial truss designs it is difficult to apply HSS (S690) in the braces also due to stability problems. This refers of course especially to the compression braces, because the member length is rather large and will lead to instability problems. Furthermore, aesthetical considerations would more likely limit reduction of the cross sectional dimensions for the tensile braces, also. However, an alternative design (e.g. (much) smaller brace length or intermediate brace supports) can make use of HSS possible in the braces also, allowing for smaller cross sections and thus, reducing the overall steel structure weight even more.

7.2 Final choice

Concluding, it is expected that due to lower dead weight (the difference is rather small but could be further increased with careful design optimization) of the hybrid truss design ("Truss 3") -in comparison to "Truss 2" and "Arch 2" designs- the economic benefits to be gained in terms of total costs highly offset the higher material costs observed in Table 7.2.

This is based on the estimation that, lower steel self-weight will have an influence on foundations (e.g. smaller piers may be required), transportation and erection costs, while smaller cross sectional areas will have a positive effect on maintenance (smaller painted area required) and fabrication (especially welding) costs, especially in small thicknesses (no preheating and no special machining equipment).

Thus, design "Truss 3" is chosen for a detailed design, as it seems that can offer further weight savings and consequent cost benefits in terms of total costs for the 'Schellingwouderbrug'.

8 Detailed truss bridge design

8.1 General

Detailed design refers to design of joints and connections. Connections have an essential role on the final decision of member design and cross sectional properties but also on costs.

Especially nowadays, fabrication and labor costs are the most costly part of construction. Designing, for example, for the lowest bridge weight, does not usually offer the most economical solution (e.g. extra costs of stiffening very slender plates).

Thus, designing and detailing must be carried out always with respect to feasibility and constructability.

8.2 Design codes and limitations

The design of connections is done according to European standards EN 1993-1-8 (design of joints) and EN 1993-1-9 (fatigue design). Additional information for HSS was found in CIDECT design guides 1 [13], 3 [14] and 8 [16] for design of hollow section joints. The joint resistances given in these guides apply up to steel grade S355.

For higher strengths it is mentioned that a reduction factor should be applied to all joint capacity equations, to account for larger deformations (an out-of plane deformation of $0.03b_0$ of the connecting RHS face, is used as the maximum deformation limit in case of S355, [Lu et al., 1994]) in case of chord face plastification occurs (thus if another failure mode governs it is rather conservative).

This reduction factor is proven to be 0.9 (verified by tests, [Liu and Wardenier, 2004]) for grades up to S460 and recommended to be 0.8 (not verified by tests though) for higher strengths (e.g. S690)

Both, EC3 and CIDECT design guides, include geometric and material limitations and thus, their application in case of HSS is considered conservative or even unsuitable in several cases. For example, in EN 1993-1-9 the fatigue details require that the geometries of the cross sections are within certain limits, and thus for this truss design they are not applicable.

In this case fatigue calculations should be based on hot spot stress ranges calculating the stress concentration factors (SCFs) for different joints and for different locations (e.g. toe, heel etc.).

8.3 Critical truss joints with RHS members

The most critical locations for each member have been found and checked only. A detail is defined as critical with respect to either its strength capacity or if it appears to be fatigue sensitive.

More specifically, only the joints/connections at the most heavily loaded member location (i.e. location on the bridge where each member must resist its maximum static load) and fatigue sensitive location (i.e. location on the bridge where each member must resist its maximum fatigue load or location with smaller fatigue load but worse fatigue detail class) have been checked.

In Figure 8.1 the numbering of joints and members is defined. In reality, all or at least all the critical details should be checked for a truss bridge. For the "Truss 3", joints 1, 2, 3, 10 and 11 (circled ones in Figure 8.1) are considered critical and are checked for joint strength capacity and fatigue.

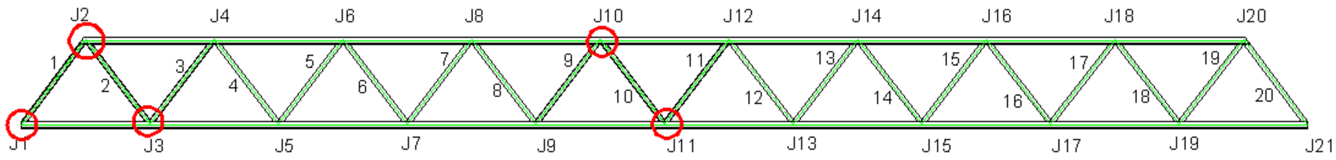


Figure 8.1 Member and joints numbering and critical joints for Truss 3 design

The joint type (Y- , K-gap-, K- overlap- joint) depends on the load transfer between the members in the joint [14].

8.4 Design of critical joints

In Figures 8.2-8.6 the critical joints between rectangular hollow section members are presented.

These joints have been designed to satisfy requirements in table 7.8 in EN 1993-1-8 and thus, they have been checked for strength (joint design axial resistance), depending on joint type, according to tables 7.10-7.12 in EN 1993-1-8. Furthermore, a reduction factor 0.8 is applied to the calculated resistances when S690 steel is used [14], [EN 1993-1-12].

It is reminded that for strength verifications, the brace members are designed pinned at both ends; therefore, in calculating the joint resistance, brace members are assumed to be subjected only to axial forces.

Thus, it must be verified that $N_{i,\,Ed} / (0.8 * N_{i,\,Rd}) \leq 1.0$

$N_{i,\,Ed}$: Design axial internal force (due to loading) in the brace member
$N_{i,\,Rd}$: Design axial resistance in the brace member

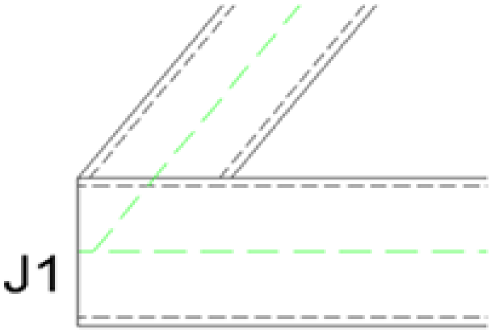
As forementioned, for fatigue verifications things are more complicated, since due to cross sectional dimensions these truss members do not comply with detail class tables in EN 1993-1-9. Thus detailed calculation to determine the specific stress concentration factors needs to be made. In this way the hot spot stress is calculated and relevant hot spot stress curves are used to determine the fatigue strength.

Determining the SCFs though requires accurate and detailed modeling which can only be made with FEM program using plate elements. This could not be done within this thesis project though since determining the exact SCFs for different joint configurations is a thesis project by itself.

Nevertheless, using SCFs' formulas and relevant hot spot stress graphs found in literature [16], a fatigue check has been performed to estimate the fatigue performance. Finally, the Miner's rule is applied to calculate the fatigue damage at each critical location caused by FLM4 on the bridge.

For more information reference is made to excel sheets for fatigue calculations and Appendix B.

Joint 1: Connection between bottom chord and end brace member



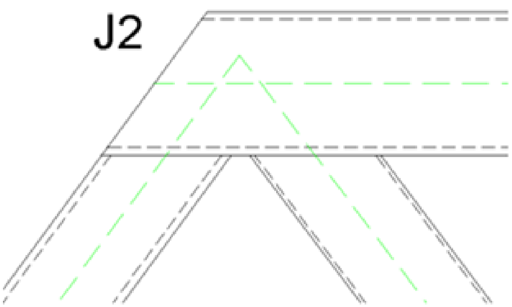
Joint 1 (J1):

Bottom chord: 500x 550x 30
End brace: 500x 400x 19
θ_1= 54°

Figure 8.2 Critical Joint 1 configuration, Y- joint

The end brace is the most heavily loaded compression brace member on the bridge. This joint is checked for strength (all members) and for fatigue (with respect to the end brace member).

Joint 2: Connection between top chord, end brace and brace 2 members



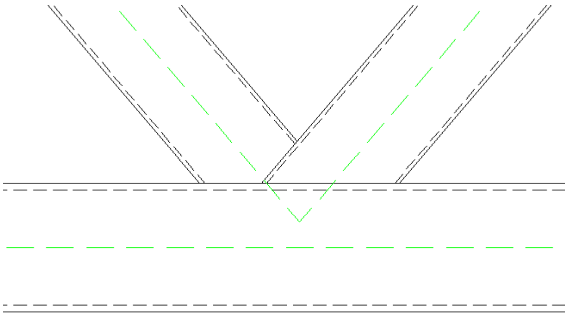
Joint 2 (J2):

Top chord: 500x 550x 30
End brace: 500x 400x 19
Brace 2: 400x 400x 15
Gap g= 70 mm
Eccentricity e= + 108 mm
θ_1= 54°
θ_2= 53°

Figure 8.3 Critical joint 2 configuration, K- gap joint

This joint is checked for strength (all members) and for fatigue (with respect to the end brace member). Brace 2 is the most heavily loaded tension brace member.

<u>Joint 3</u>: Connection between bottom chord, brace 2 and brace 3 members



J3

<u>Joint 3 (J3):</u>

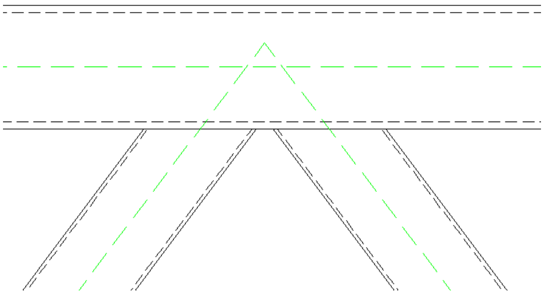
Bottom chord: 500x 550x 30
Braces 2 and 3: 400x 400x 15
Overlap Ov= 50%
Eccentricity e= -108 mm
$\theta_2 = \theta_3 = 53°$

Figure 8.4 Critical joint 3 configuration, K- overlap joint

Braces 2 and 3 are the most heavily loaded with respect to fatigue at this joint. This joint is checked for strength (all members) and for fatigue (with respect to the brace member).

<u>Joint 10</u>: Connection between top chord, brace 9 and brace 10 members

J10



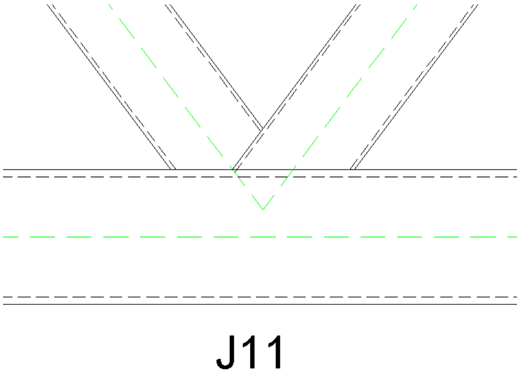
<u>Joint 10 (J10):</u>

Top chord: 500x 550x 30
Braces 9 and 10: 400x 400x 15
Gap g= 75 mm
Eccentricity e= +108 mm
$\theta_9 = \theta_{10} = 53°$

Figure 8.5 Critical joint 10 configuration, K- gap joint

This joint is chosen to be checked for strength as the top chord is most heavily loaded here. Also fatigue strength is checked with respect to the top chord as fatigue stresses are also quite high in the chord due to FLM4.

<u>Joint 11</u>: Connection between bottom chord, brace 10 and brace 11 members



<u>Joint 11 (J11):</u>

Bottom chord: 500x 550x 30
Braces 10 and 11: 400x 400x 15
Overlap Ov= 50%
Eccentricity e= -108 mm
$\theta_{10}= \theta_{11}= 53°$

Figure 8.6 Critical joint 11 configuration, K- overlap joint

This joint is chosen to be checked for strength as the bottom chord is most heavily loaded here. Also fatigue strength is checked with respect to the bottom chord as fatigue stresses are also quite high in the chord due to FLM4.

Maximum fatigue stresses in the bottom chord member are not located exactly at the vicinity of the joint but they are located a bit further to the right and specifically at the connection between bottom chord and cross beam almost at midspan. This latter connection can be made with transverse butt welds from both sides.

The truss joint configuration (i.e. joint J11) is considered more critical than the connection between cross beam and bottom chord. Higher stress concentrations are also expected to occur in the first case, especially if filet welds are to be used.

Fillet welds are usually considered for lattice girder node joints (especially at the brace toe) as they are less expensive and easier to fabricate. This is however, clearly a more fatigue sensitive detail than butt welds between two plates (i.e. butt weld between flanges of bottom chord and cross beams).

8.5 Results

8.5.1 Strength

All the joints have been proven to have sufficient strength even with the reduction factor of 0.8. The critical failure mode is punching shear for joint J1 and brace failure for all the other joints (i.e. joints J2, J3, J10 and J11).

8.5.2 Fatigue

Fatigue calculations based on SCFs and hot spot stress have been proven not to be sufficient for all joints, within the design fatigue life of $5*10^7$ cycles. This clearly shows that this type of connection between truss members (direct member connection by welding them together) is not suitable for this bridge design and the specific cross sectional dimensions (if the cross sections increase as to satisfy fatigue, then the static stress level will be low and HSS as S690 will be inefficient). Thus, alternative solutions need to be considered with respect to improve fatigue performance.

8.6 Improvement of connections with RHS members

The direct way of connecting RHS members by welding them together it is proved not to be sufficient for fatigue for this design. The connections show sufficient strength resistances due to the high strength material even with a reduction factor of 0.8. However, fatigue damage is extremely big (D >>1) and thus, the connection need to be altered and improved.

The simplest method of dealing with high fatigue stresses (especially due to bending) is to increase the overall cross sectional dimensions of the connecting members. Larger cross sectional dimensions may cause somewhat higher secondary bending moments but the overall stresses would be less. However, something like that would result also in very small static stresses making use of HSS inefficient and of course uneconomical.

In order to improve the fatigue behavior of the connection and eliminate the damage during the whole lifetime of the bridge, fatigue stresses need to be minimized (even twice as much downwards for the bottom chord) without making the use of HSS unfavorable.

The best way to do that is by moving the critical (fatigue sensitive) locations of the joint away from regions of high stresses or/and altering the joint configuration. In this way the fatigue detail can be significantly improved, leading to better results. However, in case the stresses to be compensated are not too high a local increase of the plate thickness can be also sufficient.

ALTERNATIVE 1: INCREASE MEMBER THICKNESS LOCALLY

Details in figure 8.7 (a) and (b), are to be preferred when the fatigue stresses can be reduced with a small increase of plate thickness locally (in the chord or/and in the braces). Especially when HSS is used too thick plates are not desirable, neither economical. Of course plates with lower steel grades can also be used allowing for even higher thicknesses. The plates can be connected with transverse butt welds (one or two side butt welds) at locations of same plate thickness (see Figure 8.7 (c)). The gradient in the region of thickness transition is about 1/4- 1/5.

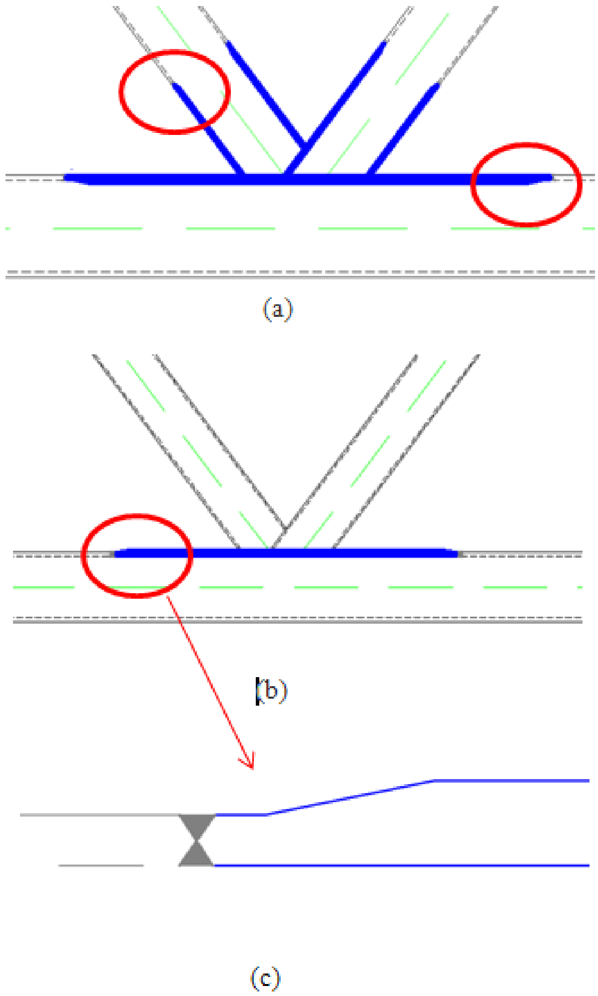


Figure 8.7 Locally thicker plates (a) when both braces and chord members are critical, (b) when the fatigue sensitive detail is located in the chord member. Plates welded together with one (or two if possible) side transverse butt welds.

This detail has the advantage that it thickens the plates in the critical locations, thus reducing the fatigue stresses (but also the static stresses) locally and the stress concentration factors may be lower especially in the chord member due to lower τ- ratios (=t_i/t_0). In addition, the one or two side butt weld that connects the plate is positioned further than the thickness gradient at a location of equal thicknesses. This requires that extra material need to be lost (removed). The final result, however, is a better detail for fatigue, since it is outside the very high stress region and the influence of the thickness gradient is negligible.

The main disadvantage is that the detail configuration is not altered, thus the critical locations and the complexity of the joint behavior remains the same, and in addition extra critical locations (butt welds) need to be checked (although probably not governing).

In case of S690 is recommended that the plate thickness can be kept at rather low values (at about 30 mm) to avoid extra costs from expensive preheating. In that case it may be better to use a lower steel grade to compensate with the extra costs. Of course for very thick plates, the yield stress decreases after a certain thickness (e.g. after t=50 mm for S690) and also through thickness properties decrease. That should be taken into account, if relevant.

Generally, another very important aspect is the stiffness (flexibility) of the connection and the source of fatigue stresses. If stresses are caused mainly by secondary bending moments, then by increasing the stiffness of the connection (higher thicknesses and/or higher brace width) the secondary bending moments will increase thus the fatigue stresses will increase. Of course, the axial stresses will decrease due to larger members' dimensions but the question still remains; what is the net value in this case?

If the final result leads to lower fatigue stresses then it is beneficial, but in any other case adding stiffness to the connection doesn't improve its fatigue behavior. In an ideal situation perfectly hinged connections between braces and chord members would eliminate secondary bending moments and therefore, increasing thickness would only reduce axial stresses and have positive results.

However, in our model it is assumed completely stiff connections between members, therefore, the flexibility and the actual stiffness of the joint is not taken into account to calculate internal moments and forces. Thus, design is being on the safe side. Being conservative in this way has the advantage that even if the thickness of the section (or any other cross sectional dimension) will increase the stresses due to bending and axial forces will always decrease (i.e. using the same forces and moments from the model to calculate the effect of the plate thickness increment, without altering the section properties in the model to recalculate new internal forces).

Especially in the design under consideration, the fatigue damage due to fatigue stresses is extremely high to be compensated by a small thickness increment. The biggest fatigue damage occurs at the bottom chord member close to midspan mainly due to bending moments (in the chord). Specifically fatigue stresses due to bending are 2-3 times higher than axial stresses.

Increasing the plate thickness 3 times (t=90mm!) and using the lowest SCF allowed in CIDECT (SCF=2.0) [16], axial and bending stresses become 3 times and 2 times smaller, respectively. However, even in this case scenario, fatigue damage although significantly reduced, still remains quite high. Moreover, if lower grades were to be used the plate thickness should then be extremely high.

Thus, this solution seems not to be suitable for this design. However, this conclusion is based only on global calculations and rough estimations. For more accurate results modeling of the connection using FEM analysis for calculation of the actual hot spot fatigue stresses needs to be performed. This is however, outside the scope of this thesis project but can be the subject for further research.

Alternative 2: Alter joint configuration using side plates (gussets)

In this alternative design, two relatively thick side (gusset) plates with a special profile (Figure 8.8 (a)) act as web plates for both braces and chord RHS members, over a certain distance. The plates are welded to the flanges of both braces and chord with butt welds. The flanges of the braces are also welded to the chord face (flange). The flanges are continuous over the whole length of the members.

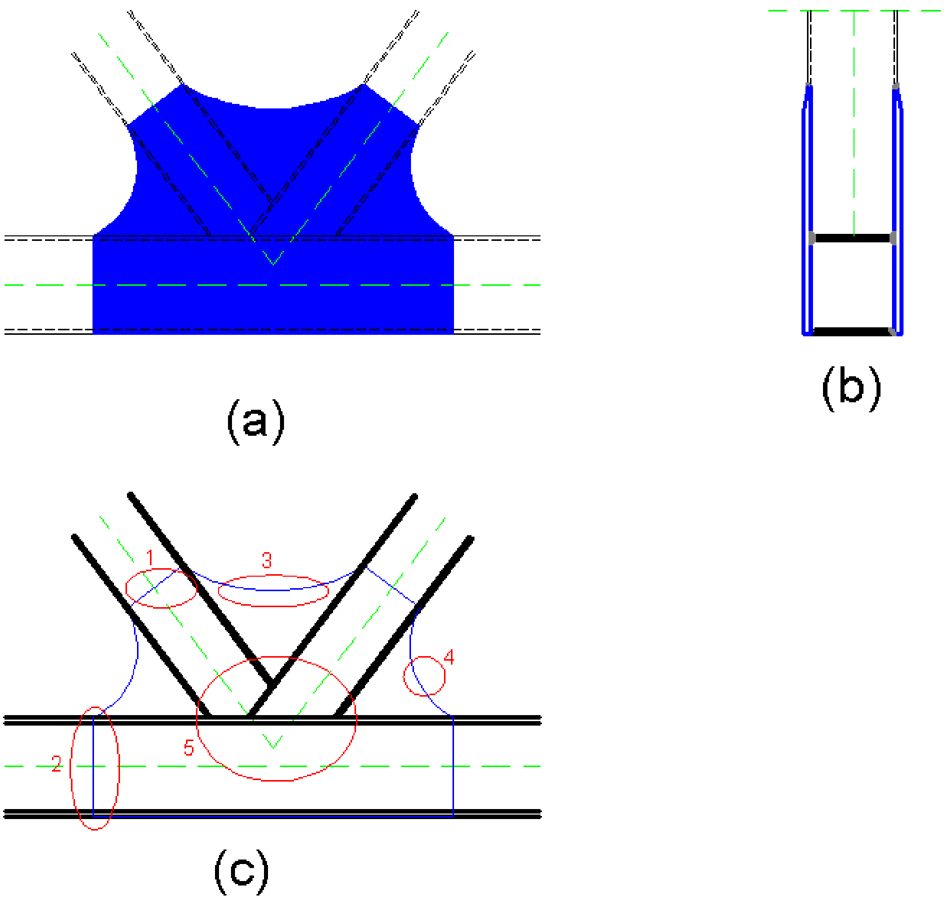


Figure 8.8 Improved connection with thick side gusset plates. (a) blue shaded area represents the side plate profile, (b) cross sectional view, (c) brace and chord flange connection and possible fatigue critical locations.

The main idea behind this solution is to make the fatigue sensitive details of RHS members (Figure 8.8 (c), location 5) non critical, by alternating the joint configuration and more specifically, the way the load is transferred from the braces to the chords. That is similar to the philosophy behind casting in circular hollow sections.

The load from the braces is assumed to pass through the welds (brace to plate connection- location 1 in Figure 8.8 (c)) to the thick side plates, and then from the plates to the chord member (through plate to chord transverse butt welds- location 2 in Figure 8.8 (c)).

A limitation of this design is that brace and chord members should have equal widths to make the welding of the plates feasible. Again, the connection between the plates, with thickness transition from one plate to the other, is similar to that in Figure 8.7 (c).

The flanges of the brace members are welded also to the chord face (e.g. location 5 in Figure 8.8 (c)) in order to shield the joint from the environment (better durability, less maintenance demand). However, at this location is expected that low stresses will occur, since the main load transfer is being made through the plates and not through the chord face.

The distance for which this plate is extended over the braces and chord at the joint location, depends on the force and moment distribution in the truss model. The concept is to choose the location of the connecting butt welds being at a less severe location (with lower stresses), away from the joint.

The advantage of doing this is that, at these locations only nominal (and lower) stresses in the members can be taken into account, and the fatigue life can be calculated by using the relevant S-N curve according to detail classification in EN1993-1-9.

Thus, changing from an unfavorable detail (lattice girder joint) to a more favorable one (plates welded together) and also moving to a region with much lower stresses, results in a much improved fatigue behavior for these connections.

For fatigue check the critical positions are now located mainly on the ends of the side plate (see Figure 8.8 (c)) where the load is transferred from and towards adjacent members through the butt welds. In the side plate localized stress concentrations may occur at the curved part (location 4) or at “passing through” welds (e.g. where chord flange is butt welded to the side plate). The actual level of stress concentration can only be determined by FEM analysis. The overall stress distribution though will be rather uniform.

In any case, the stress peaks can be minimalized by adjusting the radius (i.e. higher radius creates smoother shapes and thus lower stress concentration) and/or increasing plate thickness. Thus, fatigue check can be satisfied.

Since the brace member is connected to the flange of the bottom chord (Figure 8.8 (c), location 5), all the critical for fatigue details remain in the joint. However, they are expected not to be governing due to much lower stresses at this location in comparison to locations 1 and 2.

To avoid completely dealing with these details the design of the connection may be further improved.

A possibility is to stop the brace member at the connection with the side plates (Figure 8.9 (a)). However, high stress peaks will occur at the corners of the end bottom plate of the brace, due to higher stiffness at this location (Figure 8.9 (b)). These peaks can be significant, and thus critical for fatigue.

Therefore, a further improvement could be to profile the brace at the connection with the side plates as shown in Figure 8.9 (c). In this way the stress distribution will be more uniform due to the plate curvature. An extra thin plate can be added in the brace just to close it from the environment.

This solution has the advantage that we avoid the connection between the brace flange to the chord face. However the space between the braces, the chord and the two side plates remain open from one side, thus additional maintenance issues occur. The area should be protected (e.g. painted) and accessible for regular inspections and future repair, if necessary.

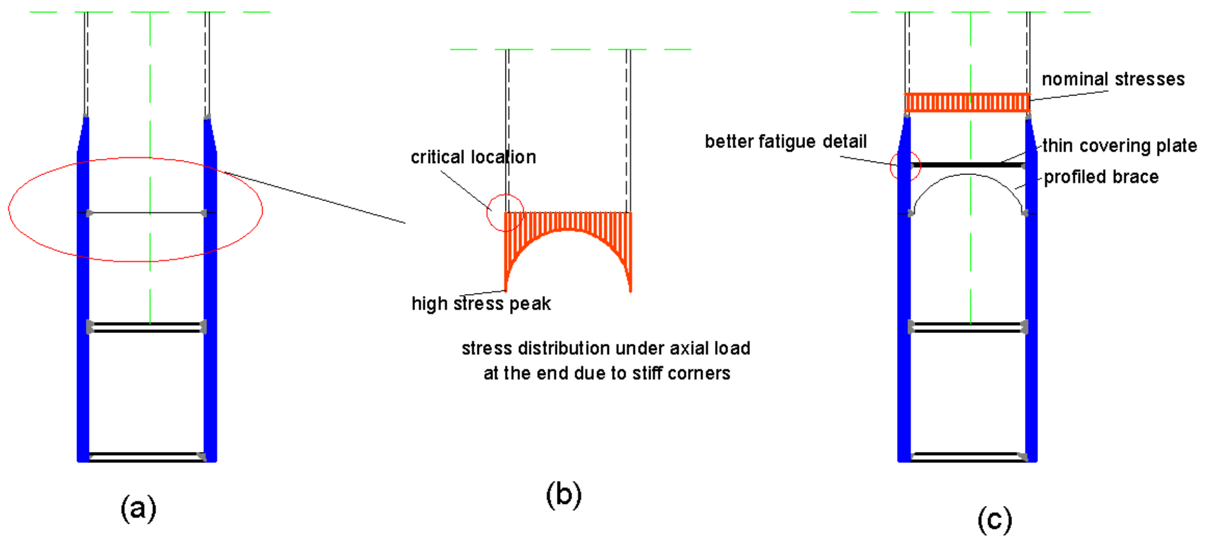


Figure 8.9 (a) end brace stops before reaching the chord face, (b) stress distribution at the end of the brace due to axial stresses- fatigue sensitive detail in the corner, (c) more uniform stress distribution by profiling the brace at the end of the brace and extra thin plate is added to shield the brace (not critical fatigue location)

In the current design in HSS, the most severe locations (locations 1 and 2 in Figure 8.8 (c)) were checked for fatigue. The butt welds are located at a distance 1.5 m away from the center of the joint in the chords (location 2) and at 2 m in the braces (location 1).

The fatigue resistance proved to be sufficient for most members at the critical joints, and the accumulated damage was well below 1 for all braces and top chord. For the bottom chord the damage was significantly reduced but still remained above 1 (D= 2). The bigger damage occurs in the high cycle fatigue region. Locally thicker flange plates and/or post weld treatment in this case is necessary to improve further the fatigue strength.

Concluding, alternative 2, with side thick plates, seems to provide an appealing solution for the truss design 3, in HSS. Extra welding and steel material will be needed to feasible this kind of solution and thus, again costs will more or less determine the final choice. However, modeling with FEM program using plate elements will of course results in more accurate conclusions.

8.7 Alternative truss hybrid design with CHS and cast joints

8.7.1 Benefits of CHS members

Circular hollow sections (CHS) may provide some benefits in comparison to rectangular hollow sections (RHS):

- Circular hollow sections (CHS) offer a pleasing shape, although, specialized profiling is needed when joining circular shapes together.

- For CHS a single longitudinal weld is required (in contrast, four welds for RHS members) reducing the overall weld volume.

- CHS members are standardized in rather large dimensions (e.g. maximum available is 660x50 for S355 steel grade) and can be ordered from stock, directly, thus costs are reduced significantly.

- Their (smooth) circular shape allows for more uniform stress distribution (i.e. no stiff corners).

- Fatigue detail category, according to EN 1993-1-9, for CHS lattice joints is higher than for RHS sections (i.e. fatigue class 90 and 71, respectively), leading to better fatigue performance.

- The design of connections (especially for fatigue) can be improved significantly by using cast joints. Cast joints are not available in case of RHS members.

- Slenderness ratios (D/t) are far more relaxed than in case of RHS (c/t), allowing for more slender cross sections. This is quite beneficial in case of HSS where due to higher yield strength (f_y), these ratios limits become even smaller.

- The overall cross sectional dimensions can be reduced.

8.7.2 Members and dimensions

A truss design with circular hollow section members (CHS) has been developed for the 'Schellingwouderbrug'. The overall bridge layout is not changed, but only the cross sectional shape and the type of joints.

More specifically for the chord members standard tubular members are used with dimensions 610x32. For all the braces again standard sections are chosen with dimensions 457x20. CHS truss members have been verified for strength and stability as in case of Truss 3 design with RHS members. For more information in cross sectional properties and for strength and stability checks reference is made to Appendix B.

The cross beams are again welded I –sections. Their dimensions are not altered to facilitate comparison and since they are sufficient for strength (U.C =0.91).

8.7.3 Cast design

For the connections casted joints are considered. They are readily available also for S690 steel grades. These joints can be properly designed to have adequate static and fatigue strength and optimal shape with the help of a FEM program. Butt welds can connect the cast steel to the member high strength steel. This is a relatively good fatigue detail and in addition as in the case of RHS with gusset plates the welds are positioned at locations of lower stresses and stress concentrations.

The static strength of the connection is ensured by making the cast joint at least as strong as the most heavily loaded member.

Fatigue strength can also be ensured by optimal and smooth shape of the casting and careful variations of local thicknesses. Thus, the locations which may be critical and need to be checked for fatigue are the butt welds at the connection of the casting to the truss members (similar to the case of RHS sections with gusset plates).

8.7.4 Costs

As the same brace member cross section is used all over the bridge repetition is allowed, which will require even a single mold for casted steel. This fact in comparison to standardized member sections will have a big influence on fabrication costs. However, plate thicknesses are somewhat bigger than in case of RHS design which may lead to increasing welding volume. On the other hand overall exposed area is smaller. Thus, maintenance cost reduction may be expected.

8.8 Truss design using S355 steel grade only

For sake of general comparison only, equivalent designs assuming steel grade S355 for all truss members (chords, braces) are considered in case of: 1) truss bridge with RHS members and gusset plates, and 2) truss bridge design with CHS members and cast joints.

Two sub-alternatives are considered for “all in S355” designs. Either a) increase only the plate thickness maintaining the other cross sectional dimensions (i.e. diameter, width, and height) constant or, b) increase the overall cross sectional dimensions keeping the plate thicknesses constant.

The scope of this is to estimate the possible benefits that a hybrid bridge construction may offer by comparing plate thicknesses, cross sectional dimensions and dead bridge weight for hybrid designs and designs in S355.

It is really important however, to make clear that this is only for obtaining a general feeling and not to compare absolute values. It would be more accurate to create a totally new design out of S355 steel grade (e.g. different truss height and/or field length, or even totally different bridge type etc.), and then compare the results. This is not possible, though, due to time limitations.

However, it is also reminded that in chapter 7 an arch bridge design out of S355 and a hybrid truss bridge using HSS S690 for the chord members (Truss 3) were compared leading to a choice for the hybrid truss bridge, mainly due to lower bridge weight.

As it is expected, the required dimensions for the chord members, in case S355 is to be used only, are almost double in comparison to S690 chord members (see also Table 9.2 and 9.4). The changes for the braces and cross beams are less apparent as they were already designed by assuming steel grade S355.

9 Total costs estimation

9.1 Final truss bridge designs

In this chapter four truss designs developed for the 'Schellingwouderbrug', are compared mainly on a costs basis by calculating material costs and estimating total costs. Similar unity check ratios are chosen for each member in different designs to make the comparison as accurate as possible.

The aim of this comparison is to conclude on the most cost effective design for the 'Schellingwouderbrug' and thus check if a hybrid construction provides more benefits.

These designs are:

a) 1. Hybrid truss design with rectangular hollow sections (RHS) members. Connections with high strength steel side (gusset) plates are assumed.

   2. An equivalent to the previous one, but "all in S355" truss design (with RHS).

b) 1. Hybrid truss design with circular hollow sections (CHS) members. Connections with high strength steel castings are assumed.

   2. An equivalent to the previous one, but "all in S355" truss design (with CHS).

It must be noted that the "all in S355" design in both cases (i.e. for RHS and CHS designs) are based on an estimation of required dimensions or plate thicknesses in order to obtain a structurally safe design for the reference bridge.

This is based only on strength and stability checks, while for fatigue they are expected to be more than sufficient due to significantly bigger cross sectional dimensions, and thus lower fatigue stress levels.

However, this only serves for an estimation of benefits that HSS can offer due to reduced dimensions and dead weight, when designing on the same basis (i.e. assume given bridge geometry, length, width, height, field length, brace angles etc.).

This does not, in any case, considered to lead to the most cost effective design made in S355 for the 'Schellingwouderbrug'.

9.2 Designs comparison on a cost basis

9.2.1 Material costs

By the term "material costs" it is meant that these are costs calculated based on bridge steel dead weight and include the self-weight of the steel members (i.e. chords, braces, cross beams) together with an extra 15% to account for welding consumables and extra steel material on the bridge (e.g. wind braces, stiffening plates, etc.). For all designs and for all truss members (braces, chords) and cross beams, the following have been determined and presented in tables for direct comparison:

- Cross sectional area A (mm^2)
- Minimum yield strength f_y (MPa) depending on the material used in each member
- Unity check ratios (U.C), especially for the truss members, are chosen as close as possible, for each member and at different designs, to make the comparison as accurate as possible.
- Steel members dead weight (tn) according to the formula:

$$\boxed{n_i*(A_i* 10^{-6}* \gamma* L_i)/10}$$

Where,
n_i: total number of member i on the bridge
A_i: cross sectional area (mm^2) of member i
γ: steel density (= 78.5 kN/m^3)
L_i: length (m) of member i

- Maximum plate thickness t (mm) per member (has important influence on welding volume required)
- Required painting area (mm^2) –assuming, conservatively, that the whole external surface of the member needs to be painted-, is calculated based on the formulas:

For RHS sections: $\boxed{ni* (2*(h_i +b_i))*L_i}$

For CHS sections: $\boxed{n_i*2*\pi*(D_i/2)*L_i}$

For cross beams: $\boxed{n_i*(2*b_i+4*t_f+(h_i-2*t_f))*L_i}$

Where,
n_i: total number of member i on the bridge
h_i: height (mm) of member i
b_i: width (mm) of member i
D_i: diameter (mm) of member i
L_i: length (mm) of member i
t_f: flange thickness (mm) of cross beam i

The painting area gives an indication on corrosion protection costs needed to satisfy maintenance requirements.

- In the tables, in order to determine material costs, the gusset plates as well as the cast joints in the CHS design are accounted for assuming an additional 10-15 % steel dead weight increment in the total bridge steel dead weight.

Although, the exact dimensions of the gusset plates are not known, their minimum required plate thickness (based on the most heavily loaded members and assuming, conservatively, constant gusset plate thickness in all joints) has been calculated according to the formulas given below (see also Figures 8.8 and 8.9):

- At the connections to the chord members: min $t_{plate,\ gusset} = (N_0/2)/\ (f_{y,\ gplate} * h_0)$,
  and $A_{plate,\ gusset} = h_0 * t_{plate,\ gusset}$, per plate.

- At the connections to the brace members: min $t_{plate,\ gusset} = (N_i/2)/\ (f_{y,\ gplate} * b_i)$,
  and $A_{plate,\ gusset} = b_0 * t_{plate,\ gusset}$, per plate.

Two gusset plates, acting locally as members web plates are assumed in each joint.

In the hybrid designs high strength steel (i.e. S690) is assumed for the gusset plates, as used for the chord members, also. In the equivalent “all in S355” truss designs with RHS members, S355 is of course also assumed for the gusset plates.

Considering the maximum normal forces in the members, similar minimum plate thicknesses as for the gusset plates, are expected to be required for the castings in design with CHS members at the ends of the casting plates. In this case, however, the thickness will more likely vary, being thicker in the middle of the casting than at the ends, for more optimum structural and fatigue behavior of the joint. Thus the actual thickness(es) can only be determined using a finite element model.

Finally, the self-weight of the concrete deck (normal weight concrete deck of 240 mm thickness is assumed for all bridge designs) is not added directly to the calculated value of the total bridge (steel) dead weight (based on steel members, welds, stiffening plates, etc.), as it does not influence the difference between the resulting values (see Tables 9.1-9.6) for costs comparison.

However, the concrete deck has an influence on the value of the final bridge dead weight in the sense that it has been applied as a load on the bridge (i.e. directly on the cross beams). Therefore, it has been taken into account when calculating the required steel members dimensions and thus their self-weight.

- Bridge steel material costs (i.e. costs calculated for the dead weight of the main steel structure- self weight of steel members plus an extra 15% for connections and additional steel-) are calculated assuming that currently, these material values (for base steel material and welding consumables) are available:

S355: 1.00 €/ kg (1.00*10^3€/ton)
S460: 1.30 €/ kg (1.30*10^3€/ton)
S690: 1.75 €/ kg (1.75*10^3€/ton)

Table 9.1 Hybrid truss design with RHS members and additional gusset plates at the connections

Hybrid design with RHS members and gusset plates								
Member	Dimensions (mm)	A (mm²)	min fy (MPa)	U.C	weight (tn)	classification	max t_{plate} (*) (mm)	$A_{paint, req}$ (mm²)
Top chord	500x550x30	59400	690	0,95	98	class 1	30	4,41E+08
Bottom chord	500x550, t_f=30,t_w= 20	49600	690	0,88	82	class 1	30	4,41E+08
End braces	500x400x16	27776	460	0,89	8	class 3	16	6,30E+07
Rest braces	500x400x16	27776	355	0,86	69	class 2	16	5,67E+08
I-cross beams	B=300,H=550,t_f=20,t_w=15	19650	355	0,62	91	class 1	20	6,98E+08
				Total(**)	**347**		Total	**2,21E+09**
Costs (10³ Euros)		steel members only			484			
		with welds and gusset plates			575			

(*) For gusset plates a minimum required plate thickness is estimated (per plate) at the connections with the truss members. For each gusset plate the min thickness is 15 mm and 32 mm for connections with the braces and the chords, respectively. Thus, for each gusset plate the min thickness is 32 mm assuming the same thickness for the whole plate.
(**) An additional 15 % of this value should be considered to account for gusset plates, additional steel (e.g. wind bracings) and welds. For this value, high strength steel grade S690 is conservatively assumed for material cost estimation.

Table 9.2 Equivalent "all in S355" truss design with RHS members and gusset plates at the connections

All in S355 truss design with RHS members and gusset plates								
a) Only member(s) plate thickness is increased								
Member	Dimensions (mm)	A (mm²)	min fy (MPa)	U.C	weight (tn)	classification	max t_{plate} (*) (mm)	$A_{paint, req}$ (mm²)
Top chord	500x550x60	111600	355	0,98	184	class 1	60	4,41E+08
Bottom chord	500x550x60	111600	355	0,87	184	class 1	60	4,41E+08
End braces	500x400x20	34400	355	0,91	9	class 2	20	6,30E+07
Rest braces	500x400x18	31104	355	0,81	77	class 2	18	5,67E+08
I-cross beams	B=300,H=550,t_f=30,t_w=15	25350	355	0,71	117	class 1	30	6,98E+08
				Total(**)	**571**		Total	**2,21E+09**
Costs (10³ Euros)		steel members only			571			
		with welds and gusset plates			657			
b) Other cross sectional dimensions (height, width) are increased								
Member	Dimensions (mm)	A (mm²)	min fy (MPa)	U.C	weight (tn)	classification	max t_{plate} (mm)	$A_{paint, req}$ (mm²)
Top chord	900x800x30	98400	355	0,92	162	class 2	30	7,14E+08
Bottom chord	900x900x30	95000	355	0,88	157	class 1	30	7,56E+08
End braces	500x400x19	32380	355	0,85	9	class 2	16	7,00E+07
Rest braces	500x400x16	27776	355	0,83	69	class 2	16	5,67E+08
I-cross beams	B=300,H=700,t_f=20,t_w=15	21900	355	0,88	101	class 1	20	7,86E+08
				Total(**)	**497**		Total	**2,89E+09**
Costs (10³ Euros)		steel members only			497			
		with welds and gusset plates			572			

(*) For gusset plates a minimum required plate thickness is estimated (per plate) at the connections with the truss members. Each gusset plate has minimum required thickness 30 mm and 60 mm for connections with the braces and the chords, respectively. Thus, for each gusset plate the min thickness is 60 mm assuming the same thickness for the whole plate.
(**) An additional 15 % of this value should be considered to account for gusset plates, additional steel (e.g. wind bracings) and welding material. For this value, normal steel grade S355 is assumed for material cost estimation

Table 9.3 Hybrid truss design with (standardized) CHS members and cast joints

Hybrid design with CHS members and cast joints								
Member	Dimensions (mm)	A (mm²)	min fy (MPa)	U.C	weight (tn)	classification	max t_{plate} (*) (mm)	$A_{paint, req}$ (mm²)
Top chord	610x32	58095	690	0,97	96	class 2	32	4,02E+08
Bottom chord	610x32	58095	690	0,81	96	class 2	32	4,02E+08
End braces	457x20	27452	460	0,92	8	class 2	20	5,02E+07
Rest braces	457x20	27452	355	0,85	68	class 2	20	4,52E+08
I-cross beams	B=300,H=610,t_f=25,t_w=15	23400	355	0,76	108		25	7,34E+08
				Total (**)	**375**		Total	**2,04E+09**
Costs (10³ Euros)		steel members only			**521**			
		with welds and castings			**619**			

(*) For casting, the plate thickness may vary. The minimum cast thickness cannot be lower than 20 and 32 mm at the connections with the brace and chords respectively.
(**) An additional 15 % of this value should be considered to account for cast joints, additional steel (e.g. wind bracings) and welds. For this value, high strength steel grade S690 is conservatively assumed for material cost estimation.

Table 9.4 Equivalent "all in S355" truss design with CHS members and cast joints

All in S355 truss design with CHS members and cast joints								
a) Only member(s) plate thickness is increased								
Member	Dimensions (mm)	A (mm²)	min fy (MPa)	U.C	weight (tn)	classification	max t_{plate} (*) (mm)	$A_{paint, req}$ (mm²)
Top chord	610x65	111600	355	0,97	184	class 1	65	4,02E+08
Bottom chord	610x65	111600	355	0,91	184	class 1	65	4,02E+08
End braces	457x25	34400	355	0,97	9	class 2	25	5,02E+07
Rest braces	457x20	31104	355	0,91	77	class 2	20	4,52E+08
I-cross beams	B=300,H=610,t_f=25,t_w=15	23400	355	0,71	108	class 1	25	7,39E+08
				Total (**)	**562**		Total	**2,05E+09**
Costs (10³ Euros)		steel members only			**562**			
		with welds and castings			**646**			
b) Other cross sectional dimensions (diameter,cross beam height) are increased								
Member	Dimensions (mm)	A (mm²)	min fy (MPa)	U.C	weight (tn)	classification	max t_{plate} (mm)	$A_{paint, req}$ (mm²)
Top chord	1000x32	97294	355	0,94	160	class 2	32	6,60E+08
Bottom chord	1100x32	107350	355	0,89	177	class 1	32	7,26E+08
End braces	559x20	33900	355	0,85	9	class 2	20	6,15E+07
Rest braces	457x20	27452	355	0,81	68	class 2	20	4,52E+08
I-cross beams	B=300,H=610,t_f=25,t_w=15	23400	355	0,77	108	class 1	25	7,39E+08
				Total (**)	**522**		Total	**2,64E+09**
Costs (10³ Euros)		steel members only			**522**			
		with welds and castings			**601**			

(*) For casting, the plate thickness may vary. The minimum cast thickness cannot be lower than 25 and 65 mm at the connections with the brace and chords respectively.
(**) An additional 15 % of this value should be considered to account for cast joints, additional steel (e.g. wind bracings) and welds. For this value, normal steel grade S355 is assumed for material cost estimation.

9.2.2 Fabrication costs

For S355, 1.50 €/kg should be in general accounted for in total costs for fabrication (preheating, welding, cutting, etc.). Considering that at small thicknesses (20-30 mm, as for these designs) there is no difference in fabrication requirements between S355 and S690 steel grade, then 1.50 €/kg can roughly be considered also in case of S690 steel.

In the design case scenario “all in S355” a), thicknesses are double as much as in case of the hybrid design. This will more likely increase the welding volume very much.

Of course the welding consumables will be more expensive in case overmatched welds are used for plates of S690 steel grades, but this has already been taken into account in material costs estimation (by accounting an extra 15% for additional steel and welding).

Furthermore corrosion protection is essential for steel bridges and influences both fabrication and future maintenance costs. For the ‘Schellingwouderbrug’, paint is considered as the chosen method for corrosion protection. An indication on costs of 3-layers painting system is 65 €/m^2 independently of the steel grade. It is obvious therefore, that bigger cross sectional dimensions lead to higher painting requirements, adding to the total costs.
In Tables 9.1-9.4 CHS hybrid design and its equivalent “all in S355 design” case (a) result in the lower painting area required and according to Table 9.5 also in lower costs for painting.

Table 9.5 Costs for fabrication (1.50 €/kg) based on bridge weight and corrosion protection (65 €/m^2)

Fabrication costs				
Truss design	**Steel weight (tn)**	**Costs (10^3 €)**	**Hybrid benefits**	**Diff. between hybrids**
RHS hybrid	347	521	43%	8% more for CHS
RHS S355 (*)	497	746		
CHS hybrid	375	563	39%	
CHS S355 (*)	522	783		
Corrosion protection costs				
Truss design	**$A_{paint,req}$ (m^2)**	**Costs (10^3 €)**	**Hybrid benefits**	**Diff. between hybrids**
RHS hybrid	2,21E+03	144	31%	8% more for RHS
RHS S355 (*)	2,89E+03	188		
CHS hybrid	2,05E+03	133	29%	
CHS S355 (*)	2,64E+03	172		

(*) Equivalent homogeneous truss design only with S355 steel grade and increased cross sectional dimensions while keeping the same plate thickness as in hybrid truss designs-case (b) in Tables 9.2 and 9.4.

9.2.3 Transportation costs

In general, transportation costs are influenced by the method (water, land), vehicle type and capacity and delivery schedule and timetables. Transportation through the land or through the sea is possible for the 'Schellingwouderbrug'. Due to the bridge dimensions (geometry) and its location, the whole bridge is possible to be completely prefabricated and then transferred as one piece to the site, on special boats. This is the same for all designs as the bridge geometry remains unaltered. However, costs for handling and lifting at the fabrication shop may be less for the hybrid designs due to lower weight.

9.2.4 Erection costs

Erection method influences the lifting and handling equipment necessary (e.g. types and number of cranes). This is of course directly related to the weight of the components or the whole bridge (if it is going to be lifted in one piece) to be lifted and their cross sectional dimensions.

As an indication, the cost for cranes are 10000 €/crane/day. Of course in case cranes with smaller lifting capacity is needed due to smaller bridge weight the price will be much lower.

The lifting capacity depends on the type of crane and if (and how much) they are inclined during lifting operation. Generally, floating cranes have larger lifting capacity than land cranes as they can be built bigger and allow for a bigger counterweight at the back.

For the 'Schellingwouderbrug' more likely 2 floating cranes will be necessary per day for one to two days for all designs. However, their lifting capacity (and thus costs) depends on the bridge dead weight. Thus, it can be expected that both hybrid designs are more economical due to significant differences in dead weight in comparison to 'all in S355' designs.

9.2.5 Maintenance costs

Inspection, maintenance and possible repairs have an important share in total costs during the lifetime of a bridge. Corrosion protection is already being considered in section 9.2.2 to be in favor of the hybrid designs due to smaller cross sectional dimensions.

In addition if a more expensive and of higher quality paint is applied, this may reduce long term maintenance (and total) costs significantly thus it should be considered as an interesting alternative.

9.3 Results interpretation

Tables 9.6 and 9.7 summarize the results of steel material costs due to differences in bridge steel weight and presented previously in tables 9.1-9.5. With red color the lower values between different truss design alternatives are indicated.

Table 9.6 Summary for truss bridge designs (for steel weight and material costs only)

#	Truss bridge design	Steel weight (tn)		Material costs (10^3 €)	
		Members only	Additional steel included	Members only	Additional steel included
1)	RHS- hybrid	347	399	484	575
1a) (*)	RHS- S355	571	657	571	657
1b)(**)	RHS- S355	497	572	497	572
2)	CHS- hybrid	375	431	521	619
2a) (*)	RHS- S355	562	646	562	646
2b)(**)	RHS- S355	522	600	522	600

Table 9.7 Summary for truss bridge designs where corrosion protection costs as calculated in Table 9.5 are also included

#	Truss bridge design	Steel weight (tn)		Material and corrosion protection costs (10^3 €)	
		Members only	Additional steel included	Members only	Additional steel included
1)	RHS- hybrid	347	399	628	719
1a) (*)	RHS- S355	571	657	715	801
1b)(**)	RHS- S355	497	572	685	760
2)	CHS- hybrid	375	431	654	752
2a) (*)	RHS- S355	562	646	695	779
2b)(**)	RHS- S355	522	600	694	772

(*) Equivalent homogeneous truss design only with S355 steel grade and increased plate thickness while keeping the same the rest of the cross sectional dimensions as in hybrid truss designs-case (a) in Tables 9.2 and 9.4.

(**) Equivalent homogeneous truss design only with S355 steel grade and increased cross sectional dimensions while keeping the same plate thickness as in hybrid truss designs-case (b) in Tables 9.2 and 9.4.

Three main criteria/parameters are considered for design comparison:

- Steel main structure dead weight (calculated)
- Steel material costs (calculated)
- Total costs (estimation)

Based on the above criteria (parameters), Tables 9.1-9.6 show directly the following:

- ✓ Hybrid design with RHS members and gusset plates has the lowest dead weight (≈8% less than CHS hybrid and up to ≈65% less than S355 equivalent -case (a)).

- ✓ Hybrid design with RHS members and gusset plates together with its equivalent "all in S355" design (case b)) have the lower steel material costs in Table 9.6 (= 575000 € and 572000 €, respectively, taking into account the 15% increment for welds and additional steel plates for connections, ≈ 8% more than CHS hybrid design). Between RHS hybrid and RHS homogeneous (case b) there is only a very small difference (0.5%) in favor of homogeneous design due to lower price of S355.

- ✓ Hybrid design with CHS members and cast joints together with its equivalent "all in S355" design (case a)) have the lower required painted area (≈ 8% more than RHS hybrid design in Table 9.5).

- ✓ However, RHS hybrid design with gusset plates shows the lower "material & corrosion protection" costs (≈ 5% less than CHS hybrid design in Table 9.7) showing again the role that dead weight plays in calculating total costs.

9.3.1 Hybrid designs vs. "all in S355" designs

Moreover, Tables 9.1-9.7 show the following:
In Table 9.2 (case (a)) the plate thickness for the chord members needs to be increased by a factor of 2, to more than 60 mm. The same holds for the thickness of the gusset plates (RHS design) and casted joints (CHS design). This naturally leads to more steel material.
Higher thickness will also lead to significantly bigger weld volumes and maybe special concerns with respect to welding procedure (e.g. higher preheating temperatures) and poorer through thickness properties of the material may occur. In addition the plate thickness exceeds 40 mm thus the min yield strength is not 355 MPa but 335 MPa. The steel dead weight in this case (including additional steel), increases at about 65% for RHS design (Tables 9.2 b) and 9.6) and 39% for CHS design (Table 9.4 b)) in comparison to the hybrid designs (Tables 9.1 and 9.3).
Material costs alone, based on steel dead weight including additional steel and welds (see also table 9.6) are 14% more for the RHS S355 design and 4% more expensive for the CHS S355 design in comparison to their equivalent RHS and CHS hybrid designs, respectively.
If also corrosion protection cost is added (table 9.7) the calculated costs are now 11% more for the RHS S355 design and ≈ 4% more expensive for the CHS S355 design in comparison to their equivalent RHS and CHS hybrid designs, respectively.

In table 9.2 (case (b)) the cross sectional dimensions need to be increased almost by a factor 2 to resist the loading. This will lead to longer welds, larger perimeter and thus, larger required painted area, plus possibly more difficult handling (associated to labor costs). The bridge dead weight (including additional steel) in this case increases to about 43% for RHS design (Tables 9.2 (a) and 9.6) and 50% for CHS design (Tables 9.4 (a) and 9.6), in comparison to the hybrid designs (Tables 9.1, 9.3 and 9.6). In addition the painting required area is increased 31% and 29% respectively (Table 9.5).
In this case however, material costs alone, based on steel dead weight including additional steel and welds (see also table 9.6) are 4% less for the RHS S355 design and 3% less

expensive for the CHS S355 design in comparison to their equivalent RHS and CHS hybrid designs, respectively.
If also corrosion protection cost is added (table 9.7) the calculated costs are now 6% more for the RHS S355 design and ≈ 4% more expensive for the CHS S355 design in comparison to their equivalent RHS and CHS hybrid designs, respectively.

Therefore, in terms of total costs the hybrid designs seem to offer a more cost effective solution than their equivalent "all in S355" truss bridge designs for the 'Schellingwouderbrug' (Tables 9.5-9.7), despite the higher price of high strength steel material.

This is obviously due to significant difference in bridge steel dead weight (was calculated to reach up to 65%!). Therefore, likewise to the corrosion protection and especially if smaller cross sectional dimensions (width, height) are considered it can be expected that also other important costs (e.g. in transportation, erection, welding volumes, maintenance requirements etc.) will be in favor of hybrid designs.

However, this conclusion is based on the current steel prices and the fact that the same design is used for both materials (equivalent design solutions -same design concept- has been considered for trusses with S355 alone and with S690 in combination with S355).

9.3.2 Hybrid RHS design vs. hybrid CHS design

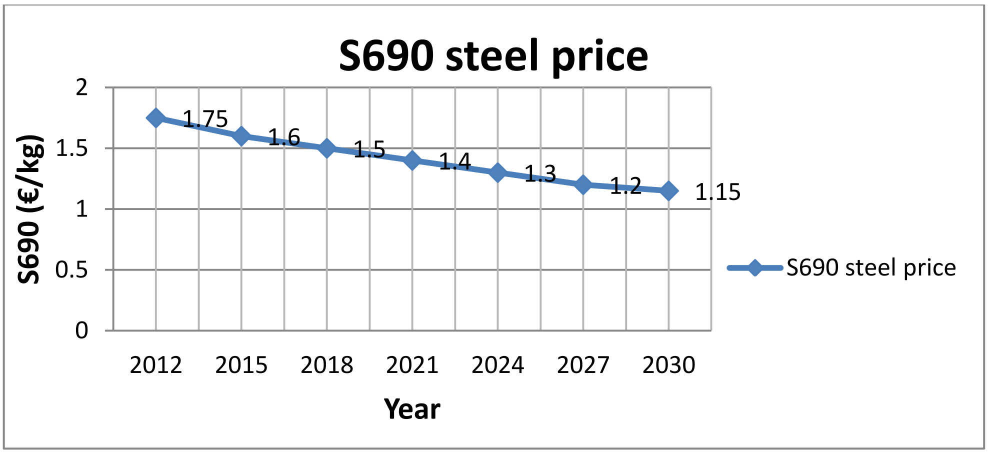
From the comparison between hybrid designs Tables 9.1, 9.3, and 9.5-9.7 seems that the hybrid design with RHS members and gusset plates, provides lower dead weight (8%), lower material costs (≈ 8% accounting only for additional steel for connections and welds and ≈ 5% if also corrosion protection costs are included, Tables 9.5-9.7), smaller maximum member thicknesses, but also slightly bigger painting required area (8%) in comparison to the hybrid design with CHS members and cast joints.

However, RHS members are welded from plate elements, while CHS members can be found available in stock. Furthermore, the cast joints can be optimized with respect to the required thickness and particular shape in specific locations (e.g. bigger cast thickness close to the center of the joint). These considerations will more likely outweigh the small differences between the two designs in favor of the CHS hybrid design.

9.4 Future trends on price of high strength steel grade S690

The price of high strength steel material is currently quite high in comparison to normal S355 steel grade (70-75% more expensive). However, this fact is expected to change in the future as the market demand for new steel grades will become higher.

A hypothetical case scenario for S690 price reduction per kg of material in the next 10-20 years is presented in Graph 9-1. The price of S355 and S460 is assumed constant to 1.00 €/kg and 1.30 €/kg, respectively.

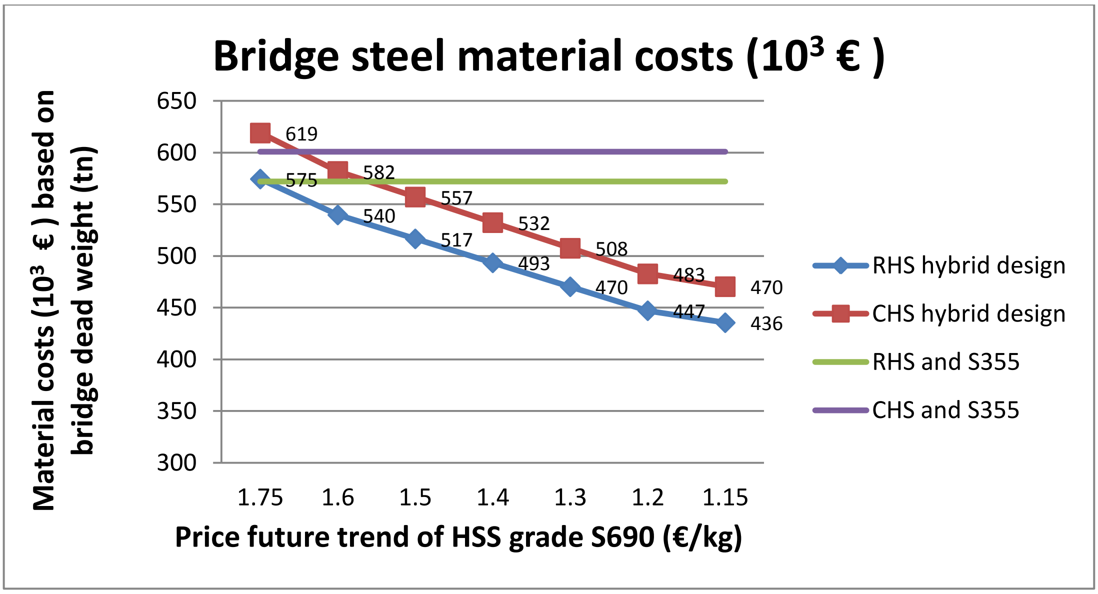


Graph 9-1 Future trend assumption for S690 steel price (€/ kg)

Based on this trend for S690 steel price in the next years, the bridge material costs (based on steel dead weight including extra steel and welds) for designs presented in Tables 9.1-9.4 are estimated and presented in Graph 9-2. Actually, a reduction of 20% in the next 20 years not only is realistic but could also be considered conservative. As it is expected, material costs (10^3 €) show a significant reduction due to lower high strength steel price.

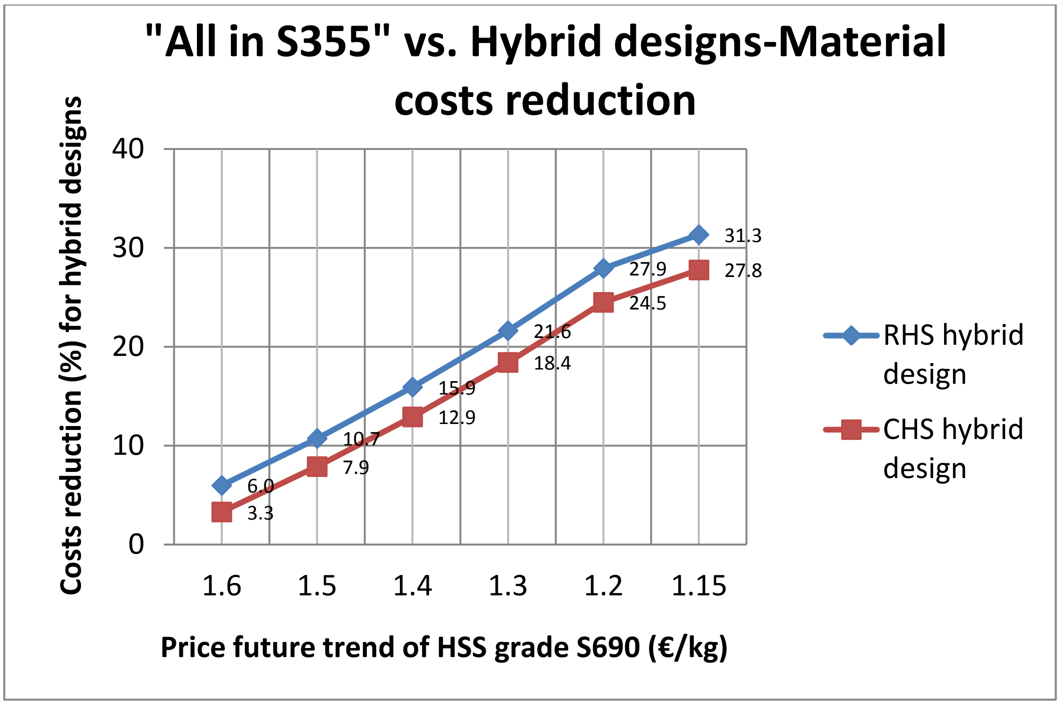
The bridge steel material costs for "all in S355" designs are presented as straight lines since the price of S355 is assumed to remain constant over the years. This serves in understanding better the cost benefits that can be gained with HSS steel grades due to weight reduction.

Similar trend can be expected for the total costs of the hybrid bridge designs, when the plate thicknesses are kept relatively low (up to 30 mm).



Graph 9-2 Material costs estimation for hybrid and all in S355 truss bridge designs assuming price reduction for S690 in comparison to S355 steel grade.

Also, in Graph 9-3 the cost reduction for the hybrid designs in comparison to "all in S355" designs (case (b)) expressed in percentage (%) is presented also as a function of the S690 future steel price reduction. It is obvious that the benefits to be gained (due to weight savings) from high strength steel material become larger as the price of the material drops.



Graph 9-3 Cost reductions (%) for hybrid designs in comparison to their equivalent "all in S355" designs (case (b): with increased cross sectional dimensions keeping the same plate thickness)

Note: In Graph 9-3, the S690 steel price starts from 1.60 €/kg and not from 1.75 €/kg (current price) as in the two previous graphs (Graph 9-1 and 9-2) because in this case the cost reduction is in favor of "all in S355"designs (i.e. cost reduction is 4% for RHS and 3% for CHS "all in S355" case (b) designs in comparison to their equivalent hybrid designs). Thus it was preferred not to include a negative value in the graph (sign '-' reflects the hybrid bridge material costs increment in this case).

It must be noted that the estimated future prices for S690 presented in Graph 9-1 cannot be considered accurate as they only serve for comparison of how the material price influences the results calculated taken as reference the 'Schellingwouderbrug'.

However, in the next years more and more experience will be gained with HSS and the cost benefits that can be gained from less steel weight (i.e. less steel material) which will also lead to higher market demands for these higher steel grades. Furthermore, more fabricators will become familiar and work with the new material and thus the price will drop.

Based on these factors, a reduction of about 15% in price of S690 steel grade in the next 20 years can not only be a realistic but even a conservative expectation.

10 Conclusions

The critical question to be answered is: Can high strength steel help reducing total costs in bridge construction?

In this thesis a literature survey together with a case study have been presented. The case study consists of preliminary hybrid -using high strength steel grade S690 (min f_y = 690 MPa) in combination with S460 (min f_y = 460 MPa) and S355 (min f_y = 355 MPa) steel grades- but also homogeneous bridge designs (the whole design in S355 steel grade) for a long span bridge (L > 100m).

From this MSc study the following can be concluded:

1. In Chapter 3 it has been shown that bridges using high strength steel grades (HSS up to S690 and usually in combination with lower steel grades-hybrid designs) can offer competitive and cost effective solutions for almost all bridge types (i.e. truss bridge, girder bridge, box girder bridge, cable stayed bridge and suspension bridge) and span length ranges (i.e. small, medium, and large), resulting mainly in significant weight savings.

2. The choice for a certain bridge type at a given location with its specific boundary conditions influences whether HSS will be favorable or not.

3. Application of high strength steel grades (mainly as quenched and tempered Q&T quality) in (hybrid) bridge design results in large weight savings (e.g. S690 steel grade in hybrid (warren) truss design can result in over 50% steel weight reduction in comparison to an equivalent homogeneous design with mild steel) especially in cases where strength governs.

4. In bridge types and in members where the governing criterion is strength, such as in truss bridges (i.e. members under tension or even compression if buckling behavior is not decisive) the use of HSS, with associated cost benefits, will be favored. Certain design improvements/changes especially for the connections (e.g. choosing for cast joints in truss bridges instead of direct member to member welded connections for better fatigue behavior), if necessary, should be considered in an early design stage.

5. Fatigue is commonly the governing criterion in steel (highway and railway) bridges especially for members under bending as in the main girders in arch bridges and in connections. This bridge type will likely not be favorable for HSS.

6. Increased bridge dead weight (for example larger bridge spans) for a given traffic load (i.e. designing for a given traffic category within a certain fatigue life) is in favor of higher steel grades as the static stresses will be higher while fatigue stress amplitude will be smaller due to the increased mass.

7. To improve fatigue behavior and allow economical use of HSS a concrete deck (in composite action or not) can be preferred over an orthotropic steel deck. The concrete deck adds to the bridge mass (and thus static stresses in the steel members are increased) and it is has less fatigue sensitive details than orthotropic steel deck. If also

in composite action, the steel members can be further reduced, thus less steel material, less steel weight and finally even less total costs (costs savings in fabrication, handling, lifting, transportation, corrosion protection, foundations etc.).

8. Considering equivalent homogeneous and hybrid bridge designs (and if the same deck type is considered, e.g. concrete) it is estimated that lower steel self-weight that can be achieved with HSS, will have an influence on foundations (e.g. smaller piers may be required especially for large spans), transportation, lifting and erection costs, while smaller cross sectional areas will have a positive effect on maintenance (e.g. smaller painted area required for corrosion protection) and fabrication (especially welding) costs, especially in small thicknesses.

9. Based on preliminary designs of a long span bridge, it has been shown that high strength steel grade S690 can provide cost effective and thus competitive bridge solutions, especially when it is used in combination with lower steel grades (in hybrid design) and in structures/members where the load is mainly transferred as axial forces like in truss bridges where strength is usually the governing criterion. In members under bending fatigue stresses at certain locations (e.g. truss joints) may be significant.

10. Nevertheless, in cases where fatigue stresses are locally quite high (e.g. fatigue sensitive details in critical joints) alternative design concepts (e.g. different joint configuration and/or locally higher plate thicknesses, moving welds away from locations of high stresses, etc.) or even post weld treatment can improve the fatigue sensitive details.

Additional conclusions based on a literature survey have been presented in Chapter 4.

Case study: 'Schellingwouderbrug'

Moreover, based specifically on the case study for the Schellingwouderbrug the following can be concluded and used as feedback for future bridge (hybrid) designs with S690 steel grade.

For members verification

- The design rules (according to EN 1993-1-1 and EN 1993-1-12) and methods of global elastic analysis for normal steel grades can be generally used for design of high strength steel (HSS) members up to S700 steel grade too.

- Hybrid designs with high strength steel allows for significant bridge steel weight reduction –up to 65% for RHS design and 50% for CHS design- in comparison to their equivalent homogeneous truss designs due to smaller plate thicknesses and/or overall cross sectional dimensions.

- The above statement implies that in order to be favored from the benefits of less steel material, the overall design philosophy (e.g. choice of bridge type, L/D ratios, detailing of connections with respect to fatigue), can be altered or adjusted to the steel material properties and vice versa. In that respect hybrid designs (combination of steel grades for different bridge regions and/or for steel members) give more weight savings and (material and total) cost benefits than homogeneous designs.

- Stability –in and out- of plane does influence the design if relatively slender and/or long members (large buckling lengths) are used. In this case static stresses are limited by the buckling strength ($f_{b, rd}$) which governs the design. This is mainly because:

- the value of flexural buckling reduction factor χ reduces as the steel grade increases (considering the same buckling curve), resulting in bigger differences between static and buckling strength values. However, the resulting buckling strength is still higher for members with high strength steel in comparison to members with mild steel grades;

- the higher the steel grade the lower the slenderness (c/t) limits (Table 5.2 EN 1993-1-1) for cross sectional classification. In result, the same cross section can be classified as class 2 for S355 and class 3 (or even class 4) for S690. This becomes more pronounced in members under pure compression where the limits are already stricter than for members in bending;

- Nevertheless, the same cross section with S690 classified for example as class 3 will result in higher member resistance against buckling (i.e. higher buckling strength) than a class 2 section made out of mild steel grades.

- Thus, high strength steel grades can still provide cost benefits for members under compression.

- Moreover, the plate thickness should be kept as low as possible to avoid extra fabrication costs (e.g. for plate thicknesses over 30 mm expensive preheating and high welding volumes will increase fabrication costs).

- If stability is the governing criterion, smaller member length (smaller field length, lower bridge height, bigger inclination for diagonal brace members for truss bridges, etc.) may be proved favorable in terms of total costs despite the more joints and the higher number of brace members that will add to fabrication costs.

- Fatigue stresses caused by bending actions are found to be 2-3 times higher than fatigue stresses due to axial forces, independent of the steel grade.

- Fatigue stresses were taken into account for the choice for the members cross section and for using HSS, and for most trial designs fatigue was not the decisive criterion. Thus, benefits due to weight savings and reduced dimensions could be gained.

- For the arch bridge design though, fatigue stresses limit the static stresses also at such levels where it was considered inefficient to use HSS grades. Thus, specifically for the ‘Schellingwouderbrug’ an arch bridge (with the given configuration) cannot provide any benefit in comparison to mild steel grade S355.

For connections verification

In this study, design of connections has only been investigated for (welded) rectangular hollow section (RHS) steel members in a Warren type truss bridge configuration for the 'Schellingwouderbrug'.

- Ensuring that the design of truss joints is within certain limits covered in section 7 in EN 1993-1-8, design resistances for lattice girder connections with RHS members can be calculated using tables and formulas provided in Eurocode 3 part 1-8 "Design of Joints". These, apply to sections at least class 2. However, a class 3 section in HSS could give more weight savings and thus costs benefits. In this case local plate buckling should be also checked.

- For strength verifications using tables in EN 1993-1-8, a reduction factor 0.9 and 0.8 should be applied to all (conservatively) the calculated design resistances for S460 and for higher (e.g. S690) steel grades, respectively, to account for larger deformations in the face of the RHS chord member.

- For fatigue verifications tables in EN 1993-1-9 for lattice girder joints and related S-N curves for nominal stress ranges, apply only to a quite small range of cross sectional dimensions, independently of the steel grade.

- FEM modeling of the connection is advised to be used for determination of actual hot spot (geometrical) stress ranges and actual stress concentration factors (SCFs).

- SCFs parametric formulas are also available (e.g. in CIDECT Design Guide No.8) but only for certain types of connections and for a range of cross sectional dimensions.

- Typical lattice girder joints with RHS members for the 'Schellingwouderbrug' were found to be unsuitable because of fatigue. High stress ranges together with high SCFs (from parametric formulas) resulted in unacceptable fatigue damage (D>> 1). Finite element modeling with plate elements is however necessary for calculating the hot spot stress ranges and the actual fatigue damage.

- For fatigue verifications a more favorable traffic category (i.e. N_{obs}/year/slow lane = $0.5*10^6$ from Table 4.5 EN 1991-2:2003), than for most highway bridge designs, has been considered based on the specific bridge location and the flow rates of lorries expected to cross it. Thus, fatigue damage can be expected to be even higher in case N_{obs}/year/slow lane = $2*10^6$ are to be considered for a given bridge design. If the choice for higher traffic category is done in combination with larger span length, thus higher bridge mass the fatigue effect will be less severe (i.e. reduced fatigue stress amplitude).

- Fatigue sensitive details can be improved by altering the design of the connections. In section 8.6 the alternative design using relatively thick gusset plates for members webs at joints location resulted in significant improvement with respect to fatigue. Damage calculation is satisfied almost for all critical joints. Here again detailed modeling of the connection with plate elements is necessary for actual fatigue assessment.

- For the design with circular hollow section (CHS) members cast joints are assumed for the connections. This is already a favorable design for fatigue based on literature findings. Once again, detailed modeling of the connection with plate elements is necessary to determine the actual shape of the casting and for accurate fatigue assessment.

For costs

- Hybrid designs show significant weight reduction (even 65% for truss with RHS members) in comparison to their equivalents "all in S355" designs.

- Calculating material costs for two hybrid truss designs and assuming that S690 is 75% more expensive than S355 it has been shown that hybrid construction shows only slightly higher material costs (4% higher with RHS members and 3% more with CHS members) in comparison to homogeneous (S355) designs.

- It was eventually not possible to calculate the total costs in detail (including fabrication, erection, transportation and maintenance). However, it is expected to be in favor of hybrid bridge designs. This is based on the estimation that, lower dead weight will have an influence on foundations (e.g. smaller piers may be required), transportation and erection costs, while smaller cross sectional areas will have a positive effect on maintenance (smaller painted area required especially for hybrid design with CHS members) and fabrication (especially welding) costs, especially in small thicknesses (thicknesses are kept relatively low- max t =30 mm for RHS hybrid design and 32 mm for CHS hybrid design- thus no preheating and no special machining equipment is necessary).

- Estimating finally that the price for HSS grade S690 will be reduced within the next decades it has been shown that the cost benefits to be gained from weight savings increase significantly as the high strength steel price reduces.

- For example, a 15% price reduction for S690 steel grade results up to 10% lower steel material costs for the 'Schellingwouderbrug' (see also Charts 9.1- 9.3).

11 Recommendations for further research

The following subjects are suggested for further investigation, as it was unfortunately not possible to cover them in this thesis project.

- Homogeneous (made completely out of a single steel grade) hollow section members have only been examined as the cross sections of bridge members. However, bridge solutions with other homogeneous but also hybrid (e.g. HSS in the flanges and lower grades for the webs) cross sectional types (e.g. built up sections, I girders, etc.) can be examined also and compared with the current designs in order to investigate the efficiency of HSS hollow section members.

- For the truss design of the 'Schellingwouderbrug' the larger available economical L/D ratio (= 15) with respect to S355 steel grades is considered for the hybrid designs as well. However most economical L/D ratios for HSS can be even bigger than normal steel grades. Therefore, further investigation is suggested together with truss optimization with respect to L/D ratio which may increase even more the cost benefits to be gained from higher steel grades.

- Design of connections is one of the most critical aspects especially when high strength steel is used as it directly influences the design of members and fabrication costs. More data from large scale test specimens for connections and even with more slender (higher class 3 or even 4) steel members are necessary in order to develop new simplified formulas for these steels.

- In addition, the estimated 0.8 reduction factor specifically for the design resistance in case of hollow section members made in high strength steel (> S460) needs to be verified by additional testing.

- Detailed fatigue design and determination of stress concentration factors (SCFs) for lattice girder connections with RHS members and gusset plates (section 8.6 alternative 2 for connections) using FEM analysis program and modeling the connection in detail is highly recommended.

- When hand (analytical) calculations are performed for fatigue verifications (as in the detailed design phase for the 'Schellingwouderbrug' with RHS members and max plate thickness 30 mm) it is possible that a relevant S-N curve for the calculated hot spot stress ranges does not exist for RHS members with plate thicknesses above 16 mm. Hot spot S-N curves therefore need to be developed and additional testing data (preferably from large scale tests) are needed for this scope.

- In order to make bridge design with HSS and hollow section members more economic (for example by using more slender sections), strength verification of connections may most of the times be out of the range of the simplified (generalized) formulas provided in EN 1993-1-8, as they cover only a limited range of geometrical parameters and are applicable for class 1 or 2 sections only. Thus, additional data are necessary to extend the formulas provided in EN 1993-1-8.

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Appendix A: Preliminary designs

A.1 Loads and load combinations

Combinations of actions (NOT for fatigue)

For ULS verifications: characteristic load combination according to EN 1990 is used.

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (6.10)$$

For deflections (SLS): frequent load combination according to EN 1990 is used.

$$E_d = E\{G_{k,j} \,;\, P \,;\, \psi_{1,1} Q_{k,1} \,;\, \psi_{2,i} Q_{k,i}\} \quad j \geq 1 \,;\, i > 1 \qquad (6.15a)$$

Load groups and traffic load models

For all the three preliminary designs two main load groups are considered for the vertical loads:

- Dead loads (self-weight+ asphalt layer) (EN 1991-1-1 for permanent actions).
- Traffic loads: LM1 (UDL+TS) and gr1a (EN 1991-2 for variable actions due to traffic).

For strength verifications (ULS)

Group load model (gr1a) consists of load model 1 (LM1) as the governing variable action which acts in combination with the vertical load q_{fk} on the cycle/foot path.

For the cycle/foot path a uniformly distributed load is applied (EN 1991-2:2003):
q_{fk}= 2+ 120/ (L+30) =2.9 kN/m^2

According again to EN 1991-2:2003 (see also Figures 1, 2 and 3), the roadway of the 'Schellingwouderbrug' (clear roadway width= 7 m) is divided into 2 notional lanes of 3 m each and a remaining part of 1 m.

Load model 1 (LM1) should be applied in each notional lane and on the remaining areas. It consists of a uniformly distributed load (UDL system) and double-axle concentrated loads (TS system) representing heavy lorries. For the assessment of general effects the TS should be assumed to travel centrally along the axes of notional lanes.

Table 4.1 - Number and width of notional lanes

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4$ m	$n_1 = 1$	3 m	w - 3 m
$5{,}4 \text{ m} \leq w < 6$ m	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_1$
NOTE For example, for a carriageway width equal to 11m, $n_1 = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.			

Figure A - 1 Division of the roadway into notional lanes, (EN 1991-2:2003)

Location	Tandem system *TS*	*UDL* system
	Axle loads Q_{ik} (kN)	q_{ik} (or q_{rk}) (kN/m^2)
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

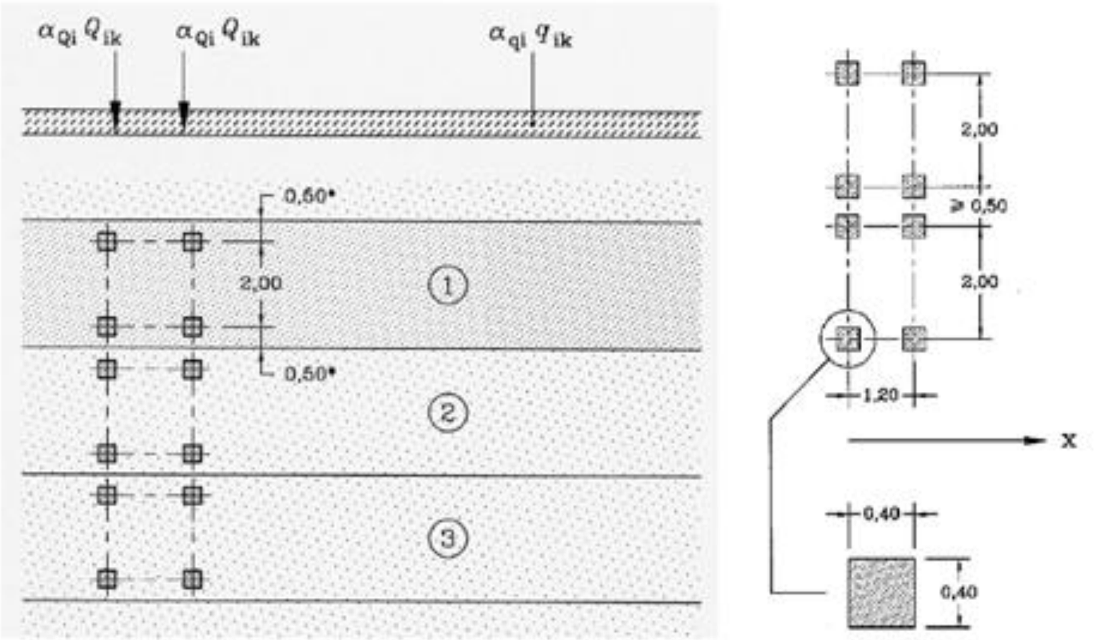


Figure A - 2 LM1-values (top) and application (bottom- left for global verifications (UDL+TS), right for local verifications (TS)), (EN 1991-2:2003).

Lane division and traffic loads for the 'Schellingwouderbrug'

According to section 4 in EN 1991-2 (2003),

Roadway: total width= 7 m

Notional Lane 1 (= 3m): q_{1k}= 9 kN/m^2 (UDL), Q_{1k}= 300 kN (axle load)
Notional lane 2 (= 3m): q_{2k}= 2.5 kN/m^2, Q_{2k}= 200 kN
Remaining area (= 1m): q_{rk}= 2.5 kN/m²

Adjustment factors a_{qi}, a_{Qi}, a_{qr} are all taken equal to 1 according to the Dutch National Annex (EN 1991-2-NA).

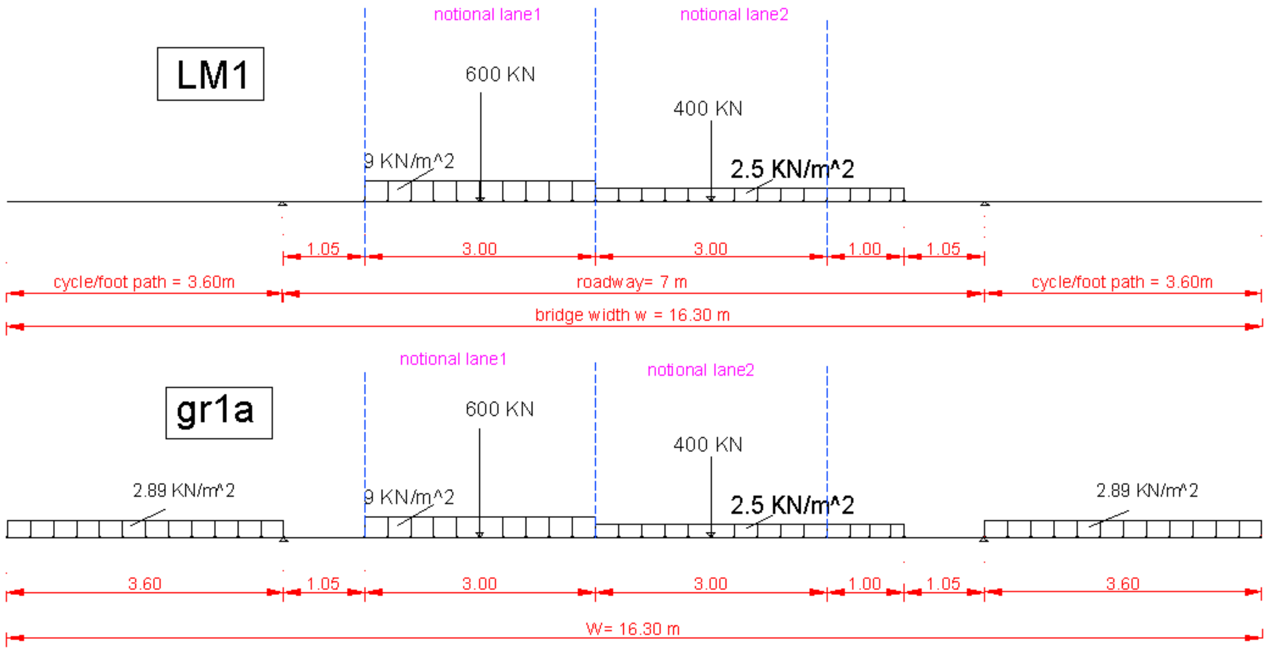


Figure A - 3 LM1 and group 1a for traffic loads on 'Schellingwouderbrug', (EN 1991-2:2003), over the bridge width.

Note: In gr1a the combination value of the cycle/foot path load is used, considering a factor ψ_0 = 0.40.

Load factors:
USL: γ_f=1.35 (DL and gr1a)
SLS: γ_f=1.00 (DL and gr1a)

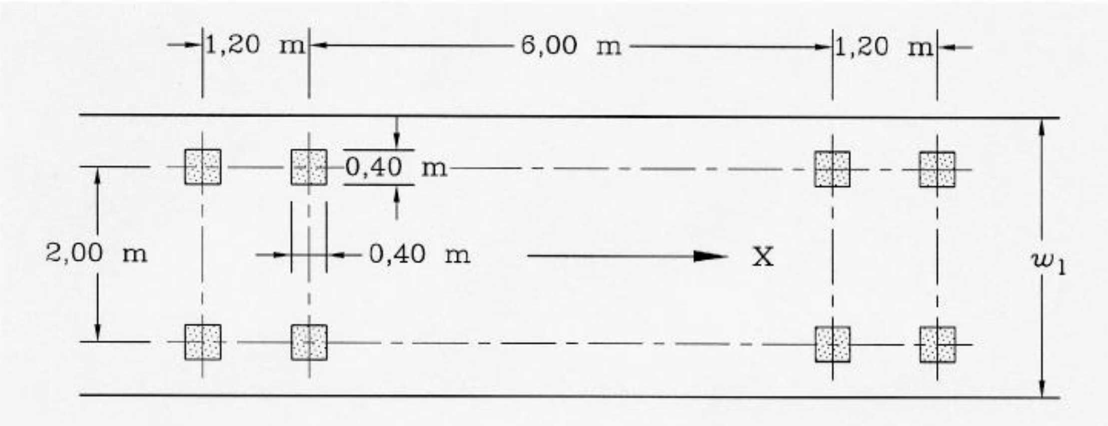
Governing combinations:
ULS: 1.35*DL+ 1.35*gr1a (according to characteristic combination, formula 6.10, EN 1990)
Deflections (SLS): 1.00*DL+ 1.00*ψ_1*gr1a (according to frequent combination, formula 6.15a, EN 1990)

For Fatigue (ULS)

In the preliminary phase, fatigue is treated as a simple global check based on maximum stress range ($\Delta\sigma$) caused by a single heavy vehicle on the bridge. This has been done only for the truss and the arch bridge in Scia Engineer FEM program.

The maximum stress range ($\Delta\sigma$) at the most critical (fatigue) location for each member is calculated by applying FLM3 (single vehicle model) according to EN 1991-2 (2003) (Figure 4). The vehicle is positioned centrally on the notional lane closer to the truss plane to cause the most severe effect on the truss members.

Longitudinally, it is positioned in 3 different locations (i.e. midspan, close to supports, random intermediate position) and the most severe load position for each member, causing the maximum (fatigue) stress level on the member is considered.



Key
w_l : Lane width
X : Bridge longitudinal axis

Figure A - 4 Fatigue load model 3

This stress level is limited by the calculated fatigue strength. This has been obtained by choosing the most relevant fatigue detail in EN 1993-1-9 and the corresponding fatigue class for our design.

The fatigue class, thus chosen is class 71 ($\Delta\sigma_{c=}$ 71 MPa at $2*10^6$ cycles). Using slope m=3 and formulas in EN 1993-1-9, the fatigue limit ($\Delta\sigma_D$ in Figure 5) is then calculated which corresponds to an allowable stress level of $\Delta\sigma_D$=52 MPa. More specifically, $\Delta\sigma_D$ is calculated using the formula in EN-1993-1-9:

$$\Delta\sigma_D{}^m*5*10^6 = \Delta\sigma_c{}^m*2*10^6,\ m=3$$

Where,
$\Delta\sigma_D$: constant amplitude fatigue limit at $5*10^6$ cycles
$\Delta\sigma_c$:maximum strength at $2*10^6$ cycles depending on detail class (Detail class 71, $\Delta\sigma_c$=71 MPa)

Ensuring therefore, at this phase, that fatigue stresses do not exceed the stress level of about 50 MPa, we obtain a design acceptable for fatigue (i.e. it is expected that fatigue strength may be sufficient when more detailed calculations are made to obtain available fatigue life).

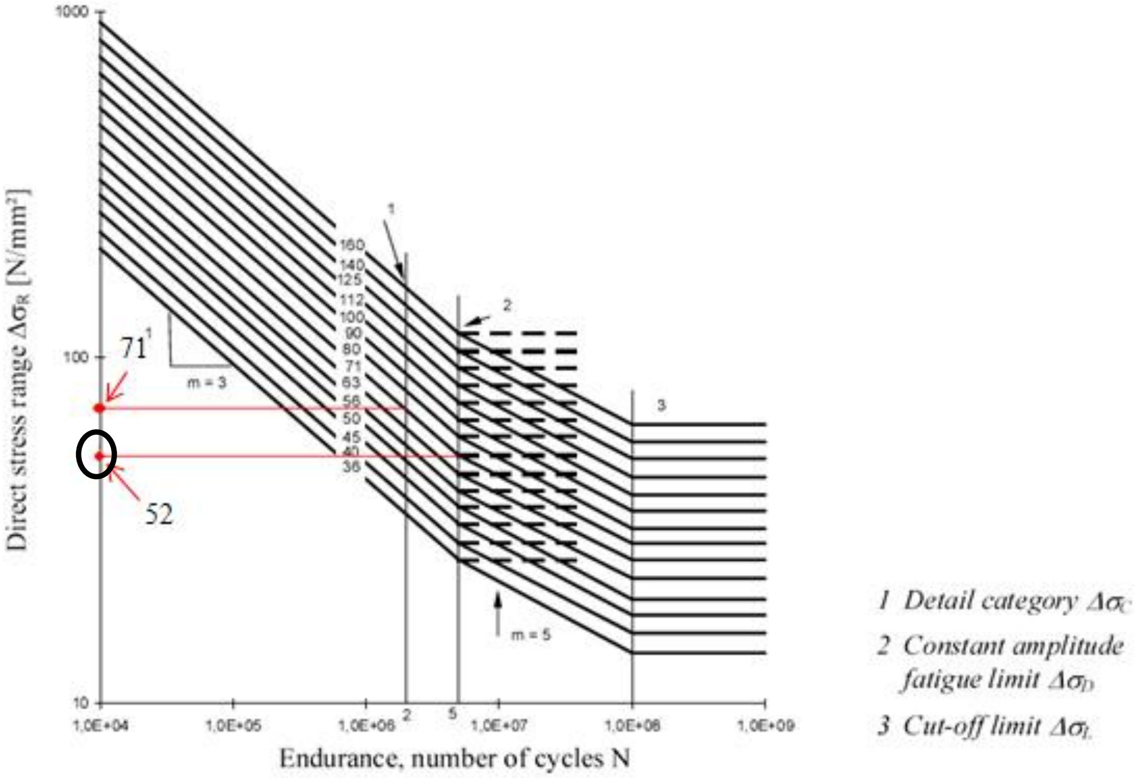


Figure A - 5 Maximum allowable stress level for fatigue verifications, $\Delta\sigma_D$= 52 MPa

Additionally, according to EN 1993-1-9, it is verified that:

$\Delta\sigma_E \leq 1.5 f_y$ and,
$\gamma_{Ff} * \Delta\sigma_E \leq (\Delta\sigma_c / \gamma_{Mf})$

Where,

$\Delta\sigma_E$: maximum stress range caused by FLM3
γ_{Ff} =1.0 load factor
$\Delta\sigma_c$ = maximum strength at $2*10^6$ cycles depending on detail class
γ_{Mf} =1.35 material factor depending on ease for inspection and maintenance (conservatively)

A.2 Trial preliminary bridge designs

a. Trial box designs

Since the box girder section is designed to be extremely slender (L/D = 64) the governing criteria initially to be checked are strength and stiffness. In addition, plate buckling and shear lag effects should also be taken into account by using an effective cross section for the box girder. Initially, however, the whole box section is assumed effective for simplicity.

Several trial box configurations have been attempted, choosing different width dimensions (with respect to vertical or inclined webs) and plate thicknesses (see also AutoCAD file "Box girder trial designs").

This is done, to investigate what are the minimum possible dimensions to give sufficient moment capacity and acceptable midspan deflections.

For the deck an orthotropic steel plate is chosen acting as the top flange of the box in all cases. A concrete deck would add extra weight which would cause even bigger deflections in an already extremely slender box section. Here, only two of the trial box designs are presented in more detail.

a1) Steel Box 1 with orthotropic steel plate

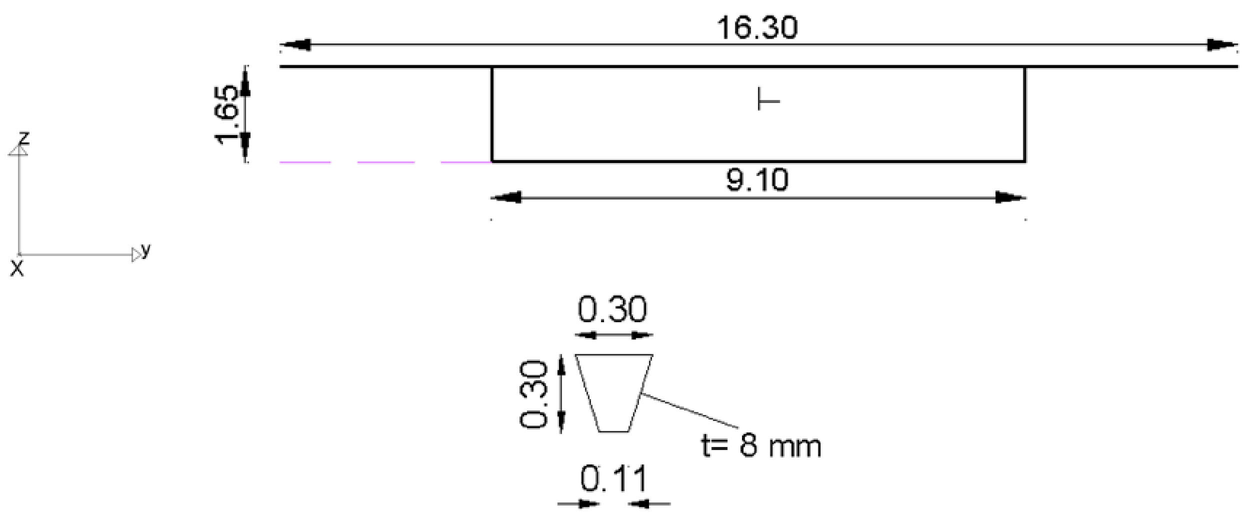


Figure A - 6 Cross section of the Box 1 girder (top) and of the longitudinal stiffeners (bottom) at the top flange of the box girder, c.t.c distance = 600 mm.

Cross sectional dimensions

Top flange: $t_{f,top}$= 30 mm (steel plate thickness is 20 mm) and steel grade assumed to be used for the top flange is S355.
Web: t_w= 20 mm (S460)
Bottom flange: $t_{f,bot}$= 30 mm (S690)

The trapezoidal stiffeners for the deck plate (Figure 6) are taken into account as an equivalent thickness equal to $t_{eq,\,ls} = A_{sl}/w$ and included in the thickness of the top flange (t_{top}).

Plate thicknesses are chosen, keeping material and fabrication costs in mind. Therefore, especially for the bottom flange (S690 grade) a limitation that the plate thickness should be kept rather small ($t \leq$ 40-50 mm), is set already from the beginning.

Box Description

Top flange: The top plate is an orthotropic steel deck with plate thickness 20 mm. The longitudinal stiffeners are distributed over the width of 16.3 m and added as an equivalent thickness on the top flange, to simplify the calculations procedure.

Hence,
$t_{top}= t_{plate}+ t_{eq,\,ls}$
$t_{eq,\,ls}= A_{ls,\,tot}/w$: equivalent thickness for Longitudinal stiffeners on the top flange

Where,
$A_{ls,\,tot}$: is the total area of the longitudinal stiffeners over the width w.
w: is the width of the top flange

This flange is in compression and since the center of gravity is closer to the top (more material and bigger area at the top flange to take the resulting force), it carries smaller stresses (<355 N/mm^2) than the bottom flange and therefore, S355 steel grade is chosen.

Bottom flange: The bottom flange is under tension. The stresses are quite high (>500 N/mm^2) and therefore steel grade S690 can efficiently take over these stresses.

Web: Vertical webs are considered with c.t.c distance of 9.10 m. The thickness is kept constant and equal to 20 mm. The stresses in the web due to global bending exceed 500 N/mm^2 close to the bottom flange. This means that even a steel grade of S460 is suitable assuming local yielding for the web.

Loads

Two types of vertical loads are considered for calculating the internal forces in the steel main structure (not for fatigue):

- Dead load

  - *Steel box*

Density for Steel: γ= 78.5 kN/m³, [EN 1991-1-1]
Self-weight= Area*density= 65.8 kN/m

- *Pavement*

Mastic asphalt is assumed over the whole bridge width with thickness t= 8 mm and density γ= 20 kN/m³ (EN 1991-1-1 suggests values between 18-22 kN/m³).
Mastic asphalt dead weight: $A_{asph.} * \gamma_{asph} = 2.6$ kN/m

- <u>Traffic loads</u>

gr1a: $q_{,k1a}$= $UDL_{,k\ (LM1)+}$ $2*0.40*q_{fk}$= 37+ 2*0,40*2.89*3.60= 40 kN/m over the bridge length
$Q_{k,1a}$= $TS_{,k\ (LM1)=}$ 1000 kN assumed to act in the centerline of the bridge

Maximum moment (DL+ gr1a, γ_f=1 .35):
$M_{y,sd}$= 247522 kNm

<u>Simplification for global analysis</u>

The position of the TS depends on the internal force to be calculated. So, in a simple supported bridge span, for obtaining the maximum global moment M_y, the most unfavorable position of the TS is at the midspan (cross section A-A). For the maximum shear force V_z, the TS is positioned very close to the supports (cross section B-B).

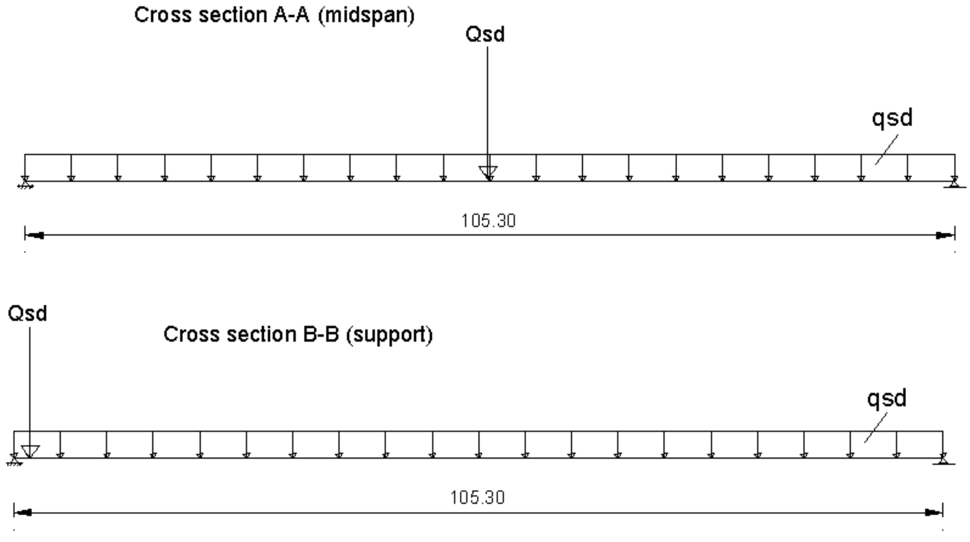


Figure A - 7 Critical positions for the TS, over the bridge span for global analysis, at the preliminary stage.

Maximum moments: $M_{y,sd} = 1.35* (DL+q_{k,1a} L^2) / 8 + 1.35* \{Q_{k,1a}*L/4\}$
Maximum shear: $V_{z,sd} = 1.35* \{DL+q_{k,1a}\}*L/2 + 1.35* Q_{k,1a}$

Stresses

Stress distribution is assumed linear over the cross section.

Table A - 1 Maximum normal stresses, section modulus W_y and unity check for strength verifications, design "Box 1".

Normal Stresses	Section modulus W_y ($*10^8$ mm³)	stress σ_{sd} (N/mm²)	Strength f_{yd} (N/mm²)	U.C. $\sigma_{sd}/ f_{yd} \leq 1$
Top flange (compression)				
top fibre	7.7	324	355	OK
bottom fibre	8.1	308	355	OK
Web(s)				
top fibre	8.1	308	460	OK
bottom fibre	4.8	523	460	NOT OK
Bottom flange (tension)				
top fibre	4.8	523	690	OK
bottom fibre	4.6	539	690	OK

Deflections

Maximum deflection at midspan caused by group traffic loads gr1a (γ_f=1, ψ_1= 0.40 for UDL and ψ_1= 0.75 for TS):
δ_{max}= 0.55 m> L/300= 0.35m NOT OK!

Results interpretation

The moment resistance at midspan, where maximum bending moment occurs, is sufficient but the maximum vertical deflection at the midspan exceeds the maximum value L/300= 0.35 m and therefore the stiffness of the cross section must be increased.

Increasing stiffness means increase the moment of inertia I_y. The stiffness can be increased in two ways:
- By adding more material (increase plate thickness and/or increase the height and the thickness of the longitudinal stiffeners).
- By increasing the construction height.

The second option is not possible due to clearance restrictions below the deck (maximum depth is limited to 1.65 m) and therefore the only option is to increase the late thickness and/or add longitudinal stiffeners at the bottom flange.

a2) Steel Box 2 with orthotropic steel plate

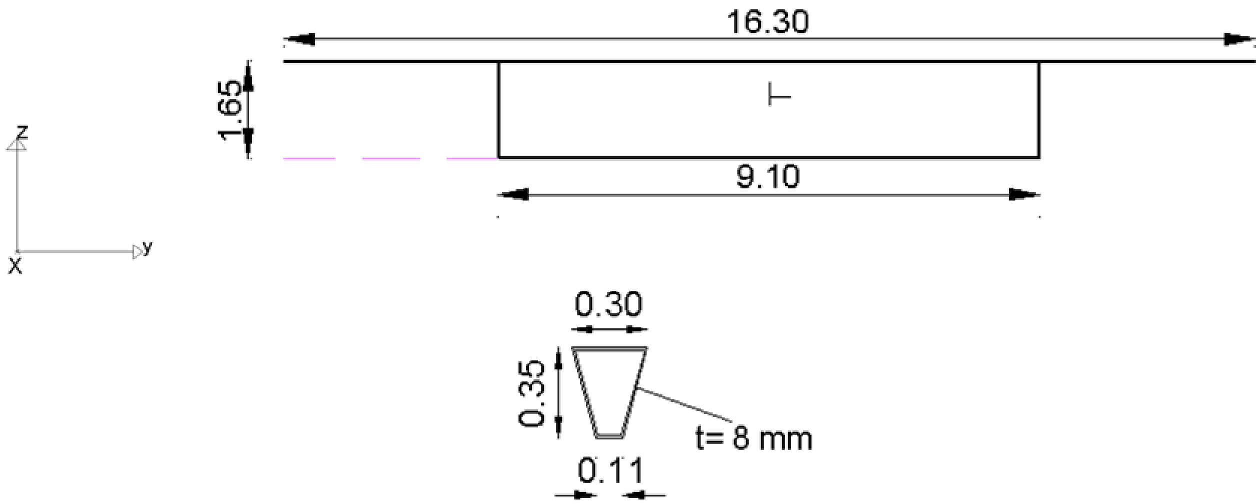


Figure A - 8 Cross section of the Box 1 girder (top) and of the longitudinal stiffeners (bottom) at the top flange of the box girder, c.t.c distance = 600 mm.

Cross sectional dimensions

Top flange: $t_{f,top}$= 50 mm (plate thickness is 37 mm) (S355)
Web: t_w= 20 mm (S355 or S460)
Bottom flange: $t_{f,bot}$= 50 mm (without stiffeners or t_{plate} = 37 with stiffeners) (S460)

Longitudinal stiffeners considered as equivalent thickness $t_{eq,ls}$.= A_{ls}/w and included in the top flange thickness $t_{f,top}$.

Loads

- Dead load

- *Steel box:*
Density for Steel: γ= 78.5 kN/m³, (EN 1991-1-1)
Self-weight= Area*density=104.6 kN/m

-*Pavement:*
Mastic asphalt is assumed over the whole bridge width with thickness t= 8 mm and density= 20 kN/m³ (EN 1991-1-1 suggests values between 18-22 kN/m³).
Mastic asphalt self-weight: A_{asph} * 20 = 2.6 kN/m
$DL_{,tot}$ = Box self-weight + Mastic asphalt

- Traffic loads

gr1a: $UDL_{,k}$= $UDL_{,k\ (LM1)}$+ 2*0.40*q_{fk}= 37+ 2*0,40*2.89*3.60= 40 kN/m over the bridge length
$TS_{,k}$= $TS_{,k\ (LM1)}$= 1000 kN assumed to act in the centerline of the bridge

Maximum moment (DL+ gr1a, γ_f=1 .35) $M_{y,sd}$= 319674 kNm

Stresses

Stress distribution is assumed linear over the cross section.

Table A - 2 Maximum normal stresses, section modulus W_y and U.C. for strength verifications, design "Box 2".

Normal Stresses	Section modulus W_y (*10^8 mm³)	stress σ_{sd} (N/mm²)	Strength f_{yd} (N/mm²)	U.C. $\sigma_{sd}/ f_{yd} \leq 1$
Top flange (compression)				
top fibre	1.2	266	355	OK
bottom fibre	13	245	355	OK
Web(s)				
top fibre	13	245	460	OK
bottom fibre	7.8	412	460	OK
Bottom flange (tension)				
top fibre	7.8	412	500	OK
bottom fibre	7.4	433	500	OK

Deflections

Maximum deflection at midspan (gr1a, γ_f=1, ψ_1= 0.40 for UDL and ψ_1= 0.75 for TS):
δ_{max}=0.349 m< L/300= 0.35 m OK!

Results interpretation

The moment resistance at midspan where the maximum bending moment is found, is sufficient and the maximum vertical deflection at the midspan is also below the maximum value L/300= 0.35 m. These plate thicknesses are already quite large especially when it comes to fabrication costs of high strength steels. In these thicknesses preheating is necessary and so extra costs will be added.

It is important however to note that the maximum vertical deflection is calculated, based only on variable load from traffic. For the dead loads it is likely that the limit will again be exceeded. Usually, in long span bridges, dead load causes much bigger deflections than traffic loading. However that does not influence the design, since it is possible to pre-camber the bridge during fabrication such that will reach the horizontal position right after erection (due to dead loads only). Therefore the only requirement is to satisfy the deflections due to vertical loads from traffic only.

All the above calculations where based on the assumption that the gross section properties can be used, thus the whole cross section is effective under the loading. Unfortunately, this is not the case here.

The box section under consideration (design “Box 2”) is classified as class 4 according to EN 1993-1-1 (Table 5.2) due to high slenderness (i.e. large c/t ratios and also because for high strength steel the slenderness limits become even smaller).

This means that the effective cross section should be used to account for premature plate buckling of the slender plates. This effective part is shown in the next figure for half the bridge width.

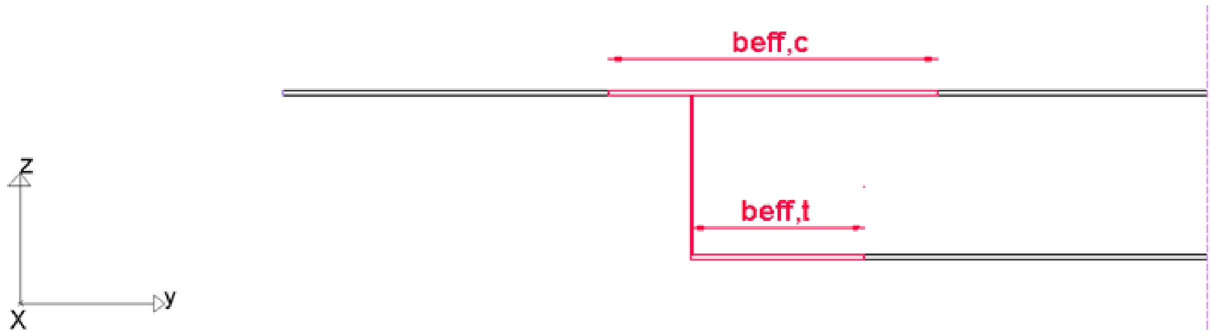


Figure A - 9 Effective cross section of design “Box 2”. Area reduction >60%.

It is already obvious from Figure 9 that the remaining (effective) cross section is quite small to withstand the loads. Specifically, the effective cross sectional area is smaller than 60% of the gross area.

In order to increase the effective cross sectional area it is essential to look for other configurations and box designs.

Possible alternatives include more webs as shown in Figure A-10.

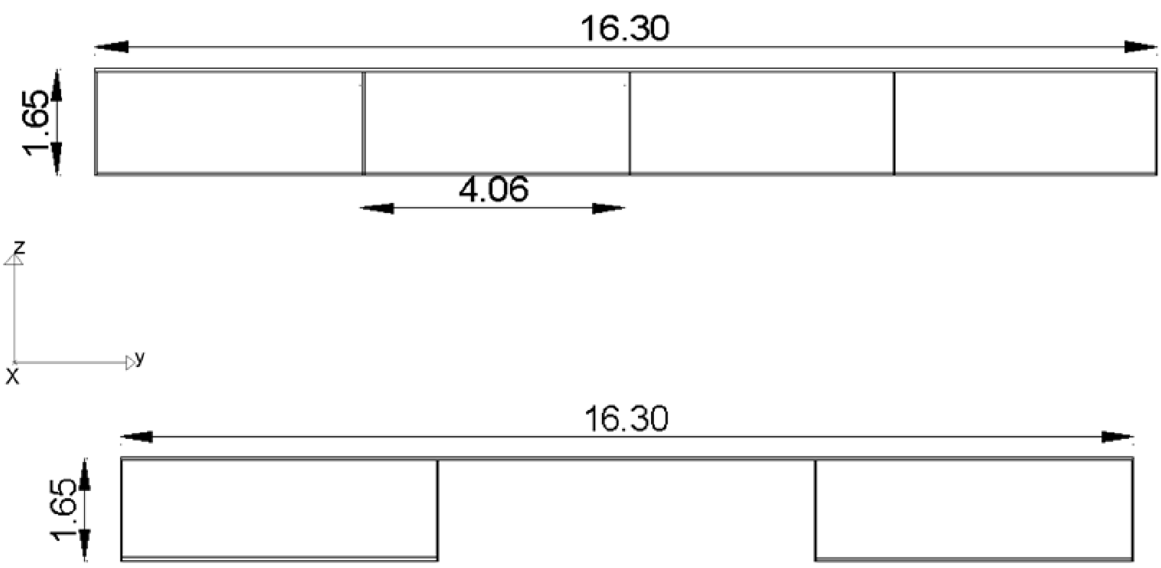


Figure A - 10 Alternative box designs with respect to increase the effective area of the cross section.

But, more webs mean higher fabrication costs and extra, unneeded, shear capacity which automatically lead to an uneconomic solution. Therefore there is no need to make extra calculations for all these alternatives.

Finally, it is concluded that a box girder bridge (alone), although it seems feasible with higher steel grades in the tension flange, will not lead to a cost effective solution for the ‘Schellingwouderbrug’.

b. Trial truss designs

Structural system

The main superstructure is the two vertical (parallel) truss planes connected with cross beams with c.t.c distance of 3m over the full length. For the truss members (braces and chords) box shaped members (welded or RHS) with welded connections (CHS members with cast joints may also be considered in a later stage for comparison). The concrete deck is resting on top of the cross beams but not in composite action. In the truss model (made in Scia Engineer) is being treated as a separate load case (dead load LC6), thus acting separately from the main structure.

Analysis and modeling

Elastic global analysis, assuming pin joints and loading directly on the cross beams, is performed in order to obtain maximum normal forces in all members (chords-braces). Pinned joints can be assumed for a relative accurate estimation of stresses for strength and stability.

This is not however the case for fatigue stresses. For fatigue stresses secondary bending moments, caused by the deformed structure, can significantly increase the fatigue stresses and thus, they should always be taken into account already in this design phase.

To account for secondary moments, therefore, the same model but with stiff joints is considered. Due to these moments, also the stresses of the brace members are increased. Generally, this may be taken into account by modeling stiff connections or by using hinged connections and allowing stresses up to 60-70% of the yield stress.

In this case it was easy to model stiff connections in the model to calculate fatigue stresses.

The modeling of the truss is made using "Scia Engineer", FEM program. Wind bracing is not included in the model since its presence does not affect the load carrying capacity of the truss under vertical loads only. However, is theoretically being considered in terms of stability.

Design criteria

The designs criteria are strength, fatigue, and stability. A check for stiffness (in terms of maximum vertical deflection at midspan) may also be considered for the final design. However, for the preliminary phase, only, the inherent stiff nature of truss bridges allows us to assume that no stiffness problems will occur. Thus, the check may be disregarded for the moment.

Loading

Two types of vertical loads are considered for calculating the internal forces in the steel main structure (not for fatigue):

- Dead loads (steel structure self-weight + 240 mm thick normal concrete deck+ 8 mm thick asphalt layer)
- Traffic loads (LM1 and gr1a)

For traffic loads again gr1a gives the governing combination. Now the positioning of traffic loads over the bridge width is such that causes the most unfavorable load effect on one truss plane. The position of the loads and the reactions at the positions of the truss planes are shown in Figure 11.

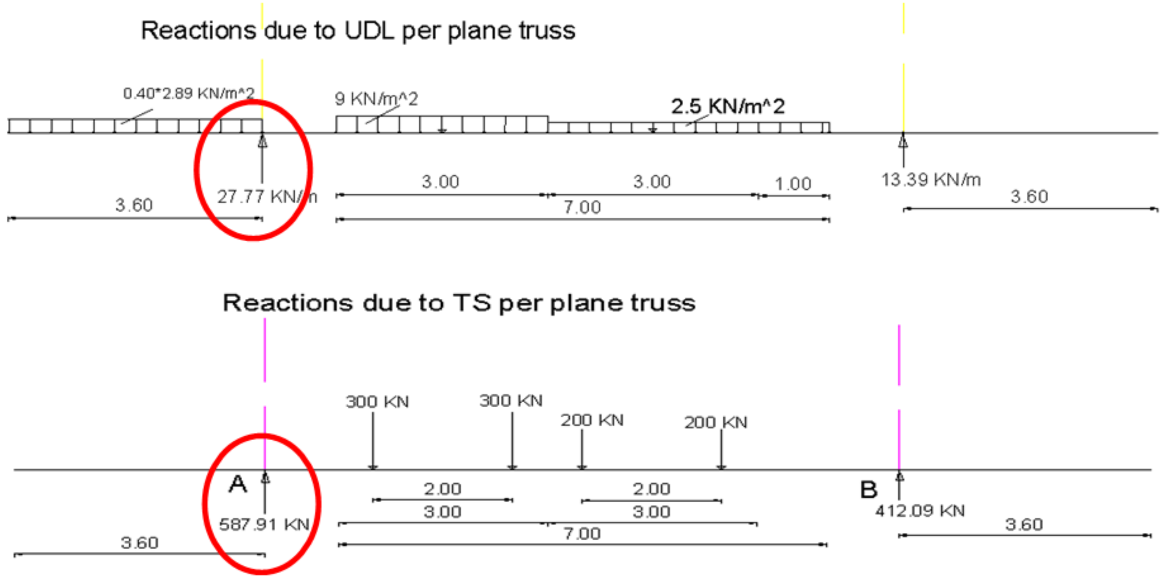


Figure A - 11 Positioning of traffic loads (gr1a) and the reactions caused separately by UDL and TS (characteristic values for LM1-for cycle/foot load $q_{f,k}$ the combination value is shown).

Truss plane A (left truss plane in Figure A-11) is in this case the more heavily loaded as can also be seen from the difference in support reaction values.

The reaction caused by the uniformly distributed loads, is applied as a uniformly distributed load (kN/m) over the whole bridge span. The reaction from the TS is once again positioned such as to cause the most unfavorable global effect (see Figure A-12).

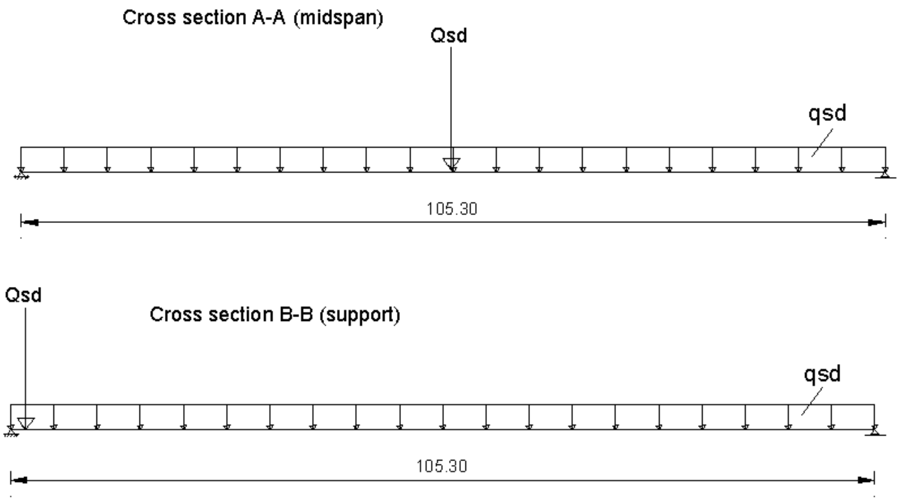


Figure A - 12Critical positions for TS (gr1a) in truss plane A causing maximum shear (top) and maximum bending moment (bottom).

Considering maximum bending moment (midspan) and maximum shear forces (supports), the most heavily loaded regions for each member can be determined.

Based on maximum normal forces (chords and brace members) and maximum in plane bending moments (cross beams and chord members only as pinned connections are assumed) in these regions, a first estimation for the dimensions of the chords, brace members and cross beams can be made.

Thus, it is customary to consider two main regions (Figure A-13) with respect to maximum internal forces in the truss members. In this way, depending on their stress level, it is possible to use higher steel grades and/or higher cross sectional dimensions (i.e. width, height, plate thickness) for the heavily loaded members and medium strength steel and or smaller cross sectional dimensions for the rest of the members.

This may provide minimum dead weight and increased members efficiency (axial design resistance/axial plastic capacity (=$N_{ax,Rd}$/ A*f_{yd})), but not necessarily to the most economic solution (fabrication of more different members dimensions and types of connections increase overall costs significantly).

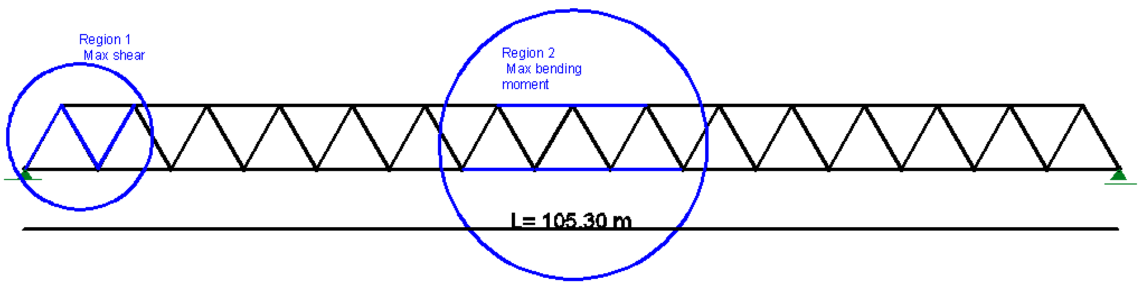


Figure A - 13 Region 1: Heavily loaded brace members close to the supports (left and right), Region 2: Heavily loaded chord members and cross beams close to midspan

Generally, there are two options to consider for the heavily loaded members:

- Higher dimensions, or

- Higher strength steel grades.

The final choice depends on strength, stability, fatigue and total costs.

Load cases for vertical loads in the truss model (Scia Engineer FEM program)

There are six load cases for calculating the internal forces in the members under vertical loads in the ULS stage (not for fatigue).

Two (LC1 and LC6) for permanent loads (dead weight and deck + pavement) and four (LC2, LC3, LC4, LC5) for variable traffic loads according to load model 1 (LM1) and group 1a (gr1a) for traffic loads on roadway bridges (EN 1991-2: 2003) (see also Figures 1, 2, 3 and 11). Analytically these are:

LC1: Self weight only (steel structure)

LC2: UDL: LM1 (on the roadway)
<u>Numerical values</u>
On edge cross beams is: Lane 1: (9*1.5) =13.5 kN/m
Lane 2: (2.5*1.5) =3.75 kN/m
Remaining part: (2.5* 1.5) =3.75 kN/m
On the others is: Lane 1: (9*3) =27 kN/m
Lane 2: (2.5*3) =7.5 kN/m
Remaining part: (2.5* 3) =7.5 KN/m

LC3: q_{lf}: cycle/foot path load (combination value, ψ_0=0.40) on the left side of the bridge only.
<u>Numerical values</u>:
On edge cross beams is: 0.40*2.89*1.5=1.73 KN/m
On the others is: 0.40*2.89*3= 3.47 KN/m

LC4: TS: LM1
<u>Numerical values</u>:
Lane 1: $Q_{s,k,1}$= 300 kN (axle load)
Lane 2: $Q_{s,k,2}$= 200 kN (axle load)

LC5: q_{lf}: cycle/foot path load (combination value) on both sides.
<u>Numerical values</u>:
On edge cross beams is: 0.40*2.89*1.5=1.7 kN/m
On the others is: 0.40*2.89*3= 3.5 kN/m

LC6: Deck + pavement: Assume normal weight concrete deck [EN 1991-1-1].
<u>Deck</u>: Concrete with thickness t= 240 mm and density γ= 25 kN/m³ (assumed).
*<u>Pavemen</u>*t: Mastic asphalt: t= 8 mm and density γ= 20 kN/m³.

For fatigue load model 3 (FLM3) is applied to obtain an indication on maximum and minimum stress values caused by traffic.

LC7: FLM3 according to the EN 1991-2:2003 (see also Figure 4).

Load Combinations

Three load combinations for vertical loads. Two for ULS (not for fatigue) and one for SLS.

- For ULS:
LCB1: 1.35* LC1+ 1.35* LC2+1.35*LC3+ 1.35*LC4, (DL + gr1a, cycle lad only on one side of the bridge).
LCB2: 1.35* LC1+ 1.35* LC4, (DL+ LM1).

- For SLS:
LCB3: 1.00* LC1+ 1.00*ψ_1* LC2+1.00* ψ_1* LC3+ 1.00* ψ_1* LC4 (only for check in a later stage)

Three sub-cases are distinguished here with respect to the positioning of the TS (LC4):

a) LC4 is positioned 3 m from the left end (longitudinal direction) to obtain the maximum support reaction (left).
b) LC4 is positioned directly on the cross beam at the end of the first truss panel, 15m from the left support. This will give the maximum normal forces in the brace members.
c) LC4 is positioned almost halfway the bridge length, at a distance 45 m (max stress at the top chord) and 54 m (max stress in bottom chord) from the left support (longitudinal direction) to obtain the maximum bending moment and normal forces in the (middle) chord members.

Note: In LCB1 the loads are applied in such a way, that the most adverse effect occurs for one truss plane in order to dimension the truss members. So notional lane 1 is closer to plane A, and the cycle load is considered only on one cantilever (next to truss plane A). In this way, plane truss A is the most heavily loaded.

b1) Truss 1

The geometrical parameters are initially chosen to be:

$$\left.\begin{array}{l} L = 105\ m \\ Li = 15\ m \\ D = 7\ m \end{array}\right\} \quad L/D = 15$$

Cross beams c.t.c distance = 3 m
Loads on the cross beams: Edge beams: q (kN/m^2)* 1.5 m (influence length)
All the others: q (kN/m^2)* 3 m (influence length)

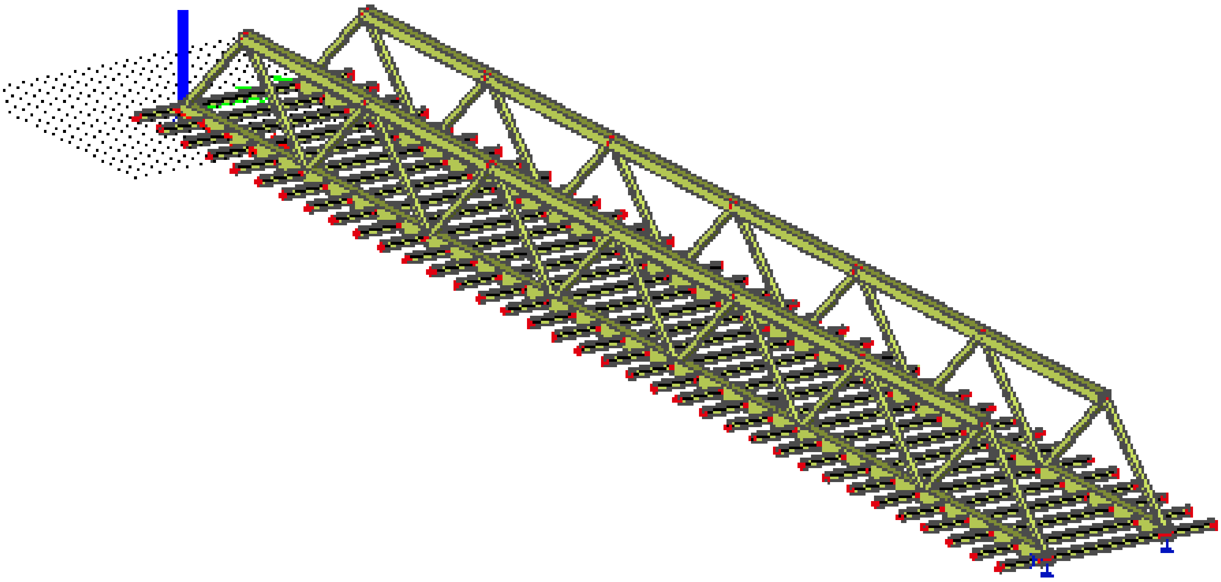


Figure 10 Truss 1: L/D= 15, 3D FEM model in “Scia Engineer”.

Cross sectional dimensions

Braces

The same cross section for all braces: B=450 mm, H= 400 mm, t_f=t_w=15 mm, A=24600 mm².

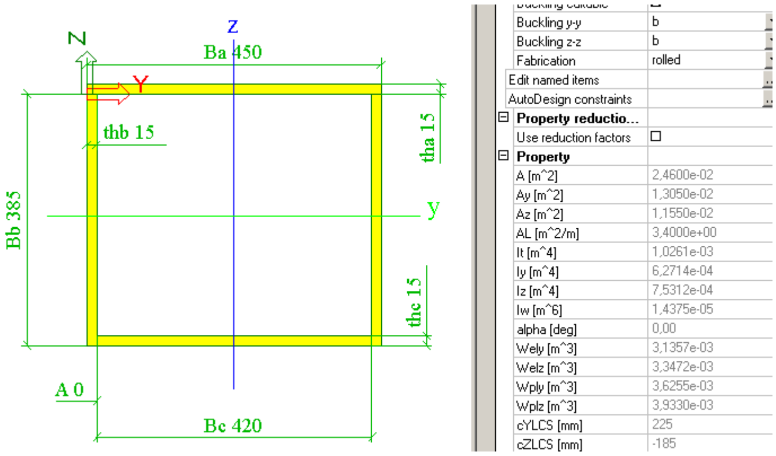


Figure A - 14 Brace cross section, Truss 1- Scia Engineer

Top Chord

The dimensions for the top chord are chosen to be: B=H=520 mm, t_f=20 mm, t_w=15 mm, A=35200 mm².

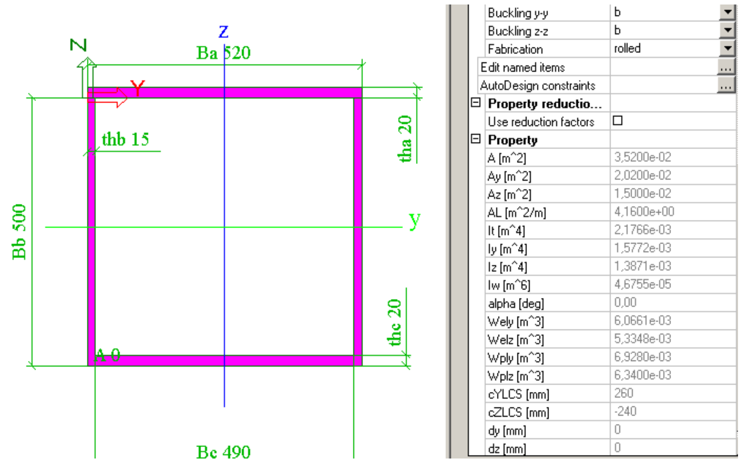


Figure A - 15 Top chord cross section, Truss 1-Scia Engineer

Bottom Chord

The dimensions for the top chord are chosen to be: B=750 mm, H=1000 mm, t_f=20 mm, t_w= 15 mm, A=58800 mm²

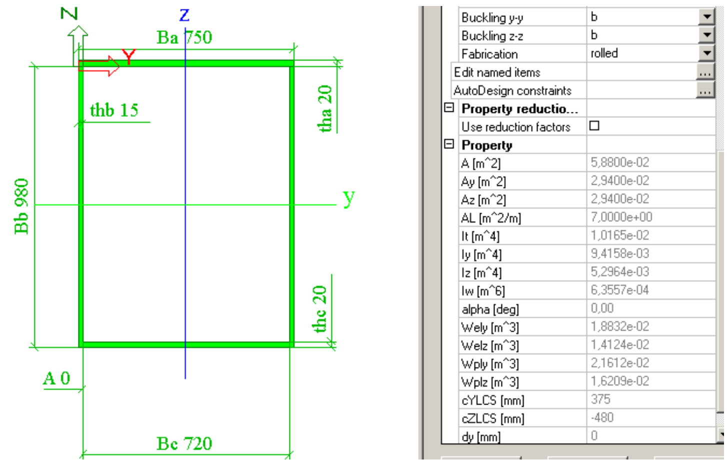


Figure A - 16 Bottom chord cross section, Truss 1-Scia Engineer

Cross beams

Initially, rolled I-section, HEB 550, is chosen for the cross beams based on maximum positive bending moments $M_{y,sd}$ at midspan.

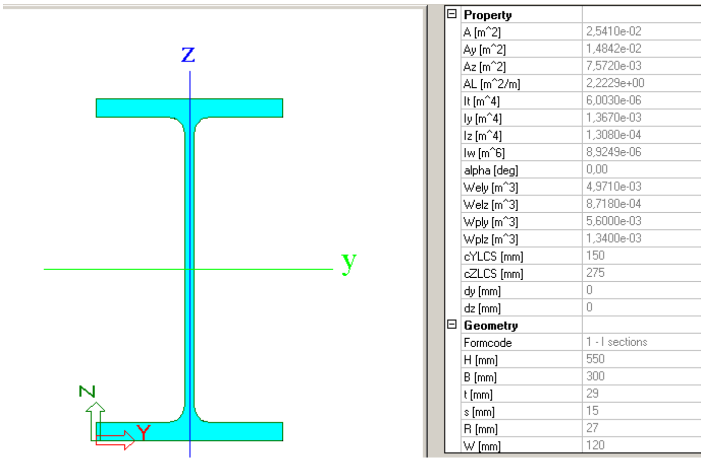


Figure A - 17 Cross beam cross section, Truss 1- Scia Engineer

Scia Engineer output

Heavily loaded truss plane A

Internal forces (static stresses)

Load combination 1 (LCB1: DL+gr1a) gives the maximum normal forces (and stresses) for the heavily loaded truss plane A under vertical loads.

For strength verifications the stresses in the members should be limited to the design yield strength (f_{yd}).

<u>Reactions</u>

Maximum reaction in vertical z-direction at the left support of truss plane A is obtained when the TS (LC4) is positioned 3m from the left support (Figure 28).

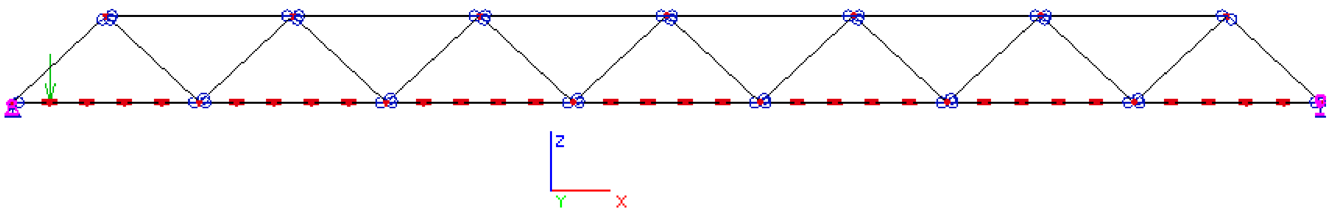


Figure A - 18 Reactions in truss plane A due to LCB1 when the TS is positioned 3m from the left support.

Brace members

Maximum normal stress (both tensile and compressive) in the brace members is obtained when the TS is positioned at the end of the first truss panel (at a distance x=15 m from the left support, see Figures 29 and 30):

- *Brace under compression* (Figure 29): $\sigma_{c,max}$= - 318 N/mm², U.C.= 0.9 (S355)

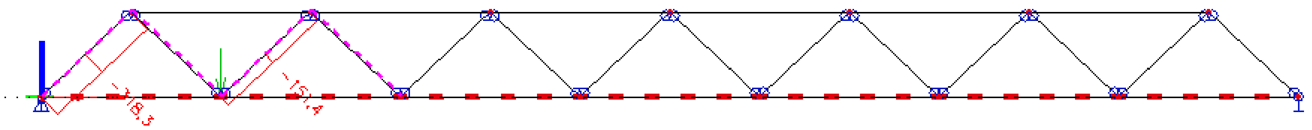


Figure A - 19 Maximum compressive stress in the brace members

- *Brace under tension* (Figure 30): $\sigma_{t,max}$= + 315 N/mm², U.C. =0.89 (S355)

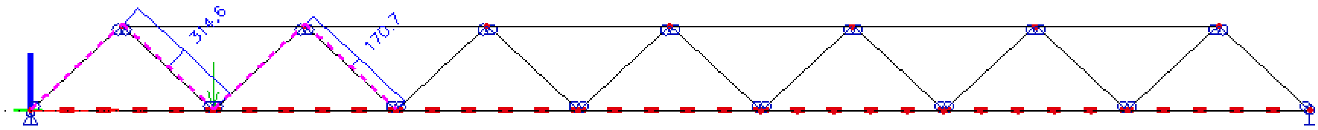


Figure A - 20 Maximum compressive stress in the brace members

The most heavily loaded brace members are the two end braces close to the supports (per truss plane) but the stresses are below the minimum yield strength of S355 steel grade. So, S355 can be used for the brace members.

Chord members

- **Top chord (in compression)**

The maximum tensile stress (tension+ bending) is obtained when the TS is positioned at x_1=45m from the left support (Figure 31): $\sigma_{c,max}$ = - 666 N/mm²

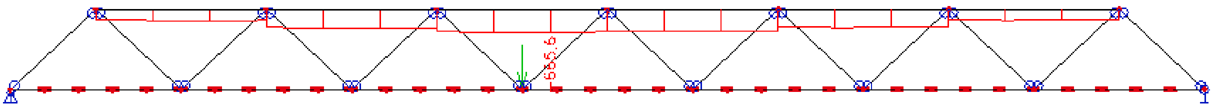


Figure A - 21 Maximum compressive stress in the top chord. The TS for LM1 is positioned directly on the cross beam at a distance x_1=45 m from the left support.

- **Bottom chord (in tension and bending)**

The maximum tensile stress (tension+ bending) is obtained when the TS is positioned at x_2= 54 m from the left support (Figure 32): $\sigma_{t,max}$ = +594 N/mm²

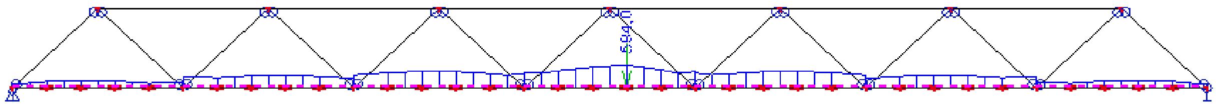


Figure A - 22 Maximum tensile stress in the bottom chord when positioning the TS (LC4) almost at the bridge centre (distance x_2=54 m from the left support).

The maximum compressive stress (bending) is obtained when the TS is positioned at x_2= 15 m from the left support (Figure 33): $\sigma_{c,max}$ = -23 N/mm²

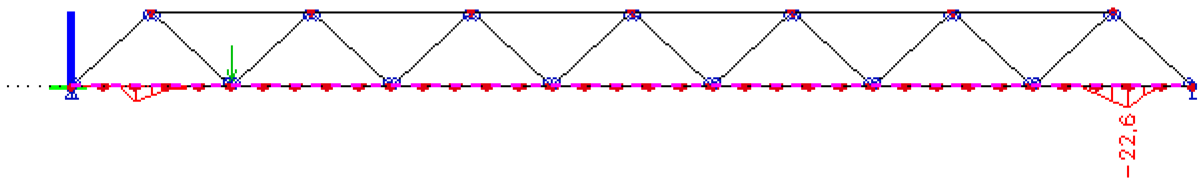


Figure A - 23 Maximum compressive stress in the bottom chord when positioning the TS (LC4) almost at a distance x=15 m from the left support.

Cross beams

The dimensions of the cross beams depend on the maximum bending moment $M_{y,sd}$ caused on the cross beams due to LCB2 (DL+ LM1).
Thus, applying only the LM1 with the TS (LC4) being at a distance x=54 m (almost at midspan) on the roadway the maximum positive bending moment (Figure 34) for the cross beam is:
M_{max} = 1948 kNm
$\sigma_{t,max} = M_{y,sd,max} / W_{pl,y}$ = 1948*(10^6) [Nmm] / 5.6*(10^6) [mm^3] = 348 N/mm^2

A steel grade with minimum yield strength 355 would be sufficient. However it is possible to achieve smaller dimensions by choosing higher steel grade (perhaps S460). This could be investigated with respect to costs.

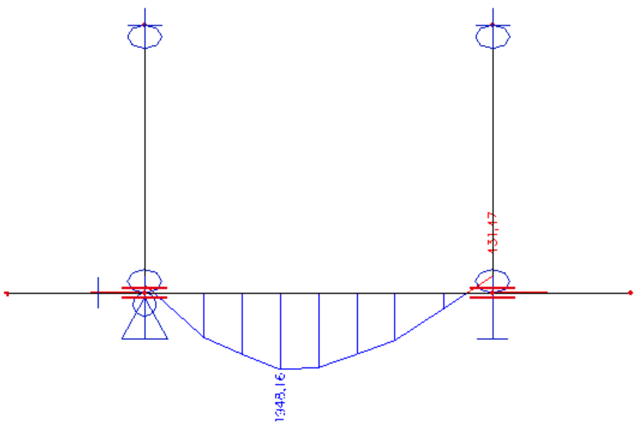


Figure A - 24 Maximum positive bending moment My in cross beam due to LCB2. Position of the LM1 is over the cross beam at x=54 m from the left support (close to midspan)

Stability check

Another important design criterion that must be satisfied for all the compression members is that instability under vertical loads should be avoided. Therefore, flexural (in and out of plane) buckling is checked. For the chord members buckling factor $K_y = K_z = 0.9$ is considered, while for the braces (hinged to the chords) $K_y = K_z = 1$.

Table A- 3 In plane buckling strength for the top chord, design "Truss 1"

Top Chord		
Area	A (mm^2)	35200
Length	L_y (mm)	15000
Buckling coefficient	K_y	0,9
Buckling length	l_y (mm)	13500
Moment of Inertia	I_y (mm^4)	$15.8*10^8$
Radius of gyration	i_y (mm)	212
Slenderness	λ_y	64
Rel. slenderness	λ_E	55
Buckling curve		b
Imperfection factor	α	0,34
Strength	f_{yd} (N/mm^2)	690
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	1,16
Phi factor	Φ	1,34
Buckling factor	χ_y	0,50
Buckling strength	$f_{b,rd}$ (N/mm^2)	344

Unity check (U.C.): Maximum compression stress in the top chord is -666 N/mm^2 > 344 N/mm^2 NOT OK!
Out of plane buckling strength is the same (square hollow section (SHS) chord member and same out of plane buckling length $l_z = l_y$).

Table A- 4 In plane buckling resistance of compression end brace member.

End brace		
Area	A (mm^2)	24600
Length	L_y (mm)	10259
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	10259
Moment of Inertia	I_y (mm^4)	6.30E+08
Radius of gyration	i_y (mm)	160.03
Slenderness	λy	64
Rel. slenderness	λ_E	58
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	620
E-modulus	E (N/mm^2)	210000

Relative slenderness	$\lambda_{y,rel}$	1.11
Phi factor	Φ	1.27
Buckling factor	χ_y	0.53
Buckling strength	$f_{b,rd}$ (N/mm^2)	329

The buckling strength of the most heavily loaded compression brace member (i.e. end brace) is only sufficient with a steel grade at least S620 (U.C.= 0.97< 1).

However, the cross section for the top chord is very small for a length of 15 m unsupported chord member. It has been calculated that only a length of 3500 mm would only be sufficient to take the compression stresses, retaining the same cross section. The other option would be to increase largely the top chord section to satisfy also the buckling criterion.

Therefore, design "Truss 1" is rejected because it does not provide sufficient buckling strength.

Fatigue stresses

Fatigue calculations although not important for "Truss 1" design (failed already in buckling) showed that fatigue stresses are well below the stress level of $\Delta\sigma_{,\,nom}$ =52 N/mm^2 (about $\Delta\sigma_{,\,nom}$= 25 N/mm^2). So fatigue is not governing.

b2) Truss 2

To improve the buckling behaviour of the top chord and compression braces, a second truss design is checked by initially increasing the cross sectional area of these members. However, also the dead weight is increased in this way which leads to stress increase in the bottom chord. The result is subsequent increase of the cross sectional dimensions of the bottom chord also.

Cross sectional dimensions

Braces

For the braces the cross section has slightly been increased in comparison to the case of Truss 1. The same cross section is used for all braces: B=H= 450 mm, t_f=t_w=17 mm, A=29444 mm².

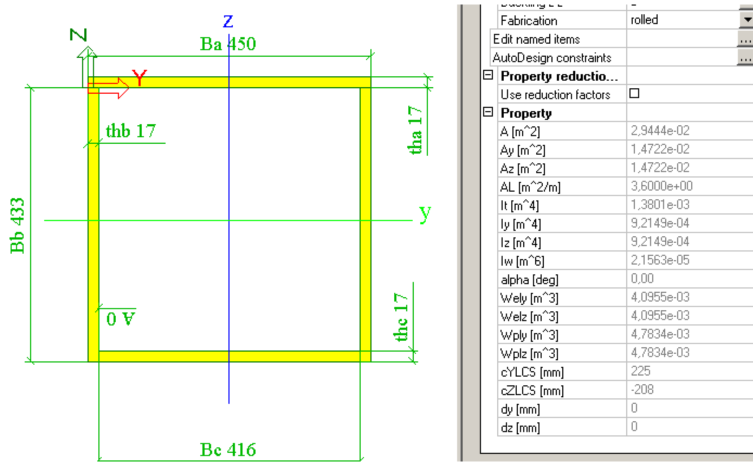


Figure A - 25 Brace cross section, Truss 2-Scia Engineer

In Truss 1 the brace cross section already satisfied the strength criterion. Now the cross section is even bigger and can sufficiently take the resulting stresses which lie below 400 MPa. Thus only buckling and fatigue resistance should be again checked.

Buckling resistance of compressive brace members

Table A -5 In of plane buckling resistance of end brace, design "Truss 2"

End brace		
Area	A (mm^2)	29444
Length	L_y (mm)	10259
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	10259
Moment of Inertia	I_y (mm^4)	9.21E+08
Radius of gyration	i_y (mm)	176.86
Slenderness	λy	58
Rel. slenderness	λ_E	58
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	620
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	1.00
Phi factor	Φ	1.14
Buckling factor	χ_y	0.59
Buckling strength	$f_{b,rd}$ (N/mm^2)	369

Out of plane buckling strength is the same (i.e. square section, same buckling length).

The maximum stress in the compression brace member is -352 N/mm^2. However, in order to satisfy also the stability criterion, S620 is the minimum steel grade to be used for the heavily loaded end braces (U.C: 352/369= 0.95 <1).

For the rest compressive (and tensile) braces however the stresses are quite smaller and steel grade S355 or S460 is sufficient for both strength and stability criteria.

Fatigue stresses in the brace members

Maximum fatigue stress in the critical brace (brace 2 connection with bottom chord) is $\Delta\sigma_{,\ nom}$ = 19 N/mm^2 << 52 N/mm^2.

Thus, this cross section provides an acceptable solution for the braces in design "Truss 2", as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Top Chord

Strength

The dimensions for the top chord have been initially increased (Figure 27) to satisfy the strength criterion (i.e. increasing the bracing dimensions increased the dead weight of the structure, thus increased the stresses in the top chord).

B=650, H=600 mm, t_f=26 mm, t_w=23 mm, A=59008 mm². In this case, sufficient strength can be obtained using at least steel grade S550 for the heavily loaded chord parts.

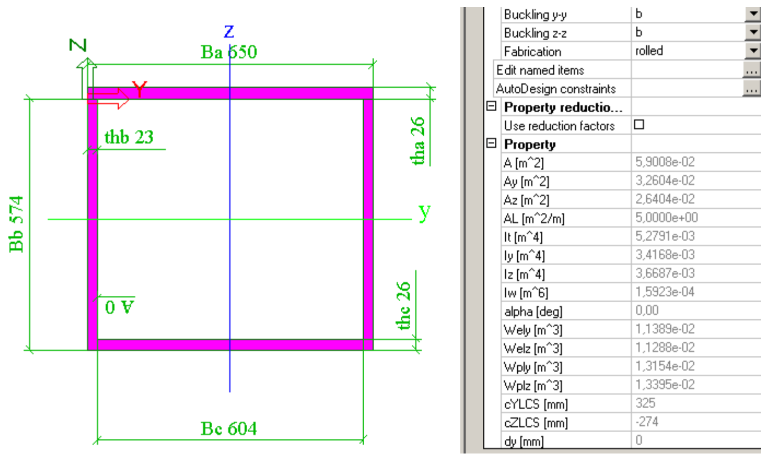


Figure A - 26 Top chord cross section, Truss 2- Scia Engineer

Buckling resistance of top (compression) chord

The top chord is subject to compression and unfortunately, global buckling is again governing in this design also for a length of 15 m (even a steel grade S690 is not sufficient for the stresses in the chord).

Thus, alternatives are considered in order to deal with stability problems. These in general include:

1. Increase the buckling resistance by increasing the top chord cross sectional area even more and thus reducing further the stresses in the chord member.
2. Decreasing the buckling length of the chord member. This is possible in two ways:

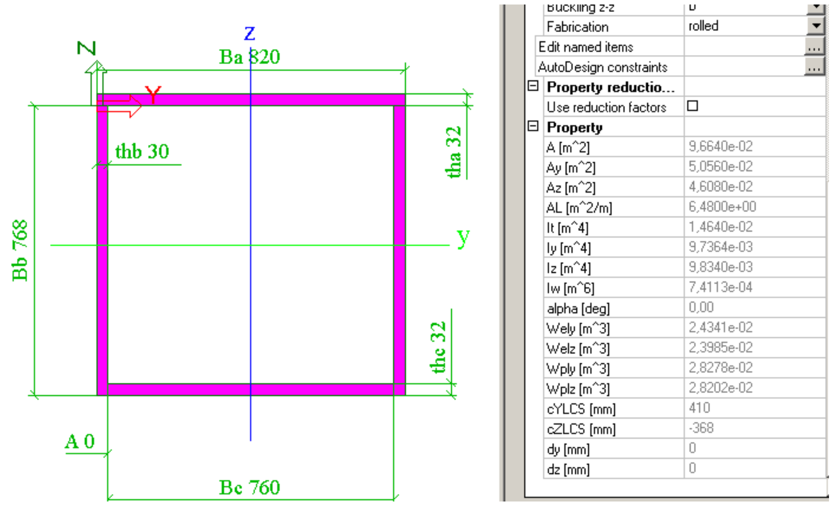
a. Add lateral (every 9.5 m) and in plane (every 7.5 m) bracing.
b. Decrease the field (panel) length from 15 m to e.g. 10 m.

Increasing the cross sectional area means adding more material and consequently more weight but also reduction of stresses which allows lower (cheaper) steel grades to be used. The final choice between these options depends of course on costs considerations. Alternative 2a is not preferred in terms of costs due to the fact that additional members and connections are needed without any cross sectional chord reduction. Thus material and fabrication costs will be increased leading to a less cost effective solution.

On the other hand reducing the field (panel) length will increase the number of connections but will also reduce the cross sectional dimensions, thus, is an interesting option to be considered as another truss design.

1. Increase the cross section

To satisfy both strength and stability criteria the cross section for the top chord is chosen to be:
B=820mm, H=800 mm, t_f=32mm, t_w=30 mm, A=96640 mm²



A [m^2]	9,6640e-02
Ay [m^2]	5,0560e-02
Az [m^2]	4,6080e-02
AL [m^2/m]	6,4800e+00
It [m^4]	1,4640e-02
Iy [m^4]	9,7364e-03
Iz [m^4]	9,8340e-03
Iw [m^6]	7,4113e-04
alpha [deg]	0,00
Wely [m^3]	2,4341e-02
Welz [m^3]	2,3985e-02
Wply [m^3]	2,8278e-02
Wplz [m^3]	2,8202e-02
cYLCS [mm]	410
cZLCS [mm]	-368
dy [mm]	0
dz [mm]	0

Figure A - 27 Increase buckling resistance using larger top chord cross section, design "Truss 2"- Scia Engineer

For buckling a grade S355 (or S420) is not sufficient and thus S460 is considered. In this case only lateral support by transverse bracing every 15 m is considered to deal with out of plane buckling while the in plane buckling is satisfied by using bigger section and steel grade with medium strength (S460). Analytically the calculations for buckling assuming a steel grade S460 are:

Table A.6 In plane buckling strength for the top chord, design "Truss 2"

Top chord		
Area	A (mm^2)	96640
Length	L_y (mm)	15000
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	13500
Moment of Inertia	I_y (mm^4)	9.73E+09
Radius of gyration	i_y (mm)	317
Slenderness	λy	43
Rel. slenderness	λ_E	67
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	460
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.63
Phi factor	Φ	0.77
Buckling factor	χ_y	0.82
Buckling strength	$f_{b,rd}$ (N/mm^2)	377

Maximum stress in the top chord is 341 N/mm^2, so in plane buckling strength is sufficient with a steel grade S460 for the top chord. U.C= 0.91 < 1

Table A - 7 Out of plane buckling strength for the top chord, design "Truss 2"

Top chord		
Area	A (mm^2)	96640
Length	L_z (mm)	15000
Buckling coefficient	K_z	0.9
Buckling length	L_z (mm)	13500
Moment of Inertia	I_z (mm^4)	9.83E+09
Radius of gyration	i_z (mm)	317
Slenderness	λ_z	43
Rel. slenderness	λ_E	67
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	460
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{z,rel}$	0.63
Phi factor	Φ	0.77
Buckling factor	χ_z	0.82
Buckling strength	$f_{z,b,rd}$ (N/mm^2)	378

Maximum stress in the truss top chord is 341 N/mm², so out of plane buckling strength is sufficient with a steel grade S460 for the top chord. U.C= 0.91 < 1

Fatigue stresses in the top chord

Maximum fatigue stress in the top chord at the critical detail (joint 10 close to midspan) $\Delta\sigma_{,nom} = 14$ N/mm^2 << 52 N/mm^2.

Thus, this cross section provides an acceptable solution for the top chord in design "Truss 2" as it satisfies all three design criteria (i.e. strength, stability, fatigue).

Bottom Chord

Strength

For the bottom chord strength is the governing criterion. So, a section with dimensions B=800 mm, H=1000 mm, t_f=20 mm, t_w= 15 mm, A=58800 mm² provides sufficient strength when using a steel grade S690 (max σ_t = 664 N/mm^2).

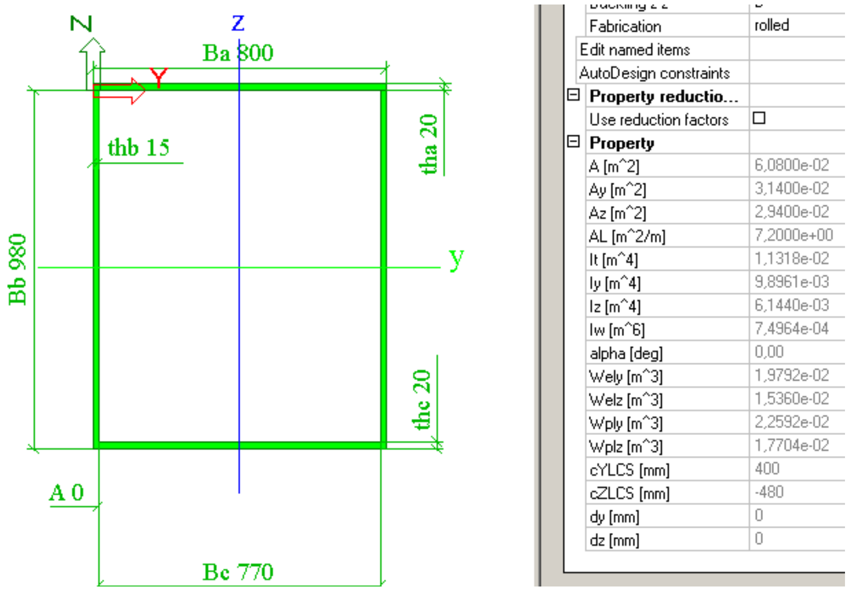


Figure A - 28 Bottom chord cross section, Truss 2- Scia Engineer

Stability check

Stability is not a problem for the bottom chord since it is in tension and in bending and the top (compression) flange is laterally supported by the cross beams (every 3 m) and also by the concrete deck (all over its length).

Fatigue

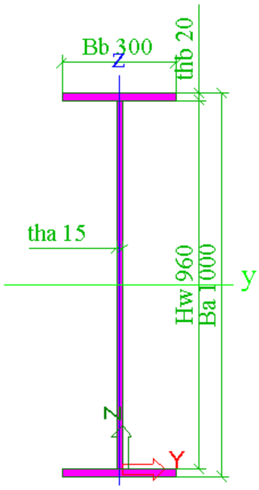
Maximum fatigue stress (mainly due to in plane bending of the bottom chord member) in the bottom chord at the critical detail (close to midspan) $\Delta\sigma_{,nom} = 30$ N/mm^2 < 52 N/mm^2.

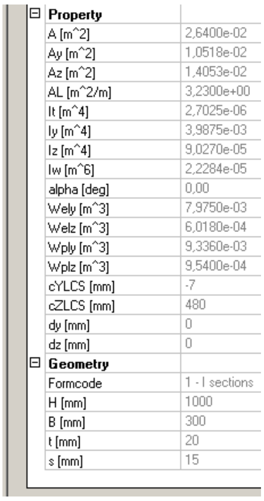
Thus, this cross section provides an acceptable solution for the bottom chord in design “Truss 2” as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Cross beams

For the cross beam I- section profile fabricated by welded plates is chosen. Fabricated sections are generally less economic than standardized profiles but they can be tailored for the specific design.

The required height to take over the vertical loads is much smaller (about 600 mm). However, a height of 1000 m is chosen as to ease the fabrication of the connection with the bottom chord. Based on resulting stresses on the cross beams (below 100 N/mm^2), a low steel grade e.g. S235 can be used for the cross beams.





Property	
A [m^2]	2,6400e-02
Ay [m^2]	1,0518e-02
Az [m^2]	1,4053e-02
AL [m^2/m]	3,2300e+00
It [m^4]	2,7025e-06
Iy [m^4]	3,9875e-03
Iz [m^4]	9,0270e-05
Iw [m^6]	2,2284e-05
alpha [deg]	0,00
Wely [m^3]	7,9750e-03
Welz [m^3]	6,0180e-04
Wply [m^3]	9,3360e-03
Wplz [m^3]	9,5400e-04
cYLCS [mm]	-7
cZLCS [mm]	480
dy [mm]	0
dz [mm]	0
Geometry	
Formcode	1 - I sections
H [mm]	1000
B [mm]	300
t [mm]	20
s [mm]	15

Figure A - 29 Cross beam cross section, Truss 2-Scia Engineer

Stability

Stability is not a problem for the cross beam since the top (compression) flange is laterally supported by the concrete deck all over its length.

Fatigue stresses in the cross beam

Maximum fatigue stress in the cross beam at the critical detail (joint at x= 15 m) $\Delta\sigma_{, nom}$ = 36 N/mm^2 < 52 N/mm^2.

Thus, this cross section provides an acceptable solution for the bottom chord in design Truss 2 as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Result

For the design "Truss 2" satisfies the structural requirements with respect to strength, fatigue and stability of the truss members. The total self-weight of the steel structure is 414 tn.

b3) Truss 3

This design differs from the other two in that the field length is now reduced to 10.5 m instead of 15 m that it was initially. This has been done to improve buckling resistance of compression members. Also the bracing system (laterally) is assumed to be closer oriented (every 10.5 m).

2.b) Decreasing the buckling length of the chord member

In order to reduce the buckling length of the top chord (alternative 2b above) a new truss design (Truss 3, Figure 5.38) is made considering smaller field (panel) length and thus more truss panels and smaller unsupported (in plane and laterally) chord length. The geometrical parameters are chosen to be in this case:

$$\left.\begin{array}{l} L= 105 \text{ m} \\ L_i= 10.5 \text{ m} \\ D= 7 \text{ m} \end{array}\right\} \quad L/D= 15$$

Cross beams c.t.c distance = 3 m
Loads on the cross beams: Edge beams: q (kN/m^2)* 1.5 m (influence length)
All the others: q (kN/m^2)* 3 m (influence length)

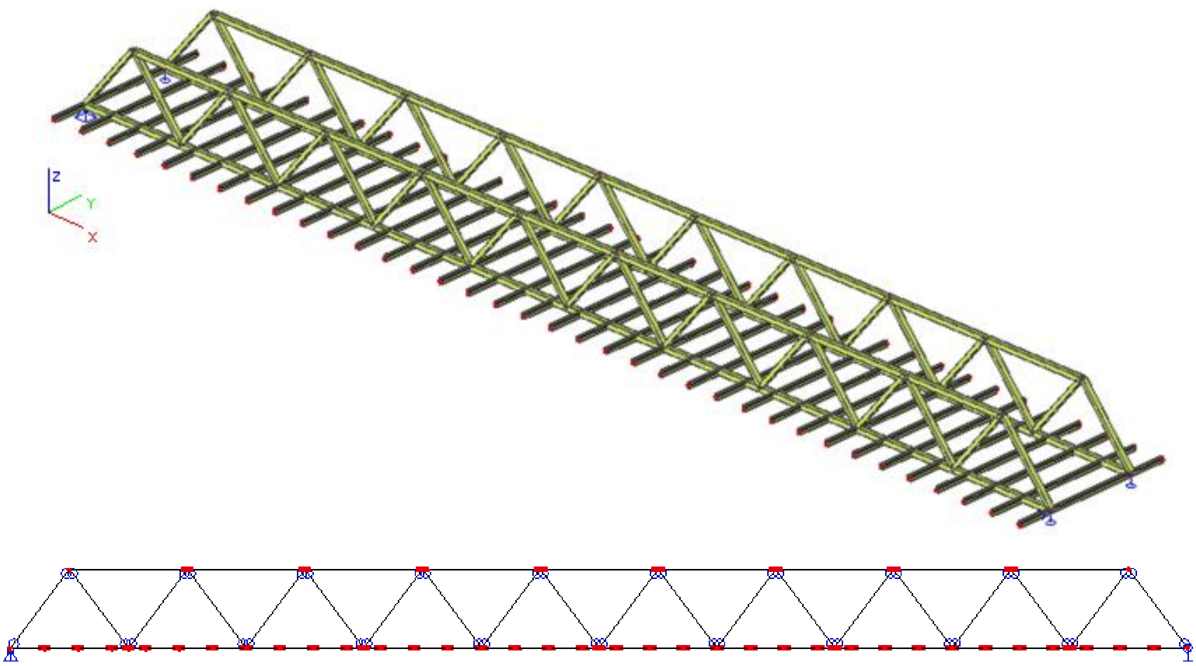


Figure A - 30 Truss 3

Cross sectional dimensions

Brace

Due to reduced member length (8.75 instead of 10.26 m) stresses are somewhat reduces in comparison to "Truss 2" thus, smaller plate thickness is possible: B = H= 450 mm, t_w=t_f= 15 mm, A= 26100 mm^2.

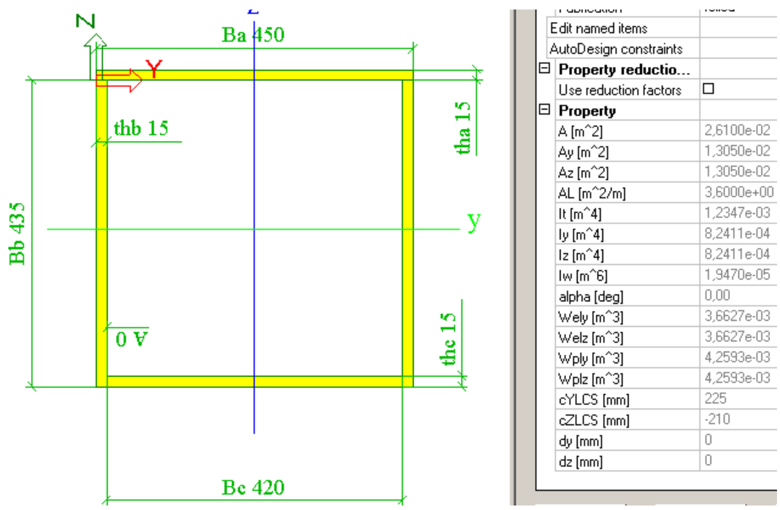


Figure A - 31 Cross beam cross section, Truss 3-Scia Engineer

Buckling calculations for compression brace

Table A - 8 In plane buckling strength end brace, design “Truss 3”

End brace		
Area	A (mm^2)	26100
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	8,24E+08
Radius of gyration	i_y (mm)	177,68
Slenderness	λy	49
Rel. slenderness	λ_E	67
Buckling curve		b
Imperfection factor	α	0,34
Strength	f_{yd} (N/mm^2)	460
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0,73
Phi factor	Φ	0,86
Buckling factor	χ_y	0,76
Buckling strength	$f_{y,b,rd}$ (N/mm^2)	352

Maximum stress in the compression brace is 312 N/mm², so in plane buckling strength is sufficient with steel grade at least S460, U.C= 0.90 < 1 . For out of plane the same strength applies (square brace section).

Table A - 9 Out of plane buckling strength end brace, design “Truss 3”

Rest compression braces		
Area	A (mm^2)	26100
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	8,24E+08
Radius of gyration	i_y (mm)	177,68
Slenderness	λy	49
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0,34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0,64
Phi factor	Φ	0,78
Buckling factor	χ_y	0,81
Buckling strength	$f_{y,b,rd}$ (N/mm^2)	289

Maximum stress in the rest compression brace is 231 N/mm², so out of plane buckling strength is sufficient with steel grade S355, U.C= 0.8 < 1.

Fatigue stresses in the brace members

Maximum fatigue stress in the critical brace (brace 2 at connection with bottom chord) $\Delta\sigma_{, nom}$ = 21 N/mm^2 < 52 N/mm^2.

Thus, this cross section provides an acceptable solution for the braces in design "Truss 3" as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Top chord

For the top chord the dimensions chosen are smaller than in truss 2 design since the unsupported chord length is also smaller (10.5 m in "truss 3" instead of 15 m in "truss 2") B= H= 690 mm, t_w =t_f= 25 mm, A= 66500 mm^2

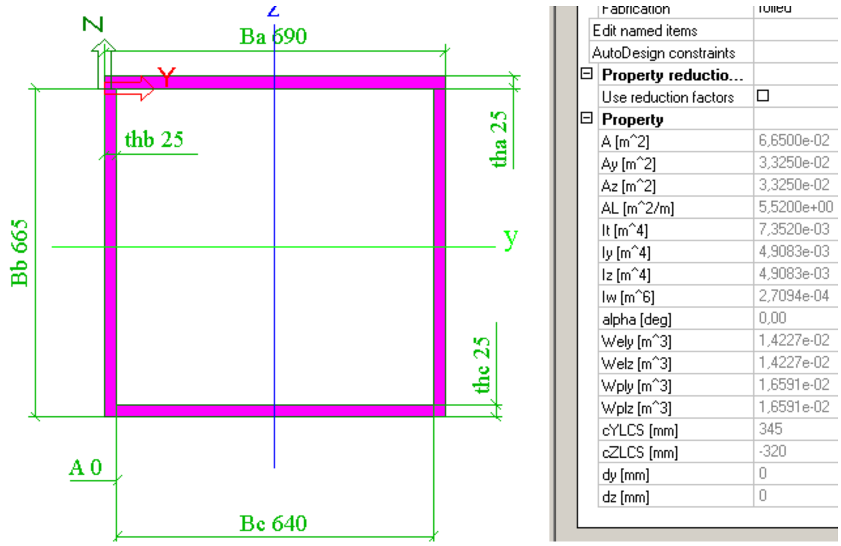


Figure A - 32 Top chord cross section, Truss 3- Scia Engineer.

Buckling calculations for the top chord

Table A - 10 Out of plane buckling strength end brace, design "Truss 3"

Top chord		
Area	A (mm^2)	66500
Length	L_y (mm)	10500
Buckling coefficient	K_y	0,9
Buckling length	l_y (mm)	13500
Moment of Inertia	I_y (mm^4)	4,91E+09
Radius of gyration	i_y (mm)	272
Slenderness	λy	50

Rel. slenderness	λ_E	55
Buckling curve		b
Imperfection factor	α	0,34
Strength	f_{yd} (N/mm^2)	690
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0,91
Phi factor	Φ	1,03
Buckling factor	χ_y	0,66
Buckling strength	$f_{b,rd}$ (N/mm^2)	453

Maximum stress in the truss top chord is 425 N/mm², so in plane buckling strength is sufficient with a steel grade S690 for the top chord. U.C= 0.94 < 1. The out of buckling strength is the same (square section, same in and out of plane buckling length).

Fatigue stresses in the top chord

Maximum fatigue stress in the top chord at the critical detail (bottom fibre in joint 10 close to midspan) $\Delta\sigma_{,\ nom}$ = 21 N/mm^2 << 52 N/mm^2.

Thus, this cross section provides an acceptable solution for the top chord in design “Truss 3” as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Bottom chord

For the bottom chord strength is the governing criterion. So, a section with dimensions B=H =700 mm, t_w= 15 mm, t_f= 20 mm A=47800 mm² provides sufficient strength when using a steel grade S690 (max σ_t = 630 N/mm^2, U.C. =0.91).

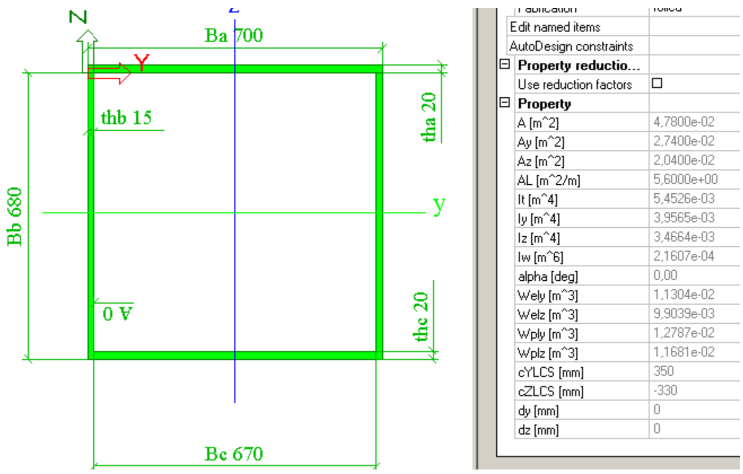


Figure A - 33 Bottom chord cross section, Truss 3- Scia Engineer

Stability

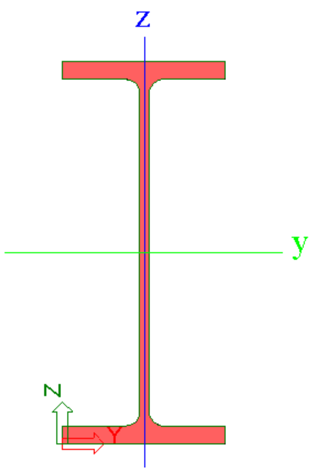
Stability is not a problem for the bottom chord since it is in tension and in bending and the top (compression) flange is laterally supported by the cross beams (every 3 m) and also by the concrete deck (all over its length).

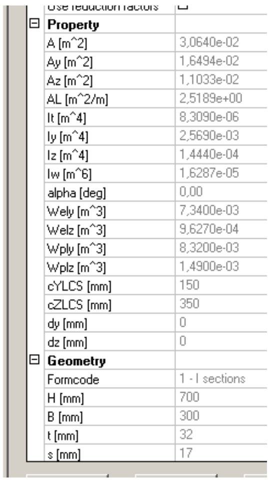
Fatigue stresses in the bottom chord

Maximum fatigue stress in the bottom chord at the critical detail (joint J11 close to midspan) $\Delta\sigma_{, nom} = 47$ N/mm$^2 < 52$ N/mm^2.
Thus, this cross section provides an acceptable solution for the bottom chord in design Truss 3 as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Cross beams

HEB 700 is chosen based on maximum positive bending moments M_{yd} at midspan (maximum bending stress is 192 N/mm^2, U.C =0.54) and fatigue stresses and to ease the connection with the bottom chord.





Property	
A [m^2]	3,0640e-02
Ay [m^2]	1,6494e-02
Az [m^2]	1,1033e-02
AL [m^2/m]	2,5189e+00
It [m^4]	8,3090e-06
Iy [m^4]	2,5690e-03
Iz [m^4]	1,4440e-04
Iw [m^6]	1,6287e-05
alpha [deg]	0,00
Wely [m^3]	7,3400e-03
Welz [m^3]	9,6270e-04
Wply [m^3]	8,3200e-03
Wplz [m^3]	1,4900e-03
cYLCS [mm]	150
cZLCS [mm]	350
dy [mm]	0
dz [mm]	0
Geometry	
Formcode	1 - I sections
H [mm]	700
B [mm]	300
t [mm]	32
s [mm]	17

Figure A - 34 Cross beam cross section, Truss 3-Scia Engineer

Stability is not a problem for the cross beam since the top (compression) flange is laterally supported by the concrete deck all over its length.

Fatigue stresses in the cross beam

Maximum fatigue stress in the cross beam at the critical detail (joint at x= 15 m) $\Delta\sigma_{, nom} = 39$ N/mm$^2 < 52$ N/mm^2.

Thus, this cross section provides an acceptable solution for the cross beam in design Truss 3 as it satisfied all three design criteria (i.e. strength, stability, fatigue).

Result

For the design "Truss 3" satisfies the structural requirements with respect to strength, fatigue and stability of the truss members. The self-weight of the steel structure is 397 tn.

c. Trial arch bridge designs

Structural system

The main superstructure is the two vertical (parallel) planes each of which consists of the arch connected to the main girder with vertical hangers. It is assumed that the connections between the hangers and both the arch and girder are pinned, so no bending moments in the hangers occur.

The main girders in both planes are connected with cross beams with c.t.c distance of 3m over the full length of the bridge. For the arch and girder members box shaped members (welded or RHS) are chosen. The concrete deck is resting on top of the cross beams but not in composite action. In the arch model (made in Scia Engineer) is being treated as a separate load case (dead load LC6), thus acting separately from the main structure.

Analysis and modeling

Elastic global analysis, assuming pin joints for the hangers and loading directly on the cross beams, is performed in order to obtain maximum normal forces in all members (arch, girders, and hangers). In addition the main girders are taken to be stiffer and have bigger dimensions than the arch (parabolic) to achieve no bending moments in the arch under full loading. The modeling of the arch is made using “Scia Engineer”, FEM program. Wind bracing is not included in the model since its presence does not affect the load carrying capacity under vertical loads. However, is theoretically being considered in terms of stability.

Design criteria

Once again, the design criteria are strength, fatigue, and stability.

Loading

Load cases for vertical loads in the truss model (Scia Engineer FEM program)

There are six load cases for vertical loads in the ULS stage (not for fatigue). Two (LC1 and LC6) for permanent loads (dead weight and deck + pavement) and four (LC2, LC3, LC4, LC5) for variable traffic loads according to load model 1 (LM1) and group 1a (gr1a) for traffic loads on roadway bridges (EN 1991-2: 2003). Analytically these are:

LC1: Steel self weight only

LC2: UDL: LM1 (on the roadway)
Numerical values
On edge cross beams is: Lane 1: (9*1.5) =13.5 kN/m
Lane 2: (2.5*1.5) =3.75 kN/m
Remaining part: (2.5* 1.5) =3.75 kN/m
On the others is: Lane 1: (9*3) =27 kN/m
Lane 2: (2.5*3) =7.5 kN/m
Remaining: (2.5* 3) =7.5 kN/m

LC3: q_{lf}: cycle/foot path load (combination value,ψ_0=0.40) on the left side of the bridge only.

Numerical values:
On edge cross beams is: 0.40*2.89*1.5=1.73 kN/m
On the others is: 0.40*2.89*3= 3.47 kN/m

LC4: TS: LM1
Numerical values:
Lane 1: $Q_{s,k,1}$= 300 kN (axle load)
Lane 2: $Q_{s,k,2}$= 200 kN (axle load)

LC5: q_{lf}: cycle/foot path load (combination value) on both sides.
Numerical values:
On edge cross beams is: 0.40*2.89*1.5=1.73 kN/m
On the others is: 0.40*2.89*3= 3.47 kN/m

LC6: Deck + pavement: Assume normal weight concrete deck [EN 1991-1-1].
Deck: Normal weight concrete with thickness t= 240 mm and density γ= 25 kN/m³ (assumed).
Pavement: Mastic asphalt: t= 8 mm and density γ= 20 kN/m³.

For fatigue load model 3 (FLM3) is applied to obtain an indication on maximum and minimum stress values caused by traffic.

LC7: FLM3 according to the EN 1991-2:2003 (see Figure 4).

11.1.1.1.1 .... Load Combinations

One load combinations for vertical loads is considered as the most critical in the ULS stage.

LCB1: 1.35* LC1+ 1.35* LC2+1.35*LC3+ 1.35*LC4, (DL + gr1a, cycle load only on one side of the bridge).

The most critical load cases in case of an arch bridge are full loading and full loading at half the length (see also Figure 5.46 a) and b) respectively). The TS is positioned at half the length of the UDL.

Full loading causes the maximum normal forces (compressive for the arch and tensile for the girders) and the maximum normal forces and stresses (tensile) in the hangers.
Full loading over half the length causes maximum bending moments and maximum normal stresses (bending + normal force) in the arch and in the girders.

Stability

The buckling length of the unsupported arch member is small (about 10 m) in comparison to the case of the truss bridge and the cross sectional area for the arch is quite bigger in comparison to the compression chord for the truss. Therefore, is assumed that buckling resistance is sufficient providing lateral bracing every 10 m. Thus, no calculations are going to be presented for the arch bridge.

Fatigue

The details of an arch bridge of this configuration are considered generally equivalent to the ones in case of the truss bridge. Again rectangular hollow sections are chosen for the main girders and the arch (as for the top and bottom chord) and the same cross beam sections are used. The hangers are also pinned at both ends (as the braces in the truss). Therefore the connections are assumed to be more or less the same for both designs.

Thus, as in the case of the truss girder bridge, considering as minimum strength for the critical detail the strength of fatigue class 71 ($\Delta\sigma$= 71 N/ mm^2) and using $\Delta\sigma$-N curves a maximum stress range level of 52 N/mm^2 (m=3) is allowed, [EN 1993-1-9].

c1) Arch 1

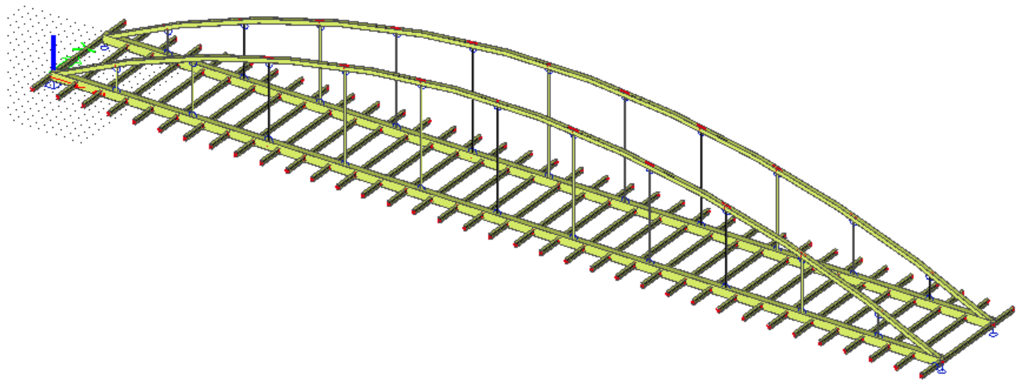


Figure A - 35 Arch 1 geometry.

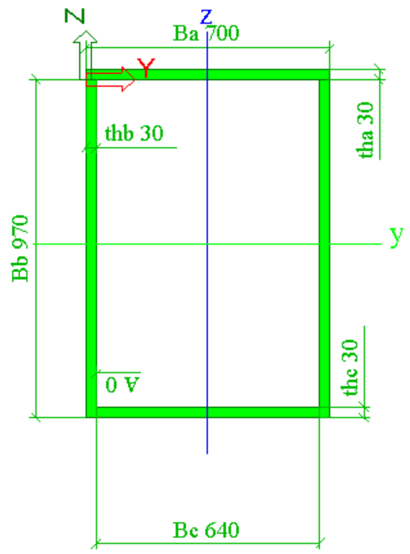
$$\left.\begin{array}{l}L = 105\text{ m}\\ f = 13\text{ m}\end{array}\right\}\quad f/L = 1/8$$

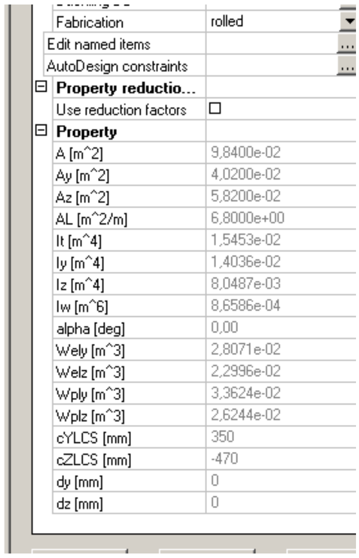
<u>Arch</u>: Parabolic shape, RHS
<u>Bottom main girders</u>: Rectangular hollow sections (RHS)
<u>Hangers</u>: 11 hangers in vertical configuration with c.t.c distance = 9 m (but 7.5 m at both ends of the span)
<u>Cross beams</u>: As in case of the truss bridge, c.t.c distance is 3m
<u>Deck</u>: Concrete deck with thickness 240 mm and 8mm mastic asphalt layer.

The arch bridge is initially considered to be a tied arch bridge (as the one shown in Figure 5.43) with stiffening main girder and more slender parabolic arch. This is preferred in order to gain higher tensile stresses in the bottom of the bridge and reduce as far as possible the bending moments in the arch.

Cross sectional Dimensions

Main Girder





Fabrication	rolled
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AutoDesign constraints	...
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Use reduction factors	☐
Property	
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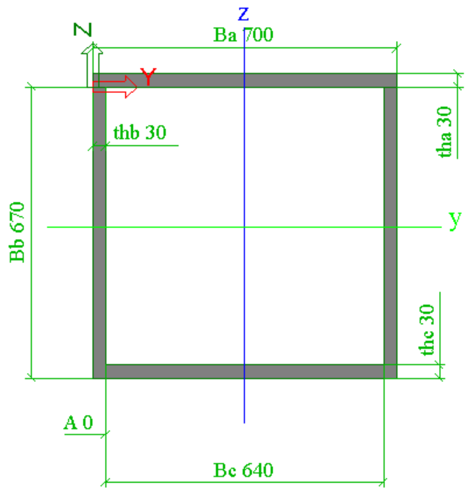
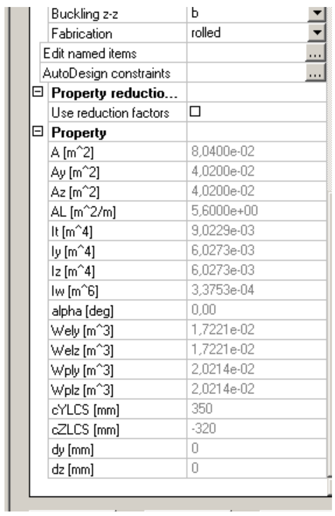
Ay [m^2]	4,0200e-02
Az [m^2]	5,8200e-02
AL [m^2/m]	6,8000e+00
It [m^4]	1,5453e-02
Iy [m^4]	1,4036e-02
Iz [m^4]	8,0487e-03
Iw [m^6]	8,6586e-04
alpha [deg]	0,00
Wely [m^3]	2,8071e-02
Welz [m^3]	2,2996e-02
Wply [m^3]	3,3624e-02
Wplz [m^3]	2,6244e-02
cYLCS [mm]	350
cZLCS [mm]	-470
dy [mm]	0
dz [mm]	0

Figure A - 36 Main girder (bottom member), Arch 1- Scia Engineer

Arch





Buckling z-z	b
Fabrication	rolled
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AutoDesign constraints	...
Property reductio...	
Use reduction factors	☐
Property	
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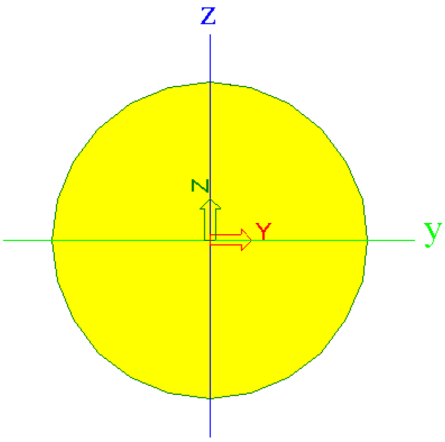
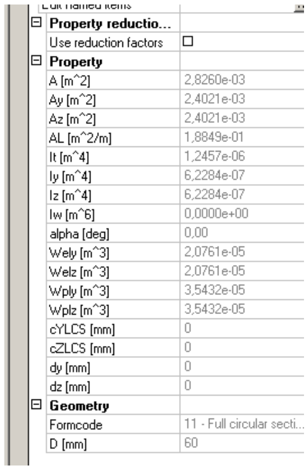
Ay [m^2]	4,0200e-02
Az [m^2]	4,0200e-02
AL [m^2/m]	5,6000e+00
It [m^4]	9,0229e-03
Iy [m^4]	6,0273e-03
Iz [m^4]	6,0273e-03
Iw [m^6]	3,3753e-04
alpha [deg]	0,00
Wely [m^3]	1,7221e-02
Welz [m^3]	1,7221e-02
Wply [m^3]	2,0214e-02
Wplz [m^3]	2,0214e-02
cYLCS [mm]	350
cZLCS [mm]	-320
dy [mm]	0
dz [mm]	0

Figure A - 37 Arch (top member), Arch 1- Scia Engineer

Hangers





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Use reduction factors	☐
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Ay [m^2]	2,4021e-03
Az [m^2]	2,4021e-03
AL [m^2/m]	1,8849e-01
It [m^4]	1,2457e-06
Iy [m^4]	6,2284e-07
Iz [m^4]	6,2284e-07
Iw [m^6]	0,0000e+00
alpha [deg]	0,00
Wely [m^3]	2,0761e-05
Welz [m^3]	2,0761e-05
Wply [m^3]	3,5432e-05
Wplz [m^3]	3,5432e-05
cYLCS [mm]	0
cZLCS [mm]	0
dy [mm]	0
dz [mm]	0
Geometry	
Formcode	11 - Full circular secti...
D [mm]	60

Figure A - 38 Vertical hangers, Arch 1- Scia Engineer

Stresses

Normal stresses
Arch: max σ = - 405 N/mm^2 (under half-length loading)
Girder: max σ = + 442 N/mm^2 (under half-length loading)
Hangers: max σ =+ 473 N/mm^2 (in hanger 6-center-, under full loading and TS in midspan)

Fatigue stresses (top fiber)
Arch: max Δσ = 57 N/mm^2 (at about L/4)
Girder: max Δσ = 83N/mm^2 (at about L/4)
Hangers: max Δσ = 32N/mm^2 (in hanger 5 and 6-center-)

Not sufficient. Very high fatigue stresses.

What happens when only decreasing the arch rise at 10 m?

$$\left.\begin{array}{l} L = 105 \text{ m} \\ f = 10 \text{ m} \end{array}\right\} \quad f/L = 1/10$$

Stresses

Normal stresses
Arch: max σ = - 458 N/mm^2 (under half-length loading)
Girder: max σ = +488 N/mm^2 (under half-length loading)
Hangers: max σ = +480 N/mm^2 (in hanger 6-center-, under full loading)

Fatigue stresses (top fiber)
Arch: max Δσ = 57 N/mm^2 (at about L/4)

Girder: max $\Delta\sigma$ = 83N/mm^2 (at about L/4)
Hangers: max $\Delta\sigma$ = 33N/mm^2 (in hanger 5 and 6-center-)

It does not improve the fatigue stresses at all but increases the stresses in the members to allow for use of higher steel grades. However fatigue is still the governing factor.
As a next step bigger arch rise decrease (f= 10 m) with simultaneous increase of the cross sectional dimensions for the main girders is attempted.
The girder cross section becomes now:

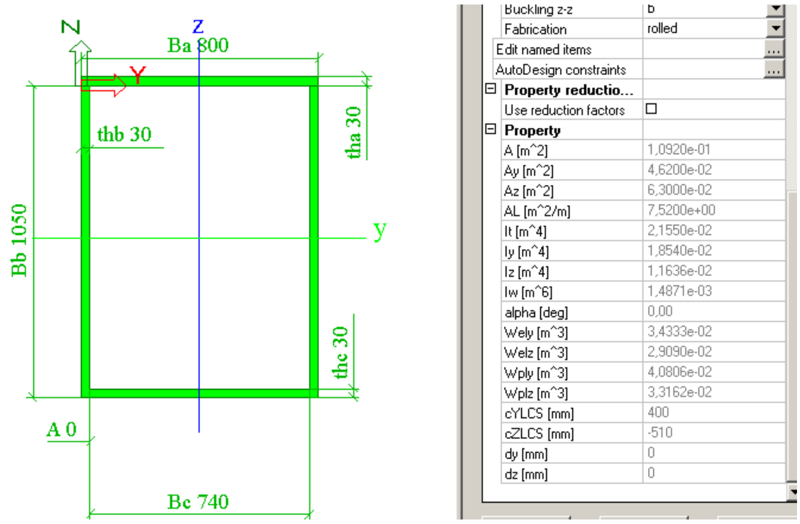


Figure A - 39 New girder cross section for “Arch 1”, with f= 10 m.

Stresses

Normal stresses
Arch: max σ = - 424 N/mm^2 (under half-length loading)
Girder: max σ = +437 N/mm^2 (under half-length loading)
Hangers: max σ = +470 N/mm^2 (in hanger 6-center-, under full loading)
Fatigue stresses (top fiber)
Arch: max $\Delta\sigma$ = 47 N/mm^2 (at about L/4)
Girder: max $\Delta\sigma$ = 72 N/mm^2 (at about L/4)
Hangers: max $\Delta\sigma$ = 28 N/mm^2 (in hanger 5 and 6-center-)

Fatigue stresses are still quite high, caused mainly by bending. So let’s try to increase the section modulus of the cross section by again increasing the height of the girder from 1080 mm to 1500 mm.

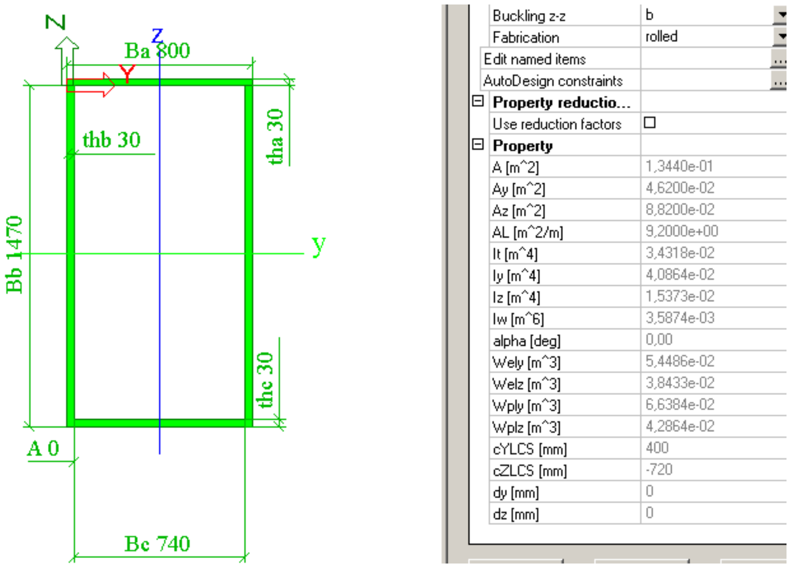


Figure A - 40 Increased main girder height, even more, to deal with high fatigue stresses mainly caused by bending, Arch 1 when f=10 m.

Stresses

Normal stresses

Arch: max σ = - 353 N/mm^2 (under half-length loading)
Girder: max σ = +348 N/mm^2 (under half-length loading)
Hangers: max σ = +450 N/mm^2 (in hanger 6-center-, under full loading)

Fatigue stresses (top fiber)

Arch: max Δσ = 26 N/mm^2 (at about L/4)
Girder: max Δσ = 53 N/mm^2 (at about L/4)
Hangers: max Δσ = 22 N/mm^2 (in hanger 5 and 6-center-)

Increasing the girder cross section the fatigue stresses are decreased but also the static stresses decrease and that does not favor the use of high strength steel grades (e.g. S690). However the stiffness of the arch in this case is bigger than the stiffness of the main girder in contrast to an arch type with a stiffening girder proposed at the beginning.
What about changing the c.t.c distance of the hangers to reduce the bending moments in the main girders? This has been done considering another design namely, "Arch 2".

C2) Arch 2

$$\left.\begin{array}{l} L = 105\text{ m} \\ f = 13\text{ m} \end{array}\right\} \quad f/L = 1/8$$

Arch: Parabolic shape, RHS
Bottom main stiffening girders: RHS sections
Hangers: 14 hangers in vertical configuration with c.t.c distance = 7.5 m.
Cross beams: As in case of the truss bridge, c.t.c distance is 3m

Deck: Concrete deck with thickness 240 mm and 8mm mastic asphalt layer.

The only difference in the arch geometry between arch bridges 1 (Figure 37) and 2 (Figure 43) is the distance between the hangers. By decreasing just the distance between the hangers allows as to use smaller hanger diameter in order to also decrease the stiffness ratio between the hangers and the main girder. However, in order to keep the fatigue stresses at low levels the arch cross section has been increased (stiffness ratio arch/girder has been increased) while the hanger diameter has been reduced (stiffness ratio hangers/girder has been reduced).

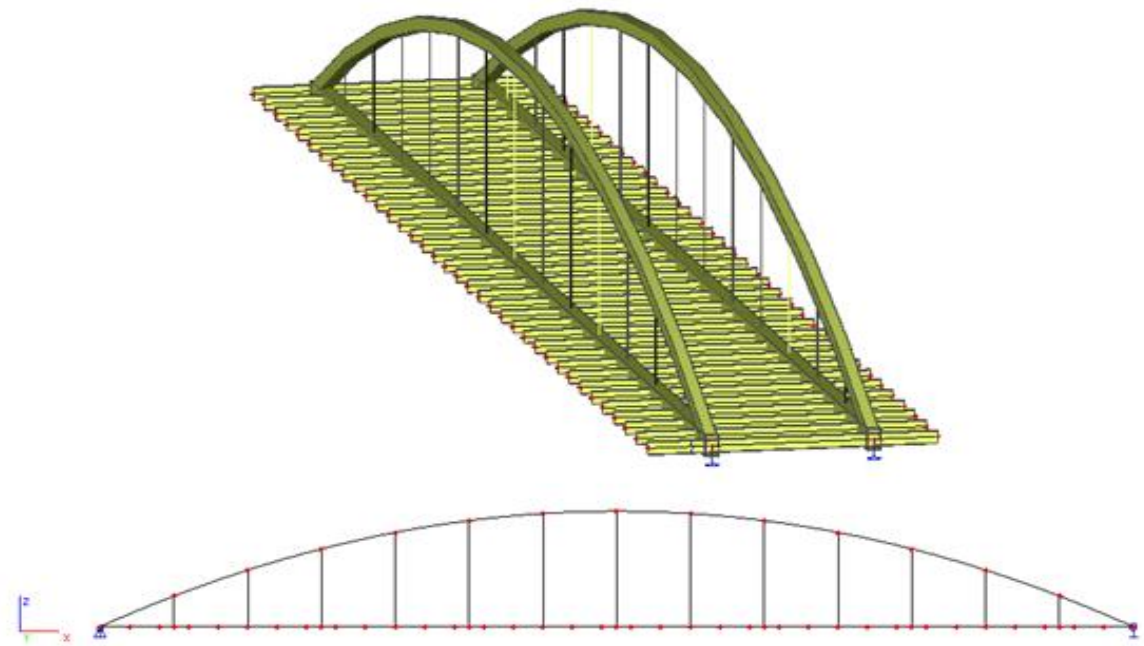
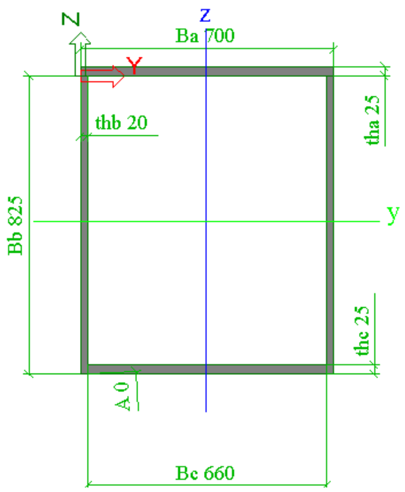


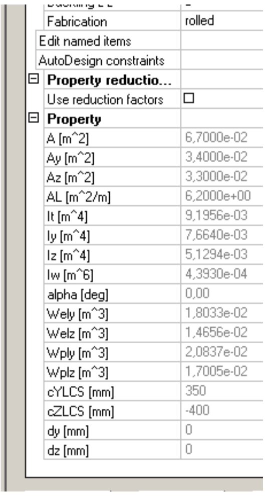
Figure A - 41 Arch 2 geometry

Cross sectional dimensions

Arch

B= 700 mm, H= 850 mm, A= 67000 mm^2, t_f= 25 mm, t_w= 20 mm





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Use reduction factors	☐
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Ay [m^2]	3,4000e-02
Az [m^2]	3,3000e-02
AL [m^2/m]	6,2000e+00
It [m^4]	9,1956e-03
Iy [m^4]	7,6640e-03
Iz [m^4]	5,1294e-03
Iw [m^6]	4,3930e-04
alpha [deg]	0,00
Wely [m^3]	1,8033e-02
Welz [m^3]	1,4656e-02
Wply [m^3]	2,0837e-02
Wplz [m^3]	1,7005e-02
cYLCS [mm]	350
cZLCS [mm]	-400
dy [mm]	0
dz [mm]	0

Figure A - 42 Arch, Arch 2- Scia Engineer

Main girder

B= 800 mm, H= 1400 mm, A= 128400 mm^2, t_f= 30 mm, t_w= 30 mm

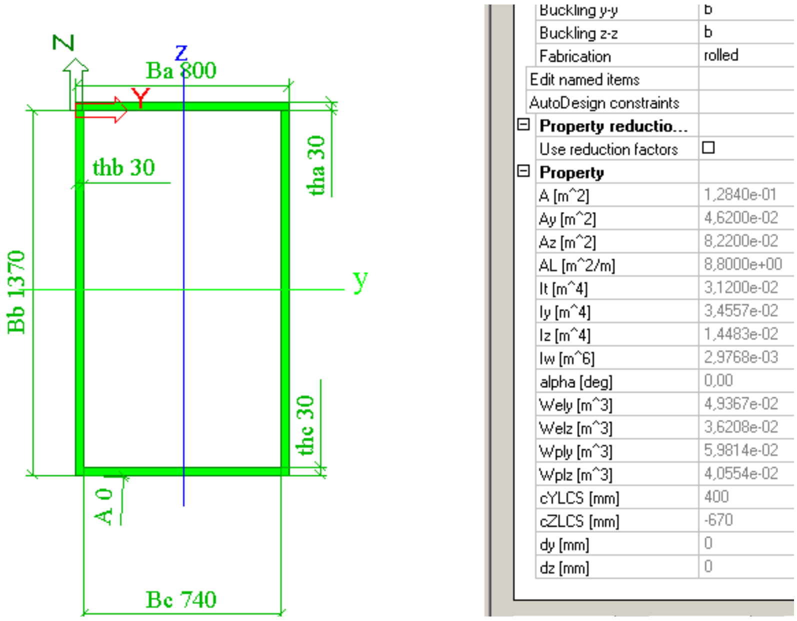


Figure A - 43 Main Girder, Arch 2- Scia Engineer

Hangers

D= 48 mm, A=1809 mm^2

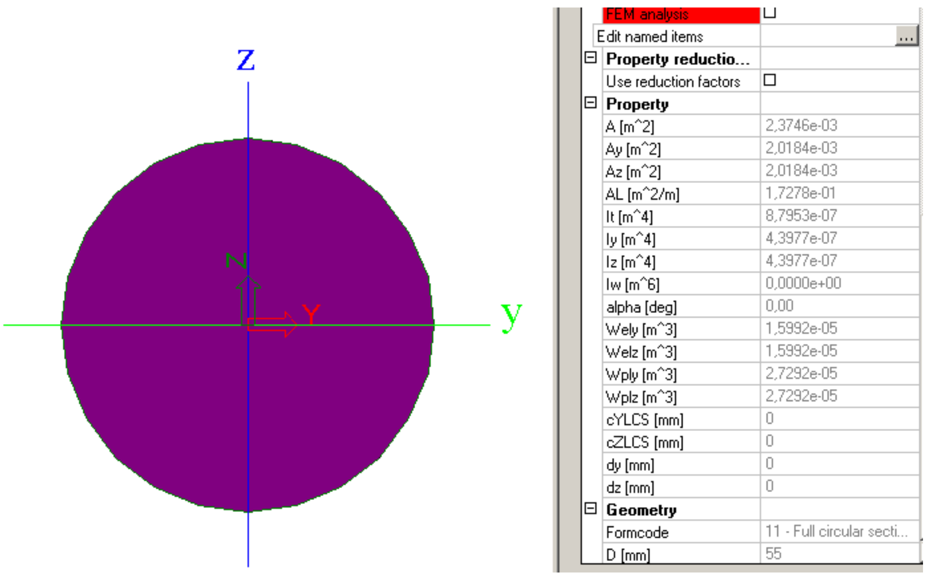


Figure A - 44 Hangers (RD55), Arch 2- Scia Engineer

Stresses

Normal stresses

Arch: max σ = - 363 N/mm^2 (under half-length loading)

Girder: max σ = +321 N/mm^2 (under half-length loading)
Hangers: max σ = +565 N/mm^2 (in hanger 6-center-, under full loading)

Fatigue stresses (top fiber)
Arch: max $\Delta\sigma$ = 27 N/mm^2 (at about L/4)
Girder: max $\Delta\sigma$ = 38 N/mm^2 (bottom fiber at about L/4)
Hangers: max $\Delta\sigma$ = 22 N/mm^2 (in hanger 6-center-)

The fatigue stresses have been reduced to acceptable levels but the static stresses in the arch and in the main girders are very small. However, the hangers (heavily loaded ones) can be made out of high strength steel S690.

Trying again altering the cross sectional dimensions and recalculating normal and fatigue stresses.

Main Girder

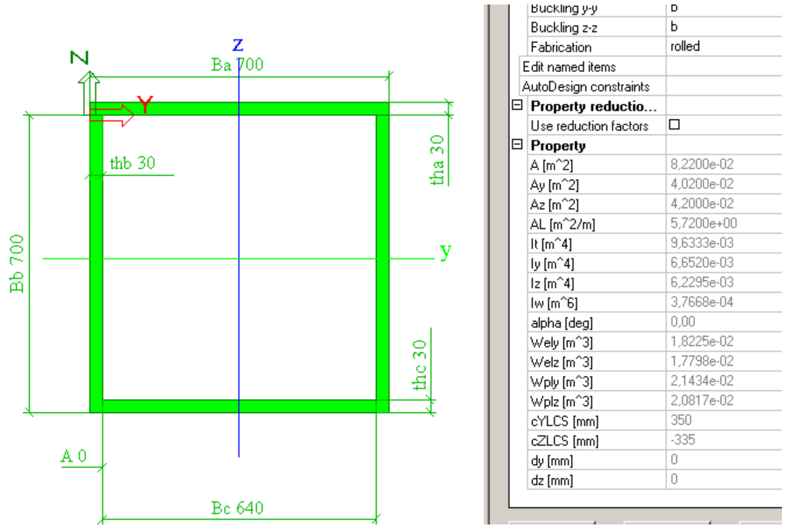


Figure A - 45 Main Girder, Arch 2- Scia Engineer

Arch

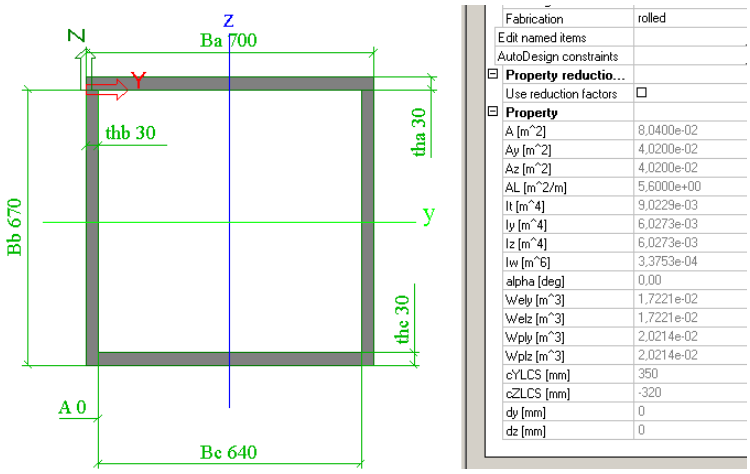
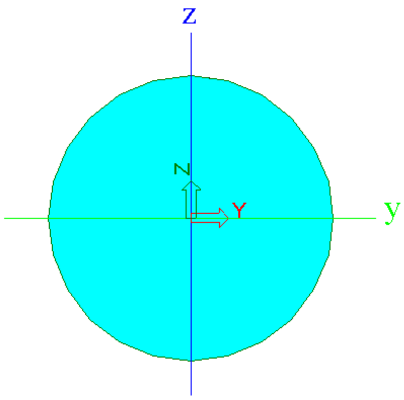


Figure A - 46 Figure 11 Arch, Arch 2- Scia Engineer

Hangers



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Use reduction factors	☐
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Ay [m^2]	1,0676e-03
Az [m^2]	1,0676e-03
AL [m^2/m]	1,2566e-01
It [m^4]	2,4606e-07
Iy [m^4]	1,2303e-07
Iz [m^4]	1,2303e-07
Iw [m^6]	0,0000e+00
alpha [deg]	0,00
Wely [m^3]	6,1515e-06
Welz [m^3]	6,1515e-06
Wply [m^3]	1,0498e-05
Wplz [m^3]	1,0498e-05
cYLCS [mm]	0
cZLCS [mm]	0
dy [mm]	0
dz [mm]	0
Geometry	
Formcode	11 - Full circular secti...
D [mm]	40

Figure A - 47 Hangers, Arch 2- Scia Engineer

Stresses

Normal stresses
Arch: max σ = - 473 N/mm^2 (under half-length loading)
Girder: max σ = +536 N/mm^2 (under half-length loading)
Hangers: max σ = +637 N/mm^2 (in hanger 6-center-, under full loading)

Fatigue stresses (top fiber)
Arch: max $\Delta\sigma$ = 85 N/mm^2 (at about L/4)
Girder: max $\Delta\sigma$ = 93 N/mm^2 (at about L/4)
Hangers: max $\Delta\sigma$ = 54 N/mm^2 (in hanger 5 and 6-center-)

The fatigue stresses remain high. Once again, the cross sections are changed to deal with (try to reduce) fatigue stresses. In addition c.t.c distance between the hangers is reduced to 6 m (Figure 123).

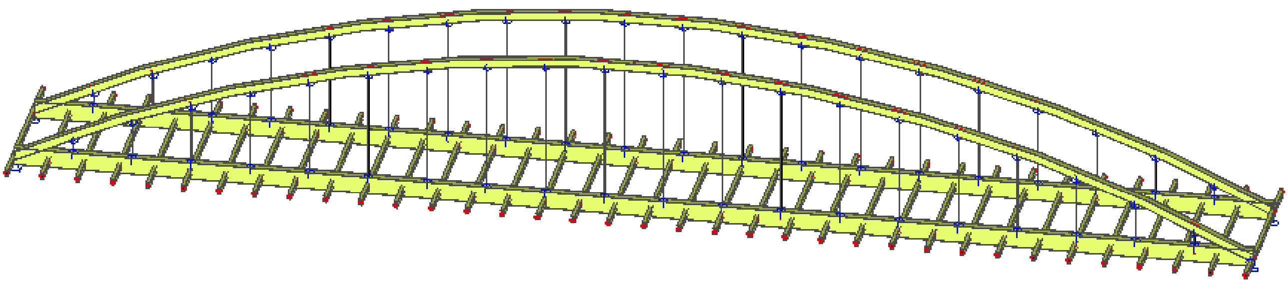


Figure A - 48 Arch 2 with c.t.c distance between hangers 6 m

Main girder

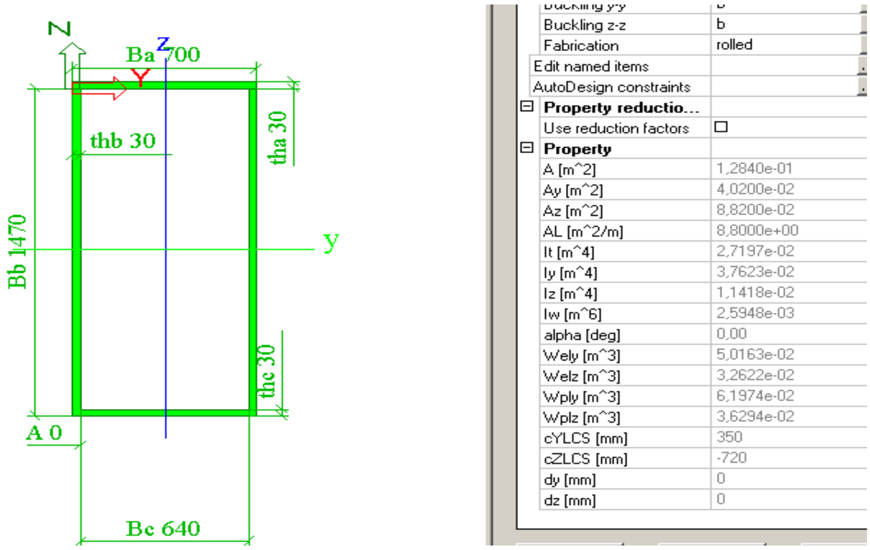


Figure A - 49 Main Girder, Arch 2- Scia Engineer

Arch

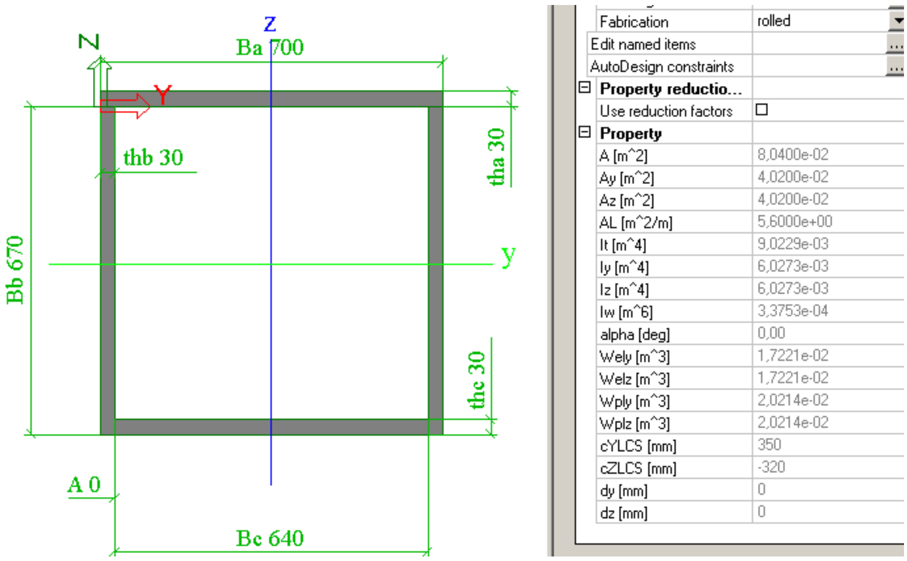


Figure A - 50 Main Girder, Arch 2- Scia Engineer

Hangers

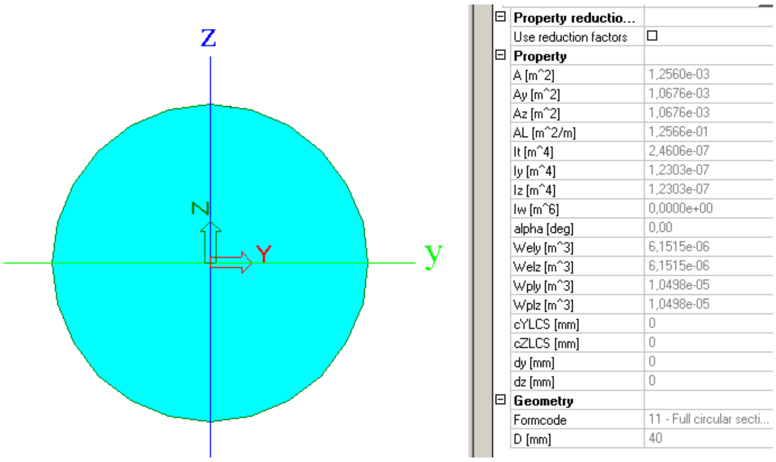


Figure A - 51 Hangers, Arch 2 - Scia Engineer

Stresses

<u>Normal stresses</u>
Arch: max σ = - 272 N/mm^2 (under half-length loading)
Girder: max σ = +333 N/mm^2 (under half-length loading)
Hangers: max σ = +563 N/mm^2 (in hanger 11-center-, under full loading)

<u>Fatigue stresses (top fiber)</u>
Arch: max Δσ = 26 N/mm^2 (at about L/4)
Girder: max Δσ = 56 N/mm^2 (at about L/4)
Hangers: max Δσ =29 N/mm^2 (in hanger 10 and 11-midspan-)

Result

High fatigue stresses, mainly in the main girder due to in plane bending, limit also the static stress level at quite low values for HSS. Thus, an arch bridge, with this configuration, for the 'Schellingwouderbrug' cannot be considered suitable for using high strength steel grades.

Appendix B: Detailed truss bridge design

This Appendix contains four truss designs for the 'Schellingwouderbrug' with altered cross sections (types, dimensions) and/or different steel grades. The overall bridge geometry remains always the same.

1) The design of typical truss connections of "Truss 3" hybrid design (i.e. S690 in the chord members, S355 in the braces and cross beams) chosen in the preliminary phase. Results and proposed alternatives for design of connections in this design are also given.
2) An equivalent truss design using RHS members (and gusset plates) and S355 steel grade only. Estimation of dimensions and total steel weight is made (for comparison).
3) A hybrid truss design (i.e. S690 in the chord members, S355 in the braces and cross beams) using CHS members and assuming cast joints for the connections, is made based on strength and stability checks for the truss members. Cast steel joints are assumed for the connections (for comparison).
4) An equivalent truss design using CHS members with cast joints and S355 steel grade only. Estimation of dimensions and total steel weight is made (for comparison).

1) Connection design of "Truss 3" hybrid design with RHS members (with typical truss joints)

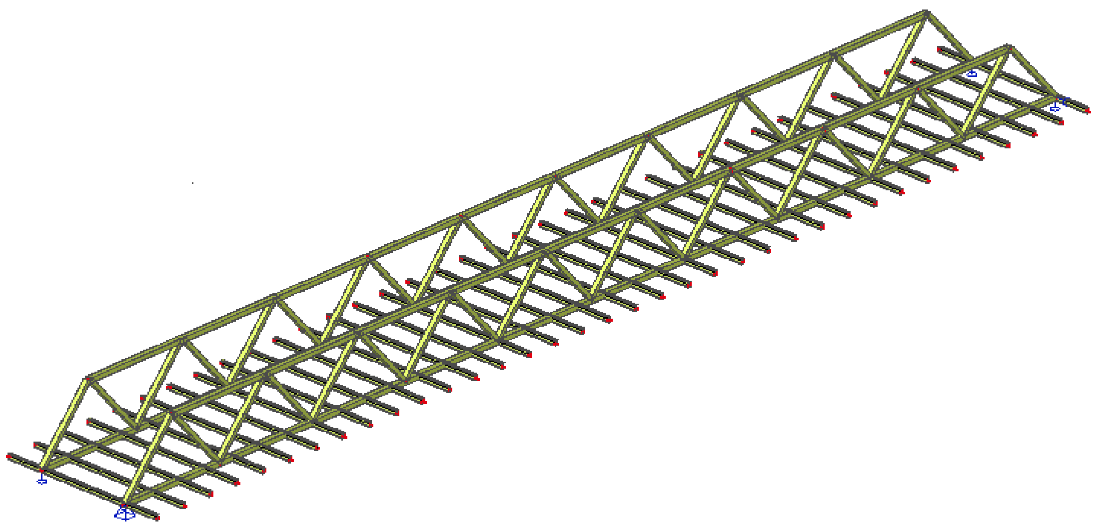
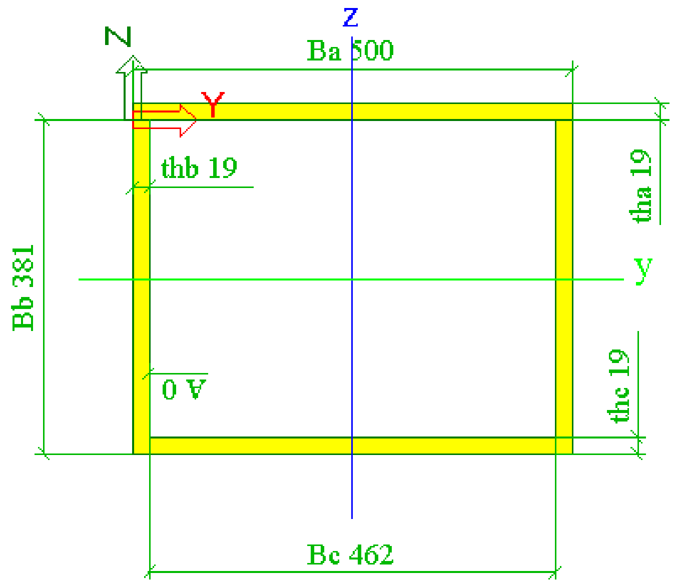


Figure B - 1 Truss 3 design with RHS members

1.a) Cross sectional dimensions

In order to satisfy the requirements for sufficient strength and rotational capacity of the connections and to be able to verify the connections based on formulas given in EN 1993-1-8, the cross sections of the truss bridge have been modified for the detailed phase, initially, to the ones shown below:

Braces



AutoDesign constraints	
⊟ **Property reductio...**	
Use reduction factors	☐
⊟ **Property**	
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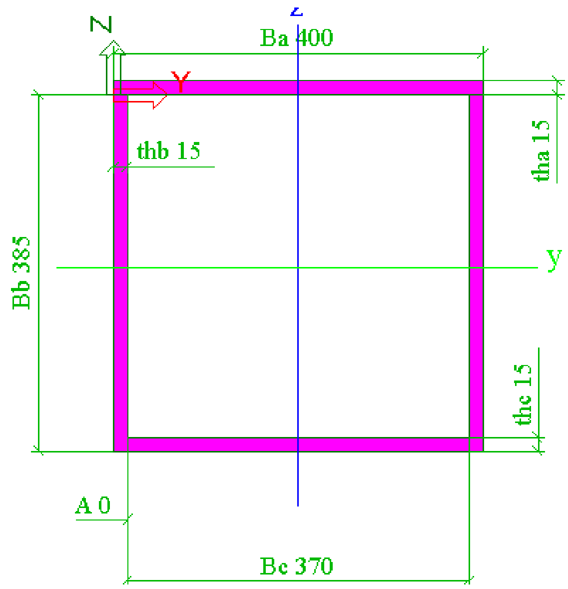
Ay [m^2]	1,8278e-02
Az [m^2]	1,4478e-02
AL [m^2/m]	3,6000e+00
It [m^4]	1,4805e-03
Iy [m^4]	8,4031e-04
Iz [m^4]	1,1919e-03
Iw [m^6]	2,2919e-05
alpha [deg]	0,00
Wely [m^3]	4,2015e-03
Welz [m^3]	4,7676e-03
Wply [m^3]	4,8644e-03
Wplz [m^3]	5,6833e-03
cYLCS [mm]	250
cZLCS [mm]	-181
dy [mm]	0
dz [mm]	0

Figure B - 2 Truss 3-detailed design-End brace (500x400x19)



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AutoDesign constraints	
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Use reduction factors	☐
⊟ **Property**	
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Ay [m^2]	1,1550e-02
Az [m^2]	1,1550e-02
AL [m^2/m]	3,2000e+00
It [m^4]	8,5600e-04
Iy [m^4]	5,7153e-04
Iz [m^4]	5,7153e-04
Iw [m^6]	1,0573e-05
alpha [deg]	0,00
Wely [m^3]	2,8577e-03
Welz [m^3]	2,8577e-03
Wply [m^3]	3,3367e-03
Wplz [m^3]	3,3367e-03
cYLCS [mm]	200
cZLCS [mm]	-185
dy [mm]	0
dz [mm]	0

Figure B - 3 Truss 3-detailed design-Other braces (400x400x15)

Top chord

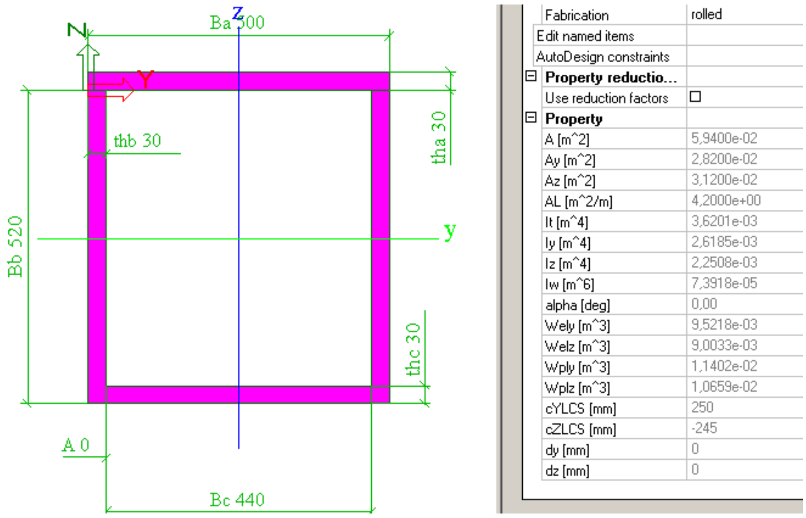


Figure B - 4 Truss 3-detailed design-Top chord (500x550x30)

Bottom chord

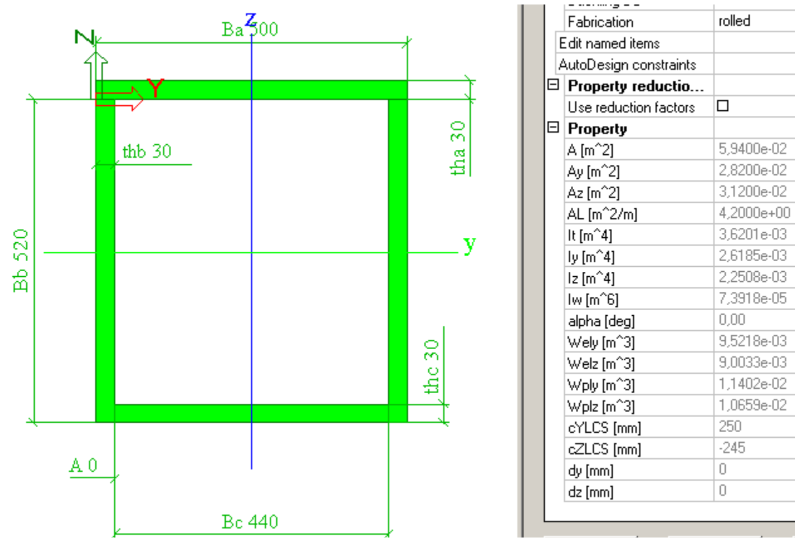


Figure B - 5 Truss 3-detailed design-Bottom chord (50x550x30)

Table B - 1 Cross sectional dimensions for the truss members in preliminary and detailed phase

Member	Preliminary phase	Detailed phase
Bottom chord	700x700 t_w= 15,t_f=20	500x550x30
Top chord	650x650x20	500x550x30
End brace	450x400x15	500x400x19
Rest braces	450x400x15	400x400x15

From Table B-1 it can be seen that the cross sections in the detailed phase have been designed more compact in comparison to the preliminary phase.
That ensures that the sections have adequate plastic deformation capacity and sufficient rotation capacity can also be achieved, thus, locally redistribution of stresses can take place. This is particularly important as hinged connections are assumed for analysis. It also assures that buckling of plate elements needs not to be checked as a failure mode.

1.b) Critical joints

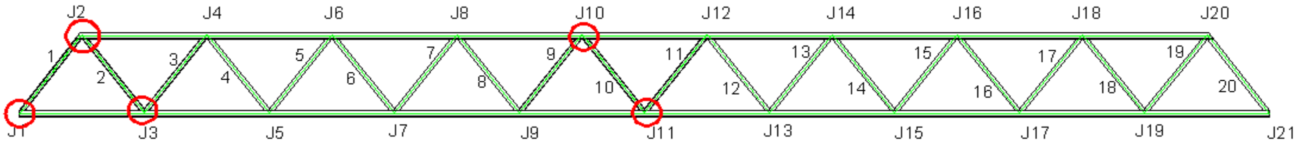


Figure B - 6 Numbering of joints and brace members per truss plane

The most critical locations (in red circles in Figure 6) for each member have been found and checked only, in order to simplify the design procedure.
By "critical" it is meant that, only the joints/connections at the most heavily loaded member location (i.e. location on the bridge where each member must resist its maximum static load) and fatigue sensitive location (i.e. location on the bridge where each member must resist its maximum fatigue load or location with smaller fatigue load but worse fatigue detail class) have been checked.
Joint type (Y- , K-gap-, K- overlap- joint) depends on the load transfer between the members in the joint (Cidect design guide 3 [3]).
For the calculations below, internal (maximum) forces, moments and stresses due to loading are taken from Scia file "Truss 3_RHS.esa" for strength verifications, and from Scia file "Truss 3_RHS_FLM4.esa" for fatigue check.

Strength

Strength (design resistance) calculations are done according to formulas for RHS members in EN 1993-1-8 (design of connections). In each case, the minimum design axial resistance is considered as the joint resistance. As high strength steel is used (S690) a reduction factor 0.8 is applied on all the design resistances independent of the failure mode [EN 1993-1-12].

Fatigue

For fatigue verification, use of nominal values and fatigue details included in table 8.7 in EN 1993-1-9 is not possible. Therefore, the hot spot stress is calculated by calculating the SCFs for each joint at different locations (toe, heel, crown etc.). Formulas for calculating the SCFs are found in Cidect design guide 8 [5]. Analytical calculations can be found in the excel file "RHS_ fatigue calculations SCFs.xlsx". Below the procedure followed for fatigue is given:

Procedure for fatigue with the hot spot stress method

(See also Figure 7 for tables and graphs used in Cidect design guide 8 [5])

1. Using Scia engineer FLM4 is applied as movable load on the bridge.

2. Nominal ranges (forces (ΔN) and bending moments (ΔM)) are collected for each member at each critical joint, caused by each vehicle separately.

3. Nominal stress ranges ($\Delta\sigma_{nom}$) are calculated from these values.

4. Stress concentrations factors (SCFs) are then calculated, separately for axial forces and bending moments, for each critical joint at different joint locations. Relevant analytical formulas and/or graphs are found in Cidect design guide 8, Appendix E (i.e. Table E.1 for RHS Y-joints, Table E.2 for RHS K-gap joints and Table E.3 for RHS K-overlap joints).

5. Hot spot stress ranges ($\Delta\sigma_{hs}$) are calculated. Relevant formulas are found again in Cidect design guide 8. General formula is: $\Delta\sigma_{hs} = SCF*\Delta\sigma_{nom}$.

6. Design fatigue life is chosen. For bridges, customary, is 100 years.

7. Traffic category for the specific bridge is chosen from table 4.5 in EN 1991-2 to be $0.5*10^6$ cycles per year per slow lane.

8. Total number of cycles is calculated $100*(0.5*10^6)$.

9. Traffic type is chosen as medium distance (column 5) from table 4.7 in EN 1993-1-9.

10. The number of cycles per vehicle (n_i) in FLM4 is calculated: lorry percentage (previous step)* total number of cycles (from step 8).

11. Relevant hot spot S-N curve is chosen. In this case it was not clear what the most suitable S-N curve is. This is because in Cidect 8 hot spot S-N curves cover plate thicknesses up to t= 16 mm for RHS members. In our case the design has maximum thickness 30 mm.
    Therefore it was decided to use the hot spot S-N curve for CHS with t= 32 mm from table 3.2 in Cidect 8 (Figure 7), in order to get a feeling on the behavior of these connections under fatigue loading.
    It must always kept in mind however, that in Eurocode RHS truss joints are classified in a lower fatigue class than the equivalent joints with CHS members.

12. Available fatigue life (N_f) is then calculated based on formulas in table 3.1 for t= 30 mm but also for t = 32 mm (according to formulas in table 3.1 in Figure 7). This is done to examine the effect of plate thickness in the formulas for the allowable number of cycles. However the results are the same for both plate thicknesses due to the very small difference.

13. Finally Miner's rule for total damage calculation is applied. Must $D= \Sigma (n_i/N_{f,i}) \leq 1$.

Note: In each joint the RHS member with higher fatigue stresses is checked in order to estimate the behavior of the connection under fatigue loading. In practice though all members and all different locations in each connection should be checked separately for fatigue.

Table 3.1 – Equations for the S_{rhs}-N_f curves for CHS joints (4 mm ≤ t ≤ 50 mm) and RHS joints (4 mm ≤ t ≤ 16 mm)

for $10^3 < N_f < 5 \cdot 10^6$	$\log(S_{rhs}) = \frac{1}{3} \cdot (12.476 - \log(N_f)) + 0.06 \cdot \log(N_f) \cdot \log\left(\frac{16}{t}\right)$ or $\log(N_f) = \dfrac{12.476 - 3 \cdot \log(S_{rhs})}{1 - 0.18 \cdot \log\left(\frac{16}{t}\right)}$
for $5 \cdot 10^6 < N_f < 10^8$ (variable amplitude only)	$\log(S_{rhs}) = \frac{1}{5} \cdot (16.327 - \log(N_f)) + 0.402 \cdot \log\left(\frac{16}{t}\right)$ or $\log(N_f) = 16.327 - 5 \cdot \log(S_{rhs}) + 2.01 \cdot \log\left(\frac{16}{t}\right)$

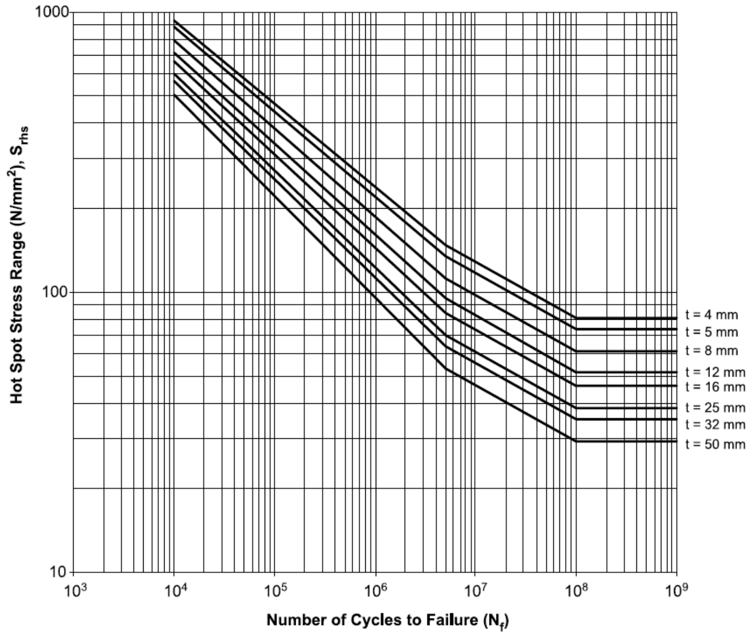


Figure 3.3 – Fatigue strength curves for CHS joints (4 mm ≤ t ≤ 50 mm) and RHS joints (4 mm ≤ t ≤ 16 mm) according to the hot spot stress method

Table 3.2 – The Constant Amplitude Fatigue Limit and Cut-Off Limit in Figure 3.3

Section Type	Thickness (mm)	Constant Amplitude Fatigue Limit (N/mm²)	Cut-Off Limit (N/mm²)
CHS & RHS	4	147	81
	5	134	74
	8	111	61
	12	95	52
	16	84	46
	25	71	39
CHS	32	64	35
	50	53	29

Figure B - 7 Tables 3.1 and 3.2 and S-N curves for the hot spot stress method for fatigue verifications in Cidect design guide 8.

1.c) Strength and fatigue calculations

Critical location 1: Connection between bottom chord and end brace member

This joint is checked for strength (all connected members) and for fatigue (with respect to the brace member). The end brace is the most heavily loaded compression brace member on the bridge.

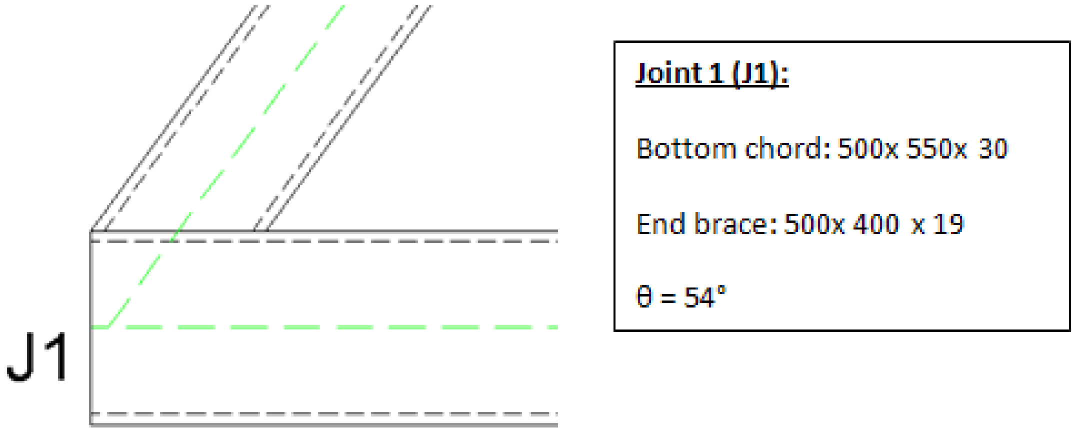


Figure B - 8 Critical Joint 1 configuration, Y- joint

Strength

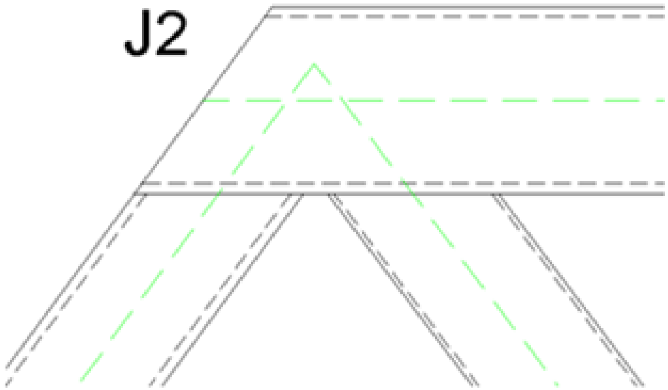
Formulas according to table 7.11 in EN 1993-1-8, using an additional reduction factor 0.8.
Additional parameters to be used for the formulas: β =1, k_n=1, η=h_i/b_0 =0.8, $\sin\theta_1$ =0.81
End brace (S460): max $N_{1, Ed}$ = 7365 kN (compression)
Bottom chord (S690): max $N_{0, b, Ed}$ =4386 kN (tension)
Axial joint resistance: min $N_{1, Rd}$ = 11826 kN (Punching shear)
U.C. = 0.62 <1 O.K. (checked for maximum normal force in the joint, i.e. $N_{i, Ed}$ =7365 kN for the end brace)

Fatigue

The end brace is checked with respect to fatigue (see also fatigue procedure above and excel file “RHS_ fatigue calculations SCFs.xlsx” for detailed numbers and calculations). Axial and in plane bending SCFs at brace toe location (due to axial load and secondary bending moments in the brace) are 3.1 and 3.3 respectively (a factor 1.4 is also included in these values to account for fillet welds).
Total damage: D = 12.2 >>1
The joint is not sufficient for the design fatigue life of $5*10^7$ cycles.

Critical location 2: Connection between top chord, end brace and brace 2 members

This joint is checked for strength (all members) and for fatigue (with respect to the end brace member). Brace 2 is the most heavily loaded tension brace member.



Joint 2 (J2):
Top chord: 500x 550x 30
End brace: 500x 400x 19
Brace 2: 400x 400x 15
Gap g= 70 mm
Eccentricity e= + 108 mm
θ_1= 54°
θ_2= 53°

Figure B - 9 Critical joint 2 configuration, K- gap joint

Strength

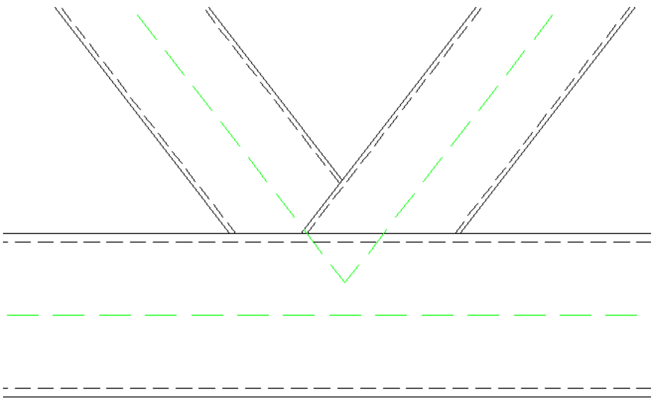
Formulas according to table 7.12 in EN 1993-1-8, using an additional reduction factor 0.8.
Additional parameters: $\beta = 0.85$, $\gamma = 8.3$, $k_n = 1$, $\sin\theta_1 = 0.81$, $\sin\theta_2 = 0.80$
Also, for chord shear failure mode:
$V_{Ed} = 185$ kN
$V_{pl,Rd} = (A_{v0}*(f_{y0}/\sqrt{3}))/\gamma_{M0} = 11951$ kN
$A_{v0} = A_0 - 2*h_{w0}*t_{w0} = 30000$ mm^2
$A_0 = 59400$ mm^2
End brace (S460): max $N_{1,Ed} = 7365$ kN (compression)
Axial resistance: min $N_{1,Rd} = 12054$ kN (Brace failure)
Brace 2 (S460): max $N_{2,Ed} = 7585$ kN (tension)
Axial resistance: min $N_{2,Rd} = 8501$ kN (Brace failure)
Top chord (S690): max $N_{0,t,Ed} = 8932$ kN (compression)
Axial resistance: min $N_{0,t,Rd} = 29630$ kN (Chord shear)
Axial joint resistance: min $N_{2,Rd} = 8501$ kN (Brace failure)
U.C. = 0.89< 1 O.K., for brace 2

Fatigue

The end brace is checked again with respect to fatigue (see also fatigue procedure above and excel file "RHS_ fatigue calculations SCFs.xlsx" for detailed numbers and calculations).
Maximum axial and in plane bending SCFs (due to axial load and secondary bending moments in the brace) in this case are 6.0 and 0.0 (negligible) respectively.
Total damage: D = 2.4 >1
The joint is not sufficient for the design fatigue life of $5*10^7$ cycles.

<u>Critical location 3</u>: Connection between bottom chord, brace 2 and brace 3 members

Braces 2 and 3 are the most heavily loaded with respect to fatigue at this joint. Also brace 2 is the most heavily loaded brace in tension.
Both braces have the same cross sectional properties. EN 1993-1-8 states that in K- overlap joints only the overlapping member (in this case brace 3) needs to be checked. The efficiency (i.e. design resistance/design plastic resistance) of the overlapped brace should be taken equal to that of the overlapping member.
In addition, the normal forces in the braces in a K- joint should not differ more than 20% [3]. In case of joint 3 (J3), however, the difference is more than 20 %.
Thus, brace 3 is checked as a K- joint, as its force is totally equilibrating from brace 2. Brace 2 is also checked as a Y- joint, as part of its force is equilibrating from bending and shear in the chord.



J3

<u>Joint 3 (J3):</u>
Bottom chord: 500x 550x 30
Braces 2 and 3: 400x 400x 15
Overlap Ov= 50%
Eccentricity e= -108 mm
$\theta_2 = \theta_3 = 53°$

Figure B - 10 Critical joint 3 configuration, partly K- overlap and partly Y- joint.

Strength

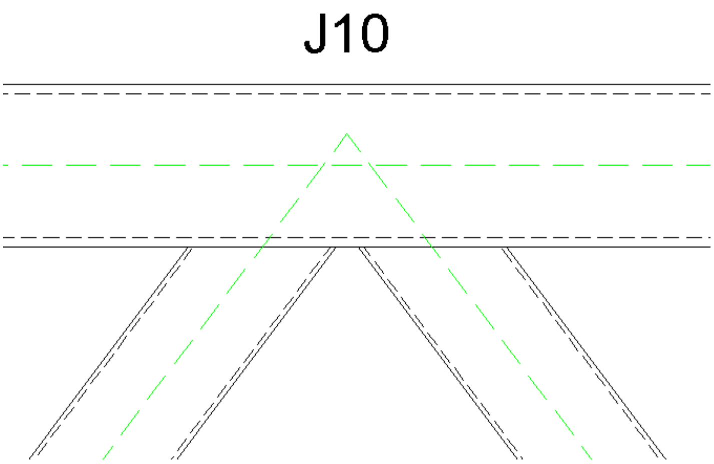
Formulas according to table 7.10 in EN 1993-1-8, using an additional reduction factor 0.8.
Additional parameters: $\beta = 0.8$, $\gamma = 8.3$, $k_n = 1$, $\sin\theta_2 = \sin\theta_3 = 0.80$, $b_{e,\,ov} = 150$ mm
<u>Brace 3</u> (S355): max $N_{3,\,Ed} = 5159$ kN (compression)
Axial resistance: min $N_{3,\,Rd} = 5683$ kN (efficiency = $N_{3,\,Rd}/\ A*f_y = 0.69$)
Axial joint resistance: min $N_{3,\,Rd} = 5683$ kN (Brace failure)
U.C. = 0.91< 1 O.K., for brace 3
Formulas according to table 7.11 in EN 1993-1-8, using an additional reduction factor 0.8.
$\beta = 0.8$, $\gamma = 8.3$, $k_n = 1$, $\eta = 0.8$, $\sin\theta_2 = \sin\theta_3 = 0.80$, $f_b = \chi * f_{y0} = 0.5*690 = 345$ N/mm^2
<u>Brace 2</u> (S460): max $N_{2,\,Ed} = 7566$ kN (tension)
Axial resistance: min $N_{3,\,Rd} = 8501$ kN (brace failure)
Axial joint resistance: min $N_{2,\,Rd} = 7566$ kN (Brace failure)
U.C. = 0.89< 1 O.K., for brace 2

Fatigue

Brace 3 is checked here with respect to fatigue as it showed higher fatigue stresses at this location (see also fatigue procedure above and excel file "RHS_ fatigue calculations SCFs.xlsx" for detailed numbers and calculations). Axial and in plane bending SCFs at brace toe location (due to axial load and secondary bending moments in the brace) are 7.8 and 0.0 (negligible) respectively.
Total damage: D = 15 >>1
The joint is not sufficient for the design fatigue life of $5*10^7$ cycles.

Critical location 4: Connection between top chord, brace 9 and brace 10 members

Here the top chord has its maximum static internal forces but also its fatigue stresses. Both braces have the same cross sectional properties thus the same resistance. Thus, only capacity with respect to the loading in brace 9 (higher load in comparison to brace 10), needs to be checked.



Joint 10 (J10):
Top chord: 500x 550x 30
Braces 9 and 10: 400x 400x 15
Gap g= 75 mm
Eccentricity e= +108 mm
$\theta_9 = \theta_{10} = 53°$

Figure B - 11 Critical joint 10 configuration, K- gap joint

Strength

Formulas according to table 7.12 in EN 1993-1-8, using an additional reduction factor 0.8.
Additional parameters: $\beta = 0.8$, $\gamma = 8.3$, $k_n = 1$, $\sin\theta_9 = \sin\theta_{10} = 0.80$
Also, for chord shear failure mode:
$V_{Ed} = 67$ kN
$V_{pl,Rd} = (A_{v0}*(f_{y0}/\sqrt{3}))/\gamma_{M0} = 11951$ kN
$A_{v0} = A_0 - 2*h_{w0}*t_{w0} = 30000$ mm^2
$A_0 = 59400$ mm^2
Brace 9 (S355): max $N_{9,Ed} = 1337$ kN (compression)
Axial resistance: min $N_{9,Rd} = 6561$ kN (Brace failure)

Top chord (S690): max $N_{0,\,t,\,Ed}$ =24427 kN (compression)
Axial resistance: min $N_{0,\,t,\,Rd}$ = 32755 kN
Axial joint resistance: min $N_{9,\,Rd}$ = 6561 kN (Brace failure)
U.C. =0.20 <1 O.K., for brace 9

Fatigue

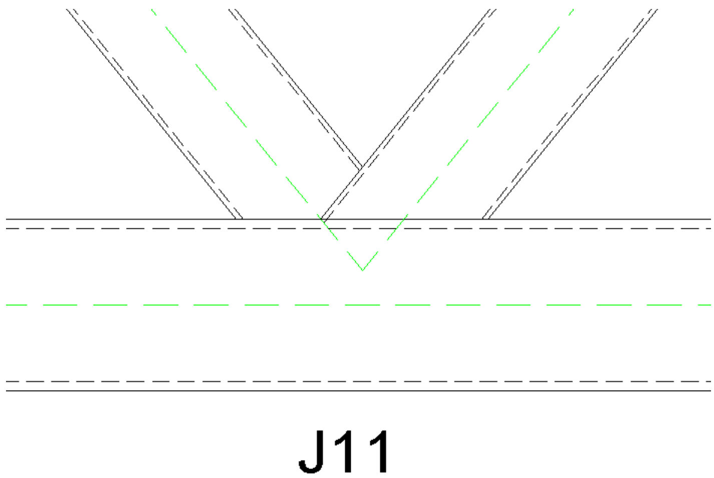
The top chord member is checked with respect to fatigue (see also fatigue procedure above and excel file "RHS_ fatigue calculations SCFs.xlsx" for detailed numbers and calculations). Maximum axial and in plane bending SCFs (due to axial load and in plane bending moments in the chord) are 3.0 and 2.7 respectively.

Total damage: D = 0.04 <1

The joint is sufficient for the design fatigue life of $5*10^7$cycles.

Critical location 5: Connection between bottom chord, brace 10 and brace 11 members

This joint is located almost at the middle of the bridge where the bottom chord suffers the highest internal forces due to traffic loading. The braces are both in tension under very small axial forces. Due to the fact that both braces are in tension the joint cannot be treated as a K-overlap joint but as two separate Y- joints. Both braces have the same cross sectional properties thus the same resistance. Thus, only capacity with respect to the loading in brace 10 (higher load in comparison to brace 11), needs to be checked.



Joint 11 (J11):
Bottom chord: 500x 550x 30
Braces 10 and 11: 400x 400x 15
θ_{10}= θ_{11}= 53°

Figure B - 12 Critical joint 11 configuration, as two separate Y-joints

Strength

Formulas according to table 7.11 in EN 1993-1-8, using an additional reduction factor 0.8.
Additional parameters β = 0.8, γ =8.3, k_n = 1, η = 0.8, $\sin\theta_{10}$ = $\sin\theta_{11}$ = 0.80, f_b = f_{y0} =690 N/mm^2
Brace 10 (S355): max $N_{10,\,Ed}$ = 1190 kN (tension)

Axial resistance: min $N_{10,\,Rd}$ =6561 kN (Brace failure)
Axial joint resistance: min $N_{10,\,Rd}$ =6561 kN (Brace failure)
U.C. = 0.18 <1 O.K. , for brace 10

Fatigue

The bottom chord member is checked with respect to fatigue (see also fatigue procedure above and excel file “RHS_ fatigue calculations SCFs.xlsx” for detailed numbers and calculations). Maximum axial and in plane bending SCFs (due to axial load and in plane bending moments in the chord) are 3.0 and 2.7 respectively.

Total damage: D = 2 >1

The joint is not sufficient for the design fatigue life of $5*10^7$ cycles.

Results

It is clear that fatigue for the ‘Schellingwouderbrug’ is not satisfied with regular joints types. Thus, alternative solutions for connections need to be made in order to improve fatigue details and enhance fatigue resistance for the bridge.

1.d) Alternative connections design with RHS members and gusset plates

An alternative design for connections (Figure 13) using side plates (gusset plates) is proposed.
In this case the cross sections of the braces have been changed. This has been done in order to obtain the same member width for the chords and the braces. This was necessary to make possible welding the gusset plates to the flanges of the braces and the chords. Thus, β- ratio is 1 for all connections.

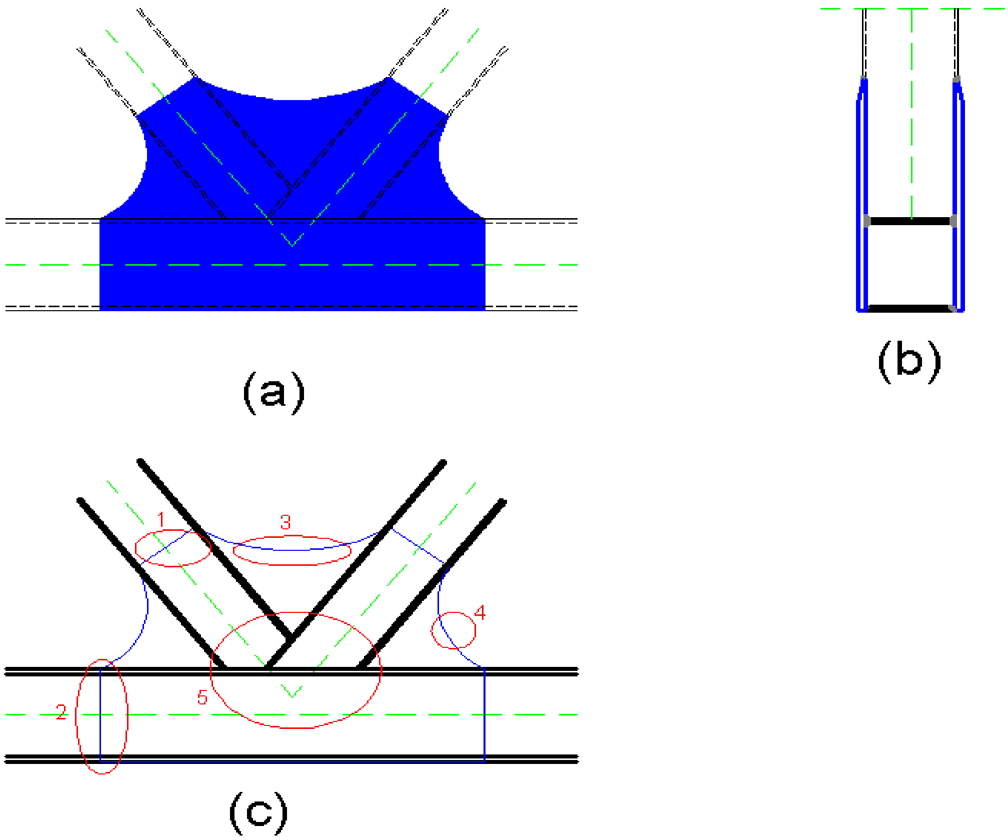


Figure B - 13 Design of connections with gusset plates (improvement for fatigue)

All braces are made now 500x400x16. Thus, the width of all braces is increased and also the thickness, except for the end brace (before it was 19 mm). The thickness has been slightly increased in order to maintain a class 2 cross section for all the compression braces (assuming and S355 steel grade). The end compression brace is now class 3 though, as S460 is assumed as the steel grade sufficient for stability requirements
The tension braces have been also altered to ease fabrication (constant brace section all over the bridge). However, thickness for the tensile braces could be remained as before (15 mm).
Moreover, the web thickness of the bottom chord could be reduced to 20 mm (from 30 mm).

Top chord

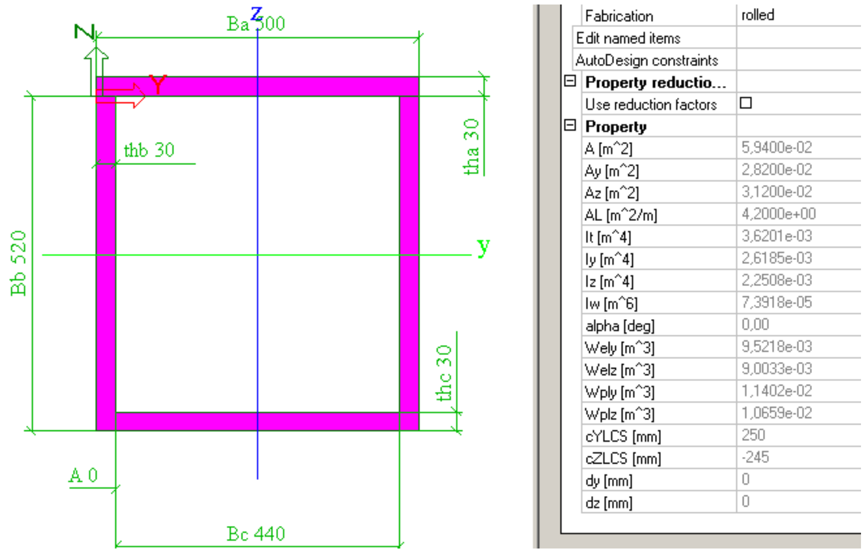


Figure B - 14 Top chord- Design with gusset plates, U.C. =0.95

Table B - 1 In plane buckling resistance of top chord in RHS hybrid design with gusset plates

Top Chord		
Area	A (mm^2)	59400
Length	L_y (mm)	10500
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	9450
Moment of Inertia	I_y (mm^4)	2.62E+09
Radius of gyration	i_y (mm)	210
Slenderness	λy	45
Rel. slenderness	λ_E	55
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	690
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.82
Phi factor	Φ	0.94
Buckling factor	χ_y	0.71
Buckling strength	$f_{b,rd}$ (N/mm^2)	491

Bottom chord

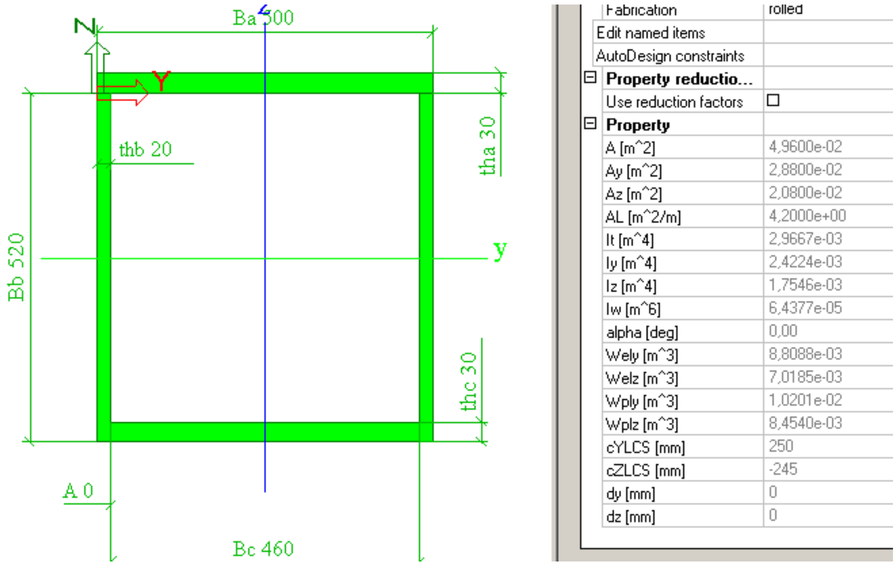


Figure B - 15 Bottom chord- Design with gusset plates, U.C. = 0.88

All braces

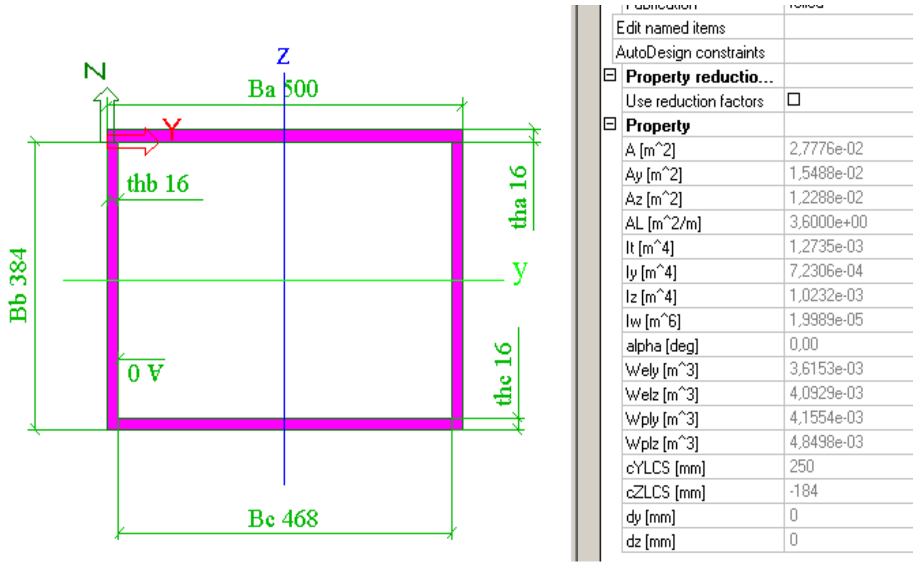


Figure B - 16 All braces-Design with gusset plates, U.C = 0.89 (end brace)

Table B - 2 In plane buckling resistance of end brace in RHS hybrid design with gusset plates

End brace		
Area	A (mm^2)	27776
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	7.23E+08
Radius of gyration	i_y (mm)	161.34

Slenderness	λy	54
Rel. slenderness	λ_E	67
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	460
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.81
Phi factor	Φ	0.93
Buckling factor	χ_y	0.72
Buckling strength	$f_{b,rd}$ (N/mm^2)	331

Table B - 3 In plane buckling resistance of rest compression braces in RHS hybrid design with gusset plates, U.C. = 0.86 (brace 3)

Rest compression braces		
Area	A (mm^2)	27776
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	7.23E+08
Radius of gyration	i_y (mm)	161.34
Slenderness	λy	54
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.71
Phi factor	Φ	0.84
Buckling factor	χ_y	0.78
Buckling strength	$f_{b,rd}$ (N/mm^2)	276

Generally, the cross sections of the braces are generally conservative with respect to their loading and thus, the stresses they need to resist are in many cases quite low (especially for the braces close to midspan, U.C.= 0.2). In all cases the maximum steel grade does not exceed S460 (only for the end braces to satisfy stability check).

Further design and member optimization (e.g. decreasing the cross sections for a number of braces or decreasing the length of the braces by decreasing the field length or the bridge height etc.) could be proven more economic. However, this procedure is time consuming. In addition, repetition and thus simplicity of fabrication is considered to be more cost effective.

Strength

The two side plates (gussets) are designed, conservatively, as they will carry the whole load the members need to resist. This relies on the assumption that these plates are thick enough, and thus stiffer, so that the load from the braces will not go through the chord face but through the side plate to the chord wall.

The thickness of the gusset plate depends on the load it must resist. The minimum required gusset plate thickness, if S690 steel grade is considered ($f_{y,\ gplate}$ = 690 N/mm^2), for these plates is calculated for top and bottom chord and for the most heavily loaded braces. If S355 steel grade is assumed, these values need to be doubled.

<u>Top chord (compression)</u>: max $N_{0,\ t,\ Ed}$ =24427 kN, $t_{gplate,\ min} = (N_{0,\ t,\ Ed}/2)/\ (h_{0,t}*f_{y,\ gplate})$= 32 mm, per gusset plate
<u>Bottom chord (tension)</u>: max $N_{0,\ b,\ Ed}$ =23738 kN, $t_{gplate,\ min} = (N_{0,\ b,\ Ed}/2)/\ (h_{0,b}*f_{y,\ gplate})$= 32 mm, per gusset plate
<u>End brace (compression)</u>: max $N_{1,\ Ed}$ = 7365 kN, $t_{gplate,\ min} = (N_{1,\ Ed}/2)/\ (b_1*f_{y,\ gplate})$ = 15 mm, per gusset plate
<u>Brace 2 (tension)</u>: max $N_{2,\ Ed}$ = 7585 kN, $t_{gplate,\ min} = (N_{2,\ Ed}/2)/\ (b_2*f_{y,\ gplate})$ = 15 mm, per gusset plate

Thus, **assuming uniform (constant) plate thickness for the gusset plates the minimum required plate thickness should be $t_{g,\ plate,\ min}$ = 32 mm.**

Although not covered in this study, design of butt welds needs to ensure they have sufficient strength (i.e. they can carry the load from the truss members to the gusset plates and vice versa). Undermatched welds will be used in connections of the gusset plates (S690) with the braces (mainly S355).

Fatigue

In this design the critical joints remain the same but the critical fatigue details in each joint have changed. Now, the connections where the butt welds are located (Figure 13, locations 1 and 2) will be critical for fatigue.
This has two main advantages. Firstly, according to EN1993-1-9, these are better fatigue details (i.e. minimum fatigue detail class is 71 if from one side butt welds is only possible, but also higher classes are possible, e.g., class 80 or 90 if both sides can be welded) and are also applicable for HSS. However, size effect for thickness > 25 mm needs to be considered. Secondly, lower nominal stresses (thus relevant nominal S-N curves from EN 1993-1-9 according to detail fatigue class) can be used, which are located further from the joint's intersection point.

Procedure for fatigue with the nominal stress method (gusset plates & butt welds)

1. Using Scia engineer <u>FLM4 is applied</u> as movable load on the bridge.

2. <u>Nominal ranges</u> at butt welds locations (forces (ΔN) and bending moments (ΔM)) are collected for each member at each critical joint, caused by each vehicle separately. Butt welds

between gusset plates and truss member (chord, braces) are assumed at about 1.5 m away from the member intersection point at the joint location.

3. Nominal stress ranges ($\Delta\sigma_{nom}$) are calculated from these values.

4. Design fatigue life is chosen. For bridges, customary, is 100 years.

5. Traffic category for the specific bridge is chosen from table 4.5 in EN 1991-2 to be $0.5*10^6$ cycles per year per slow lane.

6. Total number of cycles is calculated $100*(0.5*10^6)$.

7. Traffic type is chosen as medium distance (column 5) from table 4.7 in EN 1993-1-9.

8. The number of cycles per vehicle (n_i) in FLM4 is calculated: lorry percentage (previous step)* total number of cycles (from step 8).

9. Relevant nominal S-N curve is chosen from EN 1993-1-9. In this case it is considered that the S_N curve that corresponds to fatigue detail 71 is the most suitable.

10. Available fatigue life (N_f) is then calculated based on formulas in EN 1993-1-9 section 7.1.

11. Finally Miner's rule for total damage calculation is applied. Must $D= \Sigma (n_i/N_{f,i}) \leq 1$.

Results

Analytical fatigue calculations can be found in the excel file “RHS_fatigue calculations butt welds.xlsx”.
The fatigue resistance proved to be sufficient for most members at the critical joints, and the accumulated damage was well below 1 for all braces and top chord. For the bottom chord the damage was significantly reduced but still remained above 1 (D= 3). The bigger damage occurs in the high cycle fatigue region. Locally thicker flange plates and/or post weld treatment in this case is necessary to improve further the fatigue strength.

2) Truss bridge design with RHS members and only S355 steel grade

Just for design comparison on a cost basis performed in chapter 9, estimation on required truss member dimensions is made in case that S355 steel grade is only used for the whole bridge design. The comparison aims to show what the benefits are from a hybrid bridge construction, under the same design concept. It is out of the scope of this study to develop the most economical “all in S355” bridge design.
Two sub- cases are investigated: case (a) what is the minimum required plate thickness if all the other dimensions remain the same as for the hybrid design, and case (b) what are the minimum other cross sectional dimensions (i.e. members width, and height) required to take the stresses if the same plate thickness is considered as for the hybrid design.

<u>Case (a):</u> Increase members plate thickness only

Required members thickness to satisfy strength requirements is more or less doubled for the chord members. Unity checks are close to the case of hybrid construction. The minimum required plate thickness to take over static stresses with S355 is shown in Figures 15-17.

Top chord (S355)

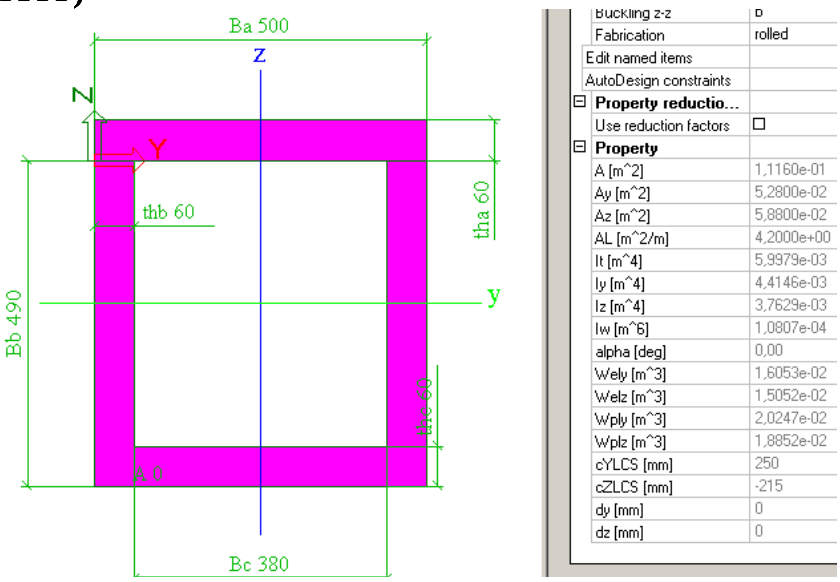


Figure B - 17 Top chord- Design with RHS members and S355 steel grade only, case (a), U.C. = 0.98

Table B - 4 In plane buckling check for top chord, “all in S355 design”, case (a)

<u>**Top Chord**</u>

Area	A (mm^2)	111600
Length	L_y (mm)	10500
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	9450
Moment of Inertia	I_y (mm^4)	4.41E+09
Radius of gyration	i_y (mm)	199
Slenderness	λy	48
Rel. slenderness	λ_E	79
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	335
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.60
Phi factor	Φ	0.75
Buckling factor	χ_y	0.83
Buckling strength	$f_{b,rd}$ (N/mm^2)	280

Bottom chord (S355)

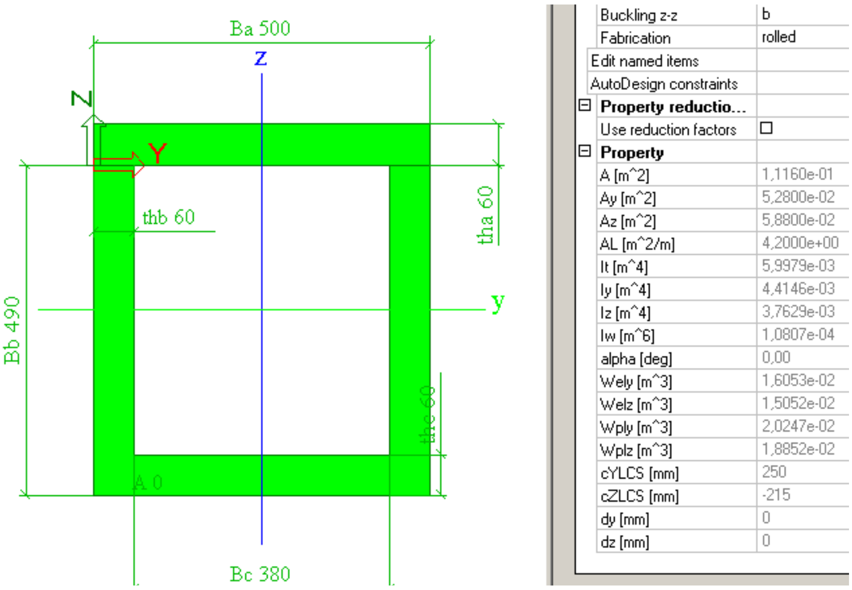


Figure B - 18 Design with RHS members and S355 steel grade only, case (a), U.C. = 0.87

Braces (S355)

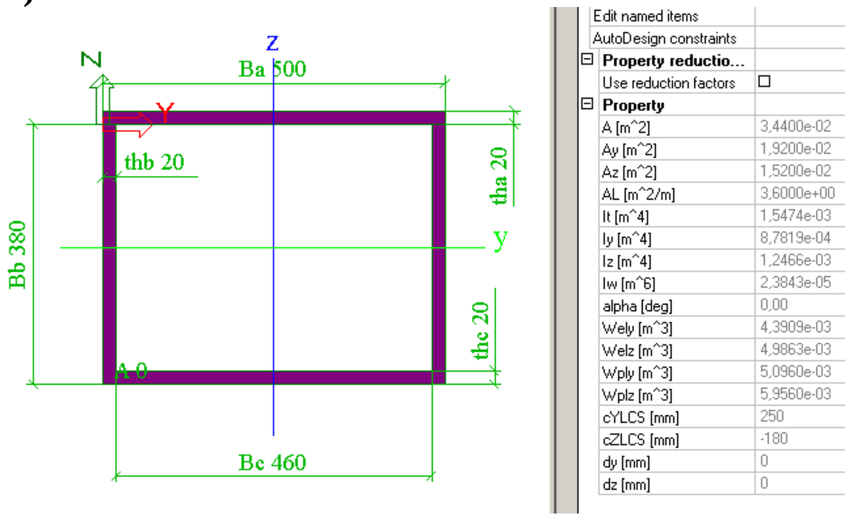


Figure B - 19 End (compression) braces, U.C. =0.91

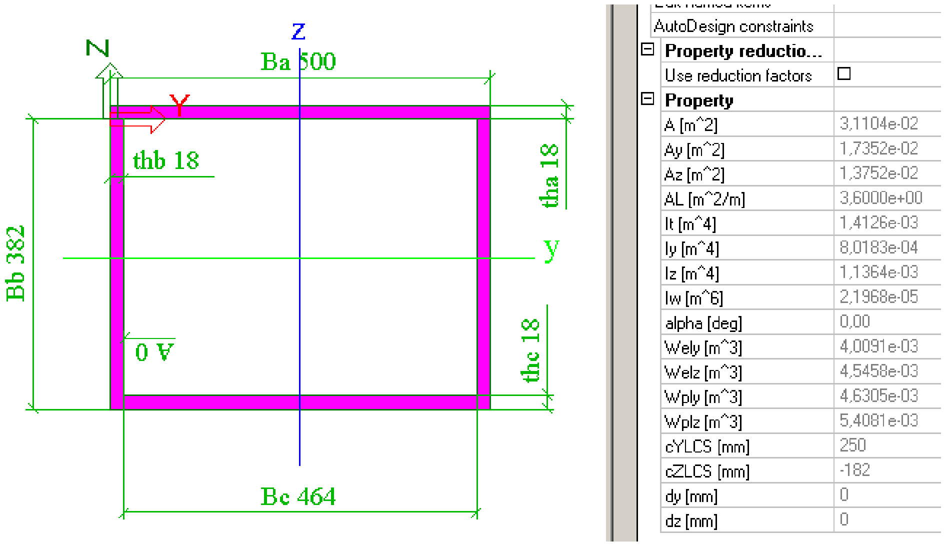


Figure B - 20 Rest braces, U.C. =0.81 (brace 3)

Table B - 5 In plane buckling check for end braces, "all in S355 design", case (a)

End brace		
Area	A (mm^2)	34400
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	8.80E+08
Radius of gyration	i_y (mm)	159.94
Slenderness	λy	55
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355

E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.72
Phi factor	Φ	0.84
Buckling factor	χ_y	0.77
Buckling strength	$f_{b,rd}$ (N/mm^2)	275

Table B - 6 In plane buckling check for rest compression braces "all in S355 design", case (a)

Rest compression braces		
Area	A (mm^2)	31104
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	8.02E+08
Radius of gyration	i_y (mm)	160.58
Slenderness	λy	54
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.71
Phi factor	Φ	0.84
Buckling factor	χ_y	0.78
Buckling strength	$f_{b,rd}$ (N/mm^2)	276

Result

Bridge steel weight has been increased to 571 tn while in hybrid RHS design was 347 tn (65% bridge dead weight increment and its associated costs!). An additional 15 % of these values should be considered to account for gusset plates, additional steel (e.g. wind bracings) and welds in each design.
The required minimum thickness for gusset plates increases to 60 mm (from 32 mm calculated for the hybrid design).

Case (b): Increase members height and width, keeping the same plate thickness

In this case the cross sectional dimensions (height (h_i), width (b_i)) have been increased to satisfy strength and stability checks. The unity checks in this case are also closer to the hybrid CHS design for better comparison.

Top chord (S355)

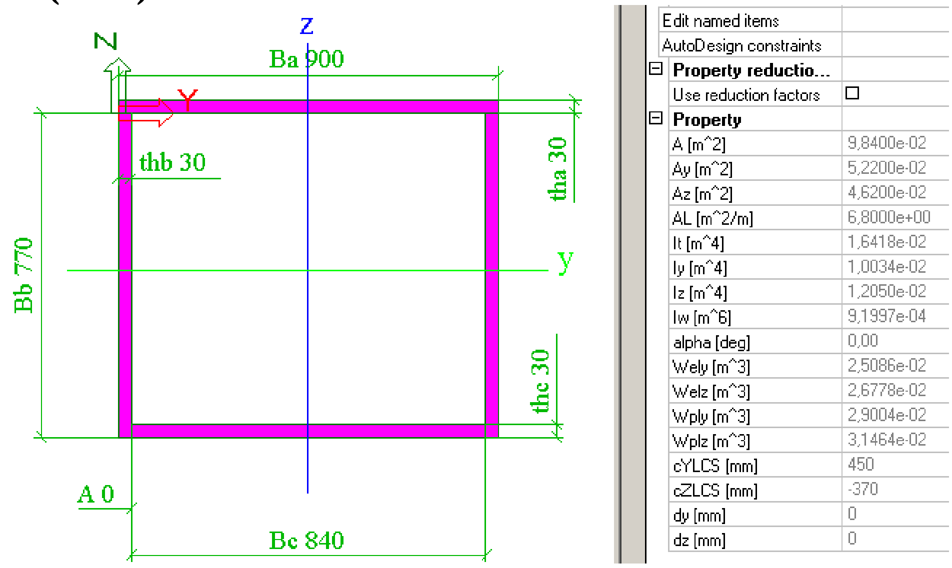
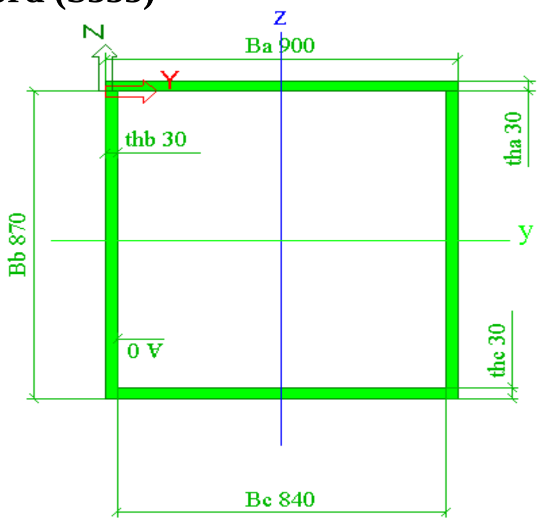


Figure B - 21 Top chord- Design with RHS members and S355 steel grade only, case (b), U.C. =0.92

Table B - 7 In plane buckling check for top chord, “all in S355 design”, case (b)

Top Chord		
Area	A (mm^2)	98400
Length	L_y (mm)	10500
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	9450
Moment of Inertia	I_y (mm^4)	1.00E+10
Radius of gyration	i_y (mm)	319
Slenderness	λy	30
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.39
Phi factor	Φ	0.61
Buckling factor	χ_y	0.93
Buckling strength	$f_{b,rd}$ (N/mm^2)	330

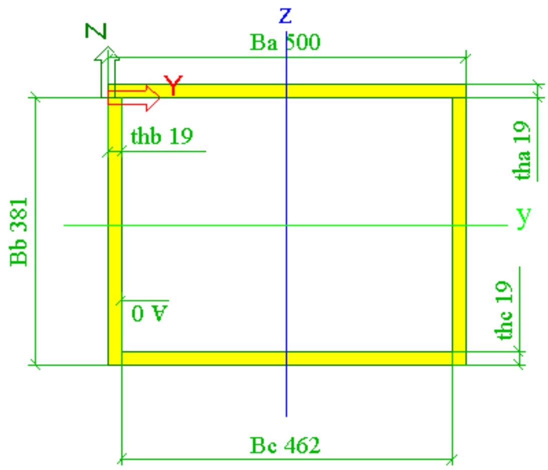
Bottom chord (S355)



Fabrication	rolled
Edit named items	
AutoDesign constraints	
Property reductio...	
Use reduction factors	☐
Property	
A [m^2]	1,0440e-01
Ay [m^2]	5,2200e-02
Az [m^2]	5,2200e-02
AL [m^2/m]	7,2000e+00
It [m^4]	1,9755e-02
Iy [m^4]	1,3186e-02
Iz [m^4]	1,3186e-02
Iw [m^6]	1,2461e-03
alpha [deg]	0,00
Wely [m^3]	2,9302e-02
Welz [m^3]	2,9302e-02
Wply [m^3]	3,4074e-02
Wplz [m^3]	3,4074e-02
cYLCS [mm]	450
cZLCS [mm]	-420
dy [mm]	0
dz [mm]	0

Figure B - 22 Bottom chord- Design with RHS members and S355 steel grade only, case (b), U.C. =0.88

Braces (S355)



AutoDesign constraints	
Property reductio...	
Use reduction factors	☐
Property	
A [m^2]	3,2756e-02
Ay [m^2]	1,8278e-02
Az [m^2]	1,4478e-02
AL [m^2/m]	3,6000e+00
It [m^4]	1,4805e-03
Iy [m^4]	8,4031e-04
Iz [m^4]	1,1919e-03
Iw [m^6]	2,2919e-05
alpha [deg]	0,00
Wely [m^3]	4,2015e-03
Welz [m^3]	4,7676e-03
Wply [m^3]	4,8644e-03
Wplz [m^3]	5,6833e-03
cYLCS [mm]	250
cZLCS [mm]	-181
dy [mm]	0
dz [mm]	0

Figure B - 23 End (compression) braces, U.C. =0.93

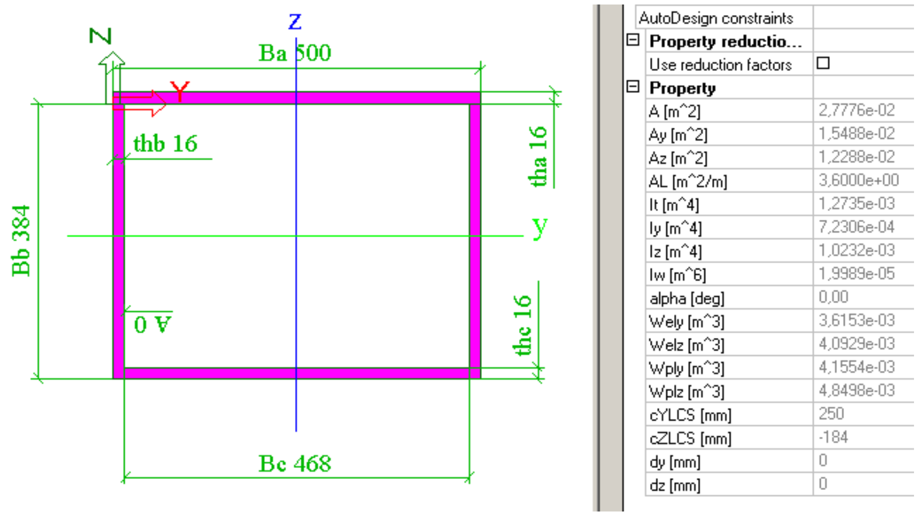


Figure B - 24 Rest (compression) braces, U.C. =0.83

Table B - 8 In plane buckling check for end braces “all in S355 design”, case (b)

End brace		
Area	A (mm^2)	32380
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	1.29E+09
Radius of gyration	i_y (mm)	199.60
Slenderness	λy	44
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.57
Phi factor	Φ	0.73
Buckling factor	χ_y	0.85
Buckling strength	$f_{b,rd}$ (N/mm^2)	302

Table B - 9 In plane buckling check for rest compression braces “all in S355 design”, case (b)

Rest compression braces		
Area	A (mm^2)	27776
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	7.23E+08
Radius of gyration	i_y (mm)	161.34

Slenderness	λy	54
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.71
Phi factor	Φ	0.84
Buckling factor	χ_y	0.78
Buckling strength	$f_{b,rd}$ (N/mm^2)	276

Result

Bridge steel weight has been increased to 497 tn while in hybrid RHS design was 347 tn (43 % bridge dead weight increment and its associated costs!). An additional 15 % of these values should be considered to account for gusset plates, additional steel (e.g. wind bracings) and welds in each design.
Furthermore, due to increased dimensions the painting required area (for corrosion protection) has been increased 31 %.

3) Hybrid truss bridge design with CHS members and cast joints

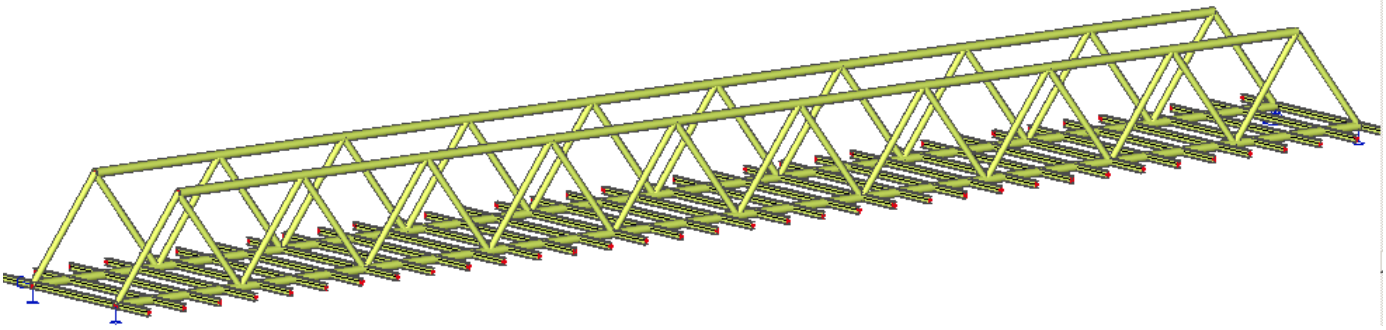


Figure B - 25 Truss design with CHS members

3. a) Cross sectional dimensions and calculations

For the design with CHS members standardized cross sections have been chosen, as they were available at the sizes required for strength and stability.
To facilitate comparison, high strength steel S690 is applied in the chord members, while S355 in the braces (S460 only in the end braces) and in the cross beams as in case of RHS members.

Global calculations for strength (maximum static stresses are limited to the yield stress-elastic analysis) and stability (see also excel file “Stability check.xls”) have been performed as in case of RHS members in the preliminary phase.
Loadings are exactly the same as in all other bridge designs (see preliminary phase-Appendix A).

The connections are assumed to be made of castings. They have not been examined in detail, however, it is possible to choose an optimal casting for these connections using FEM analysis. Castings for S690 steel grade (and even higher) are available.

Strength and stability

Top chord

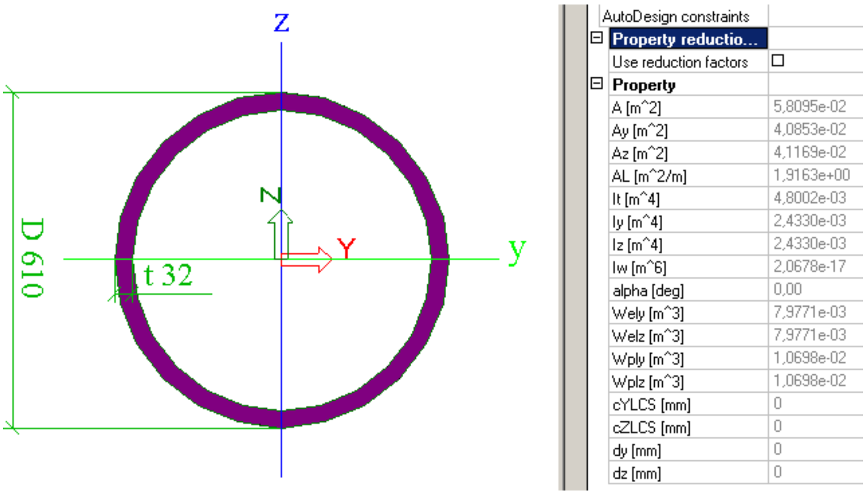


Figure B - 26 Top chord, standard sections (610x32), U.C. = 0.97

Table B - 10 In plane buckling strength for top chord, hybrid design with CHS members and cast joints

Top Chord		
Area	A (mm^2)	58095
Length	L_y (mm)	10500
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	9450
Moment of Inertia	I_y (mm^4)	2.43E+09
Radius of gyration	i_y (mm)	205
Slenderness	λ_y	46
Rel. slenderness	λ_E	55
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	690
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.84
Phi factor	Φ	0.96
Buckling factor	χ_y	0.70
Buckling strength	$f_{b,rd}$ (N/mm^2)	481

Bottom chord

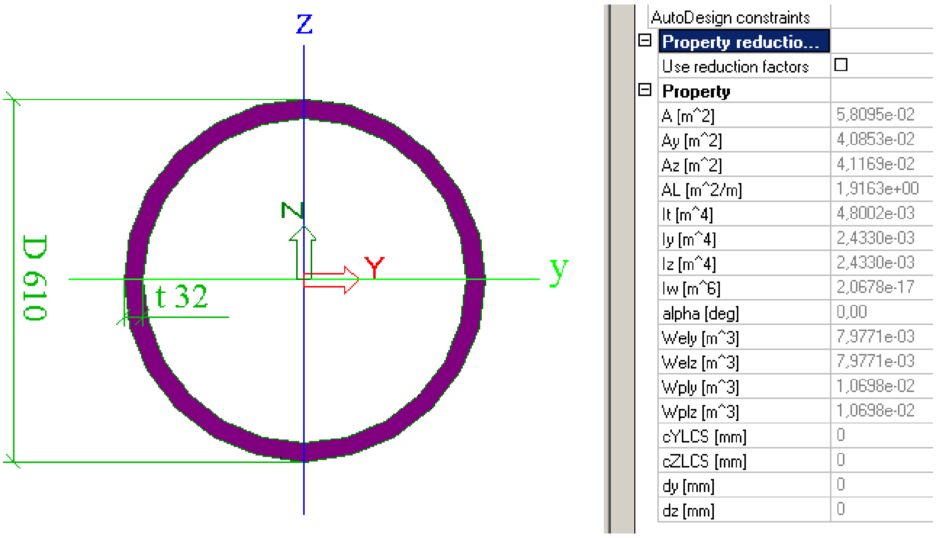


Figure B - 27 Bottom chord standard sections (610x 32), U.C. =0.81

All braces

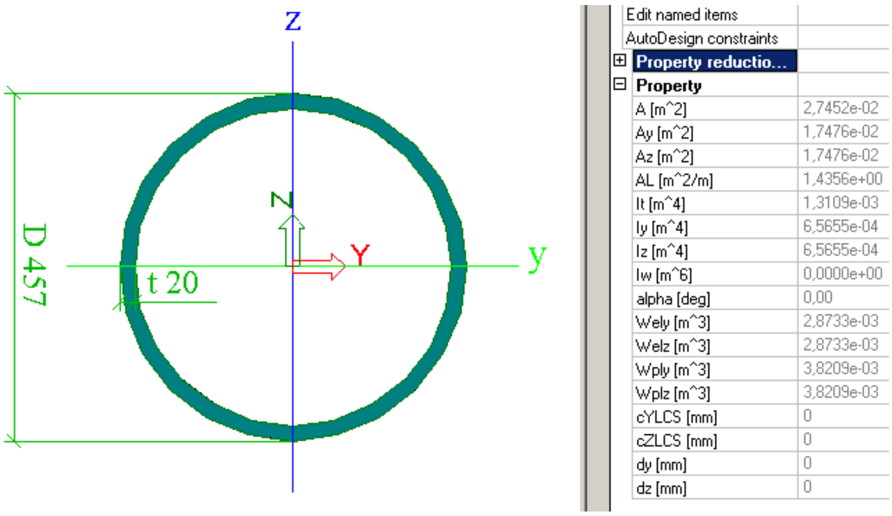


Figure B - 28 Braces standard sections (457 x 20), U.C. = 0.92 (end brace)

Table B - 11 In plane buckling resistance of compression end braces, hybrid design with CHS members and cast joints

End compressive brace		
Area	A (mm^2)	27452
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	6.57E+08

Radius of gyration	i_y (mm)	154,70
Slenderness	λy	57
Rel. slenderness	λ_E	67
Buckling curve		b
Imperfection factor	α	0,34
Strength	f_{yd} (N/mm^2)	460
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0,84
Phi factor	Φ	0,96
Buckling factor	χ_y	0,70
Buckling strength	$f_{b,rd}$ (N/mm^2)	321

Table B – 12 In plane buckling resistance of rest compression braces, hybrid design with CHS members and cast joints, U.C. = 0.85

Rest compressive brace		
Area	A (mm^2)	27452
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	6.57E+08
Radius of gyration	i_y (mm)	154.70
Slenderness	λy	57
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.74
Phi factor	Φ	0.87
Buckling factor	χ_y	0.76
Buckling strength	$f_{b,rd}$ (N/mm^2)	270

For the tensile braces maximum static stress is + 302 N/mm^2 (in brace 2). So, S355 steel grade is sufficient, U.C. = 0.85.

Fatigue

Critical fatigue locations are considered at the connections (butt welds) between cast steel and circular hollow section members. Specific calculations can be found in the excel file "CHS_ fatigue calculations butt welds.xlsx".

Generally the results showed sufficient fatigue strength (in all joints D<1) except in case of the bottom chord (in joint 11), where the total calculated damage was about D= 3.5>1.

Similar to the case of RHS members with gusset plates extra care is necessary for this connection (at joint J11). Especially if post weld treatment according to IIW recommendations is applied (in literature survey reference [63]), then fatigue stresses can be reduced by a factor 1.5. In this case scenario the calculated fatigue damage for the bottom chord is well below 1, also.

4) Truss bridge design with CHS members, all in S355 steel grade

<u>Case (a)</u>: Increase member thickness only`

Required members thickness to satisfy strength requirements is doubled for the chord members. Unity checks are a bit higher than in case of hybrid construction which means that even bigger thickness may be necessary. However, the minimum required plate thickness to take over static stresses with S355 is shown in Figures 29-32.

Top chord (S355)

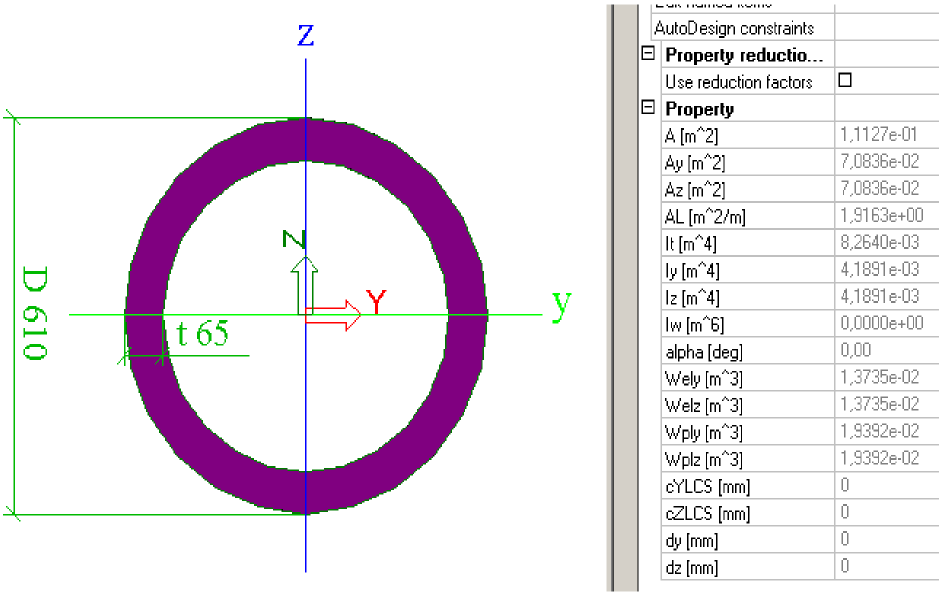


Figure B - 29 Top chord- Design with CHS members and S355 steel grade only, case (a), U.C. =0.97

Table B - 3 In plane buckling resistance of top chord, "all in S355" with CHS members and cast joints, case (a)

Top chord		
Area	A (mm^2)	111270
Length	L_y (mm)	10500
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	9450
Moment of Inertia	I_y (mm^4)	4.18E+09
Radius of gyration	i_y (mm)	194
Slenderness	λy	49
Rel. slenderness	λ_E	79
Buckling curve		b

Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	335.00
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.62
Phi factor	Φ	0.76
Buckling factor	χ_y	0.83
Buckling strength	$f_{b,rd}$ (N/mm^2)	277

Bottom chord (S355)

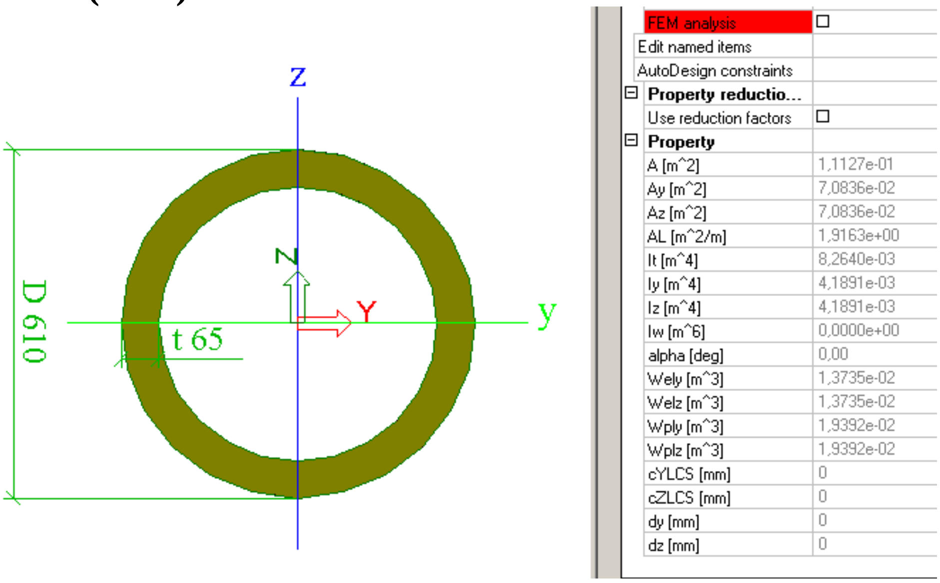


Figure B - 30 Bottom chord- Design with CHS members and S355 steel grade only, case (a), U.C. =0.91

Braces (S355)

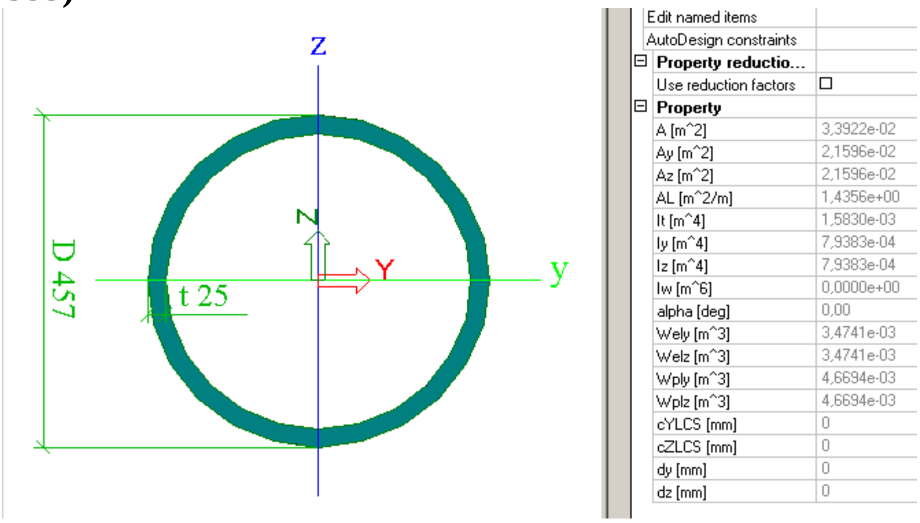


Figure B - 31 End braces, standard section (457x25)- Design with CHS members and S355 steel grade only, case (a), U.C. =0.97

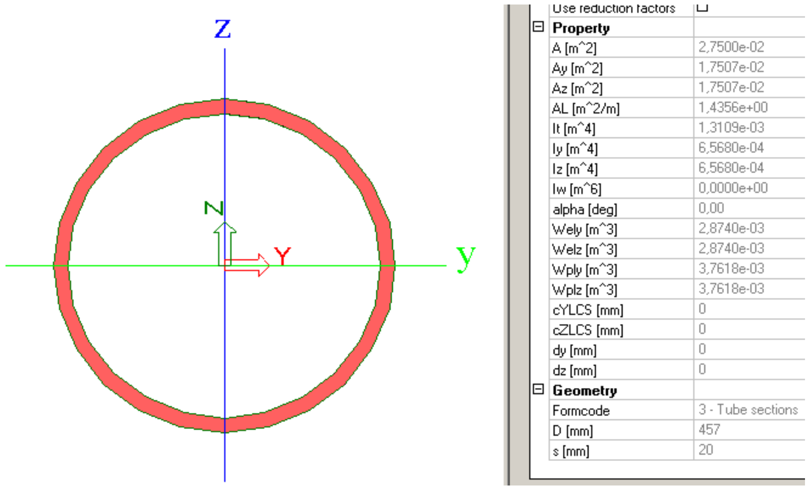


Figure B - 32 Rest braces, standard sections (457x20) - Design with CHS members and S355 steel grade only, case (a), U.C. = 0.91

Table B - 44 In plane buckling resistance of compression end braces, "all in S355" with CHS members and cast joints, case (a)

End compressive brace		
Area	A (mm^2)	33922
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	7.94E+08
Radius of gyration	i_y (mm)	152.99
Slenderness	λy	57
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.75
Phi factor	Φ	0.87
Buckling factor	χ_y	0.76
Buckling strength	$f_{b,rd}$ (N/mm^2)	268

Table B - 55 In plane buckling resistance of rest compression braces, hybrid design with CHS members and cast joints, case (a)

Rest compressive brace		
Area	A (mm^2)	27452
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	6.57E+08
Radius of gyration	i_y (mm)	154.70
Slenderness	λy	57
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.74
Phi factor	Φ	0.87
Buckling factor	χ_y	0.76
Buckling strength	$f_{b,rd}$ (N/mm^2)	270

Result

Bridge steel weight has been increased to 562 tn while in hybrid RHS design was 375 tn (50% bridge dead weight increment and its associated costs!). An additional 15 % of these values should be considered to account for cast steel, additional steel (e.g. wind bracings) and welds in each design.

<u>Case (b)</u>: Increase cross sectional area keeping the same plate thickness

In this case the cross sectional dimensions (height, width) have been increased (i.e. doubled) to satisfy strength and stability checks. The unity checks in this case are also closer to the hybrid CHS design for better comparison.

Top chord (S355)

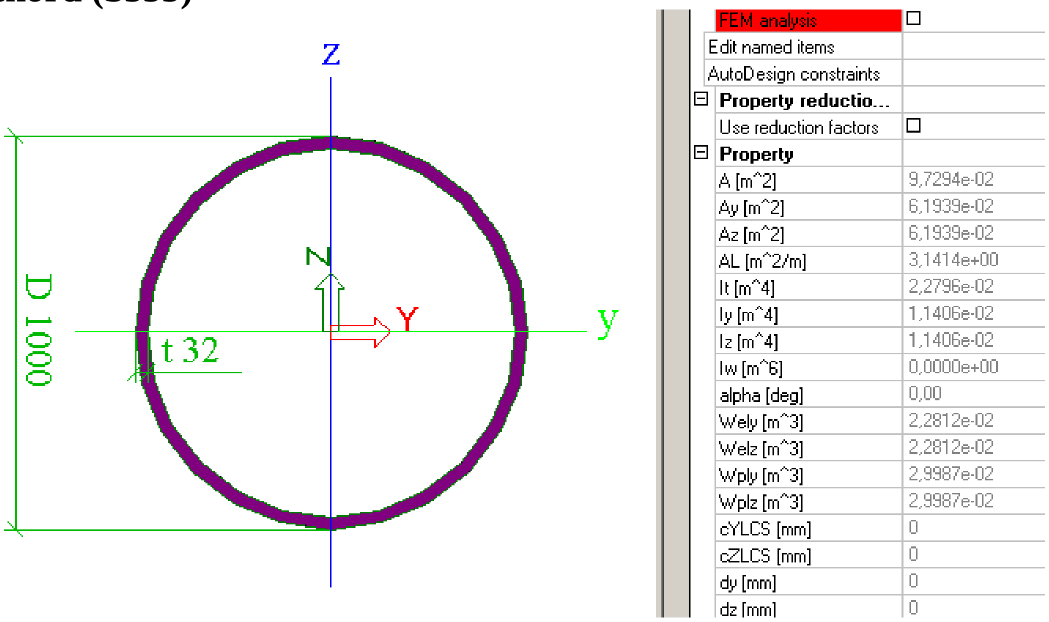


Figure B - 33 Top chord- Design with CHS members and S355 steel grade only, case (b), U.C. =0.94

Table B - 66 In plane buckling resistance of top chord, "all in S355" with CHS members and cast joints, case (b)

<u>Top chord</u>		
Area	A (mm^2)	97294
Length	L_y (mm)	10500
Buckling coefficient	K_y	0.9
Buckling length	l_y (mm)	9450
Moment of Inertia	I_y (mm^4)	1.14E+10
Radius of gyration	i_y (mm)	342
Slenderness	λy	28
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.36

Phi factor	Φ	0.59
Buckling factor	χ_y	0.94
Buckling strength	$f_{b,rd}$ (N/mm^2)	334

Bottom chord (S355)

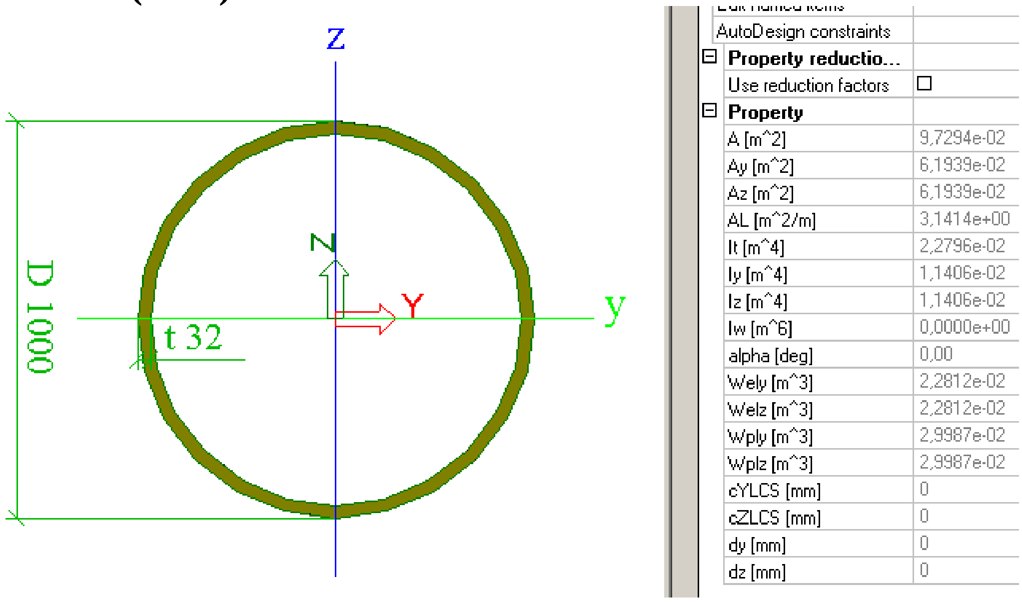


Figure B - 34 Bottom chord- Design with CHS members and S355 steel grade only, case (b), U.C. = 0.89

Braces (S355)

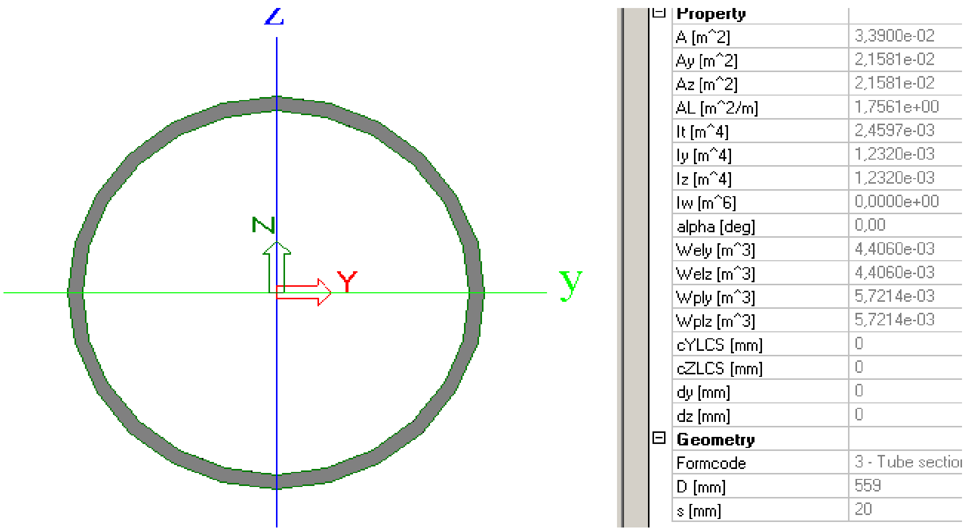


Figure B - 35 End braces - Design with CHS members and S355 steel grade only, case (b), U.C. =0.85

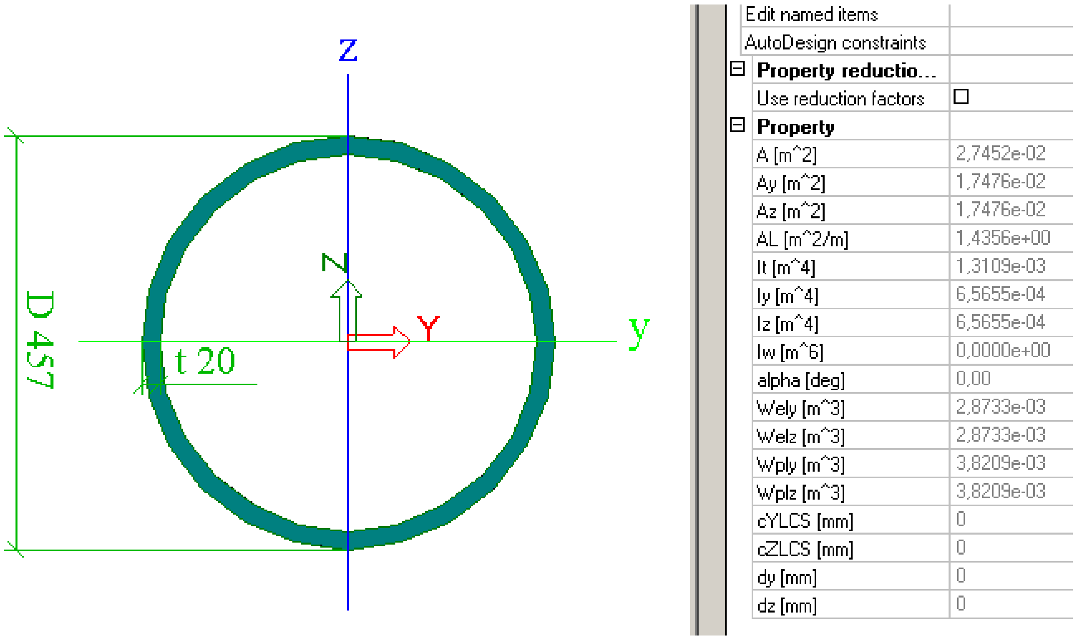


Figure B - 36 Rest braces - Design with CHS members and S355 steel grade only, case (b), U.C. =0.81

Table B - 17 In plane buckling resistance of compression end braces, "all in S355" with CHS members and cast joints, case (b)

End braces		
Area	A (mm^2)	33900
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	1.23E+09
Radius of gyration	i_y (mm)	190.48
Slenderness	λy	46
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.60
Phi factor	Φ	0.75
Buckling factor	χ_y	0.84
Buckling strength	$f_{b,rd}$ (N/mm^2)	297

Table B – 18 In plane buckling resistance of compression rest compression braces, “all in S355” with CHS members and cast joints, case (b)

Rest compressive brace		
Area	A (mm^2)	27452
Length	L_y (mm)	8750
Buckling coefficient	K_y	1
Buckling length	l_y (mm)	8750
Moment of Inertia	I_y (mm^4)	6.57E+08
Radius of gyration	i_y (mm)	154.70
Slenderness	λy	57
Rel. slenderness	λ_E	76
Buckling curve		b
Imperfection factor	α	0.34
Strength	f_{yd} (N/mm^2)	355
E-modulus	E (N/mm^2)	210000
Relative slenderness	$\lambda_{y,rel}$	0.74
Phi factor	Φ	0.87
Buckling factor	χ_y	0.76
Buckling strength	$f_{b,rd}$ (N/mm^2)	270

Result

Bridge steel weight has been increased to 522 tn while in hybrid RHS design was 375 tn (39% bridge dead weight increment and its associated costs!). An additional 15 % of these values should be considered to account for cast joints, additional steel (e.g. wind bracings) and welds in each design.
Furthermore, due to increased dimensions the painting required area (for corrosion protection) has been increased 29 %.