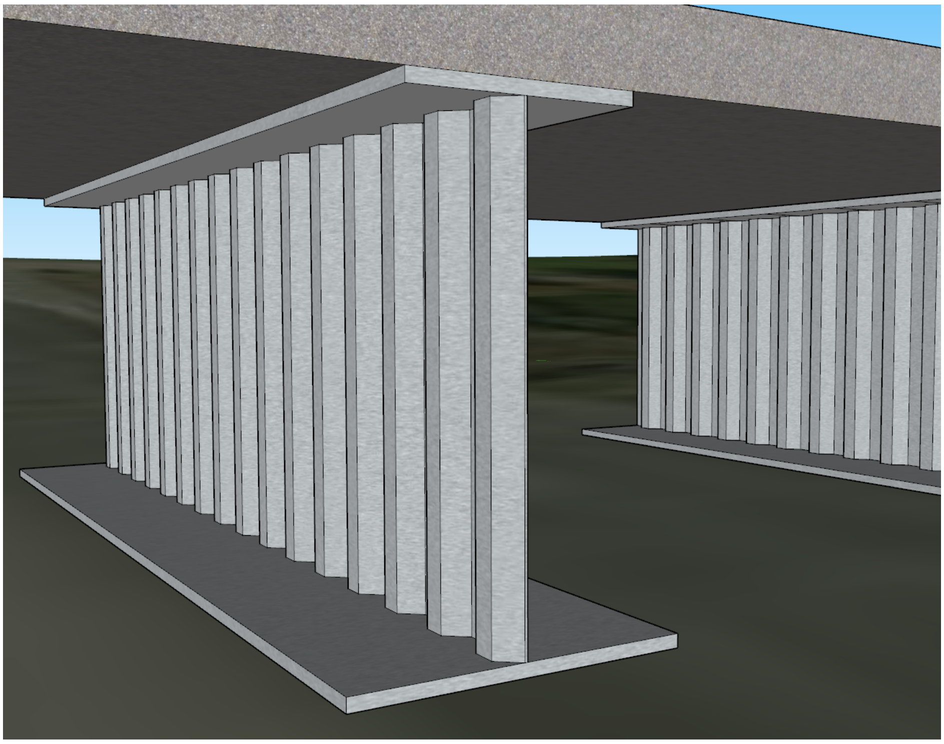


Design of Composite Steel-Concrete Bridges using Stainless Steel Girders with Corrugated Webs

A Study of the Applicability and Effectiveness of using Stainless Steel Girders with Corrugated Webs in Bridge Design

Master's thesis in Master Program Structural Engineering and Building Technology

ADAM HENRYSSON
ELLY YMAN

Department of Architecture and Civil Engineering
CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2020
www.chalmers.se

Master's thesis ACEX30

Design of Composite Steel- Concrete Bridges using Stainless Steel Girders with Corrugated Webs

A Study of the Applicability and Effectiveness of using Stainless Steel Girders with Corrugated Webs in Bridge Design

ADAM HENRYSSON
ELLY YMAN

Department of Architecture and Civil Engineering
Division of Structural Engineering
Lightweight structures
Chalmers University of Technology
Gothenburg, Sweden 2020

Design of Composite Steel- Concrete Bridges using Stainless Steel Girders with Corrugated Webs
A Study of the Applicability and Effectiveness of using Stainless Steel Girders with Corrugated Webs in Bridge Design

ADAM HENRYSSON
ELLY YMAN

Supervisor: PhD. Mohsen Heshmati, Bridge and Hydraulic department at WSP
Examiner: Senior Lecturer Mozhdeh Amani, Department of Architecture and Civil Engineering

Department of Architecture and Civil Engineering
Division of Structural Engineering
Lightweight structures
Chalmers University of Technology
SE-412 96 Gothenburg
Telephone +46 31 772 1000

Cover: Illustrates a cross-section of a steel-concrete composite bridge with stainless steel girders and corrugated webs

Department of Architecture and Civil Engineering
Gothenburg, Sweden 2020

Design of Composite Steel-Concrete Bridges using Stainless Steel Girders with Corrugated Webs
A Study of the Applicability and Effectiveness of using Stainless Steel Girders with Corrugated Webs in Bridge Design
ADAM HENRYSSON
ELLY YMAN
Department of Architecture and Civil Engineering
Chalmers University of Technology

Abstract

Steel-concrete composite bridges that are designed in Sweden today are almost exclusively conducted in carbon steel. Carbon steel is susceptible to corrosion, a consequence of which is the need to maintenance, mainly in form of repainting, surveillance, and other related work, during the service life of the bridge. A reduction of the maintenance costs is possible by instead utilising stainless steel in design of the girders. Bridges in stainless steel are nearly maintenance-free and not susceptible to corrosion. However, manufacturing of bridges using stainless steel is more costly than carbon steel and therefore, effective material utilization is desirable. In this regard, beams with corrugated webs are advantageous as larger material utilization can be achieved. Girders with corrugated webs provide high shear-buckling capacity, resulting in slenderer web plates. Further, the proof stress of stainless steel is higher than that for conventional carbon steel that is often used in bridge sector.

The aim of this study has been to evaluate the applicability and effectiveness of designing composite bridges by replacing carbon steel girders with flat webs, with stainless steel girders with corrugated webs. Further, the aim has been to highlight specific parameters that influence the design of such girders, and conclude the study by showing the possible savings in terms of material usage and cost estimation that could be achieved by implementing this concept. The latter is carried out by redesigning two existing composite bridges in a comparable manner to the original design.

The results of the case studies show that material savings of 20-30% can be achieved by implementing the suggested concept. Furthermore, if the height of the girders were allowed to be deeper, a quadratic saving of material could be obtained for every linear increase in depth. However, there exists an optimum depth from which no excessive material savings are made by further increasing the height. The study also shows that girders designed in stainless steel are subjected to larger compressive stresses due to temperature strains. This is mainly due to the discrepancy of coefficient of thermal expansion between stainless steel and concrete. Lastly, the cost estimation analysis shows nearly the same investment costs for the original design, and the alternative with stainless steel and corrugated webs thanks to the material savings and less production costs. The cost savings over the service life of the bridge makes the investigated concept extremely beneficial.

Keywords: Stainless steel, Corrugated web, Material savings, Maintenance costs, Bridge design, Steel-concrete composite bridges

Design av Samverkansbroar i Betong- och Rostfritt Stål med Korrugerade Livplåtar
En studie av tillämpningen samt dess effektivitet av rostfria stålbalkar med korrugerade livplåtar
ADAM HENRYSSON
ELLY YMAN
Institutionen för Arkitektur och Samhällsbyggnadsteknik
Chalmers Tekniska Högskola

Sammanfattning

De samverkansbroar i stål och betong som byggs, i Sverige, idag är nästan uteslutande utförda i kolstål. Kolstål är känsligt för korrosion vilket skapat ett behov av underhåll, främst i form av ommålning, övervakning och annat relaterat arbete under brons livslängd. En minskning av underhållskostnaderna är möjlig genom att istället konstruera broar i rostfritt stål. Rostfria stålkonstruktioner är nästan underhållsfria och inte utsatta för korrosion på samma sätt. Priset på rostfritt stål är dock dyrare än priset för kolstål. Det är därför viktigt att optimera designen, med hänsyn till materialanvändning, och därmed minska investeringskostnaderna. I förlängningen gör detta det föreslagna konceptet till ett mer konkurrenskraftigt alternativ. Rostfria ståltvärsnitt med korrugerade liv kan implementeras för att optimera designen med hänsyn till att en större del av tvärsnittet kan utnyttjas. Korrugerade livplåtar tillhandahåller en hög skjuvbucklingskapacitet, vilket resulterar i att tunnare livplåtar kan användas. Dessutom är flytspänningen för rostfritt stål högre än för konventionellt kolstål, vilket används inom brokonstruktion idag.

Syftet med denna studie har varit att studera användbarheten och effektiviteten för stålbalkar utförda med korrugerade liv, jämfört med tvärsnitt i kolstål och platta liv. Vidare har syftet varit att finna specifika parametrar vilka påverkar designen samt avrunda studien genom att visa vilka besparingar, gällande materialanvändningen, som kan uppnås genom att implementera det föreslagna konceptet. Det senare utförs genom att omkonstruera befintliga brokonstruktioner enligt samma dimensioneringsregler vilka gällde vid den ursprungliga dimensioneringen.

Resultaten visade att materialbesparingar mellan 20-30% kan uppnås genom att implementera det föreslagna konceptet. Om balkens höjd utförs högre fås en kvadratisk materialbesparing för varje linjär ökning av livhöjden. Det finns dock en gräns från där inga ytterligare besparingar görs. Den maximala besparing som uppnåddes vid uppökning av balkens höjd var cirka 50% jämfört med den ursprungliga designen. Studien visade också att balkar utförda i rostfritt stål utsätts för stora tryckspänningar orsakade av temperaturlaster. Dessa temperaturlaster beror på ett högre värde av värmeutvidgningskoefficienten jämfört med en konventionellt utformad samverkansbro i stål och betong. Avslutningsvis visade en kostnadsuppskattning över brons livstid att kostnaden är lägre för en bro med korrugerade livplåtar i rostfritt stål.

Nyckelord: Rostfritt stål, Korrugerade livplåtar, Materialbesparing, Underhållskostnad, Brodesign, Samverkansbroar i stål och betong

Acknowledgements

This master thesis is a part of a research project at Chalmers University of Technology, conducted in collaboration with the Bridge and Hydraulics Department at WSP, in Gothenburg. The thesis was carried out at the masters programme Structural Engineering and Building Technology at the Department of Architecture and Civil Engineering.

Firstly, we would like to thank our supervisor at WSP, PhD. Mohsen Heshmati, for his tirelessly commitment to this thesis, offering his guidance and support, regardless if it has been day or night. Further we would like to thank our examiner Senior Lecturer Mozhdeh Amani for her engagement and guidance writing this report.

Secondly, we would like to thank Associate Professor Mohammad Al-Emrani for his expertise and dedication on the subject, and willingness to help us out throughout the thesis. We would also like to thank WSP's Bridge and Hydraulics Department in Gothenburg for welcoming us into the compartment, providing us with encouragements whenever needed.

Lastly, we would like to thank Lars-Åke Persson at Stål och Rörmontage AB and Robert Hällmark at Trafikverket for their help providing us with cost estimations for our conducted case studies as well as costs for already built structures.

Adam Henrysson and Elly Yman, Gothenburg, June 2020

Contents

List of Figures

List of Tables

Nomenclature

Glossary

BaTMan	The Swedish Transport Administration's database for bridge- and tunnel structures
Cross-beam	Is the bridge member resisting lateral movements
Cross-bracing	Is a type of system resisting lateral movements
Eurocode	A collection of European standards specifying how structural design should be conducted within the European Union
Fatigue limit state	State that the structure is evaluated in in order to make sure requirements on the fatigue is fulfilled
Hogging moment	Is the negative bending moment
Krav Brobyggande	Guideline for design of new bridges, provided by Swedish Transport Administration
National Annex	Is a national annex to the Eurocodes, abbreviated as TSFS
Navier's Formula	Commonly known formula for calculation of stresses
Partial Factor Method	Method for design in the ultimate limit state. Recommended to use according to the Eurocodes
Poisson's ratio	Is a material constant, = 0.3 for steels
Serviceability limit state	State associated with the functionality of a structure
Sogging moment	Is the positive bending moment
Superstructure	Is the part of the bridge structure carrying the main load to either of the supports
Swedish Transport Administration	Authority responsible for Sweden's long- term infrastructure planning
Ultimate limit state	State associated with collapse or other related failures in the structure

Abbreviations

FAT	Fatigue Limit State
LCA	Life- cycle cost analysis
LT-buckling	Lateral-torsional buckling
NA	National Annex
SLS	Serviceability limit state
TSFS	Transportstyrelsen föreskrifter och allmäna råd om tillämpning av eurokoder
ULS	Ultimate limit state

Roman upper case letters

A_c	Area of concrete
A_{comp}	Area of the composite cross-section
A_{st}	Area of the steel
D_x,	Geometrical factor considering global shear buckling
D_z	Geometrical factor considering global shear buckling
E_s	The modulus of elasticity for steels
$E_{s,ser}$	The secant modulus of elasticity for stainless steel
E_c	The modulus of elasticity for concrete
$E_{c,eff}$	The effective modulus of elasticity for concrete
I_y	Moment of inertia, strong axis
I_z	Moment of inertia, weak axis
$I_{y.comp}$	Moment of inertia for the composite cross-section
$M_{Rd,LT}$	Bending moment capacity with respect to LT-buckling
N_{cr}	Critical buckling load
$V_{Rd,w}$	Shear capacity, corrugated web
$V_{Rd,bw}$	Shear capacity, flat web

Roman lower case letters

a_1	Flat-fold length for corrugation, see Figure 2.5
$b_{eff.slab}$	is the effective width of the concrete slab, calculated in Section 3.3.2.1
f_{cm}	Mean compressive strength of concrete
f_u	Ultimate strength
f_{ub}	Ultimate strength of connector
f_y	Yield strength/Proof strength of steel member
f_{yb}	Yield strength of connector
g	$= 9.81\frac{m}{s^2}$, is the acceleration constant
g_{st}	Self-weight of the steel cross-section
h_0	Equivalent height for the concrete slab; used for creep calculations
h_w	Height of the web
$h_{m,slab}$	Mean thickness of the concrete slab
k_T	Buckling coefficient for shear buckling, flat webs
k_σ	Buckling coefficient for plate buckling
n	The Ramberg-Osgood parameter
n_l	Modular ratio
r_c	Ratio between the corrugation's flat-fold length and corrugated length
t_w	Thickness of the web
z_{tp}	The centre of gravity

Greek letters

α	Angle for the corrugation, see Figure 2.5
α_{stud}	Correction factor for the length-diameter ratio for the shear stud
χ_{LT}	Reduction factor for LT-buckling
$\chi_{c,l}$	Reduction factor for local shear buckling
$\chi_{c,g}$	Reduction factor for global shear buckling
χ_c	Reduction factor for shear buckling
ϵ	Reference factor for cross-section classification
ϵ_{cs}	Shrinkage strains
ϵ_{temp}	Temperature strains
γ_{M0}	Partial safety factor for steels
γ_{M1}	Partial safety factor for steels
γ_{M2}	Partial safety factor for steels (both cross-section and connections)
γ_v	Partial safety factor for steels (both cross-section and connections)
$\lambda_{c,l}$	Slenderness, local shear buckling
$\lambda_{c,g}$	Slenderness, global shear buckling
λ_{LT}	Slenderness, LT-buckling
λ_p	Slenderness for plate buckling
λ_w	Slenderness of the web
$\phi(t_0, \infty)$	Final creep function
ρ	Reduction factor for calculation of effective flange width
σ	Normal stress
σ_E	Elastic buckling stress
$\sigma_\perp$	Normal stress perpendicular to the weld throat
$\sigma_\parallel$	Normal stress parallel to the weld throat
τ	Shear stress
$\tau_{cr,l}$	Critical shear stress, local shear buckling
$\tau_{cr,g}$	Critical shear stress, global shear buckling
τ_{cr}	Critical shear stress, flat web
$\tau_\perp$	Shear stress perpendicular to the weld throat
$\tau_\parallel$	Shear stress parallel to the weld throat

1 Introduction

Chapter one aims to provide an overview for the prerequisites of this thesis. First a brief background is given on the subject before the aims and objectives of the thesis are presented. Lastly the chosen approach to conduct the thesis as well as the limitations are described.

When constructing a bridge, various parameters influence the overall choice of design. The parameters to be considered could be the time it takes to erect the bridge, the self-weight of the structure being imposed on the foundation as well as material usage allowing smaller investment costs (Pipianto and De Miranda, 2016). With these parameters in mind, steel and concrete composite bridges are efficient bridge types that to some extent are being used in Sweden.

Steel and concrete composite bridges are usually designed with steel girders with flat webs that interact with a concrete slab on top. The steel girders are normally produced out of regular carbon steel that is susceptible to corrosion. In order to protect the girders against corrosion, a protective layer of paint is normally added during construction and then again whenever needed during its service life. If the carbon steel is replaced by stainless steel, the susceptibility against corrosion would be decreased, and maintenance costs could be reduced. As the price for stainless steel is approximately three times that of regular steel, it is crucial to reduce the amount of material used (Karlsson, 2018). As an example, if corrugated webs are used in a concrete box girder bridge, the self-weight of the superstructure can be reduced 10-30% compared to a traditional concrete box girder bridge (Boutillion et al., 2015).

Designing steel and concrete composite bridges with corrugated webs has mainly been used in Japan (Boutillion et al., 2015). The main benefit of replacing the traditional flat web with a corrugated one is the saving of material; the number of stiffeners as well as the thicknesses of the members could be reduced.

This master thesis was initiated by Chalmers University of Technology as part of a larger project on a new concept that incorporates lightweight steel girders with corrugated webs. The thesis is conducted at WSP's Bridge department in Gothenburg.

Hereafter the *steel and concrete* composite bridge is referred to as a composite bridge.

1.1 Aim

The aim of this master thesis is to study the applicability and effectiveness of using stainless steel girders with a corrugated web when designing composite bridges.

1.2 Objective

The objective of this thesis is to show the potential in material savings when using stainless steel girders with corrugated webs in composite bridges when comparing the concept to carbon steel girders with flat webs. More specific the study will investigate to see what most influences the design of the cross-section when they are designed in stainless steel using a corrugated web.

1.3 Limitations

- The thesis is restricted to design procedures according to the *Eurocodes* and regulations given by the *Swedish Transport Administration* only.
- The design procedure will be formulated for the steel girder only. The design of the concrete slab is disregarded.
- The studied composite bridge type is the twin-girder composite bridge.
- Only trapezoidal corrugation shapes for the web will be considered.
- Only road bridges will be covered in the study.
- The material savings will be compared to an already built bridge without any considerations of the original designer's choices.

1.4 Approach

The thesis is carried out in different stages. Firstly, a literature study is conducted. The literature study is followed by a formulation of a general design procedure for composite bridges. Further, two case studies is conducted for already built bridges, and lastly a parameter study is carried out for factors influencing the design of a stainless steel girder with corrugated webs.

1.4.1 Literature study

The literature study covers the design procedure of composite bridges, characteristics of stainless steel and the design of structures using corrugated webs instead of ordinary flat webs. The information gathered is both taken from books written on the given subjects as well as articles. However, there are not much literature combining the design of structures in stainless steel to girders with corrugated webs. In addition to the literature on the subject, a handful of calculation reports will be studied for how new bridges have been designed in the last decade by engineers in the industry.

1.4.2 General design procedure

When a good background on the subject is obtained, the general design procedure, according to the design codes for composite bridges, is described. The general design procedure is formulated for both a flat web using carbon steel and for a corrugated web in stainless steel. The design of the connection elements that transfer shear flow between the steel and the concrete is also covered. Different steps and the approaches used in the industry when designing a composite bridge are clarified.

1.4.3 Case studies

Two case studies are performed on existing structures designed according to the current standards. These studies cover a redesign of the structure adopting the concept of using corrugated webs with girders in stainless steel. Special attention is put into identifying the main differences in design, and the parameters that have the greatest influence on the outcome of the design. A comparison is made between the total material usage for the original design and the redesigned structure. Further, a cost estimation for one of the bridges is carried out, comparing the concept of using a stainless steel girder with corrugated webs to a conventional carbon steel girder with flat webs.

1.4.4 Parametric studies

The parameters that during the case studies are identified to have a great influence on the outcome of the design are further investigated. The investigation aims to provide a groundwork for future design work of stainless steel girders with corrugated webs.

2
Literature study

The literature study is providing the foundation for the subsequent work of composing a general design procedure for composite bridges with corrugated webs in stainless steel. The chapter starts with an introduction to composite bridges, followed by specifics regarding the behaviour when replacing flat webs with corrugated webs. Lastly, the specifics of stainless steel are described.

2.1 Composite bridges

A composite structure is a structure where the main load-bearing elements consist of two parts with different material properties in interaction with each other (Collin, Johansson, and Sundquist, 2008). The structural concept of composite bridges in bending is to utilize the compressive strength of concrete combined with the tensile strength of steel (Utsi and Lagerqvist, 2012). The interaction provides a stiffer behaviour than if no structural interaction takes place.

Composite bridges could be designed according to different concepts. Two examples are the *box girder bridge* or the *twin girder bridge*, see Figure 2.1 and 2.2. Composite bridges has a wide range of applications and can be designed both as simply supported and as continuous systems. One common design of a composite bridge are two steel beams, usually welded I-beams, with a concrete slab on top (Collin et al., 2008). If the steel beams has the same cross-section this system is referred to as a twin girder bridge. In order to have interaction the girders and the slab are connected via shear connectors transferring the shear flow between elements. A twin girder system is mainly used in spans ranging from 30 to 60 meters. In Figure 2.2 the cross-section of a typical twin girder bridge is presented.

One of the advantages of composite bridges in comparison to concrete bridges is the shortened erection time (Collin et al., 2008). The girders are usually prefabricated and put into place using different erection methods, see Section 2.1.3. Another benefit of using composite bridges is the reduced self-weight of the superstructure compared to a concrete bridge leading to cheaper and less foundation work for the overall structure (Vayas and Iliopoulos, 2014).

A disadvantage of the twin girder bridge is the lack of redundancy (Vayas and Iliopoulos, 2014). If either of the girders were to fail it will likely lead to a total failure of the structure. Furthermore the use of carbon steel in bridge design may cause problems as its susceptibility to corrosion can lead to high maintenance costs

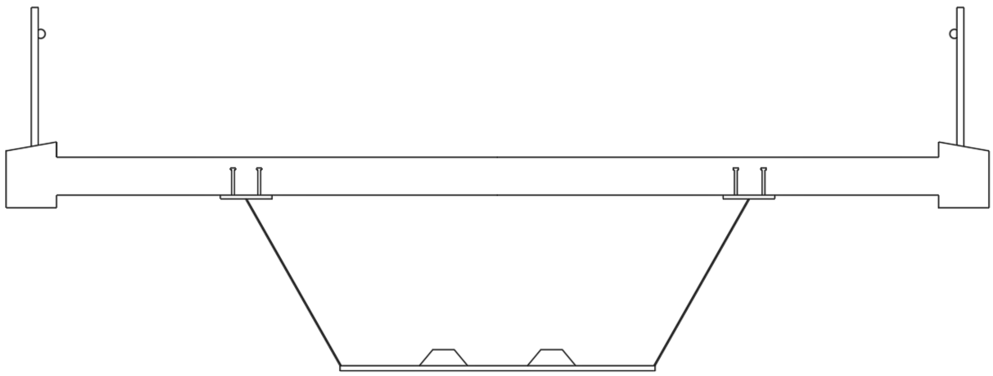


Figure 2.1: A typical box girder composite bridge

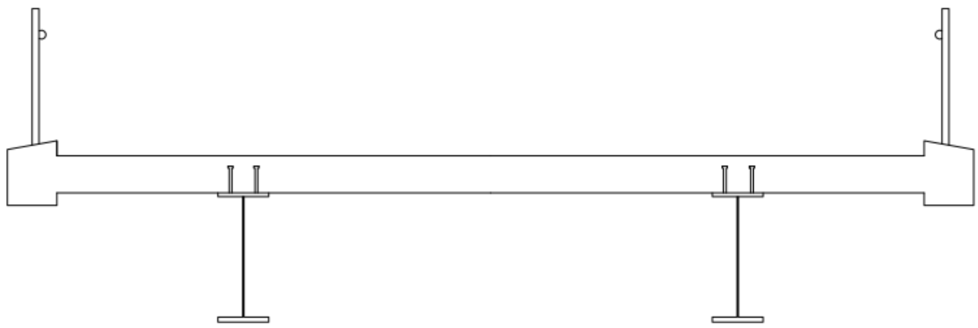


Figure 2.2: A typical twin girder composite bridge

during the service life.

2.1.1 Cross-sectional design

As mentioned in section 2.1 a twin girder composite bridge is typically designed with two symmetrical steel girders. The thickness of the concrete slab often varies from 25 to 35 centimeters and is casted on top of the steel girders (Collin et al., 2008). If the bridge is simply supported the cross-section is normally constant along its length (Vayas and Iliopoulos, 2014). For a continuous bridge the height of the girders are usually increased over supports in order to resist the interaction between hogging moments combined with high shear forces.

For a simply supported composite bridge, the utilization of the steel and concrete members is optimal with regards to their behaviour during loading (Utsi and Lagerqvist, 2012). The steel will act in tension, and the concrete in compression. However, composite bridges are also used as continuous systems where special attention must be taken to the behaviour of concrete over the support. The concrete will be in tension; hence cracking is expected. In order to limit the crack width, tensile reinforcement is needed. Note that reinforcement still might be needed in a simply supported bridge in order to localize cracks caused by compulsive stresses resulted from effects such as temperature and shrinkage.

2.1.2 Interaction between parts

The bonding between the steel flanges and the concrete slab is not enough to transfer the shear stresses that arises, in the intermediate plane, and full interaction between elements can not be ensured (Collin et al., 2008). Therefore, connection elements need to be attached to the flange. There are various types of connection elements to transfer the shear flow between the steel girders and the concrete slab. The most common connection element is welded shear studs, see Figure 2.3. The studs are welded to the flange, and once the concrete is hardened, the connection enables composite action.

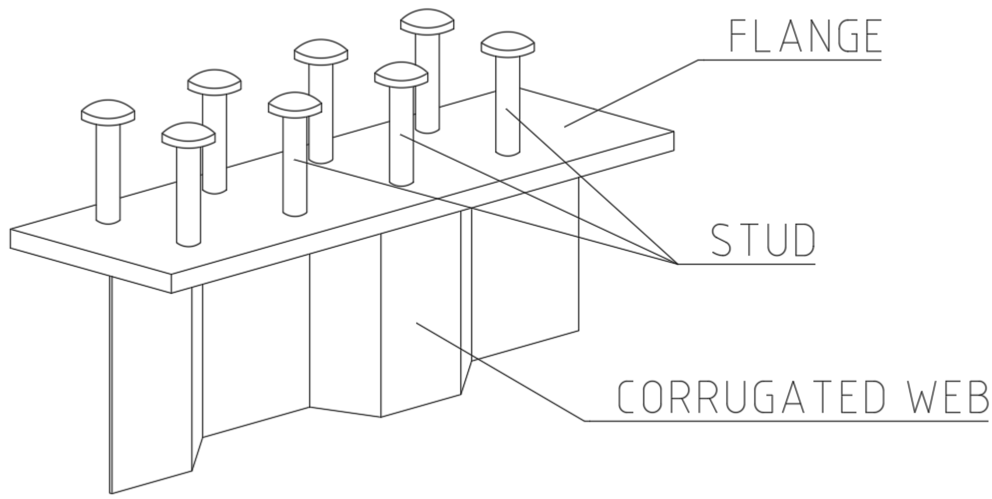


Figure 2.3: Welded shear studs on top of the flange

2.1.3 Construction phase

Depending on the bridge type and its intended design, there are several different techniques for the erection of the structure (Collin et al., 2008). The most commonly used techniques are launching the girders and lifting the girders into place. Launching is a technique for twin-girder bridges where the girders are launched from one end towards the other. If longer beams are desired, the full beam is produced by welding together smaller beams on the launching spot. Intermediate temporary lightweight supports may be used in order to reduce the length of the cantilever part of the beams. The girders are pushed forward using hydraulic jacks, and finally placed on sliding supports or big roller bearings. Lifting of the beams means that a mobile crane lifts the steel girders into place. The lifting method is the fastest and easiest erection method; however, it is limited by the lifting capacity of the cranes, meaning that it is not always the most suitable method. It is the most commonly used method for simply supported twin-girder composite bridges (Boutillion et al., 2015).

Once the beams are in place, the formwork for the concrete slab is mounted. The formwork is, in most cases, supported by the steel beams (Collin et al., 2008).

Thereafter, the placing of reinforcement and the casting of concrete procedure starts. During the design of the bridge, it is important to consider the construction phase, since the interaction between the steel and concrete elements is first achieved once the concrete is hardened. In order to reduce the risk of *lateral-torsional buckling* (LT-buckling) of the girders during casting, the steel girders could be fitted with cross-bracing. When the concrete has hardened, it will provide the needed lateral support for the top flanges.

The technique of using the steel beams as load-bearing elements for the formwork, reinforcement, and fresh concrete reduces the need for temporary supports during erection (Collin et al., 2008). This is favorable as less on-site labour is needed, and the construction time is shortened. Furthermore, it is beneficial when the construction of the bridge takes place over a road where the use of temporary support would lead to rerouting or conjunction of traffic. However, it is sometimes preferred to use temporary supports during the hardening of the concrete in order to reduce the risk of early cracking over internal supports and to reduce the need for intermediate cross-bracing due to the risk of LT-buckling (Vayas and Iliopoulos, 2014).

2.2 Corrugated webs

High load-bearing capacity is often correlated to deep cross-sections. However, deep cross-sections may lead to instability issues such as LT-buckling and out-of-plane shear buckling in the webs (Boutillion et al., 2015). If instability problems occur during the design process, several alternatives are suitable such as increasing the web thickness, adding transverse stiffeners, and, in some cases, attaching longitudinal stiffeners to the web.

Another approach for maintaining high load-bearing capacity in the web as well as reducing the overall material consumption is to replace the conventional flat web with a corrugated one (Boutillion et al., 2015). Corrugated webs have a high out-of-plane capacity due to its stiffness around the weak axis. Consequently, adopting the concept of using a corrugated web often leads to a reduced or no need for stiffeners and allows for a thinner web thickness. In bridges, the most common corrugation shape is the trapezoidal shaped webs (Karlsson, 2018). Another shape used is the sinusoidal shape. Both shapes provide high shear capacity for relatively thin plates.

A corrugated web deforms similarly to an accordion as it lacks stiffness in the longitudinal direction (Boutillion et al., 2015). The flanges of the girder will then solely resist the resulting axial forces arising from the bending moments or normal forces. The stresses arising in the web mainly comes from shear forces.

2.2.1 Manufacturing of trapezoidal shapes

The corrugation is produced by pressing flat steel plates into the desired shape (Boutillion et al., 2015). The form of the trapezoidal shape is relatively easy to produce, meaning that there are little limitations on what form to chose during the design process. However, there is an optimal shape to choose with regards to the maximum shear capacity, dependent on the bridge's overall geometry (Karlsson, 2018). Figure 2.4 illustrates the shape of a trapezoidal web.

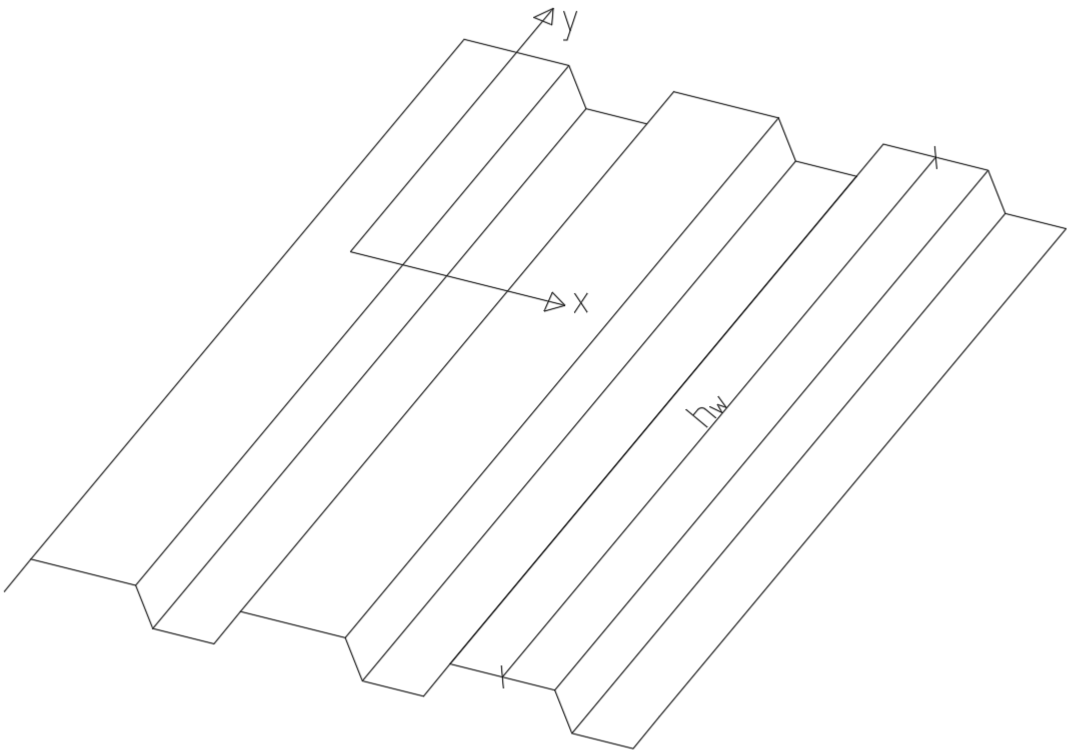


Figure 2.4: Corrugated web with trapezoidal shape

2.2.2 Design of corrugation

In a cross-sectional analysis, it is assumed that the corrugated web resists the shear forces whilst the flanges resist the bending moment. If the flanges are not fully utilized in bending, they may, however, contribute to the shear capacity (Boutillion et al., 2015). The geometrical notations for corrugated webs showed in Figure 2.5 are commonly known and used throughout the thesis.

The geometry of the corrugation has a large influence on the shear capacity of the web. In an earlier conducted master thesis by Karlsson (2018), it was concluded that it is good to perform a parametric study on the specific bridge to be designed in order to find the optimal shape to withstand shear buckling. According to the same author there are for corrugated webs three buckling modes related to shear failure; local, global and interactive. In a parametric study conducted by Karlsson (2018), it was showed that the shape of the corrugation for a certain beam affects the buckling mode that would occur. The most influencing parameters were the flat-fold length (a_1), corrugation angle (α), the height of the web (h_w), and web thickness (t_w). More specifically it was proven that there are optimal values to be chosen for a_1 and α for a given web height and web thickness. A summary of the parametric study is

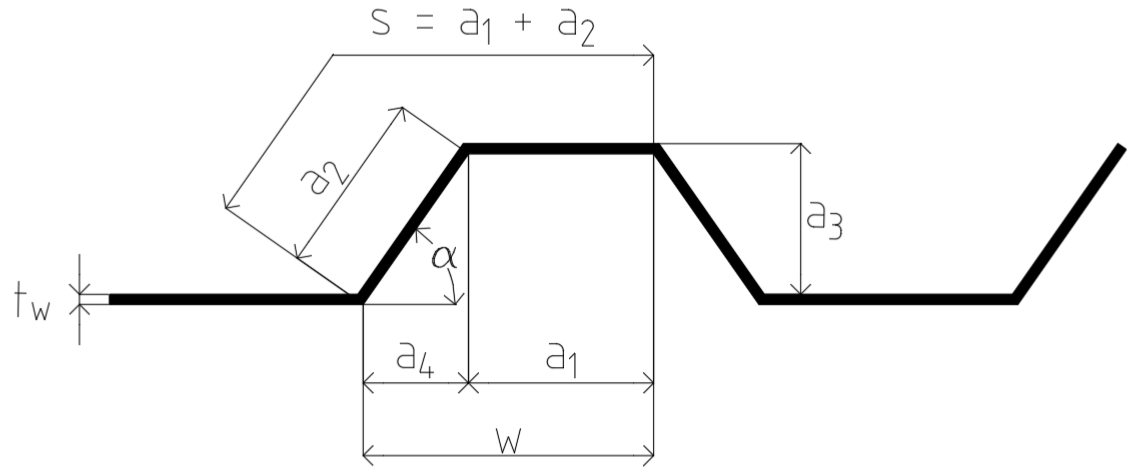


Figure 2.5: Geometric notations for corrugated webs (Karlsson, 2018).

presented in Table 2.1. The table shows how the ultimate shear capacity, V_{Rd}, and how the resulting buckling mode are affected for increased sectional dimensions.

Table 2.1: Summary of parametric study conducted by Karlsson (2018)

Description	**Ultimate shear capacity**	**Buckling mode**
Web thickness, t_w	Increases for increased t_w	Goes from local to interactive to global for increased t_w
Web height, h_w	Increases for increased h_w	Goes from local to interactive to global for increased h_w
Corrugation angle, α	Increases to a certain point for an increasing angle, α; reaches its maximum at approximately 50 degrees	Goes from global to interactive for increased α
Flat-fold length, a_1	There is an optimal length, a_1, to be found for a specific web design	Goes from global to interactive to local for increased a_1

Based on the results in Karlsson (2018), the ultimate shear capacity of a beam is mainly affected by the corrugation angle, α, and the flat-fold length, a_1. As presented in Table 2.1 an increased angle results in an increased shear capacity up to a certain point. At approximately 50 degrees, the shear capacity reaches its maximum value, however, for an angle between 30 to 50 degrees, a rather high shear capacity is achieved.

In Karlsson (2018) the optimal flat fold length was investigated for different web heights. The evaluated web heights were $h_w = 1000, 1500, 2000,$ and 2500 millimeters. The corrugation angle, α, was set constant to 32 degrees, and the web thickness was $t_w = 4$ millimeters. The highest shear capacity for the tested range of web heights was achieved for a flat fold length of 100 to 150 millimeters.

2.2.3 Existing bridges with corrugated webs

The concept of using corrugated webs in composite bridges was developed during the 1960s in Japan, although it was not implemented in bridges until 20 years later in France (Boutillion et al., 2015). France was the first country in the world to build a composite bridge with a corrugated web. The Cognac bridge is a box girder composite bridge and was completed in 1986. In 1992 Japan built their first bridge using the concept. Since then, Japan but also France, has been the frontiers in the field, and Japan has built over 140 bridges with corrugated webs. Other composite bridges using the concept has been built in South Korea and Germany. In Table 2.2, examples of the geometrical shape of the corrugation, bridge type, and longest span are described for some existing bridges (Karlsson, 2018).

Table 2.2: Existing composite bridges with corrugated web (Karlsson, 2018)

Bridge	a_1 [mm]	t_w [mm]	h_w [mm]	α [deg]	**Span length** [m]	**Bridge type**
Shinkai	250	9	1183	36.9	31	Box girder
Hondani	330	9	3315	36.5	97.2	Box girder
Cognac	353	8	1771	25.2	43	Box girder
Maupre	284	8	2650	31.9	53.5	Box girder
Dole	430	10	2546	30.7	80	Box girder

2.3 Stainless steel

Stainless steel is an attractive alternative to conventional carbon steel when there are demands on the resistance against corrosion. In structures, stainless steel has been used since it was invented more than 100 years ago (Stålbyggnadsinstitutet, 2017), although data on its implementation in bridge design is lacking. In recent years, a greater attention is paid to the life-cycle costs, not only focusing on the investment cost. Stainless steel is more expensive than carbon steel, but with lower costs for future maintenance and renovations, it is still a good alternative. Furthermore, stainless steel is fully recyclable at the end of its life-cycle, creating a new market for when the structures are outdated.

Just like carbon steel, the strength properties of stainless steels are good, see Section 2.3.1, meaning that its application is suitable in bridge design. If the content of chromium is at least 10.5%, the steel is classified as stainless steel. Stainless steels are in some manner self-healing thanks to their transparent passive layer of chromium oxides formed on the surface when oxygen is present. Its protection against corrosion increases with increased content of chromium.

The material is sub-categorized into four main steel types; ferritic, martensitic, austentic, and duplex (Stålbyggnadsinstitutet, 2017). Duplex and austentic are more suitable for bridge applications due to their high resistance against corrosion, formability, and high strength. One of the differences between the four stainless steel-categories is the chromium content, which influences the susceptibility against corrosion. The choice of steel is based upon the environment that the structure will be placed in, the investment cost, and the required strength.

In addition to replacing carbon steel with stainless steel in composite bridge girders, the reinforcement in the concrete slab could be replaced as well. Using reinforcement made in stainless steel in highly exposed areas of the slab increases the durability and decreases the life-cycle costs of the structure as the need of maintenance decreases (Dahlström and Persson, 2018).

2.3.1 Properties of Stainless steel

The stress-strain curves for stainless steels differ from the one for conventional carbon steels (Stålbyggnadsinstitutet, 2017). Carbon steel has a distinct yield point from where plastic deformation takes place. For stainless steel, the transition is more smooth as it early departs from the linear elastic behaviour due to strain hardening, see Figure 2.6. The yield stress for stainless steel is normally calculated by the stress corresponding to 0.2% permanent strain, but also dependent on the thickness of the member. In Table 2.3, nominal values for the proof strength and the ultimate strength are given. The presented steel grades are based upon recommendations for corrositivity class C4 according to the recommendations given by *The Swedish Transport Administration*, see Section 3.1.1.

Table 2.3: The Swedish Transport Administration's recommendations for stainless steel grades in bridge design and their nominal values (nominal values according SS-EN 1993-1-4:2006/A1:2015)

Type	**Grade**	$t \leq 8mm$		$t \leq 13.5mm$		$t \leq 75mm$	
		f_y [MPa]	f_u [MPa]	f_y [MPa]	f_u [MPa]	f_y [MPa]	f_u [MPa]
Austenitic	1.4401	240	530	220	530	220	520
	1.4404	240	530	220	530	220	520
	1.4571	240	540	220	540	220	520
Duplex	1.4162	530*	700*	480**	680**	450	650
	1.4362	450	650	400	650	400	630

*$t \leq 6.4mm$
**$t \leq 10mm$

2.3.1.1 Manufacturing and its influence on the strength properties

Unlike the properties of carbon steel, the manufacturing process has a large influence on the final properties of stainless steel (Stålbyggnadsinstitutet, 2017). There are

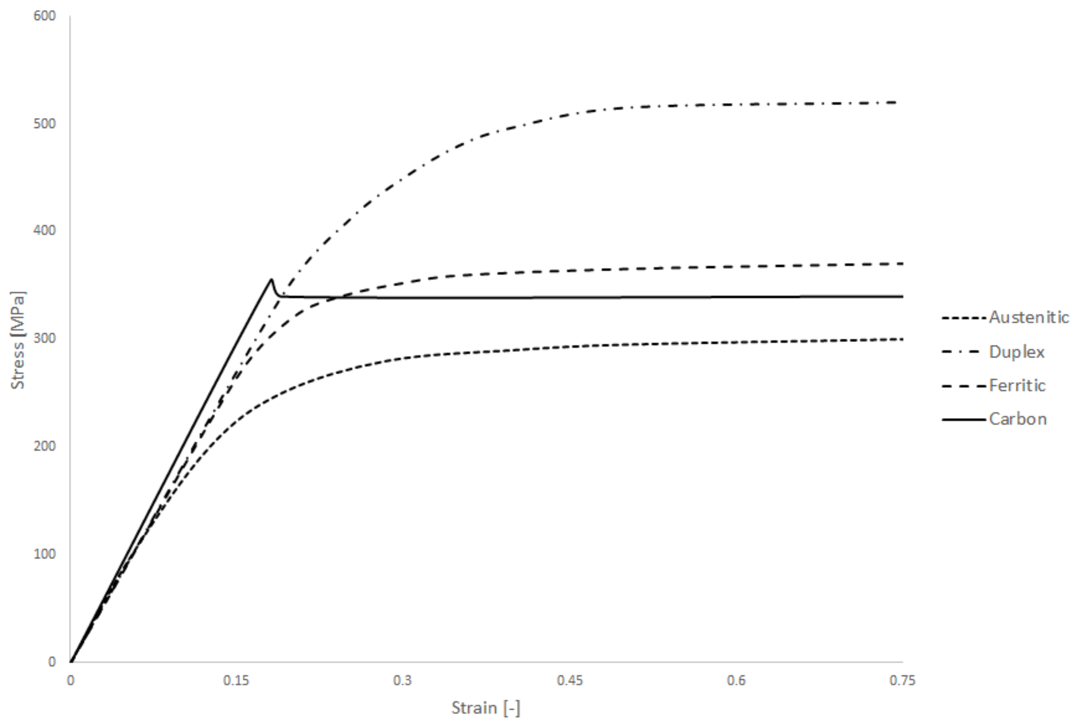


Figure 2.6: Stress-strain curve for stainless steels in relation to carbon steel

two main production techniques, annealing and cold-forming. Annealing of stainless steel means that the steel is formed to the required dimensions and then heated to change the microstructure of the material. Cold-forming of stainless steel is when the material is cold-formed into the desired dimensions in order to achieve strain hardening in the material and thereby change the material properties. Both austenitic and duplex steels achieve higher strength properties by cold-forming even if some ductility is lost. However, stainless steel is a ductile material, so the eventual loss of ductility is not a determining issue. Furthermore, cold-forming of the material is reversible at any time of the service life. By annealing the original properties can be retrieved.

Work hardening of the steel creates asymmetry in the material structure leading to anisotropy (different strength properties in tension and compression), meaning that separate stress-strain curves are needed to describe each behaviour (Stålbyggnadsinstitutet, 2017). In general, the tensile strength increases more than the compression strength during work hardening. If a cold-formed member of a structure is welded, the rise in temperature may cause it locally to anneal and so display a loss of strength locally.

2.3.1.2 Corrosion

The corrosion resistance of stainless steel is dependent on their microstructure descending from the used alloys (Stålbyggnadsinstitutet, 2017). Therefore, it is important to consider the environment the part is to be placed in, meaning that different alloys will react differently depending on the surroundings. In general, the higher corrosion resistance, the higher the material cost, with the exception that duplex steels could have increased resistance without any increase in price. The Swedish Transport Administration regulates which corrositivity resistance that is required for a certain environment (Krav Brobyggande, 2019).

2.3.2 Basics for design

The design of a cross-section in stainless steel is much like the design of a carbon steel cross-section (Stålbyggnadsinstitutet, 2017). The *height-width ratio* (c/t ratio) is determining for the parts subjected to compression stresses, which may cause the cross-section to buckle before full utilization of the ultimate strength is reached. If the c/t ratio is too large the strength of the cross-section needs to be reduced. In the same manner as the design for carbon steel, *classification of compression elements* are done in order to determine if or how much the strength needs to be reduced. There are four classes formulated in SS-EN 1993-1-4 (2006).

Class 1, The cross-section can develop a plastic hinge with the needed rotational capacity needed for plastic analysis.

Class 2, The cross-section can reach plastic moment capacity but not form a hinge.

Class 3, The parts in compression of the cross-section may reach yielding but because of the risk for buckling, the plastic moment capacity can not be reached.

Class 4, The cross-section will buckle in one or more parts before the yield stress is reached.

The classes are formulated the same for both stainless steel and carbon steel in SS-EN 1993-1-1 (2005) and SS-EN 1993-1-4 (2006). However, the limits for the c/t ratio, determining which class the part belongs to, differ. The main influencing parameters are the elastic modulus, the yield strength, and the c/t ratio. Hence, the difference in the reduction for either carbon steel cross-sections or stainless steel cross-sections needs to be determined in each case as the material properties may vary for different kinds of steels.

In bridge design the web and the flanges often have relatively large c/t ratios and therefore often ending up in cross-section class four. For parts in class four, effective widths are calculated according to SS-EN 1993-1-5 (2006) and SS-EN 1993-1-4 (2006). The principle for calculations is the same for both carbon and stainless steel with only the reduction factor, ρ, differing when calculating the reduction of parts subjected to compression.

2.3.2.1 Determination of the modulus of elasticity in SLS

Stainless steel display a non-linear stress-strain behaviour, and therefore the modulus of elasticity is not constant until plastic hinges have evolved (Stålbyggnadsinstitutet, 2017). Therefore, when calculating the deflection in the *Serviceability Limit State* (SLS), the modulus of elasticity must reflect the stress distribution over the length of beam. This is done by calculating a mean value for the secant modulus of elasticity based upon the current stresses in the flanges. The current stress is the corresponding stress for the SLS-loads calculated using *Navier's formula*. The modulus is then adjusted with regards to the current stress in the flanges, the yield strength, and the Ramberg-Osgood parameter, n. This parameter depends on the rolling direction and steel grade, and for low values, it indicates a high grade of non-linearity. Note that this method is correct as long as the secant modulus is based upon the maximal stress in the upper and lower flange and that the stresses do not exceed 65% of the 0.2% remaining strain limit. If the stress exceeds 65% of the 0.2%-limit, the modulus is much on the safe side, and so a more precise value should be calculated.

3

General design procedure

The general design procedure for designing a girder in a composite bridge with corrugated web in stainless steel is described in this chapter. Throughout the chapter, it will be clarified all steps that in any way, differ from the design of conventional flat web girders in carbon steel. The chapter will start going through the relevant standards that are to be followed during the design of a new bridge in Sweden. Thereafter all, by the standards compulsory, parts of the calculation report are presented including relevant design checks and calculation of stresses in a composite section.

3.1 Prerequisites

The prerequisites for a bridge is dependent on the requirements stated in the standards and the scope of the intended design. In section 3.1.1 the relevant guidelines for a composite bridge in stainless steel with corrugated web are presented.

3.1.1 Guidelines and Design codes

When a bridge is to be designed in Sweden today the guideline to be followed is *Krav Brobyggande*. This document regulates what to consider during the design process, and further refers to additional design codes whenever needed. The main design code that Krav Brobyggande refers to is *the Eurocodes*. The Eurocodes consists of ten standards (**Eurocode**) divided into several parts. For this study, several standards will be referred to and Table 3.1 clarifies the design codes that are relevant to the objectives of the thesis.

In addition to the Eurocodes presented in Table 3.1, Krav Brobyggande states that the *National Annex* (NA) named *Transportstyrelsens förskrifter och allmänna råd om tillämpning av eurokoder* (TSFS 2018:57), shall overrule the EN Standards whenever contradiction appear.

3.1.2 Software

During design of a composite bridge the used software needs to be carefully chosen. In the industry several computational programs are used in order to obtain load effects and design of structural parts such as reinforcement. The choice of software is dependent on the complexity of the structure. For the studied bridge type, simply supported composite bridges, the structure is statically determinate and no complex

Table 3.1: Content - European standards

Eurocode	Standard	Part
EN 1990	Basis of structural design	-
EN 1991	Actions on structures	Part 1-1: General actions Part 1-4: Wind actions Part 1-5: Thermal actions Part 2: Traffic loads on bridges
EN 1992	Design of concrete structures	Part 1-1: General rules Part 2: Concrete bridges
EN 1993	Design of steel structures	Part 1-1: General rules Part 1-4: Stainless steels Part 1-5: Plated structural elements Part 1-8: Design of joints Part 1-9: Fatigue Part 2: Steel bridges
EN 1994	Design of composite steel and concrete structures	Part 1-1: General rules Part 2: General rules for bridges

software is needed. Meaning that the theory of elasticity applies and all load effects could be calculated by hand for instance by using influence lines. It is however convenient to use a software for determination of load effects arising from traffic as the hand-calculations is time-consuming.

3.1.3 Safety class and partial safety factor

Each prospective construction is to be classified dependent on the risk for physical, economical, or social loss, for the case of structural failure. Krav Brobyggande refers to TSFS 2018:57 that presents four safety classes to chose from when designing a bridge, although there are exceptions and supplements stated directly in Krav Brobyggande. The *Partial Factor Method* could be used in design whereas the partial safety factor, γ_d, considers the targeted safety class. In Table 3.2, the safety classes, their scope, and the partial safety factor is presented.

Table 3.2: Safety classes and their corresponding partial safety factors

Safety class	Probability for severe injuries if failure	γ_d [-]
1	Low	0.83
2	Normal	0.91
3	High	1.0
4	Very high	1.1

The partial safety factor is used in load combination in the ultimate limit state, either increasing or decreasing the load dependent on the safety class.

3.2 Materials

According to SS-EN 1990 the design value for the strength properties of a material should be calculated according to Equation 3.1.

$$X_d = \eta \frac{X_k}{\gamma_m} \tag{3.1}$$

Whereas,

X_k	is the characteristic value for the strength properties of a material
η	is the mean value for the factor considering volume effects, moisture effects, temperature effects, and other relevant parameters
γ_m	is the partial factor for a certain material considering the probability of deviation in the properties, values for steels are presented in Table 3.3 and Table 3.8

3.2.1 Steels

When designing a structure in steel, the first step is to check Annex E in Krav Brobyggande which targets steel structures. Annex E specifies what environmental resistances to consider, how a certain part should be designed, and so on. For steel parts, it is determined that a thickness smaller than four millimeters is not allowed. Due to volume effects the strength of the material is dependent on the thickness of the studied part. Values for the nominal strength is listed in Table 2.3 and Table 3.9.

Dependent on the location of the structure, its parts are to be classified in different corrosivity classes considering the surrounding environment. According to Krav Brobyggande, the steel parts should be designed in at least corrosivity class C4 unless it is placed in a more hostile environment such as marine or road environment. Note that road environment only considers steel parts that are in direct contact with residues from the road such as road salts. Marine environment refers to parts that are in direct contact with water.

3.2.1.1 Stainless steel

The design of parts in stainless steel is based upon the equations given in SS-EN 1993-1-4. For the austenitic and duplex stainless steels treated in the standard, the ductility is assumed to be enough, as mentioned in Section 2.3.1.1, and no risk of brittle failure exist for temperatures above $-40°$ C. This means that the thickness of the steel plates do not need to be chosen with regards to the reference temperature unlike the case for carbon steel, which is further presented in Section 3.2.1.2.

The available steel grades and their strength properties for corrosivity class C4 is presented in Table 2.3. Partial coefficients for stainless steels are given by TSFS 2018:57 18 ch 2 §. Values are presented in Table 3.3.

Table 3.3: Partial safety factors for stainless steel

Notation	Description	Value
γ_{M0}	Resistance of cross-sections whatever the class is	1.0
γ_{M1}	Resistance of members to instability assessed by member checks	1.0
γ_{M2}	Resistance of cross-sections in tension to fracture	1.2

The elastic modulus of elasticity for determination of the cross-sectional resistances is given as E_s =200 GPa for the allowed steel grades according to SS-EN 1993-1-4. Since stainless steel displays a non-linear stress-strain behaviour the same modulus can not be used for both ULS-checks and SLS- checks, and therefore a secant modulus of elasticity is calculated according to the current stress from Equation 3.2.

$$E_{s,ser} = \frac{E_{s,1} + E_{s,2}}{2} \tag{3.2}$$

$$E_{s,i} = \frac{E_s}{1 + 0.002 \frac{E_s}{\sigma_{i,Ed,ser}} \left(\frac{\sigma_{i,Ed,ser}}{f_y} \right)^n} \tag{3.3}$$

Whereas,

$E_{s,1}$	is the secant modulus corresponding to the current stress in the tensile flange
$E_{s,2}$	is the secant modulus corresponding to the current stress in the compressed flange
$i = 1$	corresponds to the flange in tension
$i = 2$	corresponds to the flange in compression
$\sigma_{i,Ed,ser}$	is the absolute value for the largest stress
n	is the Ramberg-Osgood parameter according to Table 3.4
f_y	is the yield stress for the specific stainless steel, see Table 2.3

Table 3.4: Some values for the Ramberg-Osgood parameter, n, according to SS-EN 1993-1-4 Table 4.1, dependent on the rolling direction

Steel grade	Longitudinal	Transverse
1.4401	7	9
1.4404	7	9
1.4571	7	9
1.4362	5	5

3.2.1.1.1 Studs, screws and bolts

The studs that connects the upper steel flange to the concrete slab must be in stainless steel in order to avoid the probability of creating a galvanic cell due to either of the metals being more noble (Dahlström and Persson, 2018). The same goes for the connections of the cross-bracing. Dependent on the steel type and steel grade the steels are divided into groups, A1, A2, ... for austenitic steels and D2, D4, .. for duplex, presented in Table 3.5. The table shows what group of a certain steel grade belongs to. The groups are then divided into property classes dependent on the manufacturing method. The higher the property class, the higher the strength, whereas nominal values are listed in Table 3.6. The table shows what nominal value to choose for the connection dependent on the groups presented in Table 3.5. SS-EN 1993-1-4 provides the nominal values for the yield strength and the ultimate strength, f_{yb} and f_{ub}. The property class for austentitic steels is given by EN ISO 3506-1 depending on the method of manufacturing and size of the member, completions for duplex steel are given by (Stålbyggnadsinstitutet, 2017). The partial safety factors, γ_{M2} and γ_V, for connections in stainless steel are the same as for carbon steel presented in Table 3.11.

Table 3.5: Grouping of studs, screws and bolts in stainless steel dependent on the chosen steel grade

Type	**Grade**	**Group**
Austenitic steels	1.4401	A4
	1.4404	A4
	1.4571	A5
Duplex steels	1.4162	D4
	1.4362	D2

Table 3.6: Nominal strength values, f_{yb} and f_{ub}, for connectors dependent on the grouping and property class

Group	**Property class**	**Manufacturing method**	f_{yb} **MPa**	f_{ub} **MPa**
A4	50	Soft hot-pressed	210	500
	70	Cold-formed	450	700
	80	Cold-formed high strength	600	800
	100	Cold-formed high strength	800	1000
A5	50	Soft hot-pressed	210	500
	70	Cold-formed	450	700
	80	Cold-formed high strength	600	800
D2, D4	70	Cold-formed	450	700
	80	Cold-formed high strength	600	800
	100	Cold-formed high strength	800	1000

3.2.1.1.2 Welds

When choosing filler metal for the welds that connect the different parts it is recommended that the filler metal has at least the same properties as the parent metal (Stålbyggnadsinstitutet, 2017). An exception to this is that for austenitic steels in the cold-worked condition, the filler metal may have lower nominal strength than the parent material (ss-EN 1992-1-4:2006/A1:2015). Table 3.7 presents what filler metal to choose dependent on the parent material.

In addition to the above stated requirements on the choice of filler metal there are regulations on what tensile strength to use for cold-worked materials. For parent materials in the cold worked condition the resistance should be taken as the tensile strength of the annealed parent material (SS-EN 1993-1-4:2006/A1:2015). Furthermore for materials in the cold worked conditions the filler metals may have lower strength than the parent material, whereas the resistance should be taken as the nominal tensile strength of the filler metal.

Table 3.7: Recommended combinations for filler metals dependent on the steel grade and their references

Type	**Grade**	**EN ISO 3581:2012**	**EN ISO 14343:2009**
Austenitic steels	1.4401	19 12 3 L	
	1.4404	19 12 3 L	19 12 3 Nb
	1.4571	19 12 3 L	19 12 3 Nb
Duplex steels	1.4162	23 7 N L	22 9 3 N L
	1.4362	23 7 N L	22 9 3 N L

According to Stålbyggnadsinstitutet (2017) the permanent 0.2- proof strength can for austentitic filler materials be taken as 320 to 350 N/mm^2 with an ultimate strength of 510 to 550 N/mm^2. For duplex weld filler materials the 0.2- proof strength could be taken as approximately 450 N/mm^2 with an ultimate strength of 550 N/mm^2.

3.2.1.2 Carbon steel

Krav Brobyggande does, for carbon steel, not provide any specific steel grade to be used in corrosivity class C4. However there are requirements on the allowed plate thickness with regards to ductility. The plate thickness is calculated based on the reference temperature. The most used steel grades in bridge design are S355 and S420. The partial safety factors for carbon steel is given by TSFS 2018:57 and presented in Table 3.8. Nominal strength values for steel S355 and steel S420 are presented in Table 3.9 and design values are calculated using Equation 3.1.

Table 3.8: Partial safety factor for carbon steel

Notation	Description	Value	Reference
γ_{M0}	Resistance of cross-sections whatever the class is	1.0	TSFS 2018:57
γ_{M1}	Resistance of members to instability assessed by member checks	1.0	TSFS 2018:57
γ_{M2}	Resistance of cross-sections in tension to fracture	$\min\begin{cases} 0.9\frac{f_u}{f_y} \\ 1.1 \end{cases}$ (3.4)	TSFS 2018:57

Table 3.9: Material strengths for carbon steel

Steel category	Thickness [mm]	f_y [MPa]	f_u [MPa]	E [GPa]
S355N/NL	$t \leq 16$	355	490	210
	$16 < t \leq 40$	345	470	
	$40 < t \leq 63$	335	470	
S420M/ML	$t \leq 16$	420	520	210
	$16 < t \leq 40$	400	520	
	$40 < t \leq 63$	390	500	

3.2.1.2.1 Studs, screws and bolts

The strength classes for studs, screws and bolts are given by SS-EN 1993-1-8 and listed in Table 3.10. Partial safety factors for connectors in carbon steel are presented in Table 3.11.

Table 3.10: Material strength for studs and bolts

Strength class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb}[MPa]	240	320	300	400	480	640	900
f_{ub}[MPa]	400	400	500	500	600	800	1000

Table 3.11: Partial safety factors for studs, screws and bolts

Notation	Description	Value	Reference
γ_{M2}	Resistance of connections	1.2	TSFS 2018:57
γ_V	Resistance of shear studs	1.25	SS-EN 1994-1-1

3.2.1.2.2 Welds

The filler material of the weld should have an equivalent or higher yield strength and ultimate tensile strength than the parent material (SS-EN 1993-1-8). The partial safety factor for welded connections is presented in Table 3.12.

Table 3.12: Partial safety factor for welded connections

Notation	Description	Value	Reference
γ_{M2}	Resistance of welds	1.2	TSFS 2018:57

3.2.2 Concrete

The concrete slab is contributing with stiffness to the composite cross-section; hence relevant for the design of the girder. Further, the slab is subjected to shrinkage and creep causing internal forces. The size of these effects are determined by the concrete strength. Partial safety factors for concrete are presented in Table 3.13.

Table 3.13: Partial safety factors for concrete (SS-EN 1992-1-1)

Notation	Description	Value
γ_c	Resistance for permanent and variable loads	1.5
$\gamma_{c,E}$	Resistance at accidental loads (exceptional)	1.2
$\gamma_{c,fat}$	Resistance against fatigue loading	1.5
α_{cc}	Coefficient that considers unfavourable load application and long-term effects on the compressive strength	1.0
α_{ct}	Coefficient that considers unfavourable load application and long-term effects on the tensile strength	1.0

Krav Brobyggande does not give any requirements on the choice of concrete strength. Common choices for the concrete classes are C30/37, C35/45 or C40/50. Material strengths are given by SS-EN 1992-1-1 and presented in Table 3.14. The design value for the compressive strength of concrete is calculated according Equation 3.5.

Table 3.14: Material strengths for a selection of concrete classes

Concrete class	f_{ck} [MPa]	f_{cm} [MPa]	$f_{ctk,0.05}$ [MPa]	f_{ctm} [MPa] [MPa]	E_{cm} [GPa]
C30/37	30	38	2.0	2.9	33
C35/45	35	43	2.2	3.2	34
C40/50	40	48	2.5	3.5	35

$$f_{cd} = \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} \tag{3.5}$$

Whereas,

f_{cd} is the design compressive strength of concrete
f_{ck} is the characteristic value for the compressive strength of concrete
α_{cc} is a coefficient that considers unfavourable load application and long-term effects on the compressive strength
γ_c is the partial factor for a concrete considering the probability of deviation in the properties

The tensile design strength of concrete is calculated according Equation 3.6

$$f_{ctd} = \alpha_{ct} \cdot \frac{f_{ctk,0.05}}{\gamma_c} \tag{3.6}$$

Whereas,

f_{ctd} is the design tensile strength of concrete
$f_{ctk,0.05}$ is the characteristic value for the tensile strength of concrete
α_{ct} Coefficient that considers unfavourable load application and long term effects on the tensile strength

3.2.2.1 Creep of concrete

The concrete slab is subjected to creep leading to a change of stiffness for the cross-section. Therefore the applied loads are separated into different categories dependent on the duration. The change in stiffness is considered by calculating a creep-function, $\phi(t_0, \infty)$, described by SS-EN 1992-1-1. The creep function is dependent on the relative humidity in the ambient air, the strength of the concrete and the concrete age. When the creep function is calculated an equivalent height, h_0, for the slab is used. The equivalent height depends on the perimeter exposed to the ambient air during drying. Since the top surface of the bridge is covered with asphalt it is neglected when calculating the equivalent height. The equivalent height is calculated from Equation 3.7.

$$h_0 = \frac{2 \cdot A_c}{u} \tag{3.7}$$

Whereas,

h_0 is the equivalent thickness of the slab
A_c is the area of the concrete slab
u is the perimeter of the concrete in contact with the ambient air

Two factors related to the strength of the concrete is calculated according to Equation 3.8 and 3.9.

$$\alpha_1 = \frac{35}{f_{cm}}^{0.7} \tag{3.8}$$

$$\alpha_2 = \frac{35}{f_{cm}}^{0.2} \tag{3.9}$$

Whereas,

α_1, α_2 are factors accounting for the concrete strength
f_{cm} is the mean compressive strength of concrete [MPa]

A creep factor that accounts for the relative humidity is calculated according to Equation 3.10.

$$\phi_{RH} = \begin{cases} (1 + \dfrac{1 - RH/100}{0.1 \cdot h_0^{1/3}} \cdot \alpha_1) \cdot \alpha_2 & \text{for } f_{cm} \geq 35 \text{ MPa} \\ 1 + \dfrac{1 - RH/100}{0.1 \cdot h_0^{1/3}} & \text{for } f_{cm} < 35 \text{ MPa} \end{cases} \tag{3.10}$$

Whereas,

ϕ_{RH}	is a factor accounting for the relative humidity in the ambient air
RH	is the relative humidity in the ambient air, = 80% for outside conditions

$$\beta_{fcm} = \frac{16.8}{\sqrt{f_{cm}}} \quad \text{[MPa]} \tag{3.11}$$

$$\beta_{t,0} = \frac{1}{0.1 + (t_0)^{0.2}} \tag{3.12}$$

Whereas,

$\beta_{t,0}$	is a factor accounting for the age of concrete and its effect on the creep value
t_0	is the age of the concrete when the load is applied [days]

The creep value is different for different loads, depending on the age of concrete when the load is applied. Shrinkage is usually applied as a load at an age of two days. Permanent loads is applied at an concrete age of seven days. As these loads are acting on the structure during the whole service life of the bridge the final creep value is calculated from Equation 3.13.

$$\phi(t_0, \infty) = \phi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} \tag{3.13}$$

Whereas,

$\phi(t_0, \infty)$	is the final creep value
ϕ_{RH}	is a factor accounting for the relative humidity in the ambient air, see Equation 3.10
β_{fcm}	is a factor accounting for the effect of concrete strength on the creep value, see Equation 3.11
β_{t0}	is a factor accounting for the effect of concrete age on the creep value, see Equation 3.12

3.2.3 Modular ratios for linear elastic analysis

In order to transform the concrete into an equivalent steel cross-section, modular ratios are used and calculated according to Equation 3.14. The stiffness redistribution due to creep is considered by including the creep function for each load case when calculating the modular ratio, see Equation 3.15, 3.16 and 3.17.

$$n_0 = \frac{E_s}{E_{cm}} \tag{3.14}$$

Whereas,

n_0	is the modular ratio between steel and concrete
E_s	is the modulus of elasticity for steel
E_{cm}	is the modulus of elasticity for concrete

For composite bridges an additional creep factor, ϕ_l, is defined dependent on the durance of the load (SS-EN 1994-2). When the additional creep factor is included, the modular ratios for different load scenarios can be calculated according to Equation 3.15, 3.16 and 3.17.

$$n_{l,perm} = n_0 \cdot (1 + \phi_{l.perm} \cdot \phi_{\infty,perm}) \tag{3.15}$$

$$n_{l,cs} = n_0 \cdot (1 + \phi_{l.cs} \cdot \phi_{\infty,cs}) \tag{3.16}$$

$$n_{l.short} = n_0 \tag{3.17}$$

Whereas,

n_0	is the modular ratio between steel and concrete
$n_{l,perm}$	is the modular ratio for permanent loads
$\phi_{l,perm}$	= 1.1, is an additional creep factor dependent on the load durance for permanent loads
$\phi_{\infty,perm}$	is the final creep value after a long time for permanent loads
$n_{l,cs}$	is the modular ratio for shrinkage loads
$\phi_{l,cs}$	= 0.55, is an additional creep factor dependent on the load durance for shrinkage loads
$\phi_{\infty,cs}$	is the final creep value after a long time for shrinkage loads
$n_{l,short}$	is the modular ratio for short-term loads

3.3 System

The following section aims to describe how the stiffnesses of a composite section is calculated in the main load-bearing direction (longitudinal). Firstly the concept of cross-section classification is presented, then a description for how the effective width is calculated for both the steel flanges and the concrete slab is showed. The composite section is symmetric over the center-line of the bridge and therefore only half of the cross-section is evaluated in the system analysis.

3.3.1 Primary system, steel girder

Since the concrete slab is subjected to creep, different stiffnesses should be used for different loads. Therefore the system analysis should be carried out in different phases. The first considered phase is the one used for *the casting phase*, whereas only the steel is accounted for hence; no creep is considered. Variable loads will not be affected by creep as the duration is short, therefore these loads are included in the *short-term phase*. Permanent loads such as self-weight and other constant actions (acting after at least seven days) are included in the *permanent phase*. The creep function differs for permanent loads and shrinkage loads hence; the shrinkage loads is evaluated in a separate analysis, *the shrinkage phase*.

3.3.1.1 Cross-section classification

Steel members subjected to compression risk to buckle before yielding is attained. Therefore a cross-section classification must be carried out for all parts in order to decide what stress level each part can sustain before failure occur. The classification is dependent on the c/t ratio and the material properties of the studied part. Cross-section class 1 and 2 utilizes full plastic behaviour. Whereas, in cross-section class 3 the part can reach yielding in the outer most fibre before buckling. Parts in cross-section class 4 buckle already in the elastic region, and therefore an effective cross-section is calculated. For the effective cross-section the behaviour could be regarded as elastic.

The plate buckling depends on the slenderness of the part but also on the material properties of the steel. A reference factor, ϵ, dependent on the yield stress and its relation to a certain reference stress is calculated. The reference factor differs dependent on if stainless steel or carbon steel is used. The reference factor is calculated according to Equation 3.18 for stainless steel, and Equation 3.19 for carbon steel.

$$\epsilon = \sqrt{\frac{235}{f_y} \cdot \frac{E_s}{210}}, \quad \text{for stainless steel} \tag{3.18}$$

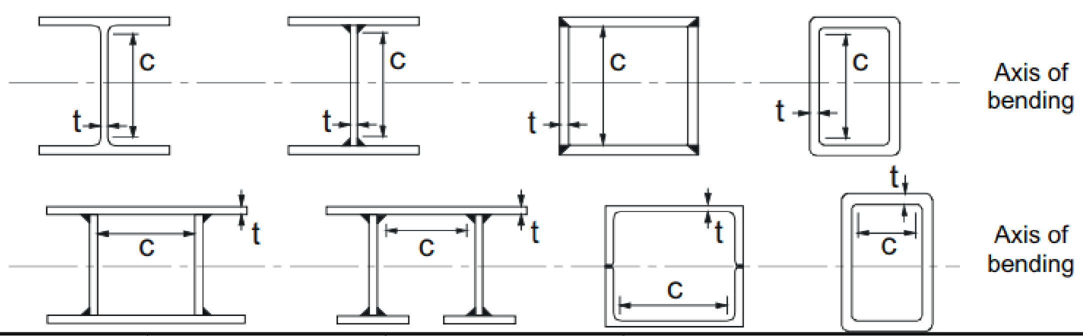
$$\epsilon = \sqrt{\frac{235}{f_y}}, \quad \text{for carbon steel} \tag{3.19}$$

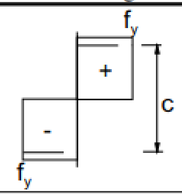
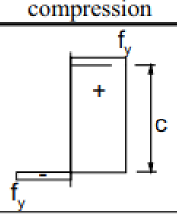
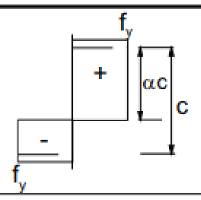
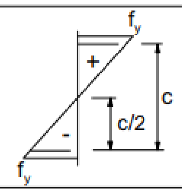
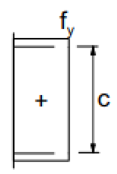
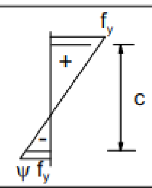
Whereas,

f_y is the yield stress for the specific material [MPa]
E_s is modulus of elasticity for stainless steel [GPa]

The cross-section classification also depends on if the part is an internal compression element or if it is an outstand compression element. The cross-section class is influenced by the type of loading. The classification for internal compression elements are made according to Figure 3.1. The classification for outstand elements are made according to Figure 3.2.

Internal compression parts

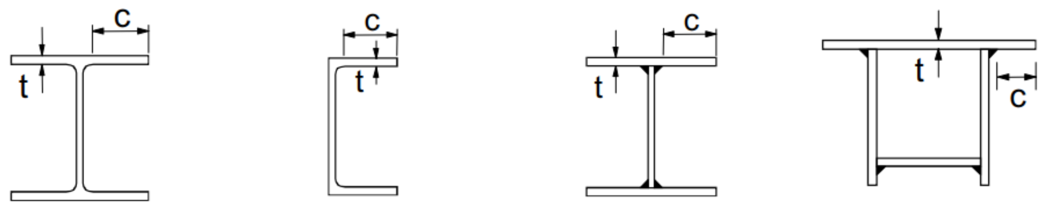


Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1^{*)}$: $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

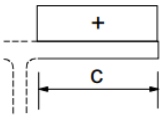
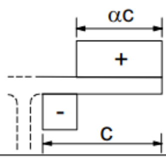
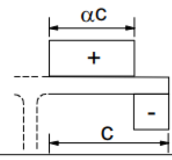
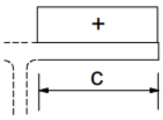
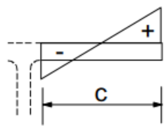
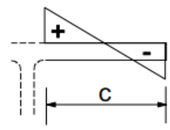
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Figure 3.1: Cross-section classification for internal compression elements (SS-EN 1993-1-1 Table 5.2)

Outstand flanges



Rolled sections — Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$	$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha}$	$c/t \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Figure 3.2: Cross-section classification for outstand compression elements (SS-EN 1993-1-1 Table 5.2)

3.3.1.2 Effective width of the compressed flange

The flange subjected to compression is during the casting phase, before the concrete has hardened, unsupported. If the flanges are in cross-section class 4 an effective width of the flange is calculated. The effective width for a stainless steel girder with corrugated web is calculated according to SS-EN 1993-1-4 and SS-EN 1993-1-5, Appendix D. If the design is made with carbon steel and a flat web the calculations are made according to SS-EN 1993-1-5. No difference is made for if stainless steel or carbon steel is used. Since the web does not contribute to the axial stiffness, hence the normal stresses are zero, only the flanges needs to be classified for girders using corrugated webs.

For girders using corrugated webs the buckling coefficient, k_σ, is defined according to Equation 3.20. This definition is taken from SS-EN 1993-1-5 Appendix D. The buckling coefficient is used in order to calculated the slenderness of the flanges.

$$k_\sigma = max(0.43 + (\frac{b}{a})^2, 0.6) \tag{3.20}$$

Whereas,

k_σ is the buckling coefficient for the compressed flange
b is the maximum width of the outstand flange from the weld toe to the free edge
a $= a_1 + 2 \cdot a_4$, is a factor related to the corrugation

Equation 3.20 states a minimum value of 0.6 for the buckling coefficient, which in application seems unreasonable. Therefore the equation is modified, instead maximizing the value for the buckling coefficient. The alternative method is calculated from the modified Equation 3.21. Further studies for the choice of buckling coefficient is studied and discussed in Chapter 5 and Chapter 6.

$$k_\sigma = min(0.43 + (\frac{b}{a})^2, 0.6) \tag{3.21}$$

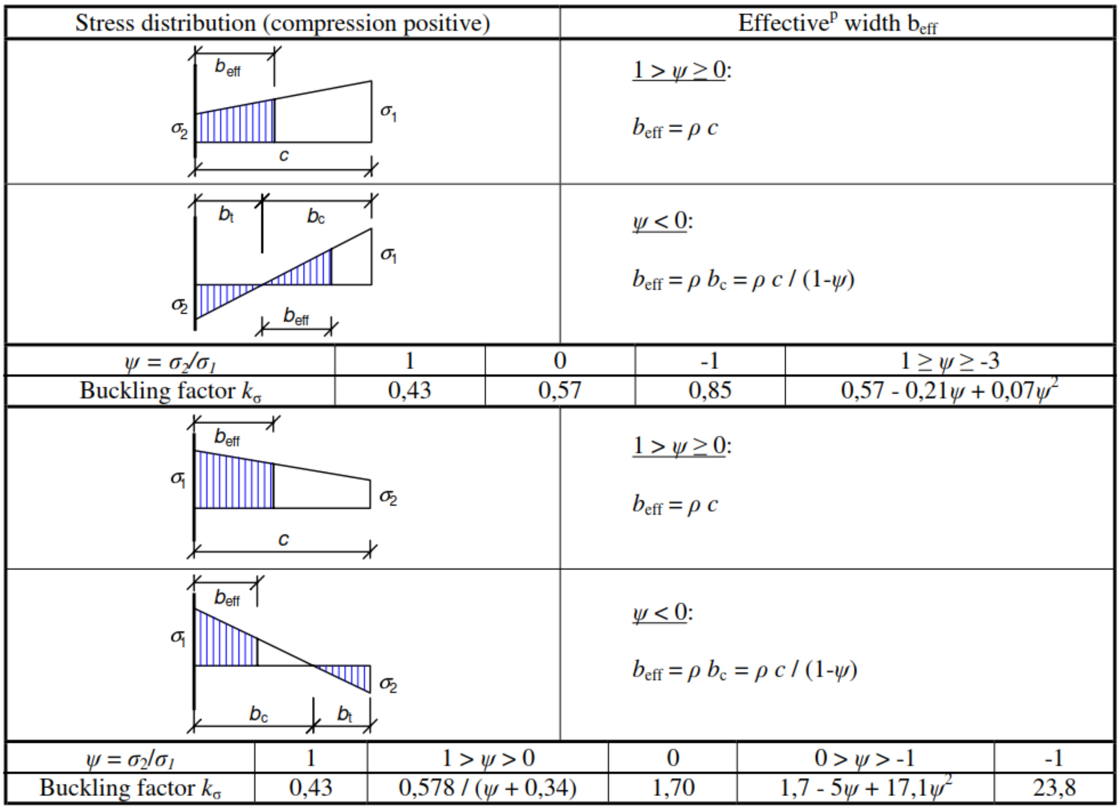
Instead of defining the distance, b, as the maximum width of the outstand flange, from the weld toe to the free edge, an alternative suggestion is presented in "Commentary and worked examples to EN 1993-1-5 Plated structural elements" (Johansson, Maquoi, Sedlacek, Müller, and Beg, 2007). It is suggested that the average outstand of the flange should be used if the criterion in Equation 3.22 is fulfilled. Note that this alternative way is not accepted by the design codes yet. Meaning that this should not be implemented in design as it contradicts with the given rules. The influence of this alternative way of defining the distance, b, is further studied in Section 5.1.

$$\frac{(a_1 + a_4) \cdot a_3}{(a_1 + 2a_4) \cdot b_f} > 0.14 \tag{3.22}$$

Whereas,

a_1 is the flat-fold length of the corrugation
a_4 is the length of the angled part
a_3 is the corrugation depth
b_f is the width of the compressed flange

For a flat web the buckling coefficient, k_σ, for the outstand flange is determined using Table 4.2 given in SS-EN 1993-1-5. The buckling coefficient is dependent on the stress distribution in the flanges.



Stress distribution (compression positive)	Effective^p width b_{eff}
b_{eff}, σ_2, σ_1, c	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\, c$
b_t, b_c, σ_1, σ_2, b_{eff}	$\underline{\psi < 0}$: $b_{eff} = \rho\, b_c = \rho\, c\, /\, (1\text{-}\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$

Stress distribution (compression positive)	Effective^p width b_{eff}
b_{eff}, σ_1, σ_2, c	$\underline{1 > \psi \geq 0}$: $b_{eff} = \rho\, c$
b_{eff}, σ_1, σ_2, b_c, b_t	$\underline{\psi < 0}$: $b_{eff} = \rho\, b_c = \rho\, c\, /\, (1\text{-}\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578\, /\, (\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

Figure 3.3: Values for the buckling coefficient, k_σ, for flat webs (SS-EN 1993-1-5, Table 4.2)

The slenderness of the flange is calculated according to Equation 3.23. In SS-EN 1993-1-5, Appendix D, the width, $\bar{b}$, is defined as half of the flange width. This is adjusted in the case studies (Chapter 4) to the maximum width of the outstand flange.

$$\lambda_p = \frac{\bar{b}/t}{28.4\epsilon \cdot \sqrt{k_\sigma}} \leq 1.0 \tag{3.23}$$

Whereas,

λ_p	is the slenderness of the flange
$\bar{b}$	is the half of the width of the flange with excluding half of the web- and weld thickness
t	is the thickness of the flange
k_σ	is the buckling coefficient for the flange subjected to compression
ϵ	is a reference factor defined by Equation 3.18 and 3.19

The reduction factor, ρ, with regards to plate buckling is for outstand compression elements calculated according to Equation 3.24 (SS-EN 1993-1-5 Equation 4.3).

$$\rho = \begin{cases} \dfrac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \lambda_p > 0.748 \\ 1.0 & \text{if } \lambda_p \leq 0.748 \end{cases} \tag{3.24}$$

The effective width for the compressed flange is calculated according to Equation 3.25.

$$b_{eff} = \rho \cdot b_f \tag{3.25}$$

Whereas,

b_{eff}	is the effective width of the compressed flange to be used in stiffness calculations
ρ	is the reduction factor for plate buckling calculated according to Equation 3.24
b_f	is the total width of the flange excluding the web and weld thicknesses

3.3.2 Secondary system, concrete slab

The secondary system consists of the concrete slab transferring the loads in the transverse direction into the girders. For the design of the girders a transformed section of the concrete is used. In order to calculate the transformed section the effective width for the concrete slab needs to be calculated. The effective width of the concrete slab is independent on the material choice for the steel and for the choice web design.

3.3.2.1 Effective width of the concrete slab

The effective width of the concrete slab that is contributing with stiffness to the composite section is calculated according to SS-EN 1994-1-1. How the effective width varies over the length of the bridge is illustrated in Figure 3.4, whereas the width if dependent on the boundary conditions. In regions close to end-supports or intermediate supports the effective width is calculated according to Equation 3.27. In regions close to mid-span the effective width is calculated according to Equation 3.26. When calculating the effective width for the support regions additional correction factors are used. The correction factors are calculated according to Equation 3.28. Figure 3.5 illustrates the notations for the variables used when calculating the effective width.

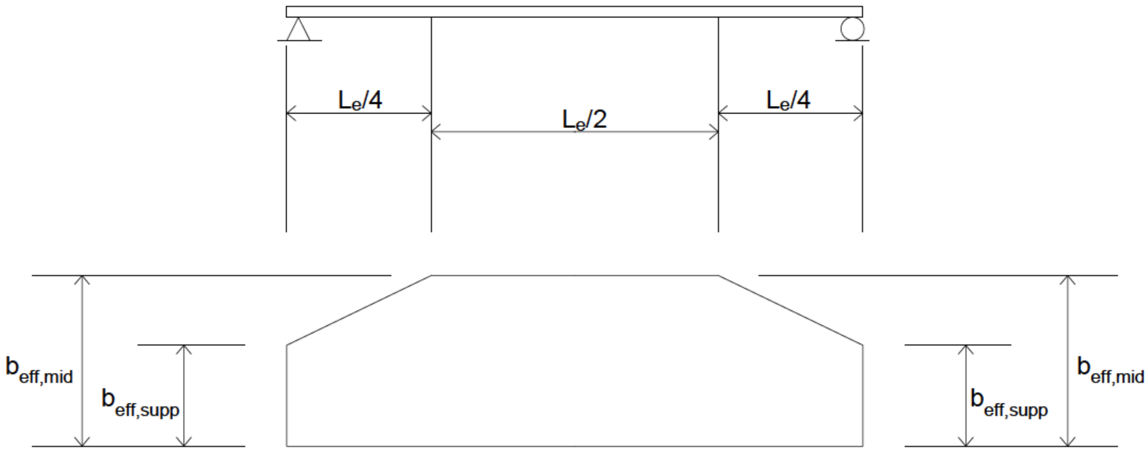


Figure 3.4: Effective width of the concrete slab

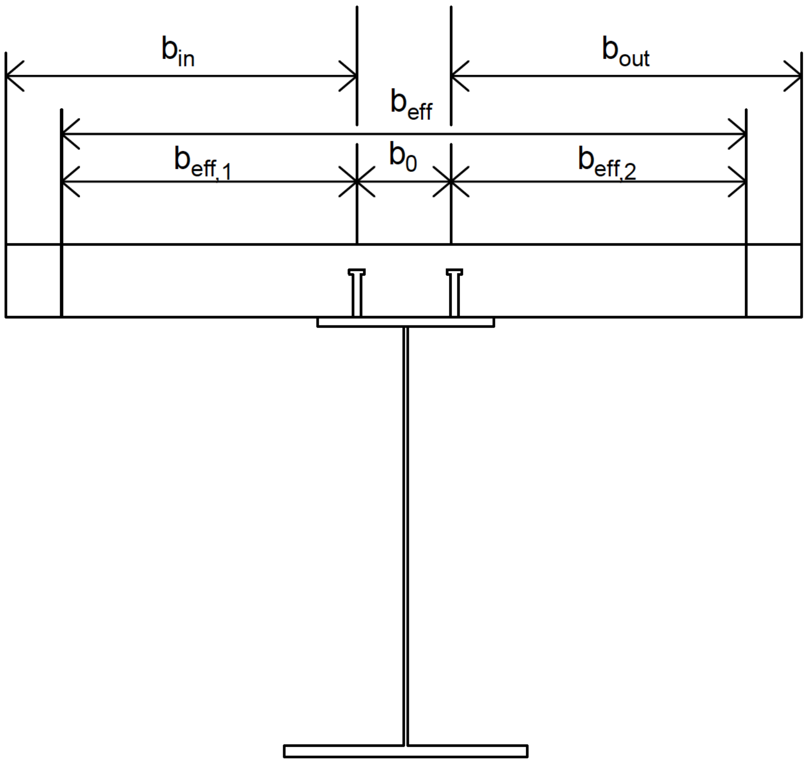


Figure 3.5: Notations for the effective width of the concrete slab

$$
\begin{aligned}
b_{eff,mid1} &= min(\frac{L_e}{8}, B_{out} - \frac{b_{0.mid}}{2}) \\
b_{eff,mid2} &= min(\frac{L_e}{8}, B_{in} - \frac{b_{0.mid}}{2}) \\
b_{eff,mid} &= b_{0.mid} + b_{eff,mid1} + b_{eff,mid2}
\end{aligned}
\tag{3.26}
$$

Whereas,

$b_{eff,mid1}$	is the effective width of the outstand part
L_e	is taken as the span length for simply supported bridges
B_{out}	is the distance between the edge beam and the centre of the web
$b_{0.mid}$	is the centre-to-centre distance between studs in the mid-region
$b_{eff,mid2}$	is the effective width of the internal part
B_{in}	is the distance between the centre line of the bridge and the centre of the web
$b_{eff,mid}$	is the effective width of the concrete slab in the mid-regions of the span

$$
\begin{aligned}
b_{eff,sup1} &= min(\frac{L_e}{8}, B_{out} - b_{0.sup}) \\
b_{eff,sup2} &= min(\frac{L_e}{8}, B_{in} - b_{0.sup}) \\
b_{eff,sup} &= b_{0.sup} + \beta_1 \cdot b_{eff,sup1} + \beta_2 \cdot b_{eff,sup2}
\end{aligned}
\tag{3.27}
$$

Whereas,

$b_{eff,sup1}$	is the effective width of the outstand part
$b_{0.sup}$	is the centre-to-centre distance between the studs in the support regions
$b_{eff,sup2}$	is the effective width of the internal part
$b_{eff,sup}$	is the effective width of the concrete slab
β_1, β_2	is the correction factors, see Equation 3.28

$$\beta_i = min(0.55 + 0.25 \cdot \frac{L_e}{b_{eff,sup,i}}, 1.0) \tag{3.28}$$

Whereas,

β_i	is the correction factor for either the internal or outstand part of the slab
$b_{eff,sup,i}$	is the effective width for either the internal or outstand part of the slab

3.3.3 Cross-sectional constants

The cross-sectional constants for the composite section is dependent on the load case. In the casting phase no composite action is achieved meaning that only the cross-section of the steel is load bearing. For the other phases; short-term, permanent and shrinkage, composite action is achieved. The cross-sectional constants for each phase are calculated for each corresponding modular ratio in order to transform the concrete into equivalent steel. The calculation of the corresponding modular ratio is described in Section 3.2.3.

3.3.3.1 Cross-section of the steel

In the casting phase only the cross-section of the steel is considered to be load bearing. Dependent on whether a corrugated web or a flat web is used the cross-sectional constants differs. The main difference originates from the accordion effect causing a lack of axial and flexural stiffness for girders with corrugated webs. When calculating the sectional constants for cross-sections with corrugated webs the contribution from the web is disregarded.

The needed sectional constants, for the design of the girder are; the cross-sectional area, the center of gravity, and the moment of inertia. The area of the steel cross-section is calculated from Equation 3.88. The center of gravity is formulated from the top of the steel flange and calculated according to Equation 3.30.

$$A_{st} = b_{fu} \cdot t_{fu} + b_{fl} \cdot t_{fl} + h_w \cdot t_w \tag{3.29}$$

Whereas,

A_{st}	is the area of the steel
b_{fu}	is the width of the upper flange
t_{fu}	is the thickness of the upper flange
b_{fl}	is the width of the lower flange
t_{fl}	is the thickness of the lower flange
h_w	is the height of the web
t_w	(= 0mm for corrugated webs), is the thickness of the web

$$z_{tp.steel} = \frac{b_{fu} \cdot t_{fu} \cdot \frac{t_{fu}}{2} + b_{fl} \cdot t_{fl} \cdot (h_{beam} - \frac{t_{fl}}{2}) + h_w \cdot t_w \cdot (t_{fu} + \frac{h_w}{2})}{A_{st}} \tag{3.30}$$

Whereas,

$z_{tp.steel}$	is the centre of gravity measured from the top flange of the steel girder
h_{beam}	is the height of the steel girder
t_w	= 0mm for corrugated webs

The moment of inertia around the strong axis (y-axis) is calculated according to Equation 3.31.

$$\begin{aligned} I_{y.steel} = \; & \frac{b_{fu} \cdot t_{fu}^3}{12} + b_{fu} \cdot t_{fu} \cdot (z_{tp.steel} - \frac{t_{fu}}{2})^2 \\ & + \frac{b_{fl} \cdot t_{fl}^3}{12} + b_{fl} \cdot t_{fl} \cdot (h_{beam} - z_{tp.steel} - \frac{t_{fl}}{2})^2 \\ & + \frac{h_w \cdot t_w^3}{12} + h_w \cdot t_w \cdot (z_{tp.steel} - \frac{h_w}{2} - t_{fu})^2 \end{aligned} \tag{3.31}$$

Whereas,

$I_{y.steel}$	is the moment of inertia for the steel
$z_{tp.steel}$	is the centre of gravity measured from the top flange and calculated according to Equation 3.30
t_w	= 0mm for corrugated webs

3.3.3.2 Composite cross-section

The cross-sectional constants for the composite section are calculated. The concrete slab is transformed, using modular ratios, into an equivalent steel section. Figure 3.6 illustrates graphically how the concrete section is transformed. Since the modular ratio depends on if creep is considered the same sectional constants can not be used for all phases. The cross-sectional area for any of the composite sections are calculated from Equation 3.32. Equation 3.33 shows the calculation for the center of gravity for the either of the composite sections.

Figure 3.6: Equivalent cross-section for a transformed concrete slab

$$A_{comp,i} = A_{st} + h_{slab} \cdot \frac{b_{eff.slab}}{n_{l,i}} \tag{3.32}$$

Whereas,

i	describes which phase that is being evaluated (short-term, permanent, or shrinkage)
$A_{comp,i}$	is the area of the composite cross-section
h_{slab}	is the thickness of the concrete slab
$b_{eff.slab}$	is the effective width of the concrete slab, calculated in Section 3.3.2.1
$n_{l,i}$	is the modular ratio for the specific phase, calculated in Section 3.2.3

$$z_{tp.comp,i} = \frac{A_{st} \cdot z_{tp.steel} + h_{slab} \cdot \frac{b_{eff.slab}}{n_{l,i}} \cdot \frac{(-h_{slab})}{2}}{A_{comp,i}} \tag{3.33}$$

Whereas,

i	describes which phase that is being evaluated (short-term, permanent, or shrinkage)
$z_{tp.comp,i}$	is the centre of gravity of the composite section measured from the top flange of the steel girder
$z_{tp.steel}$	is the centre of gravity of the steel section measured from the top flange of the steel girder, calculated from Equation 3.30

The moment of inertia around the strong axis (y-axis) is calculated according to Equation 3.34.

$$I_{y.comp,i} = I_{y.steel} + A_{st} \cdot (z_{tp.steel} - z_{tp.comp,i})^2 + \frac{h_{slab}^3 \cdot \frac{b_{eff.slab}}{n_{l,i}}}{12} + h_{slab} \cdot \frac{b_{eff.slab}}{n_{l,i}} \cdot (\frac{h_{slab}}{2} + z_{tp.comp,i})^2 \tag{3.34}$$

Whereas,

i	describes which phase that is being evaluated (short-term, permanent, or shrinkage)
$I_{y.comp,i}$	is the moment of inertia for the composite cross-section
$I_{y.steel}$	is the moment of inertia for the steel cross-section
$z_{tp.comp,i}$	is the centre of gravity for the composite cross-section measured from the top flange
$z_{tp.steel}$	is the centre of gravity for the steel cross-section measured from the top flange
$n_{l,i}$	is the modular ratio for the specific load case

3.4 Loads and load combination

The considered loads are divided into two categories; permanent- and variable loads. As the stiffness varies dependent if creep is considered, see Section 3.3.3, the loads needs to be added in different phases.

3.4.1 Permanent loads

The permanent loads to consider for a composite bridge is presented in Table 3.15. Note that the loads are, as mentioned, added in different phases.

Table 3.15: Permanent loads

Loads	Value	Reference	Phase
Steel girder	See Section 3.4.1.1	SS-EN 10088-1:2014	Casting
Form-work	50 kg/m^2*	Experience value	Casting
Reinforced concrete	25 (26)** kN/m^3	SS-EN 1991-1-1	Casting (long-term)
Asphalt covering	22 kN/m^3	Krav Brobyggande	Long-term
Crash barrier	0.25 $kN/beam$*	Experience value	Long-term
Shrinkage	See Section 3.4.1.2	SS-EN 1992-1-1	Shrinkage

*Assumed
** Self-weight of concrete is taken as a higher value if wet

3.4.1.1 Self-weight of steel girder

The self-weight of the steel girder is calculated for the full steel area, accounting for the extra length that the corrugation results in. The extra length is calculated according to Equation 3.35 using a ratio between the straight length and the corrugated length. Notations for the corrugation is illustrated in Figure 2.5. The density of different stainless steels is presented in Table 3.16.

$$r_c = \frac{a_1 + a_2}{a_1 + a_4} \tag{3.35}$$

Whereas,

a_1 is the flat-fold length
a_2 is the inclined length for the corrugation depth
a_4 is the straight length for a_2

Table 3.16: Density of different stainless steels according to SS-EN 10088-1:2014 Table E.1 and E.2

Steel grade	ρ $\frac{kg}{dm^3}$	$\frac{kN}{m^3}$
1.4401	8.0	78.5
1.4404	8.0	78.5
1.4571	8.0	78.5
1.4162	7.7	75.5
1.4362	7.8	76.5

The self-weight load is calculated according to Equation 3.36.

$$g_{st} = \rho \cdot g \cdot (t_{fu} \cdot b_{fu} + t_w \cdot h_w \cdot r_c + t_{fl} \cdot b_{fl}) \tag{3.36}$$

Whereas,

ρ	is density presented in Table 3.16
g	$= 9.81\frac{m}{s^2}$, is the acceleration constant
t_f	is the thickness of either the upper or lower flange
b_f	is the width of either the upper or lower flange
t_w	is the thickness of the web
h_w	is the height of the web
r_c	is the corrugation ratio calculated in Equation 3.35

3.4.1.2 Shrinkage force

The shrinkage force acting on the composite section is calculated for the shrinkage strains. The autogenous and the drying shrinkage is calculated according to SS-EN 1992-1-1 3.1.4. The force is acting in the center of gravity for the concrete section and applied in the center of gravity for the composite section. This results in an eccentricity causing an additional moment. The forces are stated by Equation 3.37.

$$
\begin{aligned}
F_{cs} &= \epsilon_{cs} \cdot E_{c,eff} \cdot A_c \\
M_{cs} &= F_{cs} \cdot (z_{tp,cs} + \frac{h_{slab}}{2})
\end{aligned}
\tag{3.37}
$$

Whereas,

F_{cs}	is the shrinkage force applied on the composite section
ϵ_{cs}	is the total shrinkage strain according to SS-EN 1992-1-1 3.1.4
$E_{c,eff}$	is the effective modulus of elasticity, as defined by Equation 3.38
A_c	is the area of concrete
M_{cs}	is the moment caused by the eccentricity of the shrinkage force
$z_{tp,cs}$	is the center of gravity for the shrinkage phase calculated from Equation 3.33
h_{slab}	is the thickness of the concrete slab

$$
E_{c,eff} = \frac{n_{l,short}}{n_{l,cs}} \cdot E_{cm} \tag{3.38}
$$

Whereas,

$E_{c,eff}$	is the effective modulus of elasticity
n_0	is the modular ratio as defined by Equation 3.14
$n_{l,cs}$	is the modular ratio for shrinkage, see Equation 3.16
E_{cm}	is mean modulus of elasticity for concrete

3.4.2 Variable loads

The variable loads are considered in the short-term analysis. In order to make a comparison for the redesigned bridges, all traffic loads are taken the same as the original design. Values and references are presented in Table 3.17.

Table 3.17: Variable loads

Loads	Value	Reference
Traffic load, Eurocode	$Q_{1k} = 300kN$, $Q_{2k} = 200kN$, $q_{1k} = 9kN/m^2$	SS-EN 1991-2
Traffic load, Trafikverket	$A/B = 180/300kN$	TSFS 2018:57
Acceleration, Eurocode	$0.6 \cdot \alpha_{Q1} \cdot 2 \cdot Q_{1k}$	SS-EN 1991-2
Acceleration, Trafikverket	$\min(0.35 \cdot B, 500kN)$	TSFS 2018:57
Side force	$25\% \cdot Q_{acc}$	SS-EN 1991-2
Temperature	See Section 3.4.2.2	SS-EN 1991-1-5, TSFS 2018:57
Wind load	-	TSFS 2018:57

3.4.2.1 Traffic loads

The vertical traffic loads that need to be considered are the ones formulated in SS-EN 1991-2 and TSFS 2018:57. The vertical traffic loads from Eurocode are hereafter referred to as LM1 and LM2. The traffic loads from Trafikverket are referred to as axle- and bogie loads (A/B- vehicles).

Load descending from acceleration or braking is calculated according to the Eurocodes and Krav Brobyggande. Each applied force is combined with the respective vertical traffic load. During load combination the vertical- and horizontal traffic load is combined as a multi-component load, see Section 3.4.3. The side force is calculated as 25% of the braking load.

3.4.2.2 Temperature load

The temperature load is a variable load, causing both expansion and contraction. For composite bridges there are two possible temperature scenarios; outside temperature variations and temperature differences between the composite materials. According to SS-EN 1991-1-5 Table C.1 the linear coefficient of thermal expansion for composite bridges should be taken the same for steel as for concrete. This, however, does not apply for composite bridges with stainless steel girders as the coefficient is much larger than the coefficient for concrete, see Table 3.18.

Temperature differences cause strains hence forces arise. These strains are calculated according to Equation 3.39 and the forces are calculated from Equation 3.40. An additional moment, caused by the temperature force being applied with an eccentricity in the center of gravity for the steel girder, is stated in Equation 3.41.

$$\epsilon_{temp} = \alpha_i \cdot \Delta T \tag{3.39}$$

$$F_{temp} = \epsilon_{temp} \cdot E_s \cdot A_{st} \tag{3.40}$$

$$M_{temp} = F_{temp} \cdot (z_{tp,st} - z_{tp,comp}) \tag{3.41}$$

Whereas,

ϵ_{temp}	is the resulting strain in the steel, calculated in Equation 3.44
α_i	is the coefficient of linear expansion, presented in Table 3.18
ΔT	is the temperature difference causing expansion or contraction
A_{st}	is the area of the steel
$z_{tp,st}$	is the center of gravity for the steel calculated from Equation 3.30
$z_{tp,comp}$	is the center of gravity for the composite section, calculated according to Equation 3.33

Table 3.18: Linear coefficient of thermal expansion - SS-EN 1991-1-5 Table C.1

Material	α_i $[\cdot 10^{-6}]$
Concrete	10
Stainless steel	16
Carbon steel	12
Steel-concrete composite bridge*	10

*Carbon steel

3.4.2.2.1 Outside temperature variations

Outside temperature variations are given by TSFS 2018:57 and is dependent on the location of the bridge. The equivalent temperature used in design is calculated according to Equation 3.42 and Equation 3.43.

$$\begin{aligned} T_{e.min} &= T_{min} + 4°C \\ \Delta T_{N.con} &= T_0 - T_{e.min} \end{aligned} \tag{3.42}$$

$$\begin{aligned} T_{e.max} &= T_{max} + 4°C \\ \Delta T_{N.exp} &= T_{e.max} - T_0 \end{aligned} \tag{3.43}$$

Whereas,

T_0	is the casting temperature - SS-EN 1991-1-5 A.1 (3)
T_{max}	is the maximum temperature - TSFS 2018:57 8 ch. 2§
T_{min}	is the maximum temperature - TSFS 2018:57 8 ch. 2§

3.4.2.2.2 Temperature difference between components

The temperature difference between components are stated as $\Delta T_{c2st} = 15°C$ according to SS-EN 1991-1-5 6.1.6.

3.4.2.2.3 Governing load case

Two governing combinations for the largest strain difference is distinguished. The first combination represent the load case for when there is an outside temperature drop combined with a negative temperature difference between the steel and the concrete. The next considered load case represents an increase in outside temperature, combined with a positive temperature difference between the steel and the concrete. Note that both the concrete and the steel is subjected to any temperature difference, however, they are affected differently as the coefficient of thermal expansion differ.

$$\begin{aligned} \Delta\epsilon_{max.con} &= -(\alpha_{ss} - \alpha_c) \cdot \Delta T_{N.con} - \alpha_{ss} \cdot \Delta T_{c2st} \\ \Delta\epsilon_{max.exp} &= (\alpha_{ss} - \alpha_c) \cdot \Delta T_{N.exp} + \alpha_{ss} \cdot \Delta T_{c2st} \end{aligned} \tag{3.44}$$

Whereas,

T_0	is the casting temperature - SS-EN 1991-1-5 A.1 (3)
T_{max}	is the maximum temperature - TSFS 2018:57 8 ch. 2§
T_{min}	is the maximum temperature - TSFS 2018:57 8 ch. 2§
Δ_{Tc2st}	is the temperature difference between components, see Section 3.4.2.2.2

3.4.3 Load combinations

All loads should be combined according to the guidelines, considering all relevant load cases. In Appendix A tables with correct load combination factors and references are presented for the current load combinations.

3.5 Load effects

Load effects needs to be calculated for each load. For a simply supported composite bridge the system is statically determined, meaning that no second order effects needs to be considered. Implementing this analogy no additional software is needed in order to calculate the load effects. However, using a two-dimensional software could be preferable as the worst position for each vehicle is sought, and so the process when using hand calculations could be very iterative.

3.5.1 Calculation of stresses

When calculating the load effects for different load cases the stiffness variation for different phases is considered. For the permanent loads, and the traffic loads the force is acting in the composite section, and so the resulting stresses is calculated for the composite section. A general approach for calculation of stresses in the composite section is presented in Section 3.5.1.1. However, for shrinkage and temperature loads the forces are acting in either the concrete or the steel section. This means that the stresses must be calculated for each part separately. The method for calculating these stresses are presented in Secion 3.5.1.2.

3.5.1.1 General calculation of stresses

Stresses caused by the permanent and the traffic loads are calculated using *Navier's Formula*. A general expression for the calculation of stresses is presented in Equation

3.45.

$$\sigma(z) = \frac{N}{A} + \frac{M}{I_y} z \tag{3.45}$$

Whereas,

$\sigma(z)$	is the stress at a certain point, z, from the center of gravity of the section
z	is the distance from the center of gravity for the section to the studied point
N	is the normal force acting in the cross-section
A	is the cross-sectional area
M	is the bending moment acting in the cross-section
I_y	is the moment of inertia for the cross-section, calculated according to Equation 3.34

3.5.1.2 Stresses caused by shrinkage strains

Shrinkage is a phenomenon acting only in the concrete, therefore, the stresses imposed on the steel section is recalculated according to Equation 3.46. For the recalculated load effects the stress is calculated using Equation 3.45, whereas the stiffness only the stiffnesses of the steel section is considered. Shrinkage cause a shear flow in the joint between the concrete and the steel which must be properly anchored, this is further explained in Section 3.5.1.2.1.

$$M_{st,cs} = \frac{I_{y,st}}{I_{y,comp}} M_{cs}$$

$$N_{st,cs} = F_{cs}\left(\frac{A_{st}}{A_{comp}} - \frac{A_{st}}{I_{y,comp}}\left(z_{tp,comp} - \frac{h_{slab}}{2}\right) \cdot \left(z_{tp,st} + \frac{h_{slab}}{2} - \left(z_{tp,comp} - \frac{h_{slab}}{2}\right)\right)\right) \tag{3.46}$$

Whereas,

$M_{st,cs}$	is the bending moment imposed on the steel section
$I_{y,st}$	is moment of inertia for the steel section, calculated according to Equation 3.31
$I_{y,comp}$	is moment of inertia for the composite section, calculated according to Equation 3.34
M_{cs}	is the bending moment caused by shrinkage, calculated in Equation 3.37
$N_{st,cs}$	is the normal force imposed on the steel section
F_{cs}	is the shrinkage force, calculated in Equation 3.37
A_{st}	is the area of the steel section
A_{comp}	is the area of the composite section
$z_{tp,comp}$	is the center of gravity, defined by Equation 3.33
h_{slab}	is the thickness of the concrete slab
$z_{tp,st}$	is the center of gravity, defined by Equation 3.30

3.5.1.2.1 Imposed load in the joint between the steel and the concrete caused by shrinkage

The load that is imposed on the studs in the joint between steel and concrete should be properly anchored. Concrete stresses caused by shrinkage are calculated using the same analogy as for calculating the stress in the steel caused by shrinkage forces. Equation 3.47, Equation 3.48 and Equation 3.49 shows how the force imposed on the studs is calculated.

$$F_{c,cs} = F_{cs}\left(1 - \frac{\frac{A_c}{n_{l,cs}}}{A_{comp}} - \frac{\frac{A_c}{n_{l,cs}}}{I_{y,comp}}\left(z_{tp,comp} - \frac{h_{slab}}{2}\right)^2\right)$$

$$M_{c,cs} = \frac{\frac{1}{12} \cdot \frac{b_{eff}}{n_{l,cs}} \cdot h_{slab}^3}{I_{y,comp}} \cdot M_{cs} \tag{3.47}$$

$$\sigma_1 = \frac{F_{c,cs}}{A_c} - \frac{M_{c,cs} \cdot \frac{h_{slab}}{2}}{\frac{1}{12} \cdot b_{eff} \cdot h_{slab}^3}$$

$$\sigma_2 = \frac{F_{c,cs}}{A_c} + \frac{M_{c,cs} \cdot \frac{h_{slab}}{2}}{\frac{1}{12} \cdot b_{eff} \cdot h_{slab}^3} \tag{3.48}$$

$$F_{cs,stud} = \frac{\sigma_1 + \sigma_2}{2} \cdot A_c \tag{3.49}$$

Whereas,

$F_{c,cs}$	is the force imposed in the concrete due to shrinkage
A_c	is the area of the concrete
$n_{l,cs}$	is the modular ratio considering creep and shrinkage
A_{comp}	is the area of the composite section, see Equation 3.32
$I_{y,comp}$	is moment of inertia for the composite section, see Equation 3.34
$z_{tp,comp}$	is the center of gravity, defined by Equation 3.33
$M_{c,cs}$	is the bending moment imposed on the concrete section
b_{eff}	is the effective width of concrete, see section 3.3.1.2
σ_1	is the stress in the upper edge of the concrete slab
σ_2	is the stress in the bottom edge of the concrete slab
h_{slab}	is thickness of the concrete slab
$F_{cs,stud}$	is the force imposed on the studs due to shrinkage

3.5.1.3 Stresses caused by temperature differences

The largest temperature load is obtained when primarily the steel is subjected to any temperature change, which is described in Section 3.4.2.2.3. Note that both the concrete and the steel is subjected to temperature variations, but the linear coefficient of thermal expansion differ for the two materials, causing a larger strain in the steel. Since only the stresses in the steel is of relevance when designing the girder, the load effects imposed on the section are calculated for the steel section only. For the recalculated load effects the stresses is calculated according to Equation 3.45, whereas the stiffness of the steel section is considered.

$$M_{st,temp} = \frac{I_{y,st}}{I_{y,comp}} M_{temp}$$
$$N_{st,temp} = F_{temp}(1 - (\frac{A_{st}}{A_{comp}} + \frac{A_{st}}{I_{y,comp}}(z_{tp,st} - z_{tp,comp})^2) \tag{3.50}$$

Whereas,

$M_{st,temp}$	is the bending moment imposed on the steel section
$I_{y,st}$	is moment of inertia for the steel section, see Equation 3.31
$I_{y,comp}$	is moment of inertia for the composite section, see Equation 3.34
M_{temp}	is the bending moment caused by temperature, calculated in Equation 3.41
$N_{st,temp}$	is the normal force imposed on the steel section
F_{temp}	is the temperature force, calculated in Equation 3.40
A_{st}	is the area of the steel section
A_{comp}	is the area of the composite section
$z_{tp,comp}$	is the center of gravity, defined by Equation 3.33
$z_{tp,st}$	is the center of gravity, defined by Equation 3.30

3.5.1.3.1 Imposed load in the joint between the steel and the concrete caused by temperature differences

As mentioned in Section 3.5.1.2.1 the force in the joint, between the concrete and steel, should be properly anchored using studs. The size of the force caused by temperature is calculated in the same manner as for shrinkage. Equation 3.51, 3.52 and 3.53 shows how the force imopsed on the studs is calculated.

$$F_{c,temp} = F_{temp} \cdot (\frac{\frac{A_c}{n_{l,short}}}{A_{comp}} - \frac{\frac{A_c}{n_{l,short}}}{I_{y,comp}}(z_{tp,st} - z_{tp,comp}) \cdot (z_{tp,comp} + \frac{h_{slab}}{2}))$$
$$M_{c,temp} = \frac{\frac{1}{12} \cdot \frac{b_{eff}}{n_{l,cs}} \cdot h_{slab}^3}{I_{y,comp}} \cdot M_{temp} \tag{3.51}$$

$$\sigma_1 = \frac{F_{c,temp}}{A_c} - \frac{M_{c,temp} \cdot \dfrac{h_{slab}}{2}}{\dfrac{1}{12} \cdot b_{eff} \cdot h_{slab}^3}$$
$$\sigma_2 = \frac{F_{c,temp}}{A_c} + \frac{M_{c,temp} \cdot \dfrac{h_{slab}}{2}}{\dfrac{1}{12} \cdot b_{eff} \cdot h_{slab}^3} \tag{3.52}$$

$$F_{temp,stud} = \frac{\sigma_1 + \sigma_2}{2} \cdot A_c \tag{3.53}$$

Whereas,

$F_{c,temp}$	is the force imposed in the concrete due to temperature
A_c	is the area of the concrete
$n_{l,short}$	is the modular ratio for short-term loads
A_{comp}	is the area of the composite section, see Equation 3.32
$I_{y,comp}$	is moment of inertia for the composite section, see Equation 3.34
$z_{tp,comp}$	is the center of gravity, defined by Equation 3.33
$M_{c,temp}$	is the bending moment imposed on the concrete section
b_{eff}	is the effective width of concrete, see section 3.3.1.2
σ_1	is the stress in the upper edge of the concrete slab
σ_2	is the stress in the bottom edge of the concrete slab
h_{slab}	is thickness of the concrete slab
$F_{temp,stud}$	is the force imposed on the studs due to temperature

3.6 Design of steel girder

The steel girders in a composite bridge are designed to withstand three design situations. These are the ultimate limit state (LS), the serviceability limit state (SLS) and the fatigue limit state (FAT).

3.6.1 Ultimate limit state

The ultimate limit state is evaluated in two different phases; one for the construction phase and one for the service phase of the bridge.

3.6.1.1 Construction phase

The loads that are considered during casting only results in stresses in the steel since the concrete is curing. Therefore, all deformations takes place before the concrete slab will contribute with any stiffness. The maximum stresses in the steel should be checked as well as the risk for LT-buckling of the compressed steel flange.

3.6.1.1.1 Bending moment capacity during the construction phase

The resistance against LT-buckling for girders in stainless steel are checked according to SS-EN 1993-1-4 5.4.2.1, and for girders in carbon steel according to SS-EN 1993-1-1 6.3.1.2. The compressed flange should be checked for LT-buckling around the weak axis of the girder (z-axis). For girders with corrugated webs, the stiffness contribution from the web is neglected due to lack of axial stiffness. For flat webs one third of the compressed part of the web is assumed to contribute to the overall stiffness.

For a welded I-girder in stainless steel, buckling curve *d* should be used. For girders in carbon steel the buckling curve is dependent on the thickness of the compressed flange and the buckling curve is, therefore, determined by SS-EN 1993-1-1 Table 6.2.

The reduction factor for LT-buckling, χ_{LT}, is calculated according to Equation 3.54, making use os Equation 3.55 and 3.56. The reduction factor is further used for calculating the bending moment capacity with regards to LT-buckling according to Equation 3.59.

$$\chi_{LT} = min(\frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{LT}^2}}, 1.0) \tag{3.54}$$

$$\Phi_{LT} = 0.5 \cdot (1 + \alpha_{LT} \cdot (\lambda_{LT} - 0.2) + \lambda_{LT}^2) \tag{3.55}$$

$$\lambda_{LT} = \sqrt{\frac{b_{eff} \cdot t_{fu} \cdot f_y}{N_{cr}}} \tag{3.56}$$

Whereas,

χ_{LT}	is the reduction factor for LT-buckling
Φ_{LT}	is a factor related to the reduction factor for LT-buckling
λ_{LT}	is the slenderness of compressed part
α_{LT}	is a factor dependent on the buckling curve and accounting for initial imperfections
b_{eff}	is the effective width of compressed part, calculated from Equation 3.25
t_{fu}	is the thickness of the compressed part
f_y	is the yield strength of the compressed part (differs for stainless and carbon steel)
N_{cr}	is the critical buckling load, calculated according to Equation 3.57

The critical buckling load is calculated from Equation 3.57 using the corresponding stiffness around the weak axis, calculated according to Equation 3.58.

$$N_{cr} = \frac{\pi^2 \cdot E_s \cdot I_z}{l_{cr}} \tag{3.57}$$

$$I_z = \frac{t_f \cdot b_{eff}^3}{12} \tag{3.58}$$

Whereas,

N_{cr}	is the critical buckling load
E_s	is the modulus of elasticity (differs between stainless- and carbon steel)
l_{cr}	is the critical buckling length
I_z	is the moment of inertia around the weak axis for the compressed flange
b_{eff}	is the effective width of the compressed steel flange, see Equation 3.25
t_f	is the thickness of the flange

The simplified method that is described in the Eurocodes for calculating the critical bending moment capacity is rather conservative. Therefore, a factor, k_f, can be included in the calculations according to SS-EN 1993-1-1 6.3.2.4. For a girder with corrugated webs, special attention needs to be taken considering the transverse bending moment in the flanges. The transverse bending moment arise from the use of corrugation and any shear flow in the web. The moment of resistance, M_{Rd}, during construction due to bending, is calculated according to Equation 3.59. The equation considers safety against LT-buckling.

$$M_{Rd} = \begin{cases} min \begin{cases} \dfrac{b_{eff} t_f f_{yf.r}}{\gamma_{M0}}(h_w + \dfrac{t_{fu} + t_{fl}}{2}) \\ \dfrac{b_{eff} t_{fu} k_f \chi_{LT} f_y}{\gamma_{M0}}(h_w + \dfrac{t_{fu} + t_{fl}}{2}) \end{cases} & \text{Capacity, corrugated webs} \\ \dfrac{W_{el}}{\gamma_{M0}} \cdot f_y \cdot \chi_{LT} \cdot k_f & \text{Capacity, flat webs} \end{cases} \tag{3.59}$$

Whereas,

M_{Rd}	is the bending moment capacity
$f_{yf.r}$	is the yield limit with respect to transverse moment arising in girders with corrugated webs, calculated according to Equation 3.60
b_{eff}	is the effective width of the compressed flange
t_f	is the thickness of either of
t_{fu}	is the thickness of the upper steel flange
t_{fl}	is the thickness of the lower steel flange
χ_{LT}	is the reduction factor for LT-buckling, calculated from Equation 3.54
f_y	is the yield limit for the steel (differs between stainless and carbon steel)
γ_{M0}	is a partial safety factor, listed in Table 3.3
k_f	= 1.1 (SS-EN 1993-1-1 6.3.2.4)
h_w	is the height of the web
W_{el}	is the elastic bending stiffness

$$f_{yf,r} = f_{yf} \cdot f_T \tag{3.60}$$

Whereas,

$$f_T = 1 - 0.4 \cdot \sqrt{\frac{\sigma_x(M_z)}{\frac{f_{yf}}{\gamma_{M0}}}} \tag{3.61}$$

$f_{yf.r}$	is the yield limit with respect to transverse moment arising in girders with corrugated webs
f_y	is the yield limit for the steel (differs between stainless and carbon steel)
f_T	is a reduction factor considering transverse moment in the flanges
$\sigma_x(M_z)$	is the normal stress in the flange from transverse moments caused by shear flow in the corrugated web
γ_{M0}	is a partial safety factor, listed in Table 3.3

3.6.1.2 Service phase

During the service life of the bridge the ultimate capacity of the composite cross-section is evaluated. The relevant checks that needs to be performed are the bending moment capacity, shear capacity, capacity of studs, capacity of welds, capacity of end-stiffeners and capacity of cross-beams.

3.6.1.2.1 Bending moment capacity

The bending moment capacity is checked for the stresses from each load phase that are summarized using the superposition principle, see Equation 3.62. The total stress is then compared to the corresponding yield stress. The stresses are calculated according to Section 3.5.1.

$$\sigma_{Ed,tot} = \sigma_{Ed,cast} + \sigma_{Ed,short} + \sigma_{Ed,temp} + \sigma_{Ed,perm} + \sigma_{Ed,cs} \quad \leq f_y \tag{3.62}$$

Whereas,

$\sigma_{Ed,tot}$	is the design load effect
$\sigma_{Ed,cast}$	is the stresses from the casting phase
$\sigma_{Ed,short}$	is the stresses from the short-term loads (excluding temperature stresses)
$\sigma_{Ed,temp}$	is the stresses from the temperature load
$\sigma_{Ed,perm}$	is the stresses from the permanent loads
$\sigma_{Ed,cs}$	is the stresses caused by shrinkage

3.6.1.2.2 Shear capacity

When determining the shear capacity of the girder the calculations differs between girders with corrugated webs and girders with flat webs. Calculations for corrugated webs are presented in Section 3.6.1.2.2.1 and calculations for flat webs are presented in Section 3.6.1.2.2.2. For both corrugated webs and flat webs there are differences depending on if the girder is designed in stainless or carbon steel.

3.6.1.2.2.1 Method for corrugated webs

The shear capacity is calculated according to SS-EN 1993-1-5, Appendix D. For a corrugated web, according to Eurocode, two main buckling modes should be considered; local and global buckling. The cross-section need to be checked for both modes. The risk of shear buckling is checked using the buckling reduction factors, $\chi_{c,l}$ and $\chi_{c,g}$.

Local buckling reduction factor

The local buckling reduction factor is mainly dependent on the longest flat-fold length of the corrugation and calculated in Equation 3.63. There are no difference for if stainless steel or carbon steel is used, other than the yield strength and the modulus of elasticity.

$$\chi_{c,l} = min(\frac{1.15}{0.9 + \lambda_{c,l}}, 1.0) \tag{3.63}$$

Whereas,

$\chi_{c,l}$	is the shear buckling reduction factor for local buckling, (SS-EN 1993-1-5 Equation D.5)
$\lambda_{c,l}$	is the slenderness of the web considering local buckling, calculated from Equation 3.64

$$\lambda_{c,l} = \sqrt{\frac{f_{yw}}{\tau_{cr,l} \cdot \sqrt{3}}} \tag{3.64}$$

Whereas,

$\lambda_{c,l}$	is the slenderness of the web considering local buckling, (SS-EN 1993-1-5 D.6)
f_{yw}	is the yield strength of the material in the web (differs between stainless and carbon steel)
$\tau_{cr,l}$	is a factor considering local buckling, calculated from Equation 3.65

$$\tau_{cr,l} = 4.83 \cdot E_s \cdot (\frac{t_w}{a_{max}})^2 \tag{3.65}$$

Whereas,

$\tau_{cr,l}$	is a factor considering local buckling, (SS-EN 1993-1-5 Equation D.7)
E_s	is the modulus of elasticity for the steel (differs between stainless and carbon steel)
t_w	is the thickness of the web
a_{max}	$= max(a_1, a_2)$, notations for the corrugation is illustrated in Figure 2.5

Global buckling reduction factor

The global buckling reduction factor mainly depends on the corrugation depth and is presented in Equation 3.66.

$$\chi_{c,g} = min(\frac{1.5}{0.5 + \lambda_{c,g}^2}, 1) \tag{3.66}$$

Whereas,

$\chi_{c,g}$	is the shear buckling reduction factor considering global buckling, (SS-EN 1993-1-5 Equation D.8)
$\lambda_{c,g}$	is the slenderness of the web considering global buckling, calculated from Equation 3.67

$$\lambda_{c,g} = \sqrt{\frac{f_{yw}}{\tau_{cr,g} \cdot \sqrt{3}}} \tag{3.67}$$

Whereas,

$\lambda_{c,g}$	is the slenderness of the web considering global shear buckling, (SS-EN 1993-1-5 Equation D.9)
f_{yw}	is the yield strength for the material in the web
$\tau_{cr,g}$	is a reference stress, calculated accordin to Equation 3.68

$$\tau_{cr,g} = \frac{32.4}{t_w \cdot h_w^2} \cdot (D_x \cdot D_z^3)^{1/4} \tag{3.68}$$

Whereas,

$\tau_{cr,g}$	is a factor considering global buckling, (SS-EN 1993-1-5 Equation D.10)
h_w	is the height of the web
t_w	is the thickness of the web
D_x, D_z	are geometrical factors considering global buckling, calculated from Equation 3.69 and Equation 3.70

$$D_x = \frac{E_s \cdot t_w^3 \cdot w}{12 \cdot (1 - v^2) \cdot s} \tag{3.69}$$

Whereas,

D_x	is a geometrical factor considering global buckling, (SS-EN 1993-1-5 D.2.2 (3))
E_s	is the modulus of elasticity
t_w	is the thickness of the web
v	is the *Poisson's ratio*
w	is the flat length for one corrugation segment, illustrated in Figure 2.5
s	is the length of one corrugation, illustrated in Figure 2.5

$$D_z = \frac{E_s \cdot I_z}{w} = \frac{E_s \cdot t_w^3 \cdot a_3}{12} \cdot \frac{3 \cdot a_1 + a_2}{w} \tag{3.70}$$

Whereas,

D_z is a geometrical factor considering global buckling,(SS-EN 1993-1-5 D.2.2 (3))
I_z is the moment of inertia around the weak axis for one corrugation length, given by Karlsson (2018)
a_1 is illustrated in Figure 2.5
a_2 is illustrated in Figure 2.5
w is the flat length for one corrugation segment, illustrated in Figure 2.5

Shear buckling capacity

The final reduction factor for shear buckling is calculated according to Equation 3.71, and chosen as the minimum of either the local or the global buckling reduction factor.

$$\chi_c = min(\chi_{c,g}, \chi_{c,l}) \tag{3.71}$$

Whereas,

χ_c is the shear buckling reduction factor
$\chi_{c,l}$ local buckling, calculated from Equation 3.63
$\chi_{c,g}$ global buckling, calculated from Equation **??**

The resistance for shear buckling for a girder with corrugated web is calculated according to Equation 3.72.

$$V_{Rd,w} = \chi_c \cdot \frac{f_{yw}}{\gamma_{M1} \cdot \sqrt{3}} \cdot t_w \cdot h_w \tag{3.72}$$

Whereas,

$V_{Rd,w}$ is the shear capacity of the web, (SS-EN 1993-1-5 Equation D.4)
χ_c is the shear buckling reduction factor, calculated in Equation 3.71
f_{yw} is the yield strength for the material in the web
γ_{M1} is a partial safety factor, listed in Table 3.3
h_w is the height of the web
t_w is the thickness of the web

3.6.1.2.2.2 Method for flat webs

For girders with flat webs the shear buckling capacity is calculated according to SS-EN 1993-1-5 5.2. Flat webs risk to shear buckle in between the transverse stiffeners. This risk for buckling needs to be evaluated and taken into account during design.

The risk for plate buckling for *carbon steel* is done according to SS-EN 1993-1-5 5.1 (2) and needs to be checked if Equation 3.73 is fulfilled. For *stainless steel* the criteria are presented in Equation 3.74 and carried out according to SS-EN 1993-1-4 5.6 (2) with updated limits given in SS-EN 1993-1-4:2006/2015.

$$\text{Carbon steel}\begin{cases} \dfrac{h_w}{t_w} \geq \dfrac{72\epsilon}{\eta} & \text{for an unstiffened web} \\ \dfrac{h_w}{t_w} \geq \dfrac{31\epsilon \cdot \sqrt{k_\tau}}{\eta} & \text{for a stiffened web} \end{cases} \tag{3.73}$$

$$\text{Stainless steel}\begin{cases} \dfrac{h_w}{t_w} \geq \dfrac{56\epsilon}{\eta} & \text{for an unstiffened web} \\ \dfrac{h_w}{t_w} \geq \dfrac{24.3\epsilon \cdot \sqrt{k_\tau}}{\eta} & \text{for a stiffened web} \end{cases} \tag{3.74}$$

Whereas,

h_w is the height of the web
t_w is the thickness of the web
η = 1.2, (TSFS 2018:57)
ϵ is a factor that accounts for the yield stress with regards to a reference yield stress, calculated from Equation 3.19
k_τ is a shear buckling coefficient which accounts for the size of the plate between stiffeners, calculated according to Equation 3.78

The slenderness of the web is dependent on the support conditions, if transverse stiffeners and/or if intermediate stiffeners are used, see Equation 3.75.

$$\begin{aligned} &\text{a) } \lambda_w = 0.76 \cdot \sqrt{\frac{f_{yw}}{\tau_{cr}}} \\ &\text{b) } \lambda_w = \frac{h_w}{86.4 \cdot t_w \cdot \epsilon} && \text{if transverse stiffeners are used at supports} \\ &\text{c) } \lambda_w = \frac{h_w}{37.4 \cdot \epsilon \cdot \sqrt{k_\tau}} && \text{if transverse stiffeners are used at supports} \\ & && \text{and intermediate stiffeners are used as well} \end{aligned} \tag{3.75}$$

Whereas,

λ_w is the slenderness parameter for shear buckling, (SS-EN 1993-1-5 Equation 5.3, 5.4 and 5.5)
f_{yw} is the yield strength of the web
τ_{cr} is a factor for shear buckling, calculated from Equation 3.76
ϵ is a factor that account for the yield stress with regards to a reference yield stress, calculated from Equation 3.19 and Equation 3.18
σ_E is the elastic buckling stress, calculated according to Equation 3.77
k_τ is the shear buckling coefficient that accounts for the size of the plate between stiffeners when no longitudinal stiffeners are used, see Equation 3.78 (SS-EN 1993-1-5 Equation A.5)

$$\tau_{cr} = \sigma_E \cdot k_\tau \tag{3.76}$$

$$\sigma_E = \frac{\pi^2 \cdot E_s \cdot t_w^2}{12 \cdot (1 - v^2) \cdot h_w^2} \tag{3.77}$$

$$k_\tau = \begin{cases} 5.34 + 4(h_w/a)^2 & \text{when } a/h_w \geq 1 \\ \\ 4 + 5.34(h_w/a)^2 & \text{when } a/h_w \leq 1 \end{cases} \tag{3.78}$$

Whereas,

σ_E is the elastic buckling stress, (SS-EN 1993-1-5 A.1 (2))
k_τ is the shear buckling coefficient that accounts for the size of the plate between stiffeners
E_s is the modulus of elasticity
t_w is the thickness of the web
v is the Poisson's ratio
h_w is the height of the web
a is the distance between transverse stiffeners

The reduction factor, χ_w, for shear buckling is dependent on if it has a rigid end post or a non-rigid end post. The values for χ_w is calculated using Table 3.19 for stainless steel or Table 3.20 for carbon steel.

Table 3.19: Values for χ_w for stainless steel (SS-EN 1993-1-4:2006/A1:2015, Table 5.4)

	Rigid end post	**Non-rigid end post**
$\lambda_w \leq 0.65/\eta$	η	η
$0.65/\eta < \lambda_w < 0.65$	$0.65/\lambda_w$	$0.65/\lambda_w$
$\lambda_w \geq 0.65$	$1.56/(0.91 + \lambda_w)$	$1.19/(0.54 + \lambda_w)$

Table 3.20: Values for χ_w for carbon steel (SS-EN 1993-1-5, Table 5.1)

	Rigid end post	**Non-rigid end post**
$\lambda_w < 0.83/\eta$	η	η
$0.83/\eta \leq \lambda_w < 1.08$	$0.83/\lambda_w$	$0.83/\lambda_w$
$\lambda_w \geq 1.08$	$1.37/(0.7 + \lambda_w)$	$0.83/\lambda_w$

The shear capacity for flat webs is calculated according to Equation 3.79

$$V_{Rd,bw} = \chi_w \frac{f_{yw} \cdot t_w \cdot h_w}{\gamma_{M1} \cdot \sqrt{3}} \tag{3.79}$$

Whereas,

$V_{Rd,bw}$	is the shear capacity of the web, (SS-EN 1993-1-5 Equation 5.2)
χ_w	is the shear reduction factor calculated according to Table 3.19 or Table 3.20
f_{yw}	is the yield strength (differs between stainless and carbon steel)
γ_{M1}	is a partial safety factor, listed in Table 3.3
h_w	is the height of the web
t_w	is the thickness of the web

3.6.1.2.3 Design of studs

The studs needs to be designed to transfer the shear flow between the steel and concrete, so that full composite action can be guaranteed. The studs are designed according to SS-EN 1994-2 6.6.3. The capacity is calculated according to Equation 3.80.

$$P_{Rd} = min \begin{cases} \dfrac{0.8 \cdot f_{u.stud} \cdot \pi \cdot d_{stud}^2}{4 \cdot \gamma_v} \\ \\ \dfrac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_v} \end{cases} \tag{3.80}$$

Whereas,

P_{Rd}	is the capacity of one shear stud - SS-EN 1994-2 Equation 6.18 and 6.19
$f_{u.stud}$	$= min(f_{ub}, 500MPa)$, is the ultimate strength - SS-EN 1994-2 6.6.3.1 (1)
d_{stud}	is the diameter of the stud
γ_v	is the partial factor for shear connectors, see Table 3.11
α_{stud}	is a correction factor for the length-diameter ratio of the shear stud, see Equation 3.81

$$\alpha_{stud} = \begin{cases} 0.2 \cdot (1 + \frac{h_{stud}}{d_{stud}}) & \text{if } 3 \leq \frac{h_{stud}}{d_{stud}} \leq 4 \\ \\ 1.0 & \text{if } 4 < \frac{h_{stud}}{d_{stud}} \end{cases} \tag{3.81}$$

Whereas,

h_{stud} is the length of the stud

The shear flow is calculated in the joint between the concrete and the top flange for the design shear force of each load case in the ultimate limit state, see Equation 3.82.

$$\tau_{Ed,i} = \frac{S_{y,i} \cdot V_{Ed,i}}{I_{y,i}} \tag{3.82}$$

Whereas,

$\tau_{Ed,i}$ is the shear flow for the specific load case
$S_{y,i}$ is the first moment of area for the specific load case
$V_{Ed,i}$ is shear force in the ultimate limit state for the specific load case
$I_{y,i}$ is the second moment of area for the specific load case

The needed amount of studs is calculated for the bridge's full length, expecting more studs at the support where the shear force is larger, and a lesser amount of studs in mid-span. The needed amount of studs is calculated from Equation 3.83. In order to make sure that the axial forces in the joint caused by shrinkage and temperature is fully anchored Equation 3.85 should be fulfilled. Two cases needs to be considered, one where the shrinkage and the temperature counteracts with the shear flow descending from the ULS-loads. The other case that needs to be checked is when temperature acts in the same direction as the ULS-loads (expansion) and shrinkage counteracts. For the mentioned checks an anchorage length should be calculated for the distance that is needed to transfer the loads. The anchorage length is calculated according to Equation 3.84.

$$n_{Ed} = \frac{\sum(\tau_{Ed,i}) + q_{acc}}{P_{Rd}} \leq n_{Rd} \tag{3.83}$$

Whereas,

n_{Ed} is the needed amount of studs [1/m]
$\tau_{Ed,i}$ is the shear flow for the specific load case
q_{acc} is acceleration- or braking load
P_{Rd} is the capacity of one stud, calculated in Equation 3.82
n_{Rd} is the chosen amount of studs [1/m]

$$l_{anch} = 1.5 \cdot max(B_{out}, B_{in}) \tag{3.84}$$

Whereas,

l_{anch}	is anchorage length, (SS-EN 1994-2, 6.9 (3))
B_{out}	is the distance from the centre of the web to the edge beam
B_{in}	is the distance from the centre of the web to the centerline of the bridge

$$n_{Ed,anch} = \begin{cases} \dfrac{F_{cs,stud} + F_{temp,stud}}{l_{anch} \cdot P_{Rd}} \leq n_{Rd}, & \text{Case 1} \\ n_{Ed} + \dfrac{F_{temp,stud} - F_{cs,stud}}{l_{anch} \cdot P_{Rd}} \leq n_{Rd}, & \text{Case 2} \end{cases} \tag{3.85}$$

$n_{Ed,anch}$	is the needed amount of studs due to anchorage [1/m]
$F_{cs,stud}$	is the force caused by shrinkage in the joint between concrete and steel, calculated from Equation 3.49
$F_{temp,stud}$	is the force caused by temperature in the joint between concrete and steel, calculated from Equation 3.53
P_{Rd}	is the capacity of one stud, calculated according to Equation 3.82
l_{anch}	is the anchorage length, (SS-EN 1994-2, 6.9 (3))
n_{Rd}	is the chosen amount of studs [1/m]
n_{Ed}	is the needed amount of studs [1/m]

3.6.1.2.4 Design of stiffeners

Stiffeners are used in order to avoid buckling of the cross-section in critical sections. For girders with corrugated webs the need of stiffeners (transverse) is small, only needed in the support sections. For girders with flat webs both transverse and longitudinal stiffeners can be needed in order to provide lateral support. The design of stiffeners covers cross-section classification, calculations of stiffness, and out-of-plane buckling control. The check is performed both in mid-span and at the supports. For determination of cross-section class for the stiffeners, see Section 3.3.3.

3.6.1.2.4.1 Calculation stiffness

During design of the stiffener Equation 3.86 must be fulfilled.

$$\frac{I_{st}}{I_{st.min}} \geq 1 \tag{3.86}$$

$$I_{st.min} = \begin{cases} 0.75 \cdot h_w \cdot t_w^3 & \text{when } a/h_w \geq \sqrt{2} \\ \dfrac{1.5 \cdot h_w^3 \cdot t_w^3}{a^2} & \text{when } a/h_w \leq \sqrt{2} \end{cases} \tag{3.87}$$

Whereas,

I_{st}	is the stiffness of the stiffener calculated according to Equation 3.89
$I_{st.min}$	is the minimum value for the moment of inertia that the stiffeners should have (SS-EN 1993-1-5, Equation 9.6)
h_w	is the height of the web
t_w	is the thickness of the web
a	is the distance between transverse stiffeners

When calculating the stiffness of the stiffener a part of the web is assumed to contribute. Dependent on the material choice; stainless or carbon steel, the contributing width differs. The contributing part for stainless steel is $11\epsilon t_w$ according to SS-EN 1993-1-4 5.7 (3). For carbon steel the contribution width is $15\epsilon t_w$, according to SS-EN 1993-1-5 Figure 9.1. The effective area can, however, be limited by the available width and calculated according to Equation 3.88. The effective area for carbon steel is displayed in Figure 3.7.

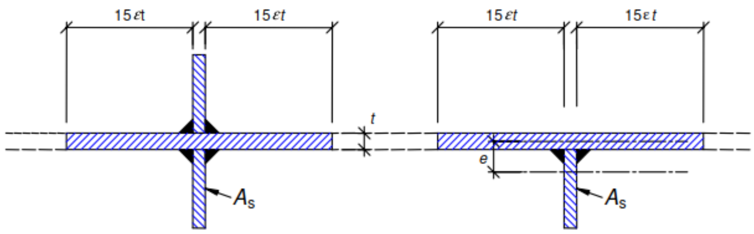


Figure 3.7: Effective area of stiffener for carbon steel (SS-EN 1993-1-5 9.1 Figure 9.1)

$$A_{st} = \begin{cases} b_{st} \cdot t_{st} + (2 \cdot 11\epsilon \cdot t_w + t_{st}) \cdot t_w, & \text{for stainless steel} \\ b_{st} \cdot t_{st} + (2 \cdot 15\epsilon \cdot t_w + t_{st}) \cdot t_w, & \text{for carbon steel} \end{cases} \quad (3.88)$$

Whereas,

A_{st}	The effective area of the stiffener
b_{st}	is the width of the stiffener
t_{st}	is the thickness of the stiffener
$11\epsilon t_w$	is the contributing part of the web for stainless steel
$15\epsilon t_w$	is the contributing part of the web for carbon steel
ϵ	is a factor that account for the yield stress with regards to a reference yield stress, calculated from Equation 3.18 for stainless steel and Equation 3.19 for carbon steel

Dependent for if the stiffeners cross-section is double symmetric or mono-symmetric the stiffness is calculated differently. For a double symmetric cross-section the moment of inertia is calculated according to Equation 3.89a and for a mono-symmetric cross-section it is calculated according to Equation 3.89b.

$$I_{st} = \begin{cases} \text{a)} & \dfrac{(2 \cdot K\epsilon \cdot t_w + t_{st}) \cdot t_w^3}{12} + 2 \cdot \dfrac{t_{st} \cdot b_{st}^3}{12} \\ \text{b)} & \dfrac{(2 \cdot K\epsilon \cdot t_w + t_{st}) \cdot t_w^3}{12} + \dfrac{t_{st} \cdot b_{st}^3}{12} + t_{st} b_{st} \cdot (\dfrac{b_{st} + t_w}{2}) \end{cases} \tag{3.89}$$

Whereas,

I_{st}	is the moment of inertia for the stiffener
K	= 11 for stainless steel, = 15 for carbon steel
h_w	is the height of the web
t_w	is the thickness of the web
a	is the distance between transverse stiffeners

3.6.1.2.4.2 Out-of-plane buckling

The stiffener should be checked for the risk of out-of-plane buckling. This check is mainly performed at the supports as the patch loading from the reaction force is large and carried by the stiffener and web. The buckling length of the stiffener is defined by SS-EN 1993-1-4 5.7 (2) for stainless steel, and SS-EN 1993-1-5 9.4 (2) for carbon steel. The requirements are the same regardless for the choice of steel. The risk of out-of-plane buckling depends on the buckling length, the radius of gyration and the slenderness. The buckling length of the stiffener is calculated according to Equation 3.90, were the ends are assumed to be laterally fixed. The radius of gyration is calculated according to Equation 3.91.

$$l_{cr,st} = 0.75 \cdot h_w \tag{3.90}$$

$$i = \sqrt{\frac{I_{st}}{A_{st}}} \tag{3.91}$$

Whereas,

$l_{cr,st}$	is the buckling length
h_w	is the height of the web
i	is the radius of gyration around the weak axis
A_{st}	is the effective area
I_{st}	is the moment of inertia

$$\lambda_{st} = \frac{l_{cr,st}}{\pi \cdot i} \cdot \sqrt{\frac{f_y}{E_s}} \tag{3.92}$$

Whereas,

λ_{st}	is the slenderness of the stiffener, (SS-EN 1993-1-1 Equation 6.50)
$l_{cr,st}$	is the buckling length of the stiffener
i	is the radius of gyration around the weak axis, calculated from Equation 3.91
f_y	is the yield stress (differs between stainless- or carbon steel)
E_s	is the modulus of elasticity (differs between stainless- or carbon steel)

The buckling reduction factor, used to calculate the buckling load (Equation 3.95) is calculated according to Equation 3.93.

$$\chi_{st} = min(\frac{1}{\Phi_{st} + \sqrt{\Phi_{st}^2 - \lambda_{st}^2}}, 1.0) \tag{3.93}$$

Whereas,

$$\Phi_{st} = 0.5 \cdot (1 + \alpha_{st}(\lambda_{st} - 0.2) + \lambda_{st}^2) \tag{3.94}$$

χ_{st}	is the reduction factor for buckling of the stiffener, (SS-EN 1993-1-1 6.3.1.3 Eq: 6.49)
α_{st}	= 0.49, buckling curve c
λ_{st}	is the slenderness of the stiffener, calculated from Equation 3.92

$$N_{Rd,b} = \frac{A_{st} \cdot f_y \cdot \chi_{st}}{\gamma_{M1}} \tag{3.95}$$

Whereas,

$N_{Rd,b}$	is the buckling load for the stiffener
χ_{st}	is the buckling reduction factor
γ_{M1}	is a partial factor, listed in Table 3.3
A_{st}	is the effective area of the stiffener, calculated from Equation 3.88
f_y	is the yield stress (differs between stainless or carbon steel)

3.6.1.2.5 Design of welds

Welded connections should be checked according to SS-EN 1993-1-8. Equivalent stresses (Von Mises stresses) is used to evaluate stresses in different directions. The design of fillet welds is presented in Section 3.6.1.2.5.1 and the design of butt welds are presented in 3.6.1.2.5.2

3.6.1.2.5.1 Fillet welds

For fillet welds the directional method is used and calculated according to SS-EN 1993-1-8 4.5.3.2. The stresses that is present in a fillet weld is shown in Figure 3.8. Whereas,

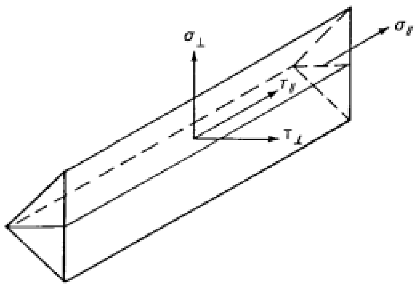


Figure 3.8: Stresses in a fillet weld

$\sigma_\perp$	is the normal stress perpendicular to the weld throat
$\sigma_\parallel$	is the normal stress parallel to the weld throat
$\tau_\perp$	is the shear stress perpendicular to the weld throat
$\tau_\parallel$	is the shear stress parallel to the weld throat

According to SS-EN 1993-1-8 4.5.3.2 (5) the normal stresses parallel to the weld throat, $\sigma_\parallel$, is not considered when checking the design resistance of the weld. The normal stress perpendicular to the weld, $\sigma_\perp$, is calculated according to Equation 3.96 for the case with a presence of a point load.

$$\sigma_\perp = \frac{P}{a \cdot l} \tag{3.96}$$

Whereas,

$\sigma_\perp$	is the normal stress perpendicular to the weld throat
P	is the point load
a	is the throat thickness
l	is the length of the weld

The shear stresses in a a fillet weld perpendicular to the weld throat is equal to the normal stresses perpendicular to the weld throat and calculated from Equation 3.97.

$$\tau_\perp = \sigma_\perp \tag{3.97}$$

Whereas,

$\tau_\perp$	is the shear stress perpendicular to the weld throat
$\sigma_\perp$	is the normal stress perpendicular to the weld throat

The shear stress parallel to the weld throat is calculated as shown in Equation 3.98.

$$\tau_{\parallel,i} = \frac{V_{Ed,i} \cdot S_{y,i}}{I_{y,i} \cdot t} \tag{3.98}$$

Whereas,

$\tau_{\parallel,i}$	is the shear stress parallel to the weld throat, for the specific load case
$V_{Ed,i}$	is the shear force for the specific load case
$S_{y,i}$	is the second moment of area for the specific phase
$I_{y,i}$	is the moment of inertia for the specific phase
t	is the thickness of the member

The design resistance is sufficient if the conditions given in Equation 3.99 are fulfilled (SS-EN 1993-1-8 4.5.3.2 (6)).

$$\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_\parallel^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$$
$$\sigma_\perp \leq \frac{0.9 \cdot f_u}{\gamma_{M2}} \tag{3.99}$$

Whereas,

$\tau_\parallel$	is the total shear stress parallel to the weld throat
$\sigma_\perp$	is the total normal stress perpendicular to the weld throat
$\tau_\perp$	is the total shear stress perpendicular to the weld throat
f_u	is the ultimate tensile strength of the weaker part joined
γ_{M2}	is a partial factor, see Table 3.3
β_w	is a correction factor found in SS-EN 1993-1-4 6.3 (1) for stainless steel and SS-EN 1993-1-8 Table 4.1 for carbon steel

3.6.1.2.5.2 Butt welds

Butt welds are designed using the directional method. Shear stresses perpendicular to the weld throat will not appear as the weld is not performed in an angle. The stresses used for calculation of the design resistance is shear stresses parallel to the weld and normal stresses perpendicular to the weld. The stresses are calculated as described in Section 3.6.1.2.5.1. The design resistance of a full penetration butt weld is taken as the design resistance of the weaker part joined. This can be verified when Equation 3.100 is fulfilled.

$$\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\parallel^2)} \leq \frac{f_u}{\gamma_{M2}} \tag{3.100}$$

Whereas,

$\tau_\parallel$	is the shear stress parallel to the weld throat
$\sigma_\perp$	is the normal stress perpendicular to the weld throat
f_u	is the ultimate tensile strength of the weaker part joined
γ_{M2}	is a partial factor, listed in Table 3.3

3.6.1.2.6 Design of cross-beams

The cross-beams are designed to withstand the resulting horizontal load descending from wind loads and side loads arising during casting. More specifically they should prevent LT-buckling before composite action is achieved. Furthermore, the support cross-beams are designed to carry the self-weight of the structure during change of bearings. An illustration is showed in Figure 3.9 for better understanding of the bearing change. The cross-beams also contribute to the torsional stiffness of the bridge.

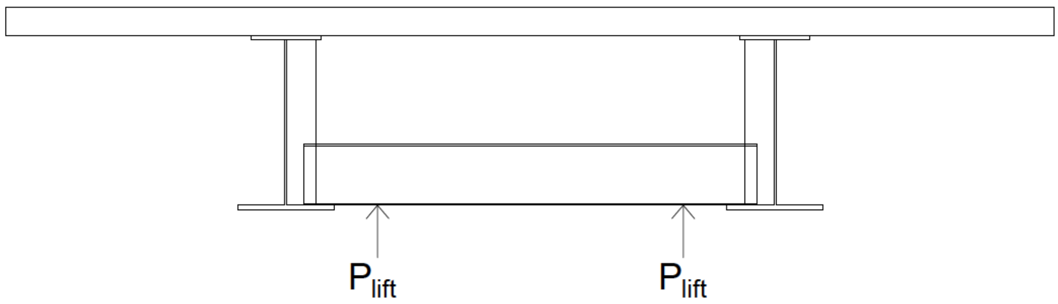


Figure 3.9: Principle figure for the bearing change

3.6.1.2.6.1 Cross-beams at support

The checks that are conducted to verify the design resistance of the cross-beams at support are the following; bending moment capacity, shear capacity, out-of-plane buckling at bearing change, and design resistance of the welds. An additional check should be carried out for if the SLS-loads are higher than the load effects from ULS. The joint between the cross-beam and the main girder should be designed and verified. Furthermore, the bending moment capacity is calculated according to Equation 3.101, and is verified against the bending moment that arise from the eccentricity of the point load at bearing change. The shear capacity is calculated calculated the same way as in Section 3.6.1.2.2.2 and is checked for the shear forces at bearing change. Out-of-plane buckling is evaluated the same way as section 3.6.1.2.4.2. The buckling length is then set to the centre-to-centre distance between the webs of the main girders. The welds of the cross-beam are designed and verified the same way as described in section 3.6.1.2.5.

$$M_{Rd,cb} = f_{y,cb} \cdot W_{el,cb} \tag{3.101}$$

Whereas,

$M_{Rd,cb}$	is the moment capacity of the cross-beam
$f_{y,cb}$	is the yield stress for the material in the cross-beam
$W_{el,cb}$	is the elastic bending resistance

3.6.1.2.6.2 Cross-beams in mid-span

The cross-beams in mid-span are usually designed as trusses. The magnitude of the horizontal forces that arise during casting from the vertical loads is calculated according to SS-EN-1993-1-1 5.3.3. The compressive force in the flange of the main girder is calculated as shown in Equation 3.102. The resulting horizontal force that is transferred to the cross-beams is calculated according to Equation 3.103.

$$N_{f,m} = \frac{M_{Ed,cast}}{h_w} \tag{3.102}$$

Whereas,

$N_{f,m}$	is the resulting compressive force in the flange due to bending moment during casting
$M_{Ed,cast}$	is the design bending moment during casting
h_w	is the height of the web

$$N_{Ed,cb} = \frac{N_{f,m} \cdot \alpha_m}{100} \tag{3.103}$$

Whereas,

$N_{Ed,cb}$	is the design force in the cross-beam during casting, (SS-EN-1993-1-1 5.3.3 (5))
m	is the number of elements that are fixed, calculated from Equation 3.104

$$\alpha_m = \sqrt{0.5 \cdot (1 + \frac{1}{m})} \tag{3.104}$$

The top bar of the truss will carry the axial force and needs to be checked for out-of-plane buckling. Out-of-plane buckling is evaluated in a similar manner as for stiffeners described in Section 3.6.1.2.4.2. A combined check for the normal force and bending moment is needed due to the eccentricity when mounting the bar. The combined check is carried out according to SS-EN 1993-1-1 6.2.9.3 and shown in Equation 3.105.

$$\frac{N_{Ed,cb}}{N_{Rd,cb}} + \frac{M_{Ed,cb}}{M_{Rd,cb}} \leq 1 \tag{3.105}$$

Whereas,

$N_{Ed,cb}$	is the design force in the cross-beam during casting, calculated from Equation 3.103
$N_{Rd,cb}$	is the normal force resistance for the cross-beam, calculated from Equation 3.95
$M_{Ed,cb}$	is the design bending moment in the cross-beam during casting
$M_{Rd,cb}$	is the bending moment resistance for the cross-beam, calculated from Equation 3.101

The diagonal bars in the truss will only transfer compressive forces and checked for out-of-plane buckling in the same manner as for the stiffeners presented in Section 3.6.1.2.4.2. The bottom bar of the truss normally has the same dimensions as the top bar and subjected to the same load effects, therefore, no specific check needs to be performed. Dependent on the type of joint that is used to connect the bars a check needs to be carried out according to SS-EN 1993-1-8.

3.6.2 Serviceability limit state

In SLS the deflection and movement descending from vertical traffic loads are determined. Furthermore, the elevation of the beam for casting loads, permanent loads and shrinkage loads is calculated so that the residual deflection is zero after loading of the structure.

As stainless steel displays a non-linear behaviour below the yield limit, a constant value for the modulus of elasticity can not be used. A secant modulus of elasticity is therefore calculated, dependent on the current stress, using the methodology presented in Section 3.2.1.1. Whereas the Ramberg- Osgood parameter, n, is taken from Table 3.4.

For the studied bridge type, simply supported, all load-effects and deflections are statically determinate. This means that no separate system analysis needs to be done for the secant modulus of elasticity, but only re-scaling the deflections from the analysis using the ordinary modulus of elasticity is needed. The total deflection is calculated according to Equation 3.106. For a girder in carbon steel the deflections could be extracted from the system analysis without any re-scaling.

$$\delta_{tot} = \Sigma(\delta_i \cdot \frac{E_s}{E_{s.ser.i}}) \tag{3.106}$$

Whereas,

δ_i	is the deflection for a certain load case such as shrinkage, permanent loads or short-term loads
E_s	is the modulus of elasticity for the chosen steel grade (stainless steel)
$E_{s.ser.i}$	is the secant modulus of elasticity for a certain stress and load case, calculated from Equation 3.2

The allowed deflection in span is calculated according to Krav Brobyggande B.3.4.2.2, and presented in Equation 3.107. Maximum allowed movement at free end is limited to 5mm. For girders with flat webs, the section is much stiffer since the web is included for the stiffness calculations. This results in smaller deflection than for girders with corrugated webs.

$$\delta_{all} = \frac{L}{400} \tag{3.107}$$

Whereas,

δ_{all} is the allowed deflection in span for vertical traffic loads
L is the span length

The elevation of the beam is chosen dependent on the deflection from the long-term loads (permanent loads and shrinkage forces).

3.6.3 Fatigue limit state

The check against fatigue is carried out according to SS-EN 1993-1-9 using the λ-*method*. The calculations do not differ dependent for if stainless or carbon steel is used. For girders with corrugated webs or flat webs the relevant checks are the same. However, the normal stresses in the web are zero for girders with corrugated webs. The main part of the fatigue evaluation is to identify the details that are susceptible to fatigue. Welded joints, notches and changes in cross-section dimensions are usually sensitive to fatigue.

The basis for the λ-method is to calculate a final λ-factor dependent on λ_1, λ_2, λ_3 and λ_4. The λ-factor is bridge specific and accounts for the bridge's length, type of loading, ratio for heavy traffic and the service life of the bridge. λ_1 depends on the bridge's length and calculated according to SS-EN 1993-2 9.5.2 Figure 9.5.

$$\lambda_1 = \begin{cases} 2.55 - 0.7 \cdot \dfrac{L-10}{70} & \text{at mid-span} \\[2ex] 2.0 - 0.3 \cdot \dfrac{L-10}{20} & \text{for } 10m \leq L \leq 30m\text{, at support} \\[2ex] 1.7 + 0.5 \cdot \dfrac{L-30}{50} & \text{for } 30m < L \leq 80m\text{, at support} \end{cases} \tag{3.108}$$

Whereas,

$$L = \begin{cases} L_{span} & \text{for bending moment and shear forces at the supports} \\[2ex] 0.4 \cdot L_{span} & \text{for shear forces at mid-span} \end{cases} \tag{3.109}$$

λ_2 accounts for the amount of heavy traffic on the bridge and is calculated according to Equation 3.110.

$$\lambda_2 = \frac{Q_{m1}}{Q_0} \cdot (\frac{N_{obs}}{N_0})^{1/5} \tag{3.110}$$

Whereas,

Q_{m1} is the mean weight of the heavy traffic in the slow lane, (Krav Brobyggande E.3.1 (f))
Q_0 is a reference weight of the heavy traffic, (SS-EN 1993-2 9.5.2 (3))
N_{obs} is the total number of heavy vehicles in the slow lane per year (SS-EN 1993-2 4.6.1 Table 4.5)
N_0 is a reference value for the number of trucks, (SS-EN 1993-2 9.5.2 (3))

The factor λ_3 accounts for the service life of the bridge and is calculated according to Equation 3.111.

$$\lambda_3 = (\frac{t_{L,d}}{100})^{1/5} \tag{3.111}$$

Whereas,

$t_{L,d}$	is the service life of the bridge [years]
100	is a reference life [years], (SS-EN 1993-2 9.5.2 (5))

According to the Swedish standard, TSFS 2018:57 27 ch. § 3, the last λ-factor is set to $\lambda_4 = 1.0$. The final λ-value is presented in Equation 3.112.

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \tag{3.112}$$

Whereas,

λ	is the final λ-factor for bending moment and/or shear forces, (SS-EN-1993-1-9 Equation 6.1)
λ_1	is the λ-factor accounting for the span length and type of loading
λ_2	is the λ-factor accounting for the amount of heavy traffic
λ_3	is the λ-factor accounting for the service life of the bridge
λ_4	is a λ-factor

The normal stress amplitude is calculated for the detail as shown in Equation 3.113 and shear stress amplitude according to 3.114.

$$\Delta\sigma_E = \lambda \cdot \Delta\sigma \tag{3.113}$$

$$\Delta\tau_E = \lambda \cdot \Delta\tau \tag{3.114}$$

Whereas,

λ	is a factor that accounts for the length of the bridge, the ratio of heavy traffic loads and service life of the bridge (varies dependent on if it is the normal stress or the shear stress that is checked, see Equation 3.112)
$\Delta\sigma_E$	is the normal stress amplitude for the detail accounting for the bridge specific fatigue loads, (SS-EN 1993-1-9 Equation 6.1)
$\Delta\sigma$	is the normal stress amplitude for the specific detail
$\Delta\tau_E$	is shear stress amplitude of the detail accounting for the bridge specific fatigue loads, (SS-EN 1993-1-9 Equation 6.1)
$\Delta\tau$	is the shear stress amplitude for the specific detail

Each detail of the bridge is checked separately. The fatigue class for a certain detail can be found in SS-EN 1993-1-9 Table 8.1-8.10. The risk for fatigue is regarded as safe if Equation 3.115 and 3.116 is fulfilled for each detail (SS-EN 1993-1-9 8 Equation 8.2).

$$\Delta\sigma_E \cdot \gamma_{Ff} < \frac{\Delta\sigma_c}{\gamma_{mf}} \tag{3.115}$$

$$\Delta\tau_E \cdot \gamma_{Ff} < \frac{\Delta\tau_c}{\gamma_{mf}} \tag{3.116}$$

Whereas,

$\Delta\sigma_E$	is normal stress amplitude for the detail accounting for the bridge specific fatigue loads, see Equation 3.113
γ_{Ff}	= 1.0, is a partial factor for the fatigue strength of the material, (SS-EN 1993-2 9.3 (1))
$\Delta\sigma_c$	is the design stress for the specific detail for a fatigue life of 2 million cycles, (SS-EN 1993-1-9 Table 8.1-8.10)
γ_{mf}	is a partial factor accounting for the consequences if failure occur and if inspections are performed, (SS-EN 1993-1-9 Table 3.1)
$\Delta\tau_E$	is shear stress amplitude for the detail accounting for the bridge specific fatigue loads, see Equation 3.114
$\Delta\tau_c$	is the design stress for the specific detail for a fatigue life of 2 million cycles, (SS-EN 1993-1-9 Table 8.1-8.10)

For the case of combined normal- and shear stresses it should be checked that the criterion in Equation 3.117 is fulfilled (SS-EN 1993-1-9 Equation 8.3).

$$(\frac{\Delta\sigma_E \cdot \gamma_{Ff}}{\Delta\sigma_c/\gamma_{mf}})^3 + (\frac{\Delta\tau_E \cdot \gamma_{Ff}}{\Delta\tau_c/\gamma_{mf}})^5 \leq 1 \tag{3.117}$$

4
Case Studies

Chapter 4 aims to present the achieved results of the conducted case studies. The chapter will start by stating the limitations that have been considered during the redesign and then continue presenting each of the studied bridges. Discussions on the results are carried out in Section 6.1. Further, cost estimations for the original and redesign of bridge 100-262-1 are presented, showing the eventual economical savings.

In order to show the potential of replacing conventional carbon steel girders with flat webs, with stainless steel girders with corrugated webs, two case studies were conducted. The case studies were carried out by performing a redesign of the steel cross-sections in existing bridge structures. The redesign was carried out by implementing the new design concept, according to the design procedure as stated in Chapter 3, and then comparing the final design with the already built structures, focusing on material savings. The bridges chosen for the case study were both designed according to Eurocode and relatively newly built. Bridge 100-262-1 was designed in 2015 and built in 2017. Bridge 100-379-1 was designed in 2016 and built in 2017. Both bridges were designed by the Bridge and Hydraulics Department at WSP. The redesign for bridge 100-262-1 is carried out in Appendix F. The redesign for bridge 100-379-1 follows the same procedure as the one presented for bridge 100-262-1.

The redesign of the cross-sections were performed for the same load effects except for the self-weight of the steel, temperature forces, and shrinkage forces. In order to get comparable results, the redesign was performed with some limitations. The limitations were made so that the obtained results, in terms of material savings, comes from the implementation of the new concept, rather than specific design choices made by the previous engineer. The stated limitations are:

- No redesign of the cross-beams has been carried out since the calculations are the same regardless of the choice of concept.
- In the new design, the cross-sections are changed at the same locations, similar to the original designs. However, it is possible that more material could be saved from changing the cross-sections at more sections along the length of the bridge.
- The height of the girders is kept constant. However, further studies for deeper webs are conducted in Chapter 5.

The corrugation shape in the new designs are chosen so that the reduction factors for local and global shear buckling are equal. This was done by adjusting the depth of the corrugation, a_3, and then choosing an appropriate flat-fold length, a_1. The corrugation shapes are presented in Table 4.5 for bridge 100-262-1 and Table 4.16 for bridge 100-379-1.

For the new designs, the temperature loads results in large compressive stresses due to a larger value for the linear coefficient of thermal expansion as mentioned in Section 3.4.2.2. This effect is further discussed in Section 5.3.

4.1 Bridge 100-262-1 over Delångersbron at Forsån, Böle

The first bridge that was evaluated was the bridge 100-262-1 shown in Figure 4.1. The bridge is a one-span twin-girder composite bridge with carbon steel girders and flat webs. The flanges are made in carbon steel grade S420ML. The web, stiffeners, and cross-beams are made in carbon steel grade S355. It is located in Böle, in the northern part of Sweden. The overall geometry and cross-section of the bridge are presented in Table 4.1 and Figure 4.2. The bridge has two lanes and a pedestrian walkway.

Figure 4.1: Bridge 100-262-1, picture taken from BaTMan

For the original bridge design, the loads in ultimate limit state are governing. The utilization ratios for the original and the new design are listed in Table 4.2. The

Table 4.1: The geometry of bridge 100-262-1

Bridge length	52 meter
Span length	51 meter
Bridge width	10.0 meter
Bridge height	2.37 meter

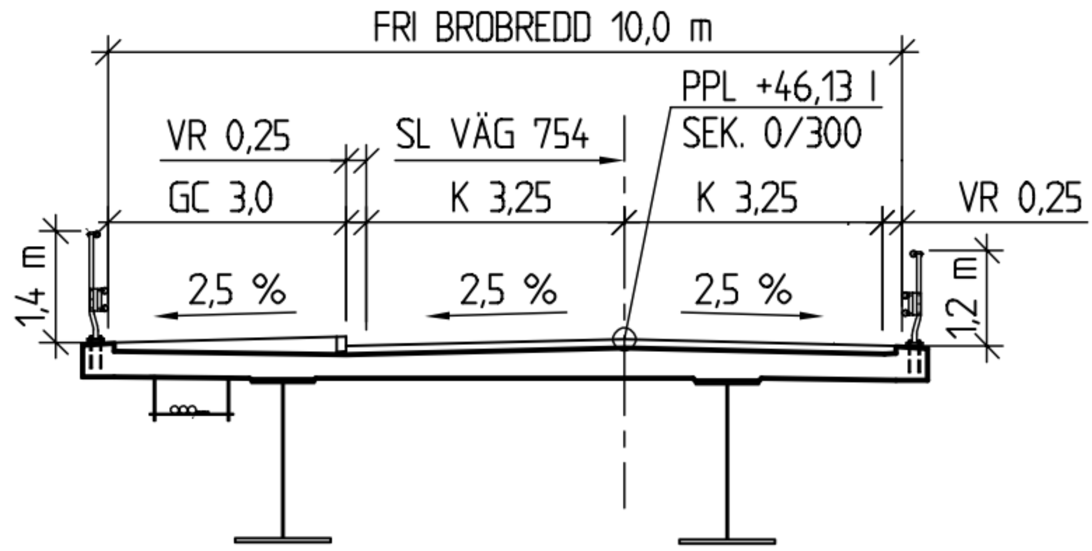


Figure 4.2: Cross-section of bridge 100-262-1

utilization ratios describes the demand over the capacity, and, therefore, imposed to be a good tool for quantifying the safety against failure for the design. In the new design, the utilization ratio for LT-buckling is considerably less than for the original design. The increase in capacity is dependent on the width of the flange, which is larger for the new design. For a larger flange width, the stiffness around the weak axis increases non-linearly, resulting in a much larger capacity against LT-buckling. Values for the flange width, and other sectional dimensions, are listed in Table 4.3 and Table 4.4.

Table 4.2: Utilization ratios for bridge 100-262-1

Governing design checks	**Original design**	**New design**
ULS, LT-buckling during casting	93%	80%
ULS, bending capacity	99%	100%
ULS, shear capacity	95%	97%

The sectional dimensions, before and after the redesign, are presented in Table 4.3 and 4.4. The redesign is carried out using duplex stainless steel, grade 1.4162. At

10.5 meters distance from either of the supports, the web thickness and the flange dimensions are changed. For the change in the flange width, the transition length is 0.8 meters. For the change of the flange thickness, the transition has a slope of 1:4. The shape of the corrugation is presented in Table 4.5.

Table 4.3: Sectional dimensions for bridge 100-262-1 at $x = 0 - 10.5$ meters (near support)

Sectional constants	**Original design** [mm]	**New design** [mm]
b_{fu}	650	765
t_{fu}	42	35
b_{fl}	1000	825
t_{fl}	48	50
t_w	20	8
h_w	1960	1965

Table 4.4: Sectional dimensions for bridge 100-262-1 at $x = 10.5 - 25.5$ meters (mid-span)

Sectional constants	**Original design** [mm]	**New design** [mm]
b_{fu}	800	850
t_{fu}	42	45
b_{fl}	1200	1225
t_{fl}	55	50
t_w	17	6
h_w	1953	1955

Table 4.5: Corrugation shape for bridge 100-262-1, notations according to Figure 2.5

Corrugation shape	
a_1	$120mm$
a_2	$119mm$
a_3	$70mm$
a_4	$96mm$
α	$36deg$
w	$216mm$
s	$239mm$

Comparing the dimensions provided in the two Tables 4.3 and 4.4, it is clear that in the new design, the thickness of the web has changed significantly compared to other dimensions. This change, or more precisely the reduction in the web thickness, results in a total weight reduction and the calculations showed that a material saving of 23% could be achieved. The material savings mainly comes from the main girders, but also from the studs and welds. The material savings are presented in Table 4.6.

Table 4.6: Material savings in percent and mass between the original and the new design

Part	**Material savings**	
	[%]	[kg]
Main girder	23%	$24 \cdot 10^3$ kg
Studs	21%	122 kg
Cross-beams	4%	175 kg
Welds	23%	11 kg
Total savings	23%	$25 \cdot 10^3$ kg

4.1.1 Cost estimation

In order to show the potential of using stainless steel girders with corrugated webs, in terms of economical savings, a cost estimation for producing and maintaining the steel girder was requested from *Stål och Rörmontage AB*. An independent party were consulted since they could appreciate the actual net present value, dependent on their purchase prices. The unit prices for the bridge's parts were separated from the overall cost estimation, these are listed in Table 4.7. As can be seen in Table 4.9, the total cost, affiliated with the production of the girders, does not differ much between the two concepts.

Table 4.7: Material cost for bridge 100-262-1 according to Stål och Rörmontage AB (Personal communication, May 22, 2020)

Part	**Original design**	**New design**
Steel, general	9/12* SEK/kg	28 SEK/kg
VKR-profile	15 SEK/kg	40 SEK/kg
Studs	50 SEK/kg	50 SEK/kg

*Steel grade S355/S420

Table 4.8: Total material consumption

Original design	**New design**
117 014 kg	88 768 kg

Table 4.9: Total cost for bridge 100-262-1 according to Stål och Rörmontage AB (Personal communication, May 22, 2020)

Part	**Original design**	**New design**
Material cost	1 800 000 SEK	2 800 000 SEK
Production cost	2 100 000 SEK	1 500 000 SEK
Total cost	3 900 000 SEK	4 300 000 SEK

Table 4.9 shows that the production cost for the original design is higher than for the new design. According to Lars-Åke Persson at Stål och Rörmontage AB (Personal communication, 26 may, 2020), the increased production cost for producing the bridge in carbon steel originates from:

- Longer welding time in the factory (labour cost increases)
- Cost of painting (stainless steel needs to be pickled, however, the cost of pickling is lesser than the cost of painting)
- The needed on-site work is faster for the new design than for the original design (welding and painting)

The total cost of the new design is 400 000 SEK greater than the cost of the original bridge. However, when evaluating the total cost of a bridge during its full service life, the need for maintenance should be considered when comparing the two alternatives. As mentioned in Chapter 1, the cost of maintaining stainless steel structures is neglectable compared to carbon steel structures. In order to create an estimation for these costs, Stål och Rörmontage AB has a tool for estimating the maintenance cost of repainting the carbon steel bridge throughout its service life. The tool is using values from tabloids stated by *the Swedish Transport Administrations* for the service periods, and cost estimations for each service measure. In these cost estimations, the same interest rate of 3.5% is used as the Swedish Transport Administration uses in their cost estimations. The costs affiliated with repainting the bridge is presented in Table 4.10. The calculated cost only considers painting costs and no costs associated with traffic delays.

Table 4.10: Maintenance costs for bridge 100-262-1 during its service life, produced in carbon steel

Year	Service measure	Cost	Net present value
10	Paint touch up	572 000 SEK	405 502 SEK
20	New cover paint	880 000 SEK	442 258 SEK
30	Paint touch up	572 000 SEK	203 791 SEK
40	Repainting	2 024 000 SEK	511 207 SEK
50	Paint touch up	572 000 SEK	102 419 SEK
60	New cover paint	880 000 SEK	111 702 SEK
70	Paint touch up	572 000 SEK	51 472 SEK
80	Repainting	2 024 000 SEK	129 117 SEK
90	Paint touch up	572 000 SEK	25 868 SEK
100	New cover paint	880 000 SEK	28 213 SEK
110	Paint touch up	572 000 SEK	13 000 SEK
	Total costs	10 120 000 SEK	2 024 549 SEK

When considering the maintenance costs presented in Table 4.10, associated with the repainting of the bridge, the total cost for the original bridge, presented in Table 4.9, increases with approximately 2 000 000 SEK. Reviewed values for the total cost of the different designs, including the maintenance cost, are listed in Table 4.11.

Table 4.11: Total cost for the steel girders in bridge 100-262-1 during the service life of the bridge

Part	Original design	New design
Material costs	1 800 000 SEK	2 800 000 SEK
Production costs	2 100 000 SEK	1 500 000 SEK
Maintenance costs	2 024 000 SEK	-
Total costs	5 924 000 SEK	4 300 000 SEK

Table 4.11 shows that the overall cost of using girders in stainless steel with corrugated webs is lesser than the cost of the original design. The difference in net present value is 1 624 000 SEK.

4.2 Bridge 100-379-1 over a gorge north of Hogsta

The second case study is performed on a simply supported twin-girder composite bridge, located over a ravine in Hogsta, Västmanland County. The bridge is originally designed with carbon steel, grade S355. The theoretical span length of the bridge is 36 meters and it is a two-lane road bridge. The bridge is shown in Figure 4.3 and the geometry of the bridge is listed in Table 4.12.

Figure 4.3: Bridge 100-379-1, picture taken from BaTMan

Table 4.12: The geometry of bridge 100-379-1

Bridge length	38 meter
Span length	36 meter
Bridge width	6.6 meter
Bridge height	2.16 meter

The original design is governed by the fatigue limit state, denoted by FAT, whereas the highest utilization ratios are obtained for the normal stress amplitude in the flanges. The redesign of the bridge is conducted with duplex stainless steel, grade 1.4162, and governed by fatigue in the lower flange, and by ULS-loads in the upper flange. The highest utilization ratios for the original and the new design are presented in Table 4.13. The normal compressive stresses in the upper flange for stainless steel girders are larger than for carbon steel girders, due to temperature effects, causing a larger utilization ratio. The bridge changes cross-section dimensions at 11 meters, thus, one of the governing checks is at the one of the notches there.

Table 4.13: Governing utilization ratios for bridge 100-379-1

Governing checks	**Original design**	**New design**
FAT, notch at 11 meters distance from either of the supports	98%	94%
FAT, joint between lower flanges in mid-span	96%	100%
ULS, normal stresses in upper flange	82%	99%

The cross-sectional dimensions for the original design and the redesign is presented in Table 4.14 and 4.15. The corrugation shape is presented in Table 4.16. Since FAT governed the design of the flanges, the redesign of bridge 100-379-1 only resulted in a change of the web thickness, keeping the same flange dimensions. The change of the flange and the web thicknesses was placed at a distance of 11 meters from either of the supports.

Table 4.14: Sectional dimensions for bridge 100-379-1 for $x = 0 - 11$ meters (near support)

Sectional constants	**Original design** [mm]	**New design** [mm]
b_{fu}	400	400
t_{fu}	25	25
b_{fl}	600	600
t_{fl}	45	45
t_w	16	5
h_w	1830	1830

Table 4.15: Sectional dimensions for bridge 100-379-1 for $x = 11 - 18$ meters (mid-span)

Sectional constants	**Original design** [mm]	**New design** [mm]
b_{fu}	400	400
t_{fu}	30	30
b_{fl}	600	600
t_{fl}	50	50
t_w	14	4
h_w	1820	1820

Table 4.16: Corrugation shape for bridge 100-379-1

Corrugation shape	
a_1	$100mm$
a_2	$102mm$
a_3	$60mm$
a_4	$83mm$
α	$36deg$
w	$183mm$
s	$202mm$

The calculations showed a material saving of 29%. This saving mainly comes from changing the dimensions of the web. All material savings are presented in Table 4.17.

Table 4.17: Material savings in percent and mass between the original and the new design, bridge 100-379-1

Part	**Material savings**	
	[%]	[kg]
Main girder	30%	$12 \cdot 10^3$ kg
Studs	35%	123 kg
Cross-beams	4%	108 kg
Welds	4%	1 kg
Total savings	29%	$12 \cdot 10^3$ kg

5
Parametric studies

In the following chapter, the results of five different parametric studies are presented. Discussions on the subjects will continue in Section 6.2.

When concluding what factors affect the design of a composite bridge built with stainless steel girders and corrugated webs, some parameters were identified as governing or interesting to further evaluate. These parameters are related to either the use of corrugated webs, the choice of material, or both.

Five analyses are conducted to more specifically determine the parameter's influence on the studied bridge concept. The analyses consider the corrugation parameter's influence on the effective width, the corrugation's influence on the shrinkage forces, the influence of the linear coefficient of thermal expansion on the stress distribution, the corrugation's influence on the elastic bending stiffness, and the web height's influence on material savings.

5.1 The corrugation's influence on the effective width of steel flanges

An effective width for the steel flanges in cross-section class four is calculated in order to make sure that the part of the flange that is assumed to be contributing with stiffness, does not risk to buckle locally under compressive loading. The effective width is both affected by the use of corrugation and the choice of material. As mentioned in Section 3.3.1.2, the effective width is only used if the part fulfills the requirements for cross-section class four. If the part is in cross-section class four, a reduction of the contributing flange is calculated. The size of the reduction is dependent on the slenderness, which then is dependent on the choice of material and whether or not corrugated webs are used.

Dependent on the corrugation geometries, the outstanding part of the flange, $\bar{b}$, will vary. For a web with a deep corrugation, the length of the outstanding flange will be larger, thus posing a greater risk of being in cross-section class four. However, the shape of the corrugation provides a stabilizing effect thanks to its high out-of-plane stiffness, meaning that a deep corrugation supports the flanges during lateral loading.

In order to evaluate the influence of the corrugation on the effective width, a study is conducted. In the study, five different cross-sections are investigated as follows:

1. A cross-section with flat webs, in carbon steel
2. A cross-section with corrugated webs, in carbon steel
3. A cross-section with flat webs, in stainless steel
4. A cross-section with corrugated webs, in stainless steel
5. A cross-section with corrugated webs, in stainless steel, using the definition described by Johansson et al. (2007) to define the outstanding part of the flange (described in Section 3.3.1.2)

The material dependency for the calculation of the effective width comes from the reference factor, ϵ, calculated according to Equation 3.18 (stainless steel) and 3.19 (carbon steel). The reference factor then affects the calculation of the slenderness (Equation 3.23), which is used to calculate the reduction factor for the flange buckling. In the study, values for stainless steel grade 1.4162 (lean duplex) and carbon steel grade S355 are used.

As mentioned, a deep corrugation provides lateral restraint to the flanges, but for a deep corrugation, the outstanding part of the flange increases, thus increasing the risk of flange buckling. Therefore, the buckling coefficient, k_σ, is used when calculating the slenderness. The factor accounts for the corrugation's out-of-plane stiffness, leading to that the buckling coefficient, k_σ, is larger for a corrugated web than for a flat web, which in many cases leads to a smaller reduction of the flange buckling capacity.

5.1.1 Formulation of the buckling coefficient, k_σ

In Section 3.3.1.2, it is stated that the equation given by SS-EN 1993-1-5 for calculation of k_σ (Equation 3.20), is incorrectly formulated. The formulation stated by the Eurocodes delimits the contribution from k_σ for corrugated webs resulting in that no reduction of the flange buckling capacity is considered. In Figure 5.1, the calculated reduction factors based on the Eurocodes, or as proposed in Section 3.3.1.2, are compared. The reduction factor is calculated for a girder in stainless steel with a varying width of the upper flange ranging from 500 millimeters to 2500 millimeters. The depth of the corrugation, a_3, is set to 75 millimeters. The reduction factor for a girder with flat webs is shown as well.

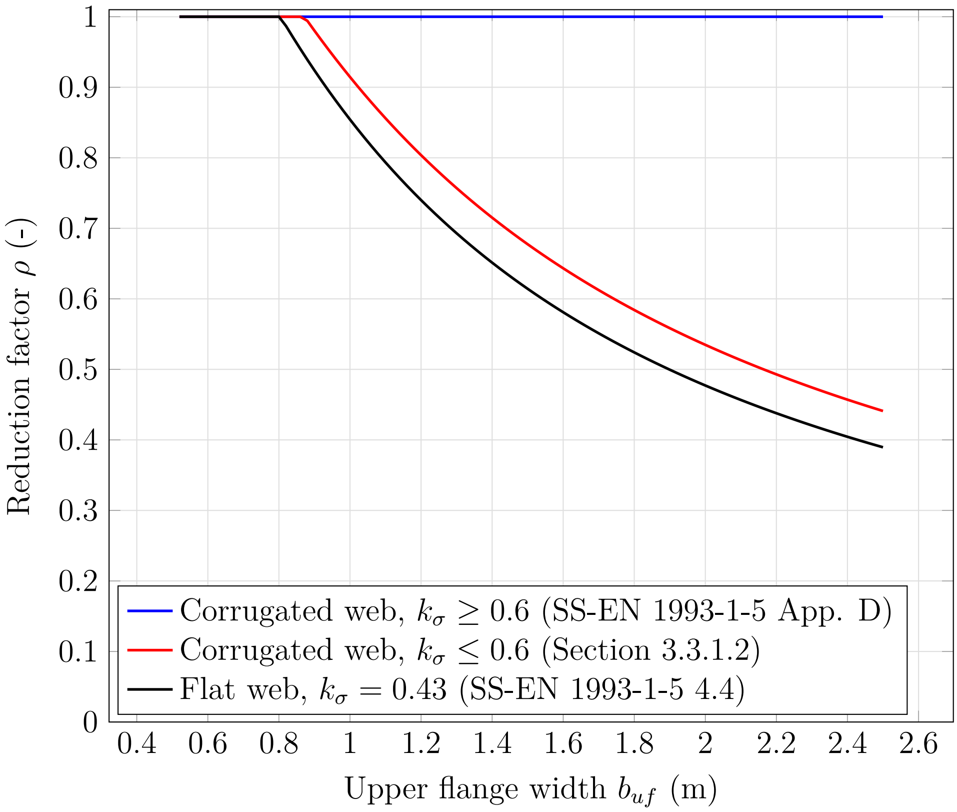


Figure 5.1: Reduction factor for varying flange width, a_3=75 mm

5.1.2 Investigation of the reduction factor for different corrugation depths

The reduction factors for different corrugation depths, a_3, ranging from 75 millimeters to 200 millimeters are investigated. The reduction factor, ρ, is calculated for an increasing width of the top flange. Figure 5.2, 5.3, and 5.4 illustrates the reduction factor dependent on the depth of the corrugation and width of the flanges. In the figures, the blue lines show the reduction factor for girders with corrugated webs. Red lines show the reduction factor for girders with flat webs, and the green lines illustrate the reduction factor using the definition of the outstanding flange defined by Johansson et al. (2007). All calculations are presented in Appendix B.

Figure 5.2 shows the relation between the reduction factor, ρ, and the flange width for a corrugation depth of 75 millimeters. Both girders with flat webs and corrugated webs are compared. In the figure, it could be seen that for the same flange width the reduction factor for a corrugated web is larger than the reduction factor for a flat web. The graph also shows that the reduction factor for carbon steel (dotted lines in Figure 5.2) is larger than the factor for stainless steel.

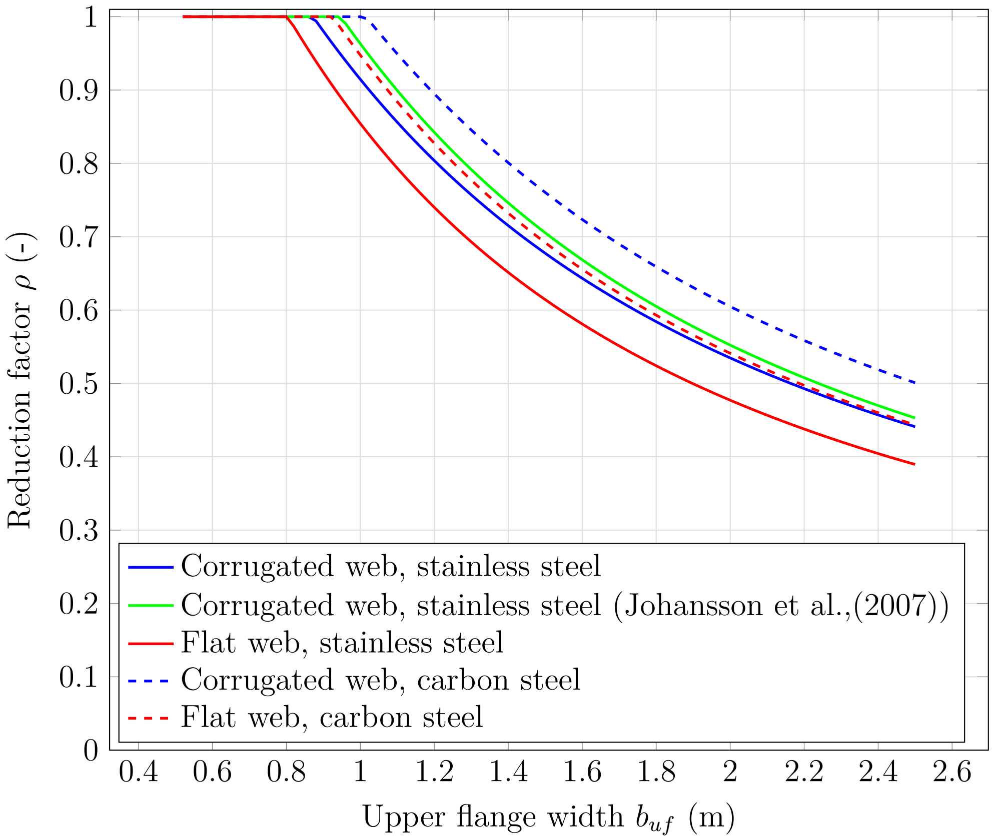


Figure 5.2: Reduction factor for varying flange width, a_3=75 mm

Exact values for the reduction factor, ρ, with the flange width 1100 millimeters and 75 millimeters corrugation depth, is presented in Table 5.1. The flange width of 1100 millimeters is chosen in order to show an arbitrary point in the curve, where all reduction factors a equal or smaller than unity.

Table 5.1: Values of ρ for $b_{uf} = 1100mm$ and $a_3 = 75mm$

Type of girder	ρ
Corrugated web, stainless steel	0.86
Corrugated web, stainless steel (Johansson et al., (2007))	0.90
Flat web, stainless steel	0.79
Corrugated web, carbon steel	0.95
Flat web, carbon steel	0.88

In Figure 5.3, the reduction factor for an increased corrugation depth is shown. The depth of the corrugation is set to 137.5 millimeters. When increasing the corrugation depth, the length of the free part of the outstanding flange increases, thus decreasing the reduction factor for a girder with corrugated webs. This results in the reduction factor, for the girder with corrugated webs, becomes less than 1.0, at a similar flange width as a girder with flat webs. The difference in the slope of the curves between the corrugated web and the flat web is coming from the difference in the buckling coefficient, k_σ, described at the beginning of this section. Exact values for the reduction factor are presented in Table 5.2.

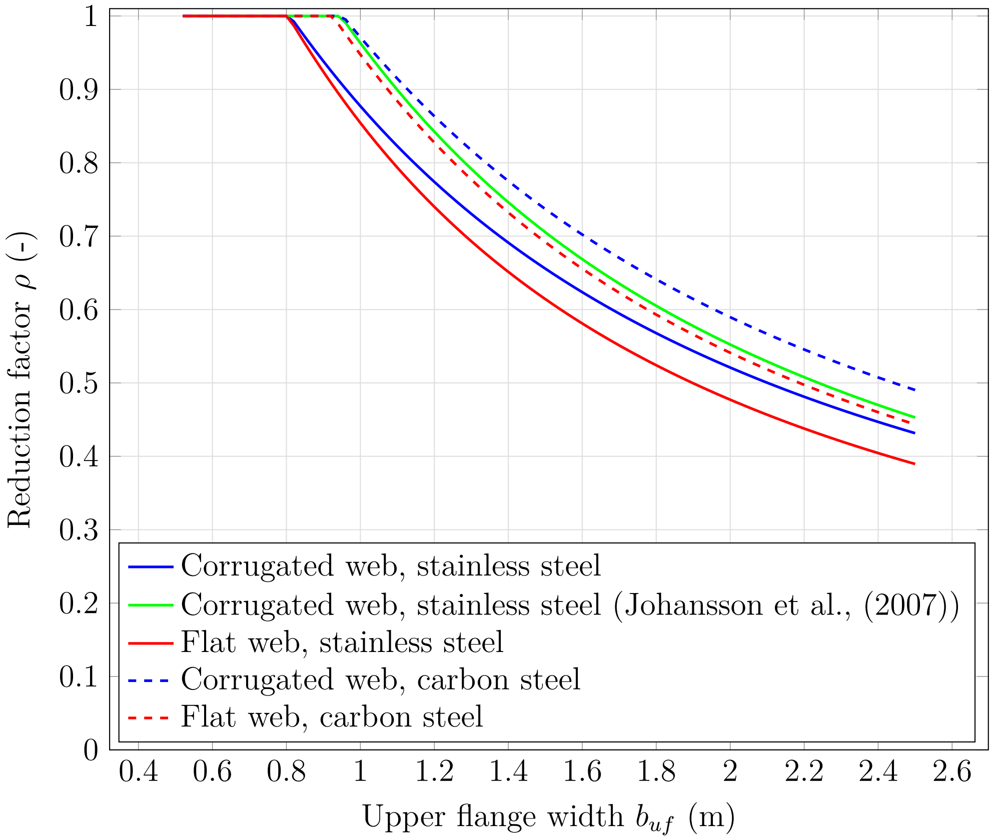


Figure 5.3: Reduction factor for varying flange width, a_3=137.5 mm

Table 5.2: Values of ρ for $b_{uf} = 1100mm$ and $a_3 = 137.5mm$

Type of girder	ρ
Corrugated web, stainless steel	0.82
Corrugated web, stainless steel (Johansson et al., (2007))	0.90
Flat web, stainless steel	0.79
Corrugated web, carbon steel	0.91
Flat web, carbon steel	0.88

The reduction factor for a corrugation depth of 200 millimeters is shown in Figure 5.4. In the figure, it could be seen that the curve is irregular for the modified way to define the outstanding part of the flange. The irregularity depends on if the criterion for the length of the outstanding flange in Equation 3.22 is fulfilled or not. In this case, the criterion is first fulfilled for a flange width of approximately 830 millimeters. For flange widths below 830 millimeters the criterion is not fulfilled and the same way to define the outstanding flange as for corrugated webs is used. Hence, the curve for the modified way and the curve for a stainless steel girder with corrugated web is plotted on top of each other, as can be seen in the figure. Once the criterion is fulfilled the reduction factor increases since the theoretical distance of the outstanding flange is reduced. Exact values for the reduction factor, ρ, for a corrugation depth of 200 millimeters is presented in Table 5.3.

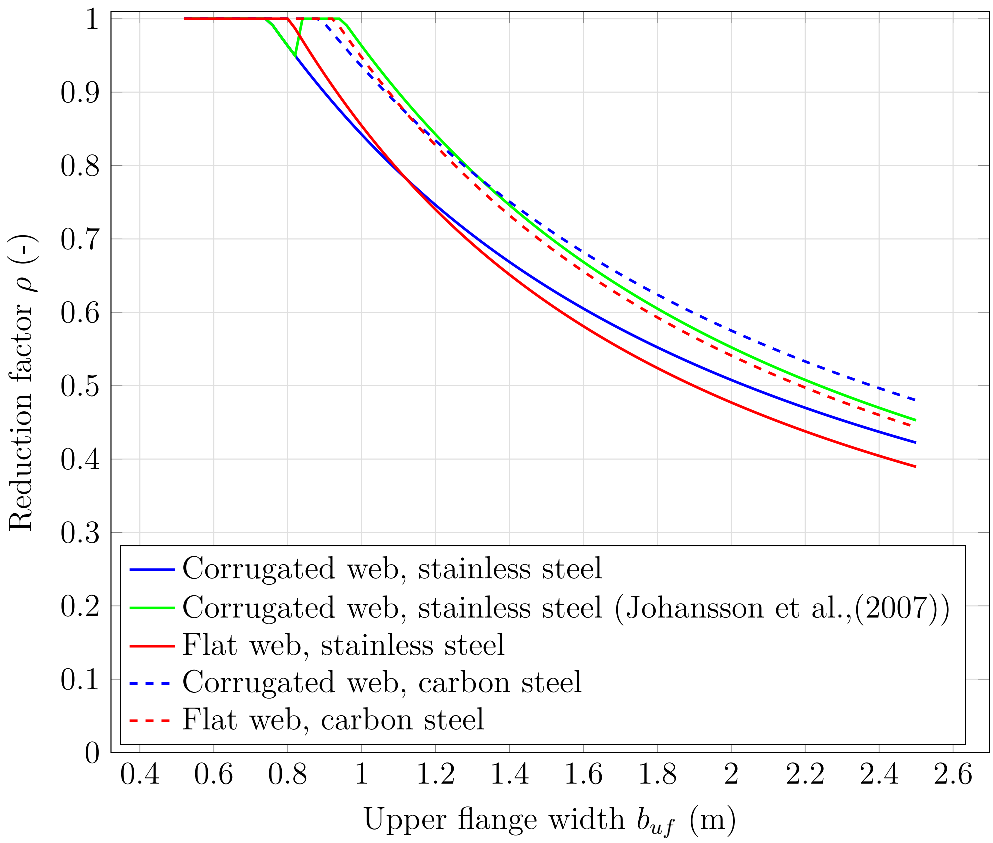


Figure 5.4: Reduction factor for varying flange width, a_3=200 mm

Table 5.3: Values of ρ for $b_{uf} = 1100mm$ and $a_3 = 200mm$

Type of girder	ρ
Corrugated web, stainless steel	0.79
Corrugated web, stainless steel (Johansson et al., (2007))	0.90
Flat web, stainless steel	0.79
Corrugated web, carbon steel	0.88
Flat web, carbon steel	0.88

From Table 5.1, 5.2, and 5.3 it could be interpreted that for deeper corrugations, the reduction factor decreases, which means that the effective width decreases as well. However, it is shown that for the modified way of defining the outstand flange, the reduction factor is kept unchanged as long as the criterion, stated in Section 3.3.1.2 (Equation 3.22) is fulfilled. When the criterion is fulfilled, the buckling coefficient, k_σ, is calculated for the average outstand of the flange. The buckling coefficient is then only dependent on the flat-fold lengths of the corrugation, hence independent on the depth, a_3. Therefore, the reduction factor is kept constant for the modified way of defining the outstand flange, which can be seen in Table 5.1, 5.2, and 5.3. The reduction factor is larger for the modified formulation thanks to the lateral restraint coming from the corrugation.

5.2 Corrugation influence on shrinkage forces

When using corrugated webs instead of flat webs in girders, the cross-sectional properties change. This influences the applied forces caused by the shrinkage of concrete since they are strictly dependent on the cross-sectional properties. If the girder is designed with corrugated webs, the stress distribution over the cross-section differs from a girder designed with flat webs. The difference is caused by the change in cross-sectional area, change of center of gravity, and change in stiffness. As mentioned in Section 3.3.3.1, the calculations for a girder with a corrugated web is conducted without any contribution from the web. When the concrete slab is subjected to shrinkage, compulsive stresses arise in the steel section. The stresses in the top flange of the steel are larger for a corrugated web than for a flat web, and therefore their influence on the design loads in ULS is of interest.

The study is conducted for the bridge 100-262-1 using the same input data as presented in Section 4.2. Forces and stresses in the steel and the concrete are calculated according to Section 3.5.1.2. The stresses are calculated to show the stress distribution over the cross-section. All calculations are presented in Appendix C.

In Figure 5.5, the stress distribution over the cross-section is presented. The blue lines represent a cross-section designed with corrugated webs, and the red lines represent a cross-section with flat webs. In order to visualize what happens when the web height is increased, the dotted lines represent the stress distribution for an increase of 25% in web height. The variable, z, shows the distance measured from the bottom steel flange for the web height $h_w = 1.0h_{w0}$. In Table 5.4 and Table 5.5, exact values for the stress distribution are presented. It can be interpreted from Figure 5.5 that the compressive stresses in the upper steel flange for a corrugated web is larger than for a flat web, although the opposite is true for the tensile stresses. The tensile stresses are, however, small and have a small influence in the ULS-loads. Furthermore, it can be interpreted that an increase in the web height does not result in any substantial change in the stress distribution.

Table 5.4: Stress distribution over the cross-section for $h_w = 1.0h_{w0} = 1955mm$, whereas the distance, z, measures the distance from the bottom steel flange

		Stresses		
	z (m)	**Corrugated web**	**Flat web**	**Deviation**
Upper concrete	2.37	0.58 MPa	0.72 MPa	−20%
Lower concrete	2.05	0.96 MPa	1.11 MPa	−13%
Upper steel flange	2.05	−34.83 MPa	−32.66 MPa	+7%
Lower steel flange	0	2.12 MPa	4.69 MPa	−55%

Table 5.5: Stress distribution over the cross-section for $h_w = 1.25h_{w0} = 2444mm$, whereas the distance, z, measures the distance from the bottom steel flange

		Stresses		
	z (m)	**Corrugated web**	**Flat web**	**Deviation**
Upper concrete	2.37	0.63 MPa	0.81 MPa	−22%
Lower concrete	2.05	0.95 MPa	1.13 MPa	−16%
Upper steel flange	2.05	−35.11 MPa	−32.39 MPa	+8%
Lower steel flange	-0.49	1.76 MPa	4.81 MPa	−63%

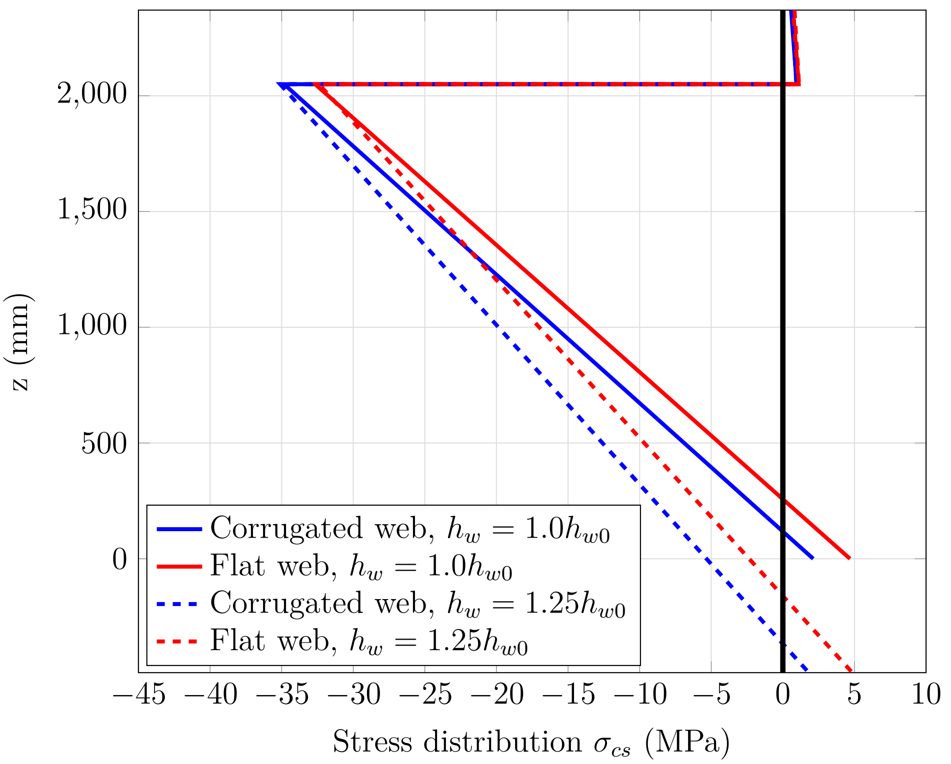


Figure 5.5: Stress distribution due to shrinkage for corrugated webs and flat webs

5.3 The influence of the linear coefficient of thermal expansion on the stress distribution

Stainless steel is more responsive to temperature variations than carbon steel, as can be seen in Table 3.18. The linear coefficient of thermal expansion for stainless steel is 33% larger than for carbon steel. This means that during the design of a composite bridge in stainless steel, considerations must be done so that the resulting stresses are kept low. Furthermore, the guidelines allow a composite bridge designed in concrete and carbon steel to use the same coefficient for both materials. The design code does not give any recommendations for the coefficient for a girder in stainless steel, and therefore a strain difference between the two materials is calculated. The calculations are carried out following the procedure presented in Section 3.4.2.2.3.

Four different studies are conducted in order to consider the influence of the thermal expansion coefficient. Stresses are calculated for the characteristic value of the load. First, the influence of corrugation and stainless steel is evaluated. Secondly, the web height is increased and two comparisons are made dependent on the design. The third study compares the compressive stresses dependent on the maximum temperature variations in Sweden. The last study evaluates the stress distribution dependent on the largest and smallest value for the temperature stress. The studies are conducted for bridge 100-262-1, and input data is taken from Section 4.2. The largest stress differences between the concrete and the steel are calculated for the load cases presented in Section 3.4.2.2.3. For the first two studies, the stress is calculated for the mean temperature variation. In the last two studies, the stress is calculated for the largest and the smallest temperatures. The maximum temperature variations are studied since expansion in the steel gives rise to compressive stresses in the upper steel flange, and the tensile stresses in the lower flange. The temperature stresses are, therefore, enhancing the ULS-loads. The studied temperature variations are listed in Table 5.6.

Table 5.6: Showing the studied temperature ranges (TSFS 2018:57)

Variable	Value	Description
$\Delta T_{N.exp}$	$25-35°C$	is the interval for the maximum temperatures in Sweden
$mean(\Delta T_{N.exp})$	30°C	is the mean maximum temperature
$T_{N.exp,min}$	25°C	is the smallest maximum temperature
$T_{N.exp,max}$	35°C	is the largest maximum temperature

5.3.1 Influence of corrugation and stainless steel

The first part of the study shows the stress distribution over the cross-section for the mean temperature, listed in Table 5.6. Stresses in the concrete and the steel parts are calculated for four different levels, placed at the top and at the bottom of each part. The comparison is made for four different cross-sections, illustrated in

Figure 5.6. The plot shows that using a corrugated web in stainless steel results in the largest compressive stresses in the upper steel flange. The smallest compressive stresses are achieved for a carbon steel girder with flat webs. The largest tensile stresses are obtained for a girder with flat webs, however, they are small. As can be seen in Figure 5.6 the corrugation has a small effect on the stresses and the main influencing parameter is the choice of material. Stresses are presented in Table 5.7.

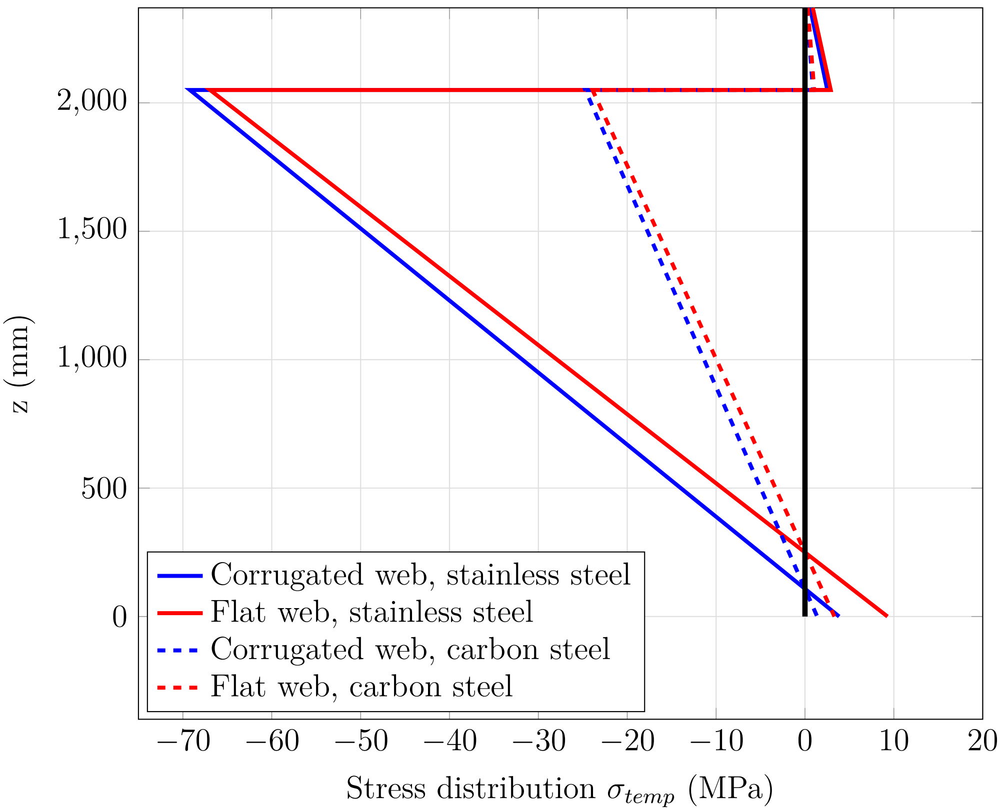


Figure 5.6: Stress distribution due to temperature differences; investigating the influence of using stainless steel girders with corrugated webs

Table 5.7: Stress distribution for four different girders, whereas the distance, z, measures from the bottom steel flange

Part	z	Stainless steel		Carbon steel	
		Corrugated	Flat	Corrugated	Flat
-	[m]	[MPa]	[MPa]	[MPa]	[MPa]
Upper concrete	2.37	0.58	0.88	0.21	0.31
Lower concrete	2.05	2.51	2.90	0.90	1.04
Upper steel flange	2.05	−69.21	−66.94	−24.72	−23.91
Lower steel flange	0	3.84	9.29	1.37	3.32

5.3.2 Influence of the height of the web

The next study is carried out with the same input data as the first. Two comparisons are performed to find out if the stresses are more dependent on the choice of girder (corrugated web or flat web) or the choice of material, considering an increase in the depth of the web. The first comparison is presented in Figure 5.7 and shows what influence the choice of girder has (exact values are shown in Table 5.8). The next comparison is presented in Figure 5.8 and shows what influence the material choice has (exact values are listed in Table 5.9). From both figures, it could be stated that for an increased web height, the stresses do not increase nor decrease considerably. Still, it could be seen that the material choice has a larger influence on the stresses than the increase in the web height.

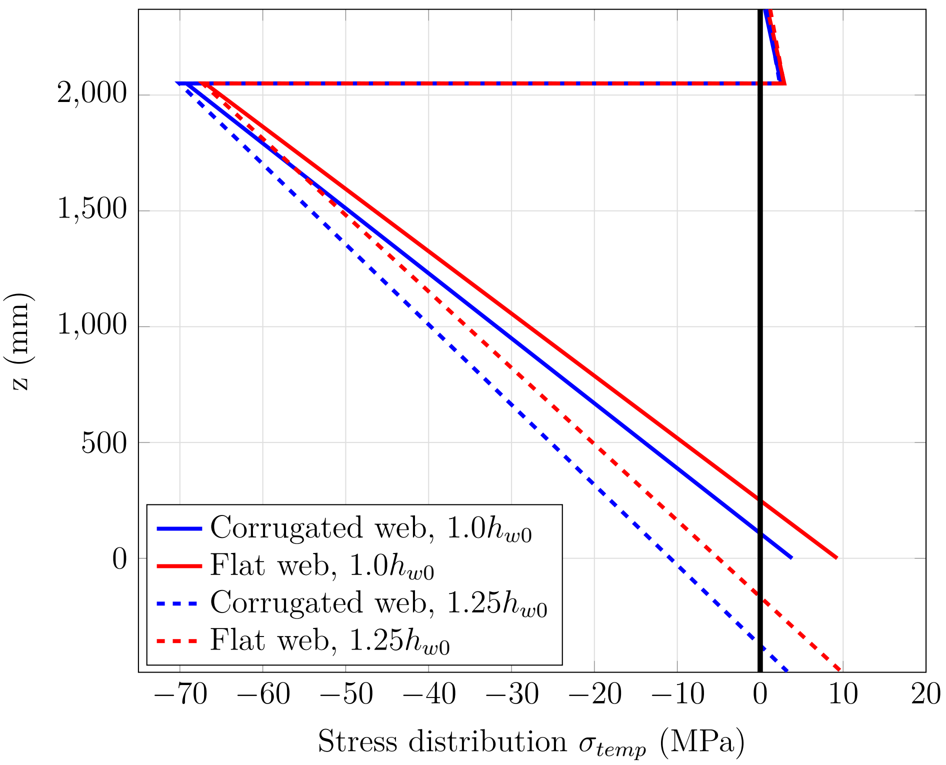


Figure 5.7: Stress distribution due to temperature variations when the web height is increased, for girders in stainless steel

Table 5.8: Stress distribution due to temperature variations when the web height is increased, for girders in stainless steel

	Corrugated web		**Flat Web**	
Part	$h_w = 1.0h_{w0}$ **[MPa]**	$h_w = 1.25h_{w0}$ **[MPa]**	$h_w = 1.0h_{w0}$ **[MPa]**	$h_w = 1.25h_{w0}$ **[MPa]**
Upper steel flange	−69.21	−70.05	−66.94	−67.24
Lower steel flange	3.84	3.27	9.29	9.77

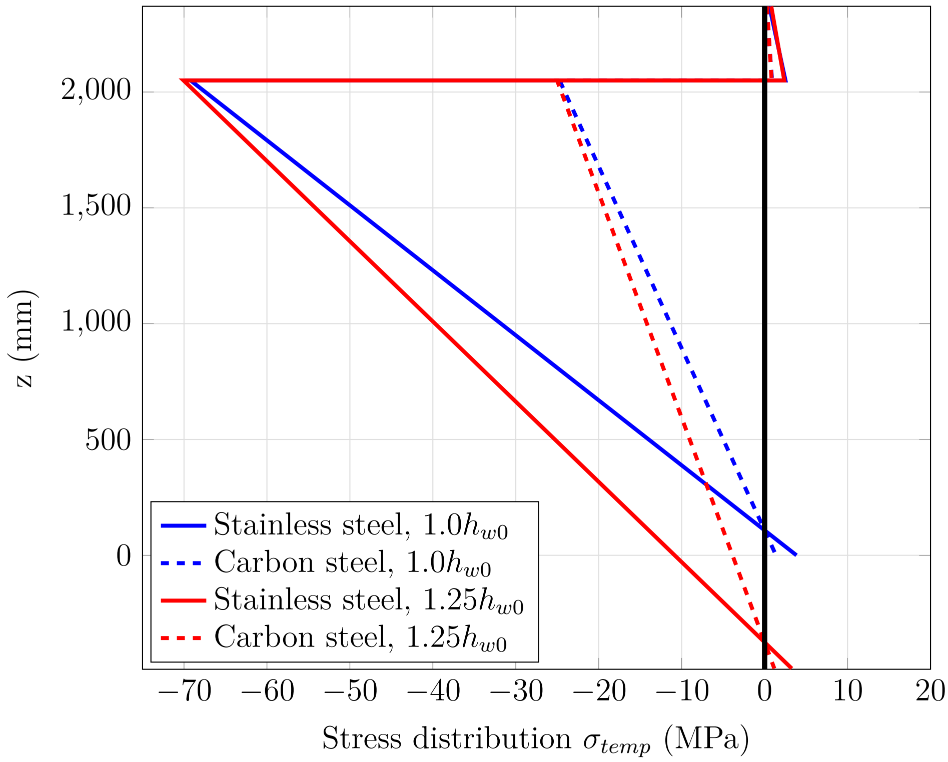


Figure 5.8: Stress distribution due to temperature variations when the web height is increased, using corrugated webs

Table 5.9: Stress distribution due to temperature variations when the web height is increased, using corrugated webs

	Stainless steel		**Carbon steel**	
Part	$h_w = 1.0h_{w0}$ **[MPa]**	$h_w = 1.25h_{w0}$ **[MPa]**	$h_w = 1.0h_{w0}$ **[MPa]**	$h_w = 1.25h_{w0}$ **[MPa]**
Upper steel flange	−69.21	−70.05	−24.72	−25.02
Lower steel flange	3.84	3.27	1.37	1.17

5.3.3 Influence of the maximum temperature variations

In order to show how the calculated maximum temperature variations influences the stresses in the upper flange, the stresses are plotted for the full temperature range in Figure 5.9. The plot mainly shows that the compressive stresses in the upper flange are larger for a girder in stainless steel. It could also be seen that the compressive stresses are insignificantly larger for a corrugated web than for a flat web. However, the difference rather comes from the choice of material. The stresses are constant for carbon steel because the linear coefficient of thermal expansion is the same for both the concrete and carbon steel.

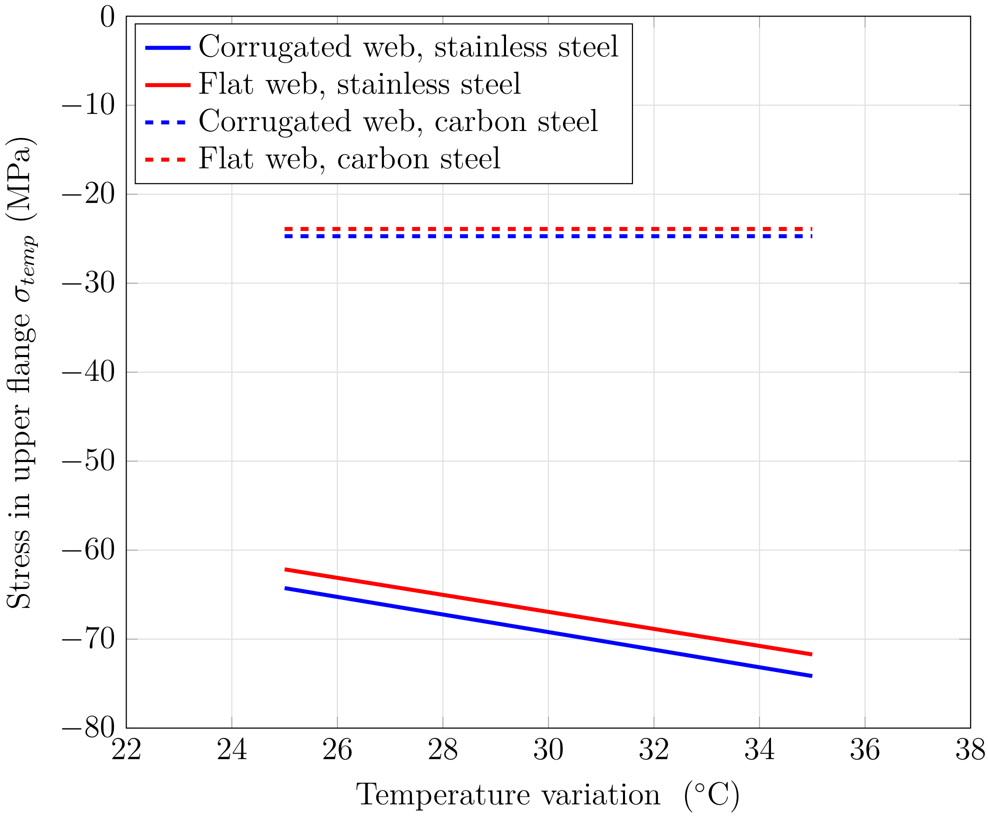


Figure 5.9: Stresses in upper flange dependent on the maximum temperature, $\Delta T_{N.exp}$

5.3.4 The magnitude of the temperature variations influence on the stress distribution

In the last study the stress distribution over the cross-section is shown for the maximum and minimum temperature variations, $\Delta T_{N.exp}$, described in Table 5.6. The results can be seen in Figure 5.10. As expected, the graph shows that a larger temperature variations gives rise to larger compressive stresses. Exact values are presented in Table 5.10, displaying a linear relationship between the stresses and the temperature variation.

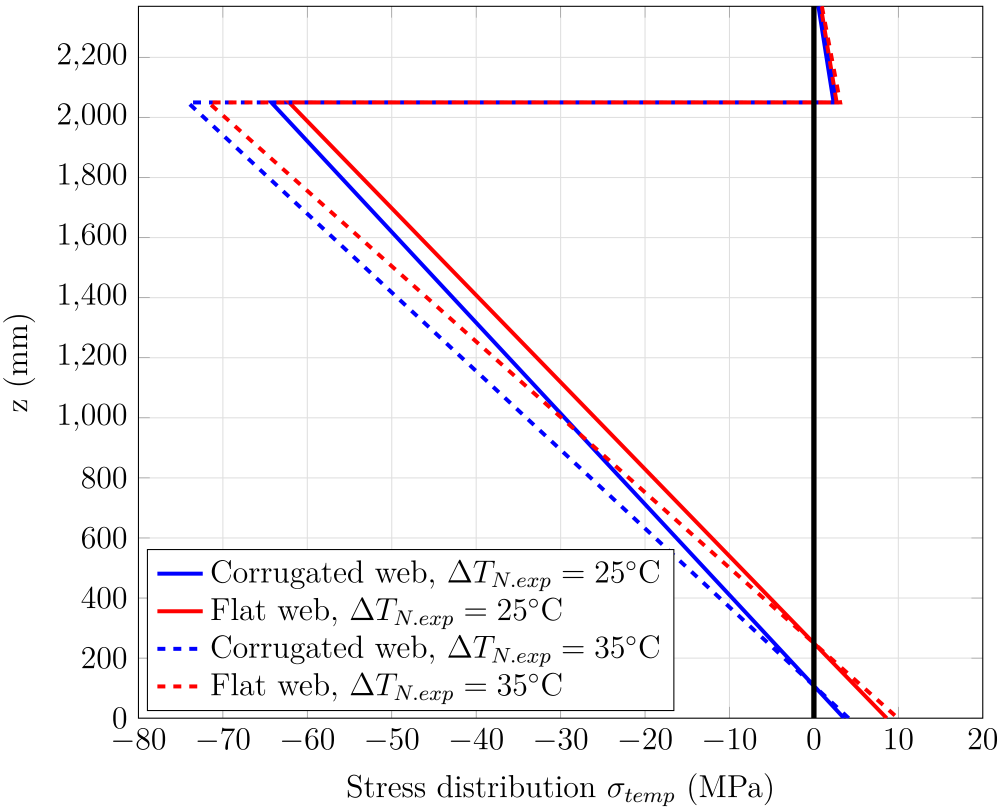


Figure 5.10: Stress distribution over the cross-section for $\Delta T_{N.exp} = 25°C$ and $\Delta T_{N.exp} = 35°C$

Table 5.10: Comparison of compressive steel stresses in the upper steel flange (MPa)

	$\Delta T_{N.exp} = 25°C$	$\Delta T_{N.exp} = 35°C$	**Deviation (%)**
Corrugated web	−64.27 MPa	−74.15 MPa	13%
Flat web	−62.16 MPa	−71.72 MPa	13%
Deviation (%)	3%	3%	

5.4 The corrugation's influence on the elastic bending stiffness

The elastic section modulus is used for calculating the sectional stresses. Girders with corrugated webs has a negligible axial stiffness in the longitudinal direction, due to the accordion effect. The web is thereby assumed not to contribute to the stiffness and therefore excluded in the calculation of the center of gravity and the moment of inertia. This leads to decreased elastic section modulus. Three studies are performed to investigate the influence of neglecting the web. The first study is performed on a steel-only girder. The second study is carried out for a composite section, and the third is performed on a composite section with an increased web height of 25%. The elastic section modulus is calculated according to Equation 5.1.

$$W_{el} = min(\frac{I_y}{z_{tp}}, \frac{I_y}{h_w - z_{tp}}) \tag{5.1}$$

Whereas,

W_{el}	is the elastic section modulus
I_y	is the moment of inertia
z_{tp}	is the distance from the center of gravity
h_w	is the height of the web

5.4.1 Influence on the steel cross-section

The study is performed on four different types of girders to investigate the difference in elastic section modulus when comparing flat webs with corrugated webs. The dimensions of the studied girder are presented in Appendix E. The examined sections for the steel cross-section are listed as:

1. Steel cross-section with corrugated web
2. Steel cross-section with flat web, 10 millimeter web
3. Steel cross-section with flat web, 16 millimeter web
4. Steel cross-section with flat web, 20 millimeter web

The difference in elastic bending stiffness for girders with corrugated webs and girders with flat webs are plotted in Figure 5.11, and shows that there is a deviation which increases for an increasing web thickness and web height. The figure shows the elastic bending stiffness for varying web height and for an upper flange width of 800 millimeters. The elastic section modulus calculated according to Equation 5.1, where the distance to the centre of gravity has an influence. In the studied cases, the center of gravity for the steel cross-section is located closer to the bottom flange than to the top flange because of its dimensions being larger. Meaning that the sizes of the flanges has an influence on the section modulus as it affects the distance to the centre of gravity. However, the main affecting factor is the web height.

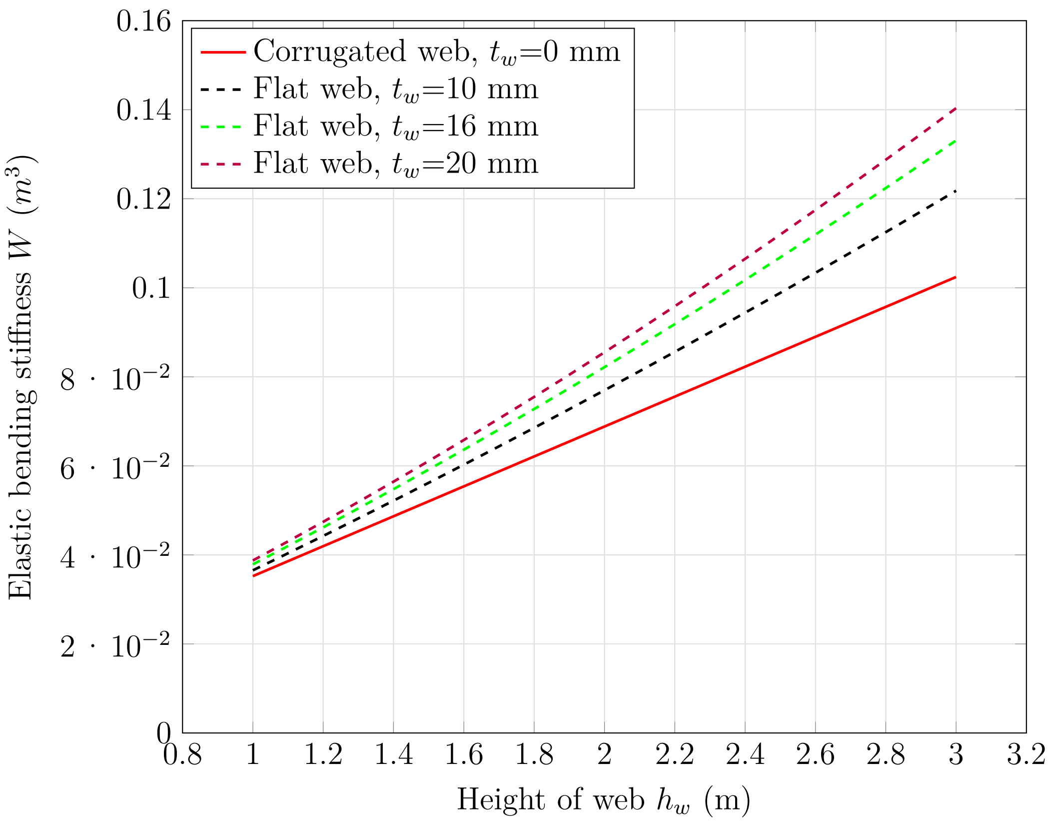


Figure 5.11: The elastic bending stiffness for varying web height, $b_{uf} = 800mm$

If, instead, the width of the flange is studied, the deviation in stiffness for the studied cross-sections changes, as showed in Figure 5.12. In the figure, it can be seen that a change in slope occurs for all girder types. For all girders with flat webs the change of slope is located at a flange width of around 850 millimeters. The change of slope depends on the effective width. For corrugated webs a larger width of 900 millimeters is allowed before the slope is reduced. The deviation in stiffness between girders with flat webs and girders with corrugated webs, for different dimensions, is presented in Table 5.11. The results changes if the shape of the corrugation changes.

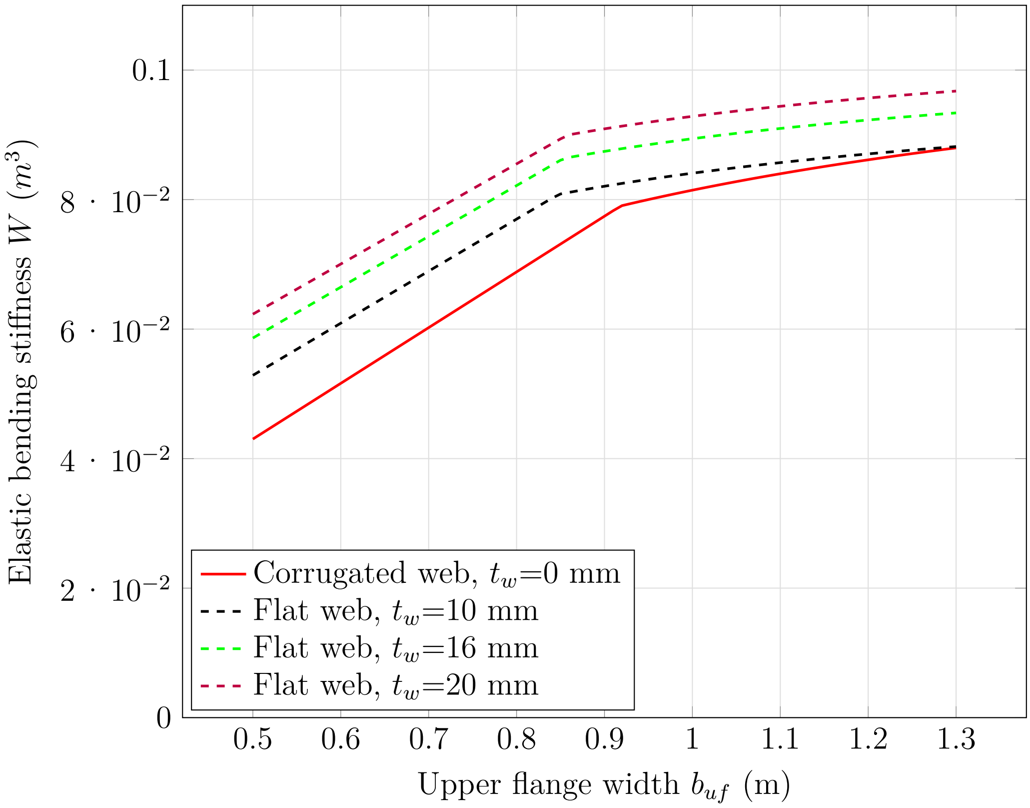


Figure 5.12: The elastic bending stiffness for varying flange width, $h_w = 2000mm$

	η (%)		
	$t_w = 10mm$	$t_w = 16mm$	$t_w = 20mm$
$b_f = 0.7m$, $h_w = 1m$	8%	11%	14%
$b_f = 0.8m$, $h_w = 1m$	6%	10%	12%
$b_f = 0.9m$, $h_w = 1m$	2%	5%	8%
$b_f = 0.7m$, $h_w = 2m$	14%	20%	24%
$b_f = 0.8m$, $h_w = 2m$	12%	18%	21%
$b_f = 0.9m$, $h_w = 2m$	7%	13%	16%
$b_f = 0.7m$, $h_w = 3m$	19%	27%	31%
$b_f = 0.8m$, $h_w = 3m$	17%	24%	28%
$b_f = 0.9m$, $h_w = 3m$	12%	19%	23%

Table 5.11: Deviation in elastic section modulus between girders with flat webs and corrugated webs, $\eta = 1 - \dfrac{W_{el,corr}}{W_{el,flat}}$

5.4.2 Influence on the composite cross-section

The same study that was carried out for the steel section is performed on the composite section. For the composite section, the distance from the centre of gravity to the top flange is greater than the distance to the bottom flange. Therefore, in Equation 5.1, the part below the center of gravity is governing. The examined sections for the composite cross-sections are listed as:

1. Composite cross-section with corrugated web
2. Composite cross-section with flat web, 10 millimeter web
3. Composite cross-section with flat web, 16 millimeter web
4. Composite cross-section with flat web, 20 millimeter web

Similar behavior as for the steel cross-section can be seen for the composite section. The deviation between the cross-sections is smaller for the composite section compared to the steel section. This can be seen in Figure 5.13 and the deviation is listed in Table 5.12.

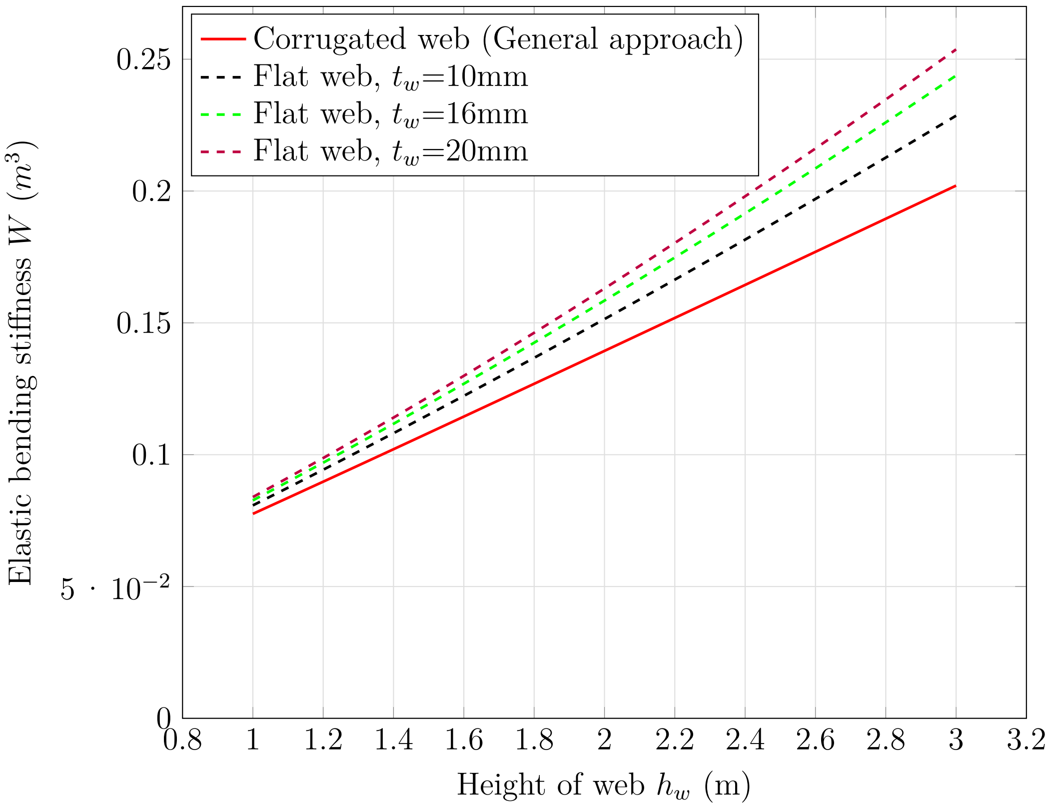


Figure 5.13: The elastic bending stiffness for a varying web height

	η (%)		
	$t_w = 10mm$	$t_w = 16mm$	$t_w = 20mm$
$b_f = 0.8m$, $h_w = 1m$	4%	6%	8%
$b_f = 0.8m$, $h_w = 2m$	8%	12%	15%
$b_f = 0.8m$, $h_w = 3m$	12%	17%	20%

Table 5.12: Relative difference in stiffness between a girder with flat webs and a girder with corrugated webs, $\eta = 1 - \dfrac{W_{el,corr}}{W_{el,flat}}$

5.4.3 Influence on the composite cross-section with an increased web height

High out-of-plane stiffnesses in the web are obtained when corrugated webs are used. These stiffnesses yield possibilities to have very slender and deep girders without needing any thick web plates. For a flat web girder an increased web height demands thicker web plates, and more stiffeners in order to obtain the needed shear buckling capacity, hence, not a suitable solution. Therefore, a study is carried out for a case where a girder with a corrugated web has an increased web height of 25%, compared to three girders with flat webs (thicknesses ranging from 10 to 20 millimeters) where the web height is kept unchanged. In order to illustrate the possibility to have a deeper web for the corrugated web girder, the curve for the elastic section modulus of the corrugated web is translated 25% to the left in Figure5.14.

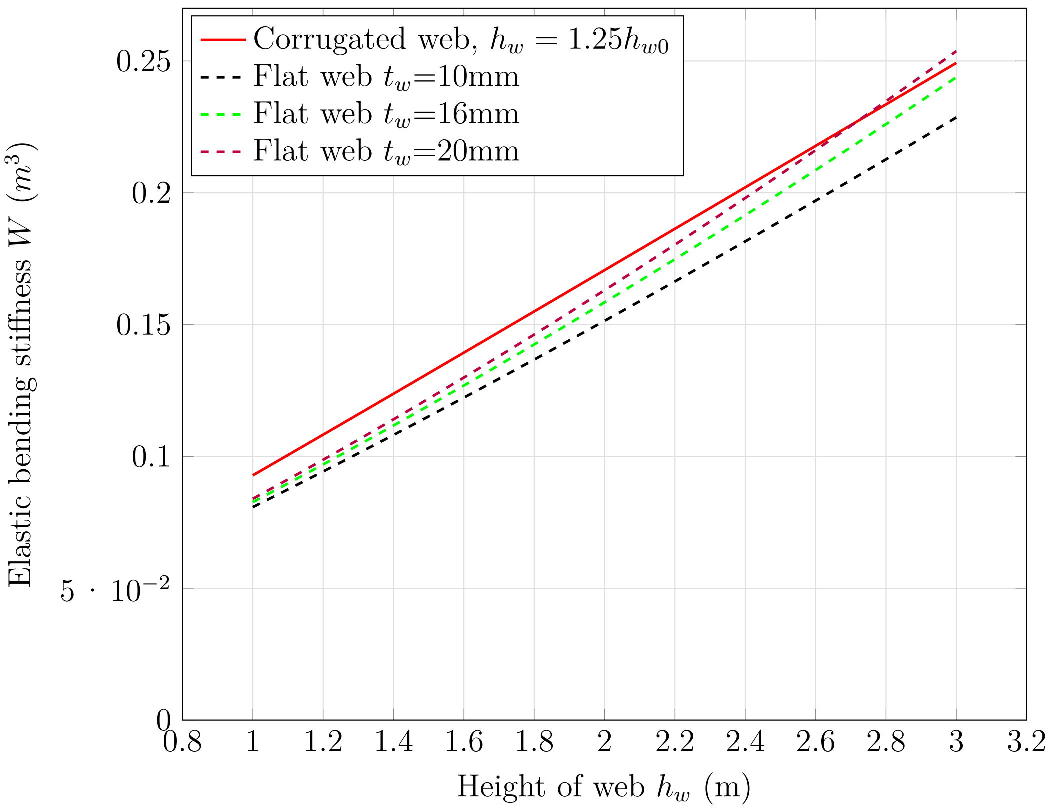


Figure 5.14: The elastic bending stiffness for a varying web height

For an increase of web height of 25% for the corrugated web girder, it obtains comparable or greater bending stiffness than the girders conducted with flat webs. A comparison for the stiffnesses are presented in Table 5.13. Negative values means that the deeper corrugated web girder has a greater bending stiffness.

	η (%)		
	$t_w = 10mm$	$t_w = 16mm$	$t_w = 20mm$
$b_f = 0.8m$, $h_w = 1m$, ($h_{w.cw} = 1.25m$)	-15%	-12%	-11%
$b_f = 0.8m$, $h_w = 2m$, ($h_{w.cw} = 2.5m$)	-13%	-8%	-5%
$b_f = 0.8m$, $h_w = 3m$, ($h_{w.cw} = 3.75m$)	-9%	-2%	2%

Table 5.13: Comparison for the difference in bending stiffness when increasing the web height with 25%, $\eta = 1 - \dfrac{W_{el,corr}}{W_{el,flat}}$

5.5 Influence of web height on the material savings

One of the advantages of using girders with corrugated webs is the increase in shear capacity, since the risk of shear buckling decreases. If the risk of shear buckling decreases the depth of the web can be increased. When the web height is increased the stiffness of the cross-section increases and the flange dimensions could be decreased, as can be seen in Section 5.4. Further, the web thickness can be decreased. With this in mind material savings could be obtained with regards to a deeper web. In most cases the free height beneath the bridge is governing the design. A study is conducted for the case when it does not govern the design to show how much larger the material saving could be, if a deeper web is used.

The study is done for the bridge 100-262-1. The bridge is appropriate to study since the span length is long and it is positioned over water, hence allowing for a deeper web. Six different designs are evaluated for different web heights. The same input data are used and the design is then optimized with regards to the largest material savings.

The angle of the corrugation is kept constant to 36 deg. The corrugation depth, a_3, and the flat-fold length, a_1, is chosen so that the reduction factors for local and global shear buckling are equal. When a good design for the corrugation is achieved the rest of the girder is redesigned, see Table 5.14 and Table 5.15. Utilization ratios are chosen so that they are alike the original design and comparable for different girder heights. The ratios are listed in Table 5.16.

The calculated material savings are presented in Table 5.17 and Figure 5.15, and calculated with regards to the original design (carbon steel girder with flat webs). The results show that for an increased web height, increased material savings are possible to a certain point. Using a prediction for the material savings it could, for the studied bridge, be interpreted that no additional savings can be made after an increase in web height for a factor of 2.1. When comparing the optimal depth of the web to the original design (carbon steel girder with flat webs) a material saving of $\sim 47\%$ is possible. The material savings for different web heights together with the prediction is plotted in Figure 5.15.

Table 5.14: Chosen cross-sectional design for $X = 0m - 10.5m$

	Cross-section dimensions (mm)								
	h	b_{fu}	t_{fu}	h_w	t_w	b_{fl}	t_{fl}	a_1	a_3
1.0	2050	765	35	1967	9	850	48	120	70
1.25*	2560	725	35	2475	7	700	50	124	73
1.375	2820	675	35	2735	7	600	50	135	80
1.5	3075	580	35	2990	7	550	50	153	90
1.75**	3590	600	35	3510	6	600	45	185	110
2.0**	4100	575	30	4025	6	525	45	215	125

*Change of cross-section at $X = 11.5m$
**Change of cross-section at $X = 12.5m$

Table 5.15: Chosen cross-sectional design for $X = 10.5m - 26m$

	Cross-section dimensions (mm)								
	h	b_{fu}	t_{fu}	h_w	t_w	b_{fl}	t_{fl}	a_1	a_3
1.0	2050	830	45	1953	6	1180	52	120	70
1.25*	2560	725	45	2460	5	925	55	124	73
1.375	2820	675	45	2720	5	825	55	135	80
1.5	3075	675	40	2980	5	750	55	153	90
1.75**	3590	700	35	3505	4	700	50	185	110
2.0**	4100	575	35	4015	4	625	50	215	125

*Change of cross-section at $X = 11.5m$
**Change of cross-section at $X = 12.5m$

Table 5.16: Utilization rates for ULS-checks

	Utilization ratios (%)			
	$\eta_{\sigma.u,max}$ (casting)	$\eta_{\sigma.u,max}$ (service life)	$\eta_{\sigma.l,max}$	$\eta_{V,max}$
1.0	71	100	100	93
1.25	69	100	98	98
1.375	69	100	100	96
1.5	70	100	99	87
1.75	65	100	100	97
2.0	76	98	96	90

Table 5.17: Material savings for an increased girder height

Web height increase factor (-)	**Percentage (%)**	**Mass (kg)**
1.0	22.9%	25000 kg
1.25	31.8%	35000 kg
1.375	35.4%	39000 kg
1.5	39.1%	43000 kg
1.75	43.2%	47000 kg
2.0	46.6%	51000 kg

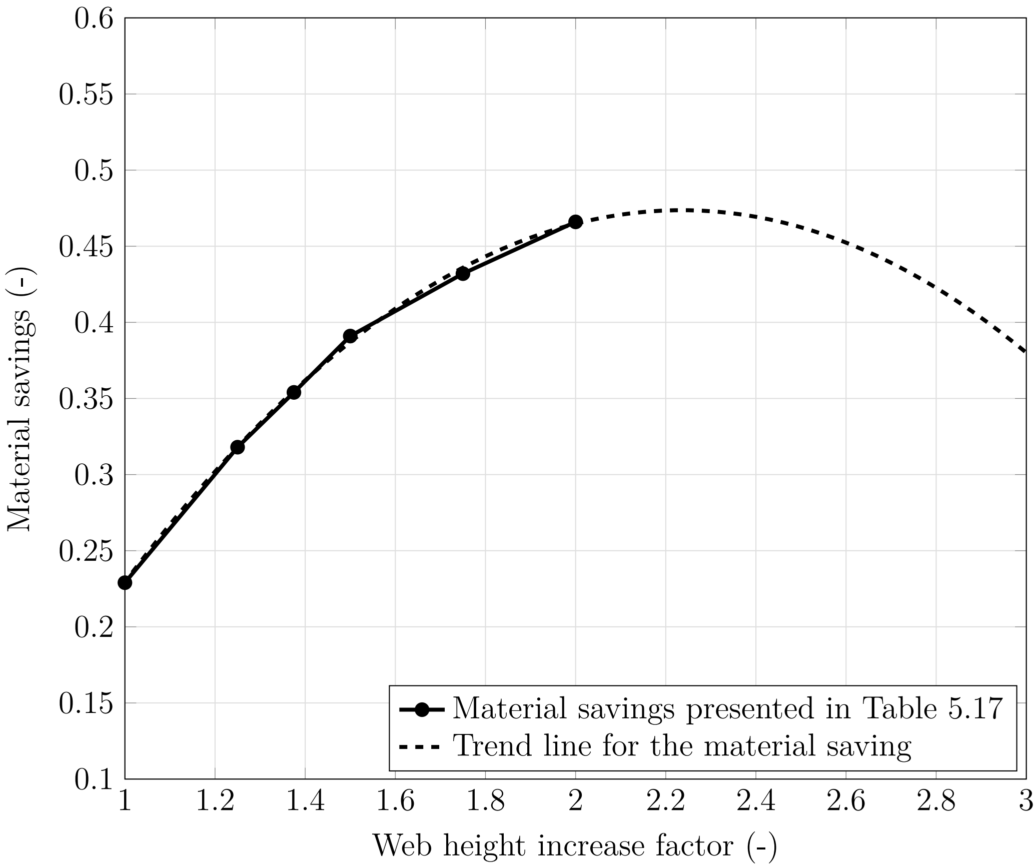


Figure 5.15: Material savings with regards to an increase in web height

6
Discussion

The sixth chapter aims to discuss the results that were obtained for the conducted case and parameter studies. Throughout the chapter the concept of using stainless steel girders with corrugated webs will be discussed and any source of errors will be clarified.

6.1 Case studies

The case studies that have been carried out in Chapter 4 showed that by redesigning existing bridges using the concept of stainless steel girders with corrugated webs, material savings of 20-30% were obtained. The saving mainly comes from the possibility to decrease the web thickness as the risk of shear buckling decreases when using corrugated webs. Both of the studied bridges had relatively deep webs in their original design which then demanded thicker webs in order to avoid buckling. Therefore large material savings could be achieved in the redesign of the structures.

The first bridge that was studied was bridge 100-262-1, whereas the governing check is the ULS-loads, both before and after the redesign. This lead to that the web could be redesigned, making use of the increase in the shear buckling capacity obtained from the use of corrugation. Thereafter, the flanges were optimized with regards to higher yield stress coming from the use of stainless steel (cross-section class three resulted in that no effective width needed to be calculated). The next bridge was 100-379-1, whereas the normal stresses in the lower flange in the fatigue limit state governed, both before and after the redesign. This meant that only an increase in stiffness could decrease the normal stresses, which by extension would mean that the material usage would increase. Since the shear stresses in the fatigue limit state were not governing, the web thickness was made thinner with regards to the ULS-loads hence decreasing the material usage.

The concept of using stainless steel girders with corrugated webs has only been tested on bridges with relatively deep webs. Therefore, it is not anticipated to achieve similar results for girders with less depths. As mentioned the main parameters governing the material savings is the dimensions of the web. It could, therefore, be stated that the possibility to save material is strictly dependent on that either of these parameters being allowed to be adjusted. The minimum allowed plate thickness is four millimeters according to Krav Brobyggande, as mentioned in Section 3.2.1. This means that girders depending on a thinner web thickness for the material saving will not gain as much by using this concept. However, if there are no restrictions on the height of the bridge, deeper webs can be used, leading to that the flanges could be optimized, thus save more material. Discussions on this subject is continued in Section 6.2.5.

For bridge 100-262-1, the compressive stresses caused by temperature variations govern the design in ULS. This results in a stress magnitude of around 30% of the total yield stress in the upper flange. The large temperature stresses results in that the dimensions for the upper flange is slightly larger for the new design using stainless steel compared to the original design using carbon steel. When comparing the utilization ratios for the original and the new design, it seems that the gain in strength descending from using stainless steel is lost in the top flange due to the increased temperature stresses. Meaning that with regards to the design of the upper flange, almost no gain is obtained when using stainless steel girders instead of carbon steel girders.

In the two conducted studies, it could be argued that the majority of the savings come from using corrugated webs rather than stainless steel. For bridge 100-262-1 the flanges are made in carbon steel, grade S420. This means that the material savings that could be gained from a higher yield stress when using stainless steel are smaller compared to a girder in carbon steel, grade S355. The bridge 100-379-1 is designed using steel grade S355 but since it is governed by the fatigue strength, the design strength is independent of the steel grade. However, the saving that has been presented for bridge 100-379-1 depends mainly on the fact that the webs in the original design are made of thick plates.

The cost estimation for bridge 100-262-1, made by *Stål och Rörmontage AB*, shows that the suggested design concept can be economically motivated for the new design. When accounting for the costs of repainting, the net present value of both alternatives clearly shows the benefit of using stainless steel girders with corrugated webs. However, it should be mentioned that this is a simple cost analysis, not accounting for all economical aspects that should be considered if an LCC-analysis were to be conducted. Further economical aspects to consider in order to get a better evaluation of the concepts are traffic conjunctions during service measures, the size of the needed cranes in the construction phase (hence the price of the cranes), and the rate of recycling at the end of the service life. These aspects would, however, be more beneficial for the new design concept since it is maintenance-free in terms of repainting, lighter, and 100% recyclable in the end of its service life (Stålbyggnadsinstitutet, 2017).

6.2 Parametric studies

The parametric studies that have been carried out in Chapter 5 have shown both the advantages and the disadvantages of designing stainless steel girders using corrugated webs. The advantages have been savings in terms of material usage, meanwhile the disadvantages have been large stresses due to temperature loads that might be inevitable for this type of design.

In the parametric studies, no effective cross-section is considered for the flat web girders. More specific the web is not reduced, even if it fulfills the criterion for cross-section class four. This is a *source of error*, which has an impact on the elastic section modulus. Meaning that, in the parametric studies, when comparing flat web girders to corrugated girders, a larger stiffness, for flat web girders, than allowed is accounted for. This affects the studies for the shrinkage and the temperature stresses, shown in Section 5.2 and 5.3. If the effective height of the flat web girders were to be calculated, larger stresses are expected, since the stiffness decreases.

6.2.1 Effective width of the compressed flange

The calculated effective width is different for a girder with a corrugated web compared to a girder with a flat web. The difference mainly depends on the depth of the corrugation. As shown in Section 5.1, the reduction factor, ρ, for corrugated webs are larger for shallow corrugation depths when following the design codes. For increasing corrugations depths the reduction factor for corrugated webs decreases. However, the method suggested by Johanssons et al., (2007) results in constant values for the reduction factor, even if the corrugation gets deeper.

The yield strength of steel is dependent on the size of the plates, more specifically the thicknesses. This is clearly shown in Table 2.3 (stainless steel) and Table 3.9 (carbon steel), whereas smaller plate thicknesses has higher strength parameters. For the flanges in corrugated web girders, larger values for the reduction factor, ρ, are achieved, which is shown in Section 5.1. For a larger reduction factor, thinner flanges can be used, thus, resulting in a higher yield strength for the specific part. This means that a higher material utilization is obtained. However, the increase in yield strength is different dependent on the choice of steel. For stainless steel the limits for the yield stress are changed at thicknesses of 8, 13.5, and 75 millimeters (6.4, 10, and 75 millimeters for steel grade 1.4162). For carbon steel the limits are changed at 16, 40, and 63 millimeters. Therefore, in the conducted case studies, no increased yield strength for the stainless steel girders were achieved as the limits were far from either of the parts thicknesses.

Furthermore, the effective width has an influence on the LT-buckling in the casting phase. For flanges in corrugated web girders, the value for the reduction factor is larger, hence, the calculated effective width is larger. When using the simplified method, presented in Section 3.6.1.1.1, the dimensions of the compressed flange is used for calculating the moment of inertia around the weak axis. Since the moment of inertia around the weak axis is highly dependent on the width of the flange, a small increase in the effective width results in a cubic increase in stiffness. This increase in stiffness has a large influence on the LT-buckling capacity.

Lastly, the equations given by Appendix D in SS-EN 1993-1-5 should be discussed regarding their applicability in bridge design. As shown in Figure 5.1, the reduction factor is not on the safe side for the evaluated cross-sections. Calculations showed that no reduction should be made, even for very wide flanges, as long as corrugated webs are used. In the case studies, this was considered by limiting the buckling coefficient, k_σ, when calculating the reduction factor.

6.2.2 Influence of corrugation on the shrinkage stresses

Shrinkage induces compressive stresses in the upper steel flange. These have a substantial influence on the overall ULS-loads, since they sometimes are larger than the multi-component traffic load (Appendix F). Figure 5.5 shows that the compressive stresses in the upper steel flange are larger for a corrugated web than for a flat web. The difference descends from the negligence of the web for the section using corrugation. When neglecting the web, the sectional area and the bending stiffness decreases, but the sectional forces decrease as well. However, the sectional constants decrease with a higher rate than the sectional forces causing an increase of compressive stresses in the upper flange. Note that the study for the shrinkage stresses is independent of the material, meaning that only the concept of using corrugation influences the stresses.

In Section 5.5, it is shown that for an increased web height, material savings could be made. Therefore, the stresses for a 25% deeper web are calculated. However, Figure 5.5 shows that there is a negligible difference in the stress magnitude if the height is increased. The stresses are kept close to constant because of the sectional forces increasing with the same rate as the sectional constants.

6.2.3 Influence of the material choice and corrugation on the temperature stresses

For bridge 100-262-1, the temperature stresses in the upper flange are the design load case. Temperature is governing since the linear coefficient of thermal expansion is much larger for stainless steel than for carbon steel. Furthermore, the thermal coefficient for composite bridges using carbon steel is recommended, by the design codes, to be taken the same as for concrete, which is even smaller than the coefficient for carbon steel. The coefficient for stainless steel is 60% larger than for concrete. The study, presented in Section 5.3 mainly aims to show the influence of the linear coefficient of thermal expansion, but also to highlight the difference in stress when using corrugated webs instead of flat webs.

Figure 5.6 shows that the compressive stresses in the upper steel flange are much larger when using stainless steel instead of carbon steel. There is a marginal difference for the stresses dependent on if the girder is built with corrugated webs or flat webs. The compressive stresses are caused by the strain difference between the steel and the concrete. Only the strain difference is studied since the bridge is simply supported and, therefore, only the internal differences between parts is of relevance. For a continuous bridge, both the strain difference and the resulting restraining forces would have an impact. The strain difference is dependent on the maximum temperatures in the surrounding air and an additional temperature difference between parts defined by SS-EN 1991-1-5. For girders in carbon steel, only the latter is of relevance, since the concrete and the steel will expand or contract the same, thus, the strain difference will be zero, which cause the stresses for stainless steel to be much larger.

Since the calculations are based on the report of a previously designed bridge, the study follows the same presumptions. However, the method of applying a strict temperature difference between parts could sometimes be rather conservative, as it assumes that there is a distinct temperature difference in the joint between the concrete and the steel. The other method presented in the guidelines assumes a linear temperature difference over the cross-section. No study has been carried out using this method, but further studies should aim to investigate the full effect of temperature differences in composite bridges with stainless steel girders.

In order to see how the stress distribution changes for a deeper web, a study is conducted for an increase of 25% of the girder height. The investigation conducted in Section 5.5 shows that an increase in the web height resulted in substantial savings of material. However, it should be noted that it exists a optimum web height from where no excessive material saving was obtained. The temperature stresses do not seem to be strongly dependent on the depth of the web. In the calculations, it rather looks like the applied forces on the section increase as much as the sectional constants, hence, resulting in only a slight increase of stresses, see Figure 5.7 and Figure 5.8.

The temperature stresses are also dependent on the location of the bridge, therefore, it was investigated if stainless steel structures are more suitable in the northern part of Sweden. Surrounding temperatures in the environment are given by TSFS 2018:57, whereas the maximum temperature variations cause the worst load case for a simply supported composite bridge with stainless steel girders. The highest temperatures are found in the southern part of Sweden, even if it was shown that the relative difference between the south and the north is small. Figure 5.10 compares the largest temperature variations to the smallest temperature variations. For a girder with corrugated webs in stainless steel, the difference in the compressive stresses is 13% (Table 5.10), for the highest and the lowest maximum temperature variations. The difference between these stresses further makes up for 2% of the yield stress for the characteristic loads, meaning that the location of the bridge does not matter as much as the overall use of a temperature-responsive material, thus stainless steel.

6.2.4 Corrugation's influence on the elastic section modulus

The study on the corrugation's influence on the elastic section modulus showed that the stiffnesses varied as expected. When calculating the stiffnesses for a girder with corrugated webs, the webs are neglected, and therefore the bending stiffness is smaller compared to a flat web. However, one of the main advantages of using corrugated webs over flat webs is the possibility to increase the web height without increasing the risk of buckling. Therefore, the study showed that for an increased web height of 25% for the girder with corrugated webs, the elastic section modulus was comparable or larger than the girder with flat webs. Note that the height of the flat web was not increased as it is limited by the risk of buckling. This means that the loss of bending stiffness when neglecting the web could be regained by increasing the web height for a girder using corrugated webs.

6.2.5 Girder height's influence on the material savings

In the first sections of this chapter, it has been discussed that there are positive effects if the web height is increased when using the concept of stainless steel girders with corrugated webs. The elastic bending stiffness increases for a deeper web, and the temperature stresses, which are the design load case for bridge 100-262-1, are kept close to constant for an increased height. Therefore, the material savings with regards to an increased depth of the web is studied. The savings that could be achieved by increasing the height of the girder spans from approximately 25% to 50% for bridge 100-262-1, see Section 5.5.

The study showed that material savings could be achieved for girder heights up to a factor of 2.1 in comparison to the original design. However, in most cases, it is not reasonable to have such deep webs, and therefore material saving of 50% is not possible in practice. Neither does any supplier have that large plates in their standard assortment, meaning that high costs might overcome the saving itself. Nevertheless, it is worth stressing that the study showed that for every linear increase in the depth of the web resulted in a quadratic saving with regards to material usage, which means that as long as the free height beneath the bridge is not compromised the objective should always be to maximize the depth of the web.

During the design of the six girders, the same utilization ratios were aimed so that the results were comparable, and a trend line could be obtained. When using standard dimensions, this is not always possible since a change in a single millimeter in the web thickness could result in a 20% larger or smaller capacity for thin web thicknesses ($\sim 4-6mm$). In order to be able to decrease the web thickness wherever needed, the section for where the web changes thickness was displaced. By moving the section, a larger material saving could be obtained.

The design of the girders starts with a redesign of the corrugation. The corrugation is chosen so that one segment length, s, is as short as possible, leading to less material usage. The angle of the corrugation, α, and the depth of the corrugation, a_3, are the two governing parameters controlling the segment length. For a fixed angle, the depth of the corrugation should be chosen as small as possible. In Table 5.14 and Table 5.15, it is shown that the depth needs to be increased for an increasing web height.

The study that has been done for the material savings is very much bridge specific. The correlation between the girder height and the material savings is dependent on the overall geometry, the applied loads, but most important the governing checks. However, it shall still be noted that the savings will behave quadratic for any bridge when increasing the depth of the web. Dependent on the governing design check, different material savings are expected for different bridges.

7
Conclusions

The following chapter will conclude and respond to the aims and objectives that were set up at the beginning of this thesis.

The aim of this thesis has been to study the *applicability* and *effectiveness* of using the concept of stainless steel girders with corrugated webs when designing steel-concrete composite bridges. The effectiveness was studied with regard to any material savings that could be achieved by applying the concept. Further, the objective was to identify the differences that most influences the design.

Applicability, during the design of a composite bridge with stainless steel girders and corrugated webs, some struggles arose as the design procedure was formulated. First, it was noticed that the formulation of calculating the effective width of the steel flanges was not applicable for the design of corrugated webs. Therefore, before applying the method formulated by the Eurocodes, further studies should aim to investigate the behaviour of the girder during compressive loading. Next, the compressive stresses in the upper steel flange arising from strain differences due to temperature variations should be more thoroughly studied as these are one of the governing load cases for both of the studied bridges. For instance, a finite element model should be set up for the girder, studying the influence on the stresses dependent on how the temperature strains are implemented.

Effectiveness, the concept of using stainless steel girders with corrugated webs reduces the material usage of composite bridges compared to if a traditional flat web design in carbon steel is used. Material savings between 20-30% can be expected if the girder height is kept unchanged. The material savings are, however, highly dependent on the bridge height. Without any restrictions on the free height beneath the bridge, the material savings approaches 50% compared to the original design. It is concluded that a linear increase in the depth of the web results in a considerable saving of material up to a certain point. The effectiveness, from an economical aspect, of the suggested concept is in the case studies, showed to be competitive compared to the investment cost of the original concept. The cost estimation, including the net present value, showed an economical saving of 27% (representing 1 624 000 SEK), for the redesigned bridge, bridge 100-262-1. Thus, it is expected that further economical savings are expected if the estimation were to be carried out for the deeper girders.

In conclusion it should be stressed that the concept of using stainless steel girders with corrugated webs has good prospects to be implemented in the industry today.

References

Boutillion, L., Combault, J., Ikeda, S., Imberty, F., Mori, T., Nagamoto, N., … Saito, K. (2015). *Corrugated-steel-web bridges*. Fédération internationale du béton.

Collin, P., Johansson, B., & Sundquist, H. (2008). *Steel Concrete Composite Bridges*. Stockholm: Royal Institute of Technology and Luleå Technical University.

Dahlström, S. E., & Persson, J. (2018). Implementation of Stainless Steel Reinforcement in Concrete Bridges Redesign of Reinforcement using Stainless Steel to Increase Durability and Profitability in Bridge Design .

European Commission. (no date). *SS-EN 1993-1-8 Design of steel structures, Part 1-8 Design of joints*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2003). *SS-EN 1991-2 Actions on structures, Part 2 Traffic loads on bridges*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2005a). *SS-EN 1993-1-1 Design of steel structures, Part 1-1 General rules*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2005b). *SS-EN 1993-1-9 Design of steel structures, Part 1-9 Fatigue*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006a). *SS-EN 1991-1-1 Actions on structures, Part 1-1 General actions*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006b). *SS-EN 1991-1-4 Actions on structures, Part 1-4 Wind actions*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006c). *SS-EN 1991-1-5 Actions on structures, Part 1-5 Thermal actions*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006d). *SS-EN 1992-1-1 Design of concrete structures, Part 1-1 General rules*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006e). *SS-EN 1992-2 Design of concrete structures, Part 2 Concrete bridges*. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006f). *SS-EN 1993-1-4 Design of steel structures, Part 1-4 Stainless steels*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006g). *SS-EN 1993-1-5 Design of steel structures, Part 1-5 Plated strucural elements*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006h). *SS-EN 1993-2 Design of steel structures, Part 2 Steel bridges*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006i). *SS-EN 1994-1-1 Design of composite steel and concrete structures, Part 1-1 General rules*. Joint research center. Retrieved from https://eurocodes.jrc.ec.europa.eu

European Commission. (2006j). *SS-EN 1994-2 Design of composite steel and concrete structures, Part 2 General rules for bridges*. Joint research center.

Johansson, B., Maquoi, R., Sedlacek, G., Müller, C., & Beg, D. (2007). Commentary and Worked Examples To En 1993-1-5 " Plated Structural Elements ". *October*, *3*(October).

Karlsson, E. (2018). Stainless Steel Bridge Girders with Corrugated Webs Efficiency , stability and life-cycle cost analysis.

Pipianto, A., & De Miranda, M. (2016). *Innovative Bridge Design Handbook*. Oxford: Butterworth Heinemann.

Stålbyggnadsinstitutet. (2017). *Dimensionering av konstruktioner i rostfritt stål* (4th edition). Stockholm.

Trafikverket. (2016a). *Krav brobyggande*.

Trafikverket. (2016b). *Råd brobyggande*.

Transportstyrelsen. (2018). *Transportstyrelsens föreskrifter och allmäna råd om tillämpning av eurokoder*.

Utsi, S., & Lagerqvist, O. (2012). *Samverkanskonstruktioner stål - betong* (1st editio). Stockholm: Stålbyggnadsistitutet.

Vayas, I., & Iliopoulos, A. (2014). *Design of Steel-Concrete Composite bridges to Eurocodes*. Boca ranton: CRC Press.

A

Load combinations

A.1 Ultimate limit state

Table A.1: Load combinations, ultimate limit state - (EQU) 6.10

Permanent loads	sup	inf	Max	Min	Reference
Self-weight	1.0	1.0	$\gamma_d \cdot 1.1 \cdot sup$	$0.9 \cdot inf$	SS-EN 1990 4.1.2 (5)
Asphalt cover	1.1	0.9	$\gamma_d \cdot 1.1 \cdot sup$	$0.9 \cdot inf$	SS-EN 1991-1-1 5.2.3 (3) , TSFS 2018:57 5 ch. 3§
Shrinkage	1.0	1.0	$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (3), SS-EN 1992-1-1 2.4.2.1 (1)
Variable loads	Ψ_0		Main	Other	Reference
Traffic	0.75		$\gamma_d \cdot 1.5$	$\gamma_d \cdot 1.5 \cdot \Psi_0$	TSFS 2018:57 4 ch. 8 § Table 4.2
Acceleration	0.75		$\gamma_d \cdot 1.5 \cdot 0.6$	$\gamma_d \cdot 1.5 \cdot 0.6 \cdot \Psi_0$	SS-EN 1990:2002 Table A2.1
Temperature	0.6		$\gamma_d \cdot 1.5$	$\gamma_d \cdot 1.5 \cdot \Psi_0$	SS-EN 1990:2002 Table A2.1

Table A.2: Load combinations, ultimate limit state - (STR/GEO) 6.10a

Permanent loads	sup	inf	Max	Min	Reference
Self-weight	1.0	1.0	$\gamma_d \cdot 1.35 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (5)
Asphalt cover	1.1	0.9	$\gamma_d \cdot 1.35 \cdot sup$	$1.0 \cdot inf$	SS-EN 1991-1-1 5.2.3 (3) , TSFS 2018:57 5 ch. 3§
Shrinkage	1.0	1.0	$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (3), SS-EN 1992-1-1 2.4.2.1 (1)
Variable loads	Ψ_0		Main	Other	Reference
Traffic	0.75		$\gamma_d \cdot 1.5 \cdot \Psi_0$	$\gamma_d \cdot 1.5 \cdot \Psi_0$	TSFS 2018:57 4 ch. 8 § Table 4.2
Acceleration	0.75		$\gamma_d \cdot 1.5 \cdot 0.6\Psi_0$	$\gamma_d \cdot 1.5 \cdot 0.6 \cdot \Psi_0$	SS-EN 1990:2002 Table A2.1
Temperature	0.6		$\gamma_d \cdot 1.5 \cdot \Psi_0$	$\gamma_d \cdot 1.5 \cdot \Psi_0$	SS-EN 1990:2002 Table A2.1

Table A.3: Load combinations, ultimate limit state - (STR/GEO) 6.10b

Permanent loads	sup	inf	Max	Min	Reference
Self-weight	1.0	1.0	$\gamma_d \cdot 0.89 \cdot 1.35 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (5)
Asphalt cover	1.1	0.9	$\gamma_d \cdot 0.89 \cdot 1.35 \cdot sup$	$1.0 \cdot inf$	SS-EN 1991-1-1 5.2.3 (3) , TSFS 2018:57 5 ch. 3§
Shrinkage	1.0	1.0	$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (3), SS-EN 1992-1-1 2.4.2.1 (1)
Variable loads	Ψ_0		Main	Other	Reference
Traffic	0.75		$\gamma_d \cdot 1.5 \cdot$	$\gamma_d \cdot 1.5 \cdot \Psi_0$	TSFS 2018:57 4 ch. 8 § Table 4.2
Acceleration	0.75		$\gamma_d \cdot 1.5 \cdot 0.6$	$\gamma_d \cdot 1.5 \cdot 0.6 \cdot \Psi_0$	SS-EN 1990:2002 Table A2.1
Temperature	0.6		$\gamma_d \cdot 1.5 \cdot$	$\gamma_d \cdot 1.5 \cdot \Psi_0$	SS-EN 1990:2002 Table A2.1

A.2 Serviceability limit state

Table A.4: Load combinations, serviceability limit state - Frequent - 6.15b

Permanent loads	sup	inf		Max	Min	Reference
Self-weight	1.0	1.0		$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (5)
Asphalt cover	1.1	0.9		$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1991-1-1 5.2.3 (3) , TSFS 2018:57 5 ch. 3§
Shrinkage	1.0	1.0		$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (3), SS-EN 1992-1-1 2.4.2.1 (1)
Variable loads	Ψ_0	Ψ_1	Ψ_2	Main	Other	Reference
Traffic	0.75	0.75	0	Ψ_1	Ψ_2	TSFS 2018:57 4 ch. 8 § Table 4.2
Acceleration	0.75	0.75	0	-	-	SS-EN 1990:2002 Table A2.1
Temperature	0.6	0.6	0.5	Ψ_1	Ψ_2	SS-EN 1990:2002 Table A2.1

Table A.5: Load combinations, serviceability limit state - Quasi permanent - 6.16b

Permanent loads	sup	inf		Max	Min	Reference
Self-weight	1.0	1.0		$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (5)
Asphalt cover	1.1	0.9		$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1991-1-1 5.2.3 (3) , TSFS 2018:57 5 ch. 3§
Shrinkage	1.0	1.0		$1.0 \cdot sup$	$1.0 \cdot inf$	SS-EN 1990 4.1.2 (3), SS-EN 1992-1-1 2.4.2.1 (1)
Variable loads	Ψ_0	Ψ_1	Ψ_2	Main	Other	Reference
Traffic	0.75	0.75	0	Ψ_2	Ψ_2	TSFS 2018:57 4 ch. 8 § Table 4.2
Acceleration	0.75	0.75	0	Ψ_2	Ψ_2	SS-EN 1990:2002 Table A2.1
Temperature	0.6	0.6	0.5	Ψ_2	Ψ_2	SS-EN 1990:2002 Table A2.1

B

Corrugation parameters influence on the effective width of steel flanges

Appendix E Influence of corrugation on elastic bending stiffness

E.1 Geometry

$t_{uf} := 42\ mm$ Thickness of upper flange

$b_{uf} := 800\ mm$ Width of upper flange

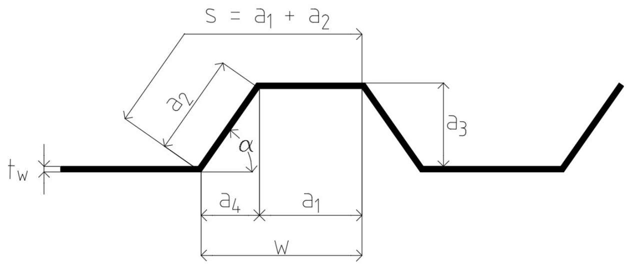
$t_{lf} := 55\ mm$ Thickness of lower flange

$b_{lf} := 1150\ mm$ Width of lower flange

$t_w := \begin{bmatrix} 10\ mm \\ 16\ mm \\ 20\ mm \end{bmatrix}$ Thickness of web

$a_{weld} := 4\ mm$ Weldthroat thickness

E.1.1 Corrugation shape



$a_{c1} := 110\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 75\ mm$ Corrugation depth

$a_{c2} := \dfrac{a_{c3}}{sin\left(\alpha_c\right)} = 128\ mm$ legnth of hypopythis

$a_{c4} := \dfrac{a_{c3}}{tan\left(\alpha_c\right)} = 103\ mm$ Length of angled part

$s_c := a_{c1} + a_{c2} = 238\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 213\ mm$ Straight length

$\dfrac{s_c}{w_c} = 1.11$ Ratio corrugation/straight length

$$c_{uf}\left(t_w, b_{uf}\right) := \frac{b_{uf}}{2} + \frac{a_{c3}}{2} - \frac{t_w}{2} - \sqrt{2} \cdot a_{weld}$$

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

E.2 Corrugated web stiffness

E.2.1 Plate buckling of compressive flange corrugated - SS-EN 1993 1-4 5.2.3

$$h_{beam}\left(h_w\right) := h_w + t_{uf} + t_{lf}$$

$$a_{bend} := a_{c1} + 2\ a_{c4}$$

SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$k_{\sigma u1}\left(t_w, b_{uf}\right) := 0.43 + \left(\frac{c_{uf}\left(t_w, b_{uf}\right)}{a_{bend}}\right)^2$$

SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$k_{\sigma u2} := 0.6$$

SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$k_{\sigma u}\left(t_w, b_{uf}\right) := min\left(k_{\sigma u1}\left(t_w, b_{uf}\right), k_{\sigma u2}\right)$$

SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$\lambda_{pu}\left(t_w, b_{uf}\right) := \frac{\dfrac{c_{uf}\left(t_w, b_{uf}\right)}{t_{uf}}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_{\sigma u}\left(t_w, b_{uf}\right)}}$$

Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho_u\left(t_w, b_{uf}\right) := \begin{cases} 1 & \text{if } \lambda_{pu}\left(t_w, b_{uf}\right) \le 0.748 \\ min\left(\dfrac{\lambda_{pu}\left(t_w, b_{uf}\right) - 0.188}{\lambda_{pu}\left(t_w, b_{uf}\right)^2}, 1\right) & \text{else} \end{cases}$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

$$b_{effu\_corr}\left(t_w, b_{uf}\right) := \rho_u\left(t_w, b_{uf}\right) \cdot b_{uf}$$

SS-EN 1993-1-5 Table 4.1

E.2.2 Stiffness steel beam

$$z_g(h_w, t_w, b_{uf}) := \overrightarrow{\frac{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}(h_w) - \frac{t_{lf}}{2}\right)}{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} + b_{lf} \cdot t_{lf}}}$$

$$I_y(h_w, t_w, b_{uf}) := \overrightarrow{\frac{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf}^3}{12} + b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2} - z_g(h_w, t_w, b_{uf})\right)^2 + \frac{b_{lf} \cdot t_{lf}^3}{12} + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}(h_w) - \frac{t_{lf}}{2} - z_g(h_w, t_w, b_{uf})\right)^2}$$

$$W_{gu\_corr}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{I_y(h_w, t_w, b_{uf})}{z_g(h_w, t_w, b_{uf})}}$$

$$W_{gu\_corr2}(h_w, t_w, b_{uf}) := \overrightarrow{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} \cdot \left(h_w + \frac{t_{uf} + t_{lf}}{2}\right)}$$

Appendix D calculation

$$W_{gl\_corr}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{I_y(h_w, t_w, b_{uf})}{h_{beam}(h_w) - z_g(h_w, t_w, b_{uf})}}$$

$$W_{gl\_corr2}(h_w) := \overrightarrow{b_{lf} \cdot t_{lf} \cdot \left(h_w + \frac{t_{uf} + t_{lf}}{2}\right)}$$

E.2.3 Stiffness composite section

$$A_s(t_w, b_{uf}) := \overrightarrow{t_{uf} \cdot b_{effu\_corr}(t_w, b_{uf}) + t_{lf} \cdot b_{lf}}$$

$$A_{slab.fic} := 1.5\ m^2$$

$$h_{m.slab} := 0.32\ m$$
$$b_{eff\_mid} := 4.85\ m$$

$$A_{sl\_short}(t_w, b_{uf}) := \overrightarrow{A_s(t_w, b_{uf}) + \frac{A_{slab.fic}}{n_{L_2}}}$$

Area of composite section for shortterm loads

$$z_{tp\_short}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{\frac{A_{slab.fic}}{n_{L_2}} \cdot \frac{h_{m.slab}}{2} + t_{uf} \cdot b_{effu\_corr}(t_w, b_{uf}) \cdot \left(h_{m.slab} + \frac{t_{uf}}{2}\right) + t_{lf} \cdot b_{lf} \cdot \left(h_{m.slab} + t_{uf} + h_w + \frac{t_{lf}}{2}\right)}{\frac{A_{slab.fic}}{n_{L_2}} + t_{uf} \cdot b_{effu\_corr}(t_w, b_{uf}) + t_{lf} \cdot b_{lf}}}$$

Centre of gravity from the top

$$I_{y\_short}(h_w, t_w, b_{uf}) := \overrightarrow{I_y(h_w, t_w, b_{uf}) + A_s(t_w, b_{uf}) \cdot \left(\left(z_g(h_w, t_w, b_{uf}) + h_{m.slab}\right) - z_{tp\_short}(h_w, t_w, b_{uf})\right)^2 + \frac{h_{m.slab}^{\ 3} \cdot \frac{b_{eff\_mid}}{n_{L_2}}}{12} + \frac{A_{slab.fic}}{n_{L_2}} \cdot \left(\frac{h_{m.slab}}{2} - z_{tp\_short}(h_w, t_w, b_{uf})\right)^2}$$

$$W_{comp.corr.u}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{I_{y\_short}(h_w, t_w, b_{uf})}{z_{tp\_short}(h_w, t_w, b_{uf})}}$$

$$W_{comp.corr.l}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{I_{y\_short}(h_w, t_w, b_{uf})}{\left(h_{beam}(h_w) + h_{m.slab}\right) - z_{tp\_short}(h_w, t_w, b_{uf})}}$$

E.3 Flat web stiffness

E.3.1 Plate buckling of compressive flange flat we - SS-EN 1993 1-4 5.2.3

$$c_{uf}(t_w, b_{uf}) := \frac{b_{uf}}{2} - \frac{t_w}{2} - \sqrt{2} \cdot a_{weld}$$

$$k_{\sigma u} := 0.43$$

$$\lambda_{pu}(t_w, b_{uf}) := \frac{\frac{c_{uf}(t_w, b_{uf})}{t_{uf}}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_{\sigma u}}}$$

$$\rho_u(t_w, b_{uf}) := \begin{array}{l} \text{if } \lambda_{pu}(t_w, b_{uf}) \le 0.748 \\ \quad \| \ 1 \\ \text{else} \\ \quad \| \ min\left(\frac{\lambda_{pu}(t_w, b_{uf}) - 0.188}{\lambda_{pu}(t_w, b_{uf})^2}, 1\right) \end{array}$$

$$b_{effu}(t_w, b_{uf}) := \rho_u(t_w, b_{uf}) \cdot b_{uf}$$

E.3.2 Stiffness steel beam flat web

$$z_{g\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + t_w \cdot h_w \cdot \left(h_{beam}\left(h_w\right) - t_{lf} - \frac{h_w}{2}\right) + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}\left(h_w\right) - \frac{t_{lf}}{2}\right)}{b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf} + b_{lf} \cdot t_{lf} + t_w \cdot h_w}}$$

$$\begin{aligned} I_{y\_flat}\left(h_w, t_w, b_{uf}\right) := &\ \overrightarrow{\frac{b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf}^{3}}{12} + b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2} - z_{g\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2} \\ & + \frac{t_w \cdot h_w^{3}}{12} + t_w \cdot h_w \cdot \left(z_{g\_flat}\left(h_w, t_w, b_{uf}\right) - t_{uf} - \frac{h_w}{2}\right)^2 \\ & + \frac{b_{lf} \cdot t_{lf}^{3}}{12} + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}\left(h_w\right) - \frac{t_{lf}}{2} - z_{g\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2 \end{aligned}$$

$$W_{gu\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_flat}\left(h_w, t_w, b_{uf}\right)}{z_{g\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

$$W_{gl\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_flat}\left(h_w, t_w, b_{uf}\right)}{h_{beam}\left(h_w\right) - z_{g\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

E.3.3 Stiffness composite section

$$A_{s\_flat}\left(t_w, b_{uf}, h_w\right) := \overrightarrow{t_{uf} \cdot b_{effu}\left(t_w, b_{uf}\right) + t_{lf} \cdot b_{lf} + t_w \cdot h_w}$$

$$A_{sl\_short\_flat}\left(t_w, b_{uf}, h_w\right) := \overrightarrow{A_{s\_flat}\left(t_w, b_{uf}, h_w\right) + \frac{A_{slab.fic}}{n_{L_2}}}$$

Area of composite section for shortterm loads

$$z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{\frac{A_{slab.fic}}{n_{L_2}} \cdot \frac{h_{m.slab}}{2} + t_{uf} \cdot b_{effu}\left(t_w, b_{uf}\right) \cdot \left(h_{m.slab} + \frac{t_{uf}}{2}\right) + t_{lf} \cdot b_{lf} \cdot \left(h_{m.slab} + t_{uf} + h_w + \frac{t_{lf}}{2}\right) + h_w \cdot t_w \cdot \left(t_{uf} + \frac{h_w}{2}\right)}{\frac{A_{slab.fic}}{n_{L_2}} + t_{uf} \cdot b_{effu}\left(t_w, b_{uf}\right) + t_{lf} \cdot b_{lf} + t_w \cdot h_w}}$$

Centre of gravity from the top

$$\begin{aligned} I_{y\_short\_flat}\left(h_w, t_w, b_{uf}\right) := &\ \overrightarrow{I_{y\_flat}\left(h_w, t_w, b_{uf}\right)} \\ & + A_{s\_flat}\left(t_w, b_{uf}, h_w\right) \cdot \left(\left(z_{g\_flat}\left(h_w, t_w, b_{uf}\right) + h_{m.slab}\right) - z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2 \\ & + \frac{h_{m.slab}^{3} \cdot \frac{b_{eff\_mid}}{n_{L_2}}}{12} + \frac{A_{slab.fic}}{n_{L_2}} \cdot \left(\frac{h_{m.slab}}{2} - z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2 \end{aligned}$$

$$W_{comp.flat.u}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_short\_flat}\left(h_w, t_w, b_{uf}\right)}{z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

$$W_{comp.flat.l}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_short\_flat}\left(h_w, t_w, b_{uf}\right)}{\left(h_{beam}\left(h_w\right) + h_{m.slab}\right) - z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

$$W_{comp.flat}\left(t_w, b_{uf}\right) := \left\| \begin{array}{l} \text{for } i \in 1 \,..\, length\left(h_w\right) \\ \quad \left\| \, out_i \leftarrow min\left(W_{comp.flat.l}\left(h_{w_i}, t_w, b_{uf}\right), W_{comp.flat.u}\left(h_{w_i}, t_w, b_{uf}\right)\right) \right. \\ out \end{array} \right.$$

$$W_{comp.flat}\left(x, t_w, b_{uf}\right) := linterp\left(h_w, W_{comp.flat}\left(t_w, b_{uf}\right), x\right)$$

E.4 Comparisson Steel cross-section

$$h_w := \begin{Vmatrix} \text{for } i \in 1..21 \\ \quad \Vert \; out_i \leftarrow 1\ m + 0.1\ m \cdot (i-1) \\ out \end{Vmatrix} \qquad b_{uf.plot} := \begin{Vmatrix} \text{for } i \in 1..81 \\ \quad \Vert \; out_i \leftarrow 0.5\ m + 0.01\ m \cdot (i-1) \\ out \end{Vmatrix}$$

W_{gu_corr} Elastic bending stiffness corrugated web- W=I/z

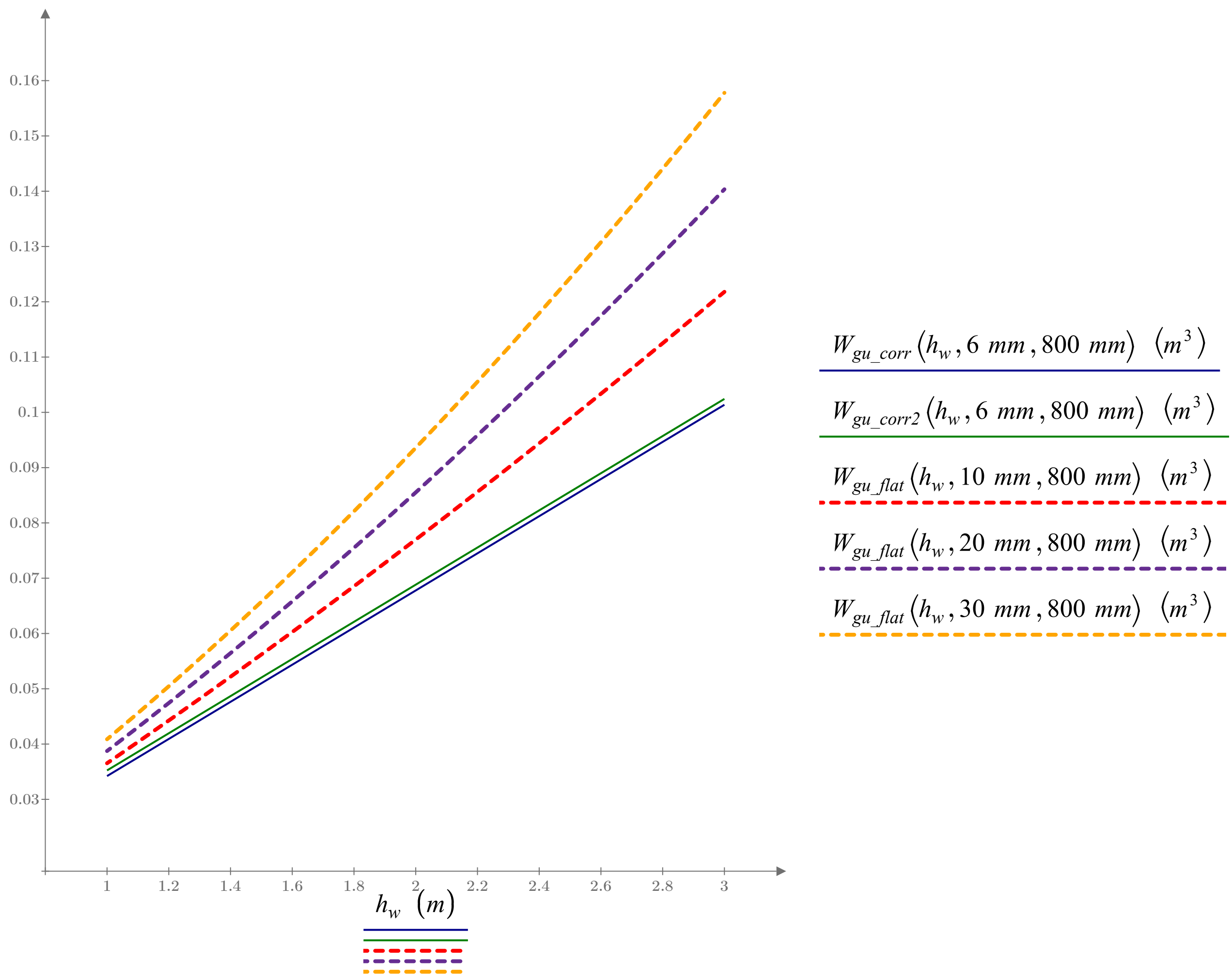
W_{gu_corr2} Elastic bending stiffness corrugated web- SS-EN 1993-1-5 Appendix D calculation

W_{gu_flat} Elastic bending stiffness flat web- W=I/z

\- - - - - - = Flat Web

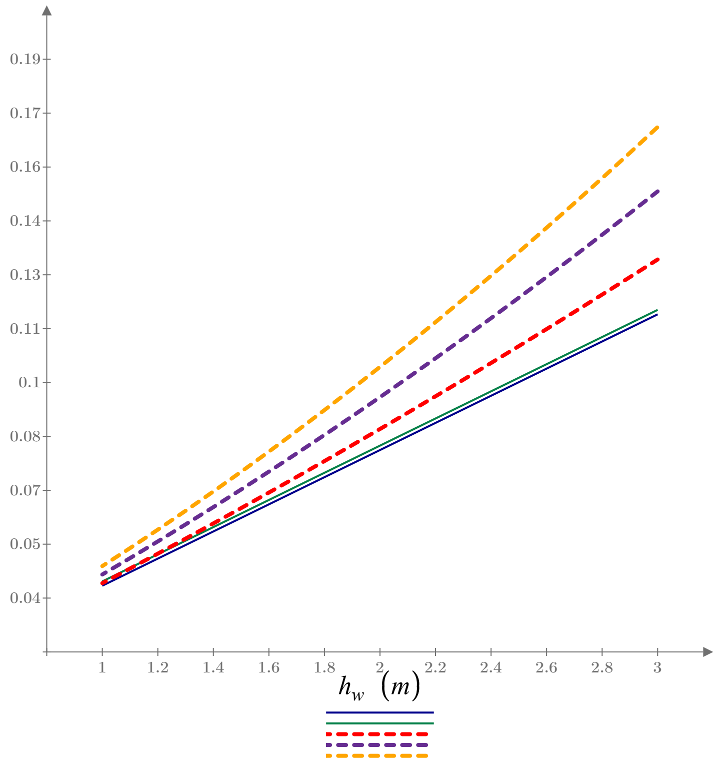
________ = Corrugated web

Flange width 800mm



12/05/2020

Flange width 900mm



$W_{gu\_corr}(h_w, 6\ mm, 900\ mm)\ (m^3)$

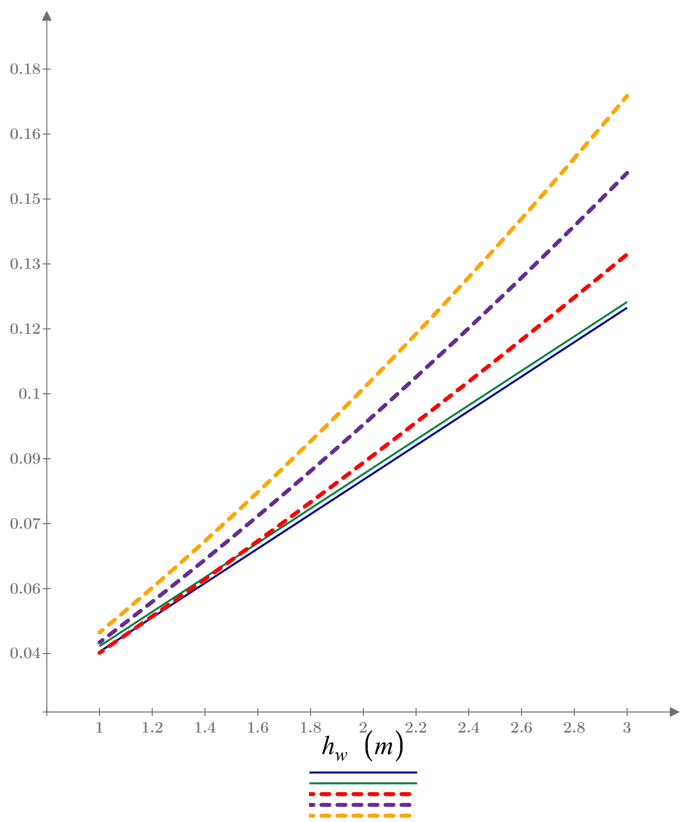
$W_{gu\_corr2}(h_w, 6\ mm, 900\ mm)\ (m^3)$

$W_{gu\_flat}(h_w, 10\ mm, 900\ mm)\ (m^3)$

$W_{gu\_flat}(h_w, 20\ mm, 900\ mm)\ (m^3)$

$W_{gu\_flat}(h_w, 30\ mm, 900\ mm)\ (m^3)$

Flange width 1000mm



$W_{gu\_corr}(h_w, 6\ mm, 1000\ mm)\ (m^3)$

$W_{gu\_corr2}(h_w, 6\ mm, 1000\ mm)\ (m^3)$

$W_{gu\_flat}(h_w, 10\ mm, 1000\ mm)\ (m^3)$

$W_{gu\_flat}(h_w, 20\ mm, 1000\ mm)\ (m^3)$

$W_{gu\_flat}(h_w, 30\ mm, 1000\ mm)\ (m^3)$

12/05/2020

E.4.1 Comparisson Steel cross-section, deviation

Web thickness 10mm

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(1\ m, 10\ mm, 700\ mm)} = 8\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(1\ m, 10\ mm, 800\ mm)} = 6\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(1\ m, 10\ mm, 900\ mm)} = 2\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(2\ m, 10\ mm, 700\ mm)} = 14\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(2\ m, 10\ mm, 800\ mm)} = 12\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(2\ m, 10\ mm, 900\ mm)} = 7\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(3\ m, 10\ mm, 700\ mm)} = 19\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(3\ m, 10\ mm, 800\ mm)} = 17\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(3\ m, 10\ mm, 900\ mm)} = 12\%$$

Web thickness 16mm

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(1\ m, 16\ mm, 700\ mm)} = 11\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(1\ m, 16\ mm, 800\ mm)} = 10\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(1\ m, 16\ mm, 900\ mm)} = 5\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(2\ m, 16\ mm, 700\ mm)} = 20\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(2\ m, 16\ mm, 800\ mm)} = 18\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(2\ m, 16\ mm, 900\ mm)} = 13\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(3\ m, 16\ mm, 700\ mm)} = 27\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(3\ m, 16\ mm, 800\ mm)} = 24\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(3\ m, 16\ mm, 900\ mm)} = 19\%$$

--Web thickness 20mm--

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(1\ m, 20\ mm, 700\ mm)} = 14\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(1\ m, 20\ mm, 800\ mm)} = 12\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(1\ m, 20\ mm, 900\ mm)} = 8\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(2\ m, 20\ mm, 700\ mm)} = 24\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(2\ m, 20\ mm, 800\ mm)} = 21\%$$

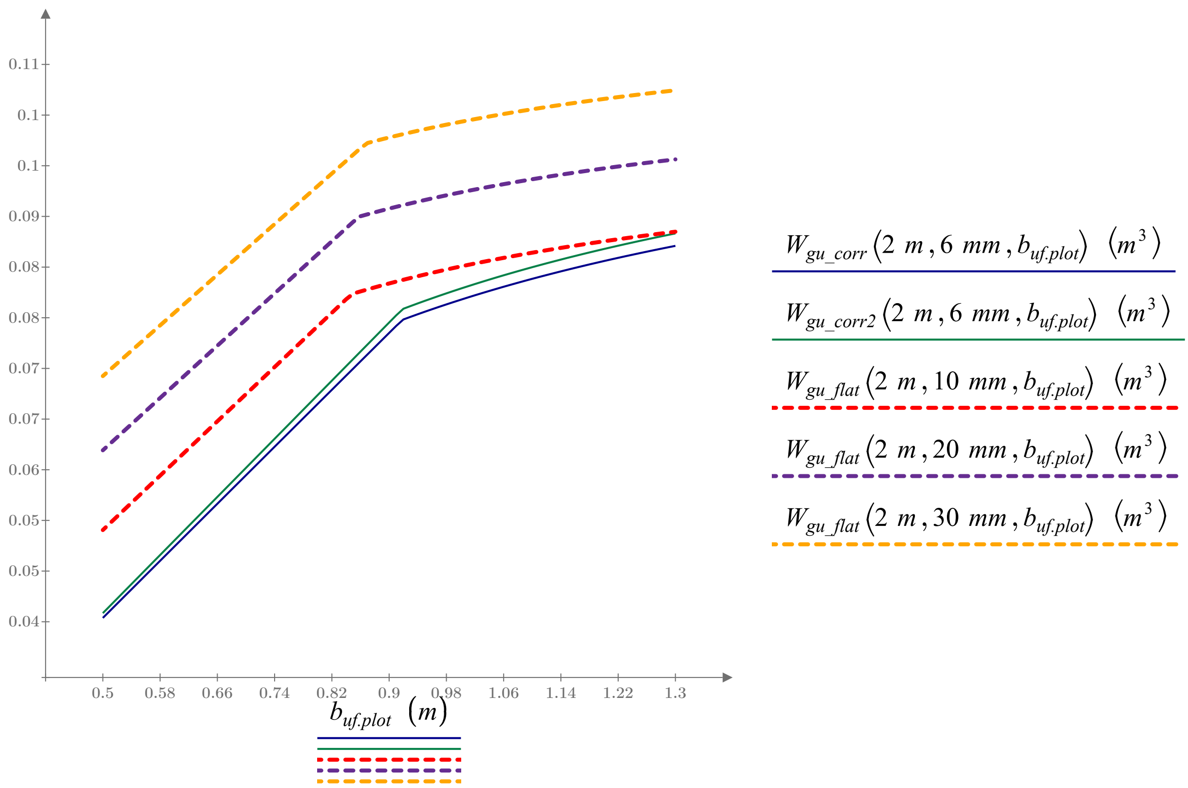
$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(2\ m, 20\ mm, 900\ mm)} = 16\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(3\ m, 20\ mm, 700\ mm)} = 31\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(3\ m, 20\ mm, 800\ mm)} = 28\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(3\ m, 20\ mm, 900\ mm)} = 23\%$$

E.4.2 Comparisson Steel cross-section, flange width

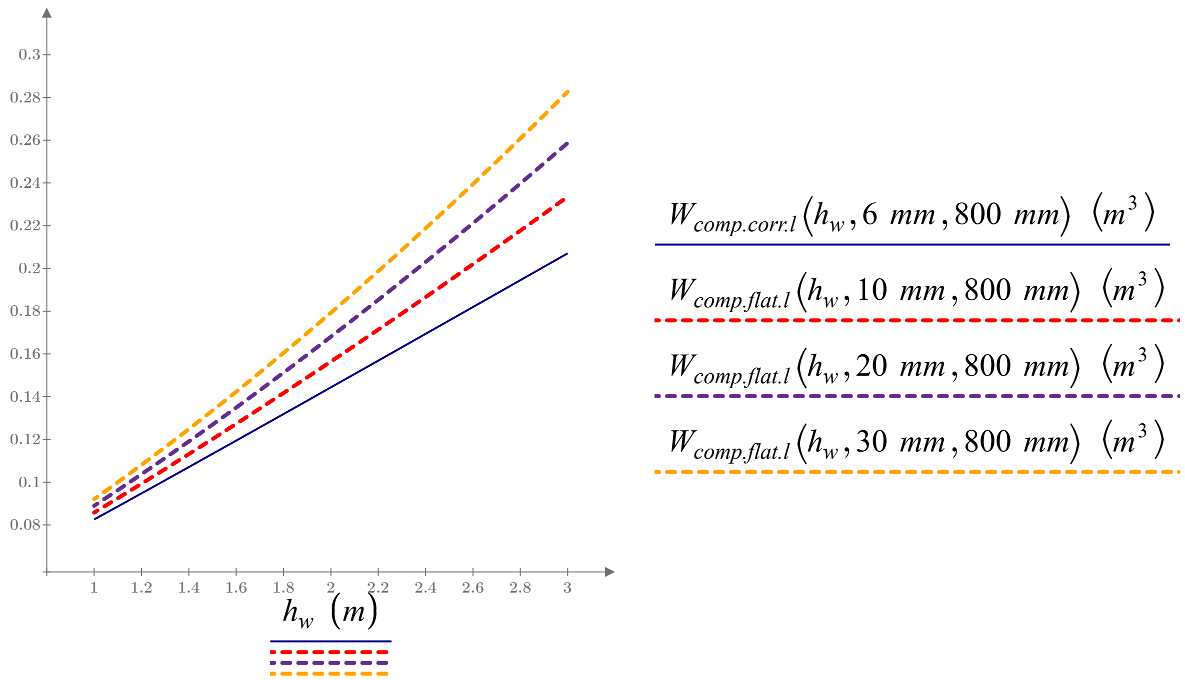


E.5 Comparisson composite cross-section

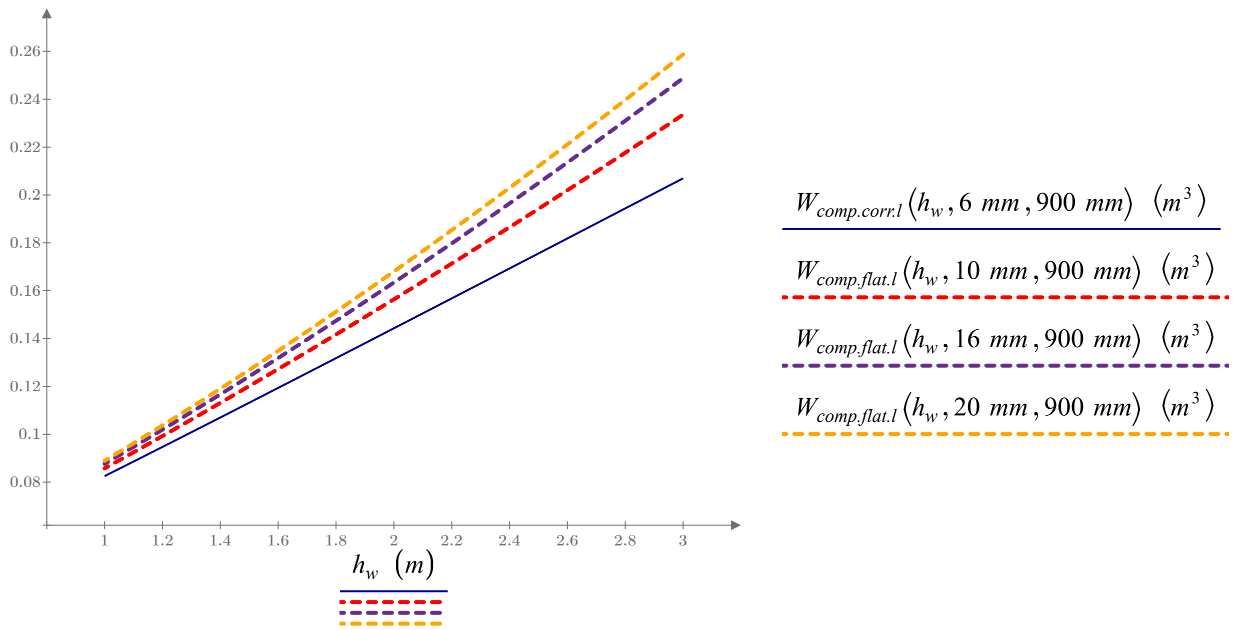
$W_{comp.corr.l}$ Elastic bending stiffness corrugated web

$W_{comp.flat.l}$ Elastic bending stiffness flat web

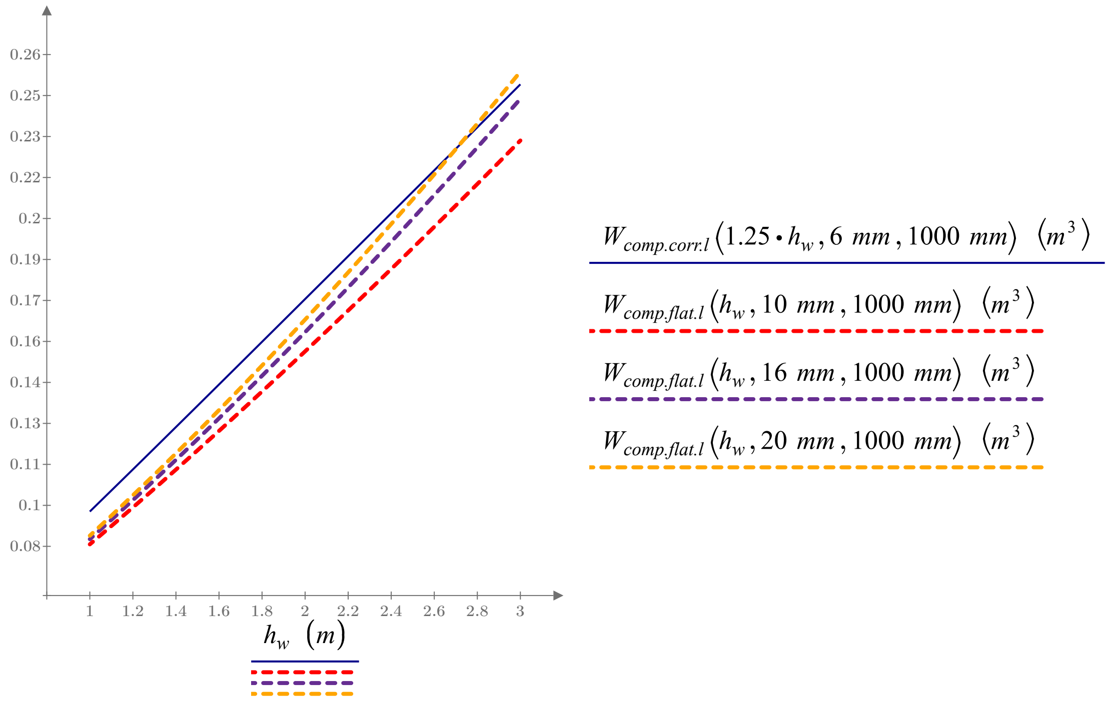
Flange width 800mm



Flange width 900mm



Flange width 1000mm



E.5.1 Comparisson composite cross-section, deviation

$$\eta_{1\_comp} := 1 - \frac{W_{comp.corr.l}(1\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(1\ m, 10\ mm, 800\ mm)} = 4\%$$

$$\eta_{1\_comp} := 1 - \frac{W_{comp.corr.l}(1\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(1\ m, 16\ mm, 800\ mm)} = 6\%$$

$$\eta_{2\_comp} := 1 - \frac{W_{comp.corr.l}(2\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(2\ m, 10\ mm, 800\ mm)} = 8\%$$

$$\eta_{2\_comp} := 1 - \frac{W_{comp.corr.l}(2\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(2\ m, 16\ mm, 800\ mm)} = 12\%$$

$$\eta_{3\_comp} := 1 - \frac{W_{comp.corr.l}(3\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(3\ m, 10\ mm, 800\ mm)} = 12\%$$

$$\eta_{3\_comp} := 1 - \frac{W_{comp.corr.l}(3\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(3\ m, 16\ mm, 800\ mm)} = 17\%$$

$$\eta_{1\_comp} := 1 - \frac{W_{comp.corr.l}(1\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(1\ m, 20\ mm, 800\ mm)} = 8\%$$

$$\eta_{2\_comp} := 1 - \frac{W_{comp.corr.l}(2\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(2\ m, 20\ mm, 800\ mm)} = 15\%$$

$$\eta_{3\_comp} := 1 - \frac{W_{comp.corr.l}(3\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(3\ m, 20\ mm, 800\ mm)} = 20\%$$

C

Corrugation influence on shrinkage forces

C Corrugation influence on shrinkage forces

$\varepsilon_{cs} := 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4, Equation 3.8

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab

$n_{L\_cs} := n_{L_3} = 14.9$ Modular ratio considering creep

C.1 Cross-sectional constants

Functions are created for calculation of the cross-sectional constants.

Steel section

$$A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := b_{fu} \cdot t_{fu} + h_w \cdot t_w + b_{fl} \cdot t_{fl}$$

$$z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := \frac{b_{fu} \cdot t_{fu} \cdot \frac{t_{fu}}{2} + h_w \cdot t_w \cdot \left(t_{fu} + \frac{h_w}{2}\right) + b_{fl} \cdot t_{fl} \cdot \left(t_{fu} + h_w + \frac{t_{fl}}{2}\right)}{A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}$$

$$\begin{aligned} I\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := {} & \frac{b_{fu} \cdot t_{fu}^3}{12} + b_{fu} \cdot t_{fu} \cdot \left(z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) - \frac{t_{fu}}{2}\right)^2 \\ & + \frac{t_w \cdot h_w^3}{12} + t_w \cdot h_w \cdot \left(\frac{h_w}{2} + t_{fu} - z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \\ & + \frac{b_{fl} \cdot t_{fl}^3}{12} + b_{fl} \cdot t_{fl} \cdot \left(\frac{t_{fl}}{2} + h_w + t_{fu} - z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \end{aligned}$$

Composite section

$$A_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) + \frac{A_{slab.fic}}{n_{L\_cs}}$$

$$z_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := \frac{\frac{A_{slab.fic}}{n_{L\_cs}} \cdot \frac{-h_{m.slab}}{2} + b_{fu} \cdot t_{fu} \cdot \frac{t_{fu}}{2} + h_w \cdot t_w \cdot \left(t_{fu} + \frac{h_w}{2}\right) + b_{fl} \cdot t_{fl} \cdot \left(t_{fu} + h_w + \frac{t_{fl}}{2}\right)}{\frac{A_{slab.fic}}{n_{L\_cs}} + A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}$$

$$\begin{aligned} I_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := {} & I\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) + A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) \cdot \left(z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) - z_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \\ & + \frac{h_{m.slab}^3 \cdot \frac{b_{eff}}{n_{L\_cs}}}{12} + \frac{A_{slab.fic}}{n_{L\_cs}} \cdot \left(\frac{h_{m.slab}}{2} + z_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \end{aligned}$$

Dimensions

$b_{fu} := 850\ mm$ $\quad t_{fu} := 45\ mm$ Dimensions upper flange

$h_w := 1955\ mm$ $\quad t_w := 17\ mm$ Dimensions web - note that for the corrugated web the thickness is set to zero

$b_{fl} := 1225\ mm$ $\quad t_{fl} := 50\ mm$ Dimension lower flange

$h_{tot}(t_{fl}, h_w, t_{fu}) := h_w + t_{fu} + t_{fl}$ Total height of steel girder

C.2 Shrinkage force

$E_{c.eff} := \dfrac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

Shrinkage force on composite section

$$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5202\ kN$$

$$M_{cs}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) := F_{cs} \cdot \left(z_{comp}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) + \frac{h_{m.slab}}{2}\right)$$

Forces acting in the steel

$$F_{cs\_st}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) := F_{cs} \cdot \left(\frac{A(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})}{A_{comp}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})} - \frac{A(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})}{I_{comp}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})} \cdot \left(z_{comp}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) + \frac{h_{m.slab}}{2}\right) \cdot \left(z(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) + \frac{h_{m.slab}}{2} - \left(z_{comp}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) + \frac{h_{m.slab}}{2}\right)\right)\right)$$

$$M_{cs\_st}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) := \frac{I(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})}{I_{comp}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})} \cdot M_{cs}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl})$$

$$F_{cs\_cor} := F_{cs\_st}(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}) = 1214\ kN$$

$$F_{cs\_fw} := F_{cs\_st}(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}) = 1441\ kN$$

$$\sigma := \frac{F_{cs\_cor}}{F_{cs\_fw}} = 84\%$$

$$M_{cs\_cor} := M_{cs\_st}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}\right) = 1702\ kN \cdot m$$

$$M_{cs\_fw} := M_{cs\_st}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) = 1938\ kN \cdot m$$

$$\sigma := \frac{M_{cs\_cor}}{M_{cs\_fw}} = 88\%$$

Stress in upper flange due to shrinkage

$$\sigma_{cs\_uf}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := \frac{-F_{cs\_st}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}{A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)} - \frac{M_{cs\_st}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) \cdot z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}{I\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}$$

$$\sigma_{cs\_cor.uf} := \sigma_{cs\_uf}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}\right) = -34.8\ MPa$$

$$\sigma_{cs\_fw.uf} := \sigma_{cs\_uf}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) = -32.7\ MPa$$

$$\sigma := \frac{\sigma_{cs\_cor.uf}}{\sigma_{cs\_fw.uf}} = 107\%$$

Stress in lower flange due to shrinkage

$$\sigma_{cs\_lf}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := \frac{-F_{cs\_st}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}{A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)} + \frac{M_{cs\_st}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) \cdot \left(h_{tot}\left(t_{fl}, h_w, t_{fu}\right) - z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)}{I\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}$$

$$\sigma_{cs\_cor.lf} := \sigma_{cs\_lf}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}\right) = 2.12\ MPa$$

$$\sigma_{cs\_fw.lf} := \sigma_{cs\_lf}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) = 4.7\ MPa$$

$$\sigma := \frac{\sigma_{cs\_cor.lf}}{\sigma_{cs\_fw.lf}} = 45\%$$

Forces acting in the concrete

$$F_{cs\_c}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) := F_{cs} \cdot \left(1-\frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{comp}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})}-\frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{comp}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})} \cdot \left(z_{comp}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})+\frac{h_{m.slab}}{2}\right)^2\right)$$

$$M_{cs\_c}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) := \frac{\frac{1}{12}\frac{b_{eff}}{n_{L\_cs}} \cdot h_{m.slab}^3}{I_{comp}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})} \cdot M_{cs}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})$$

Stress in upper part of slab

$$\sigma_{cs\_us}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) := \frac{F_{cs\_c}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})}{A_{slab.fic}} - \frac{M_{cs\_c}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} b_{eff} \cdot h_{m.slab}^3}$$

$$\sigma_{cs\_cor.us} := \sigma_{cs\_us}(b_{fu},t_{fu},h_w,0\ mm,b_{fl},t_{fl}) = 0.58\ MPa$$

$$\sigma_{cs\_fw.us} := \sigma_{cs\_us}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) = 0.7\ MPa$$

$$\sigma := \frac{\sigma_{cs\_cor.us}}{\sigma_{cs\_fw.us}} = 80\%$$

Stress in lower part of slab

$$\sigma_{cs\_ls}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) := \frac{F_{cs\_c}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl})}{A_{slab.fic}} + \frac{M_{cs\_c}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} b_{eff} \cdot h_{m.slab}^3}$$

$$\sigma_{cs\_cor.ls} := \sigma_{cs\_ls}(b_{fu},t_{fu},h_w,0\ mm,b_{fl},t_{fl}) = 1.0\ MPa$$

$$\sigma_{cs\_fw.ls} := \sigma_{cs\_ls}(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}) = 1.1\ MPa$$

$$\sigma := \frac{\sigma_{cs\_cor.ls}}{\sigma_{cs\_fw.ls}} = 87\%$$

C.3 Stress distribution

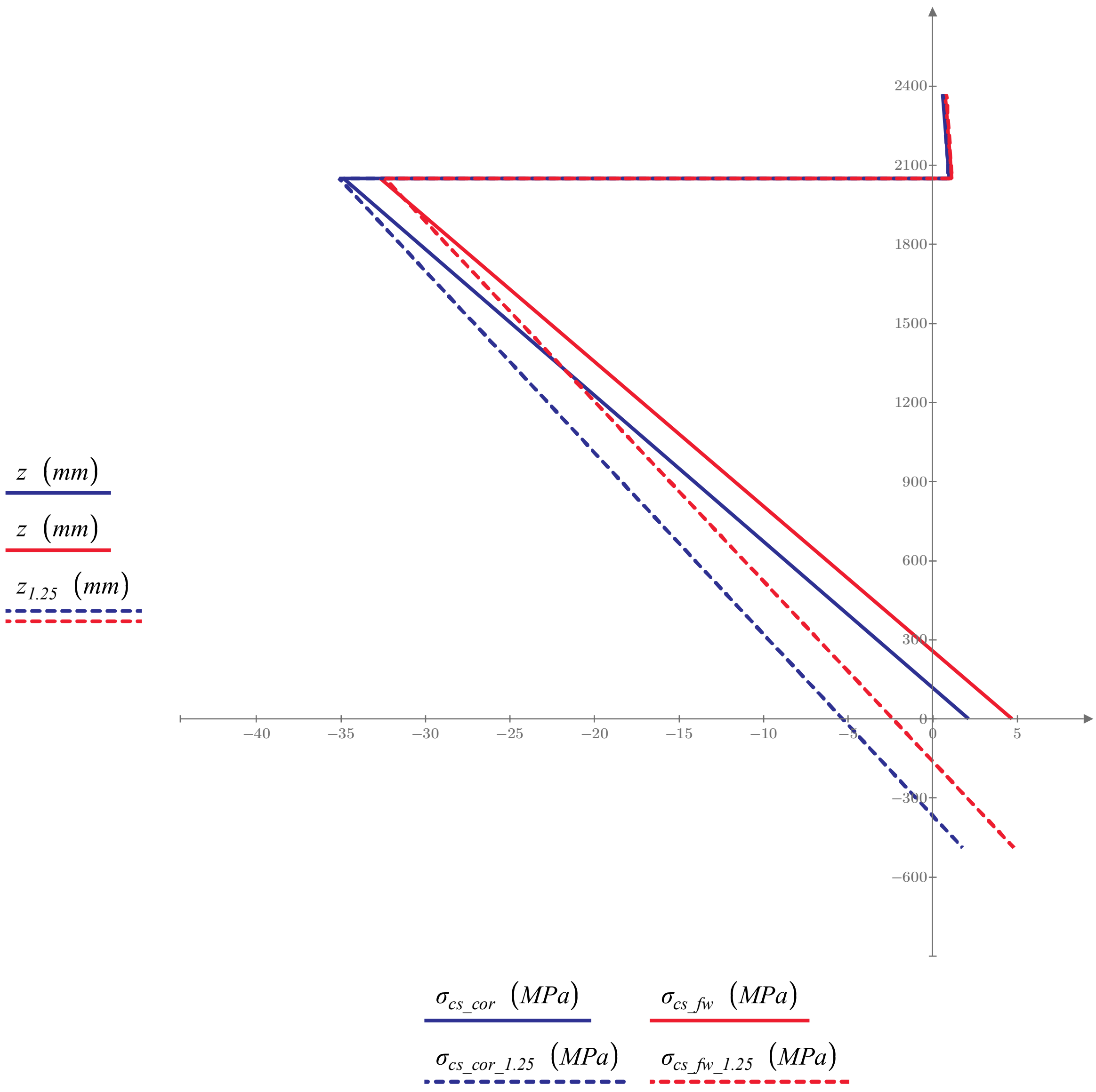
Red line = **flat** web

Blue line = **corrugated** web

______ = 1.0 h_w

\- - - - - - = 1.25 h_w

The below plot displays the stresses over the height of the cross-section whereas the top coordinates represents the stresses in the concrete (tensile stresses) and the bottom coordinates represents the stresses in the steel. The dotted line shows the stresses for if the web height is increased with 25%.



D

Influence of linear coefficient of thermal expansion

D Influence of linear coefficient of thermal expansion

For composite bridges conducted with carbon steel the thermal expansion coefficient could be taken the same for both concrete and steel. This means that for a simply supported bridge no strain difference caused by external temperature variations will take place between the two components. For steel girders in stainless steel however the coefficient is 60% larger for stainless steel compared to concrete, this means that strain differences will occur between the sectional parts.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

$T_0 := 10$ A.1(3)

$T_{min} := T(-48, -20)$ Interval for minimum temperature in Sweden - TSFS 2018:57 - 8 ch 2 §

$T_{max} := T(30, 40)$ Interval for maximum temperature in Sweden - TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min}$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0$ Expansion - Equation 6.1

$\Delta T_{c2st} := 15$ Temperature difference steel and concrete - SS-EN 1991-1-5

Material parameters

$f_{ylf} = 450\ MPa$ Yield strength - lower flange, stainless steel

$f_{y\_cs} := 335\ MPa$ Yield strength - lower flange, carbon steel (cs) $(40\ mm < t \le 63\ mm)$

D.1 Strains

Strains are calculated for two load cases; one dependent on the strain difference between components caused by local temperature variations. The other is given in SS-EN 1991-1-5 as a temperature difference between components.

D.1.1 Stainless steel

$\Delta\varepsilon_{LC.1\_con}\left(\Delta T_{N.con}\right) := -\left(\alpha_{ss}-\alpha_c\right)\cdot\Delta T_{N.con}$

Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp}\left(\Delta T_{N.exp}\right) := \left(\alpha_{ss}-\alpha_c\right)\cdot\Delta T_{N.exp}$

Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss}\cdot\Delta T_{c2st} = 24.0\ 10^{-5}$

Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c\cdot\Delta T_{c2st} = 15.0\ 10^{-5}$

Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st},\Delta\varepsilon_{LC.2\_c}\right) = 24.0\ 10^{-5}$

Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp.ss\_con} := -\Delta\varepsilon_{LC.2}+\Delta\varepsilon_{LC.1\_con}\left(\Delta T_{N.con}\right)$

Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp.ss\_exp} := \Delta\varepsilon_{LC.2}+\Delta\varepsilon_{LC.1\_exp}\left(\Delta T_{N.exp}\right)$

Maximum strain difference; temperature raises and the steel heatens up even higher

D.1.2 Carbon steel

$\varepsilon_{temp.cs} := \alpha_c\cdot\Delta T_{c2st} = 15.0\ 10^{-5}$

$\varepsilon_{temp.carb} := \varepsilon_{temp.cs}\cdot\frac{\Delta T_{N.exp}}{\Delta T_{N.exp}}$ Creating a vector

$\Delta\varepsilon_{LC.1\_con}\left(\Delta T_{N.con}\right) := -\left(\alpha_{cs}-\alpha_c\right)\cdot\Delta T_{N.con}$

Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp}\left(\Delta T_{N.exp}\right) := \left(\alpha_{cs}-\alpha_c\right)\cdot\Delta T_{N.exp}$

Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{cs}\cdot\Delta T_{c2st} = 18.0\ 10^{-5}$

Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c\cdot\Delta T_{c2st} = 15.0\ 10^{-5}$

Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st},\Delta\varepsilon_{LC.2\_c}\right) = 18.0\ 10^{-5}$

Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp.cs\_con} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con}\left(\Delta T_{N.con}\right)$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp.cs\_exp} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp}\left(\Delta T_{N.exp}\right)$ Maximum strain difference; temperature raises and the steel heatens up even higher

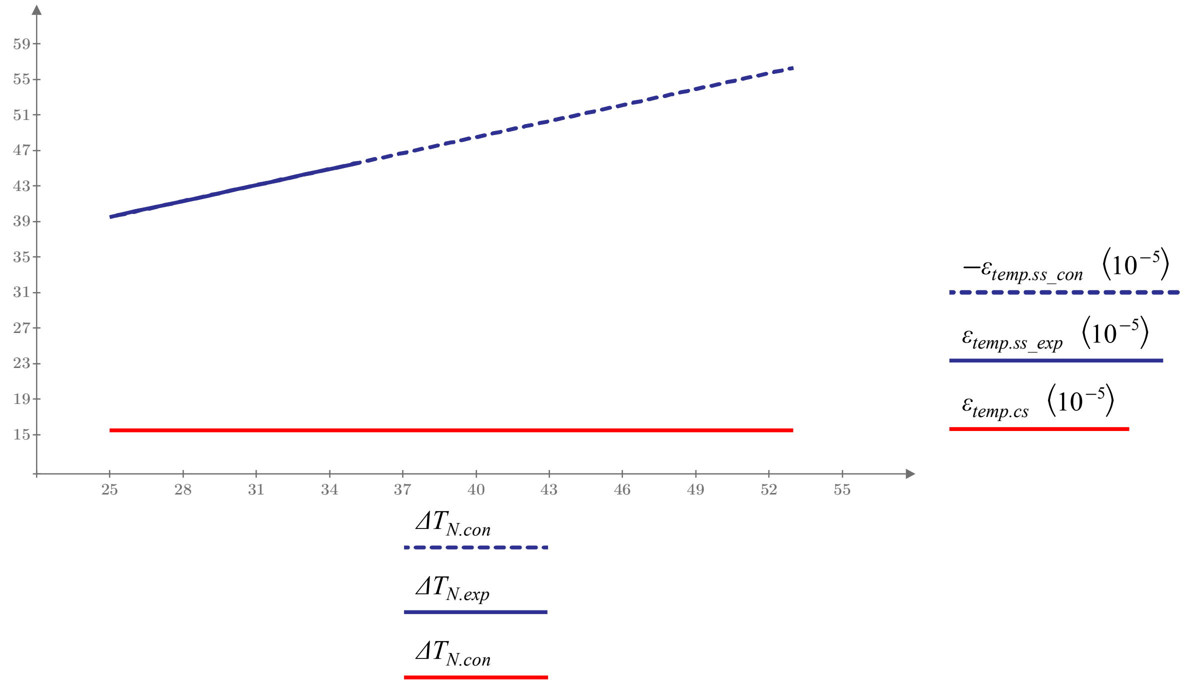


Figure. Showing how the strain varies dependent for if stainless steel or carbon steel is used, as well as if expansion or contraction takes place

D.2 Cross-sectional constants

Functions are created for calculation of the cross-sectional constants.

Steel

$$A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := b_{fu} \cdot t_{fu} + h_w \cdot t_w + b_{fl} \cdot t_{fl}$$

$$z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := \frac{b_{fu} \cdot t_{fu} \cdot \frac{t_{fu}}{2} + h_w \cdot t_w \cdot \left(t_{fu} + \frac{h_w}{2}\right) + b_{fl} \cdot t_{fl} \cdot \left(t_{fu} + h_w + \frac{t_{fl}}{2}\right)}{A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}$$

$$\begin{aligned} I\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := & \frac{b_{fu} \cdot t_{fu}^3}{12} + b_{fu} \cdot t_{fu} \cdot \left(z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) - \frac{t_{fu}}{2}\right)^2 \\ & + \frac{t_w \cdot h_w^3}{12} + t_w \cdot h_w \cdot \left(\frac{h_w}{2} + t_{fu} - z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \\ & + \frac{b_{fl} \cdot t_{fl}^3}{12} + b_{fl} \cdot t_{fl} \cdot \left(\frac{t_{fl}}{2} + h_w + t_{fu} - z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \end{aligned}$$

Composite section

$$A_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) + \frac{A_{slab.fic}}{n_{L\_short}}$$

$$z_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := \frac{\frac{A_{slab.fic}}{n_{L\_short}} \cdot \frac{-h_{m.slab}}{2} + b_{fu} \cdot t_{fu} \cdot \frac{t_{fu}}{2} + h_w \cdot t_w \cdot \left(t_{fu} + \frac{h_w}{2}\right) + b_{fl} \cdot t_{fl} \cdot \left(t_{fu} + h_w + \frac{t_{fl}}{2}\right)}{\frac{A_{slab.fic}}{n_{L\_short}} + A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)}$$

$$\begin{aligned} I_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) := & I\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) + A\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) \cdot \left(z\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right) - z_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \\ & + \frac{h_{m.slab}^3 \cdot \frac{b_{eff}}{n_{L\_short}}}{12} + \frac{A_{slab.fic}}{n_{L\_short}} \cdot \left(\frac{h_{m.slab}}{2} + z_{comp}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}\right)\right)^2 \end{aligned}$$

$b_{fu} := 850\ mm$ $\quad t_{fu} := 45\ mm$ — Dimensions upper flange

$h_w := 1955\ mm$ — Height of web

$b_{fl} := 1225\ mm$ $\quad t_{fl} := 50\ mm$ — Dimension lower flange

$h_{tot}\left(t_{fl}, h_w, t_{fu}\right) := h_w + t_{fu} + t_{fl}$ — Total height of steel girder

D.2.1 Temperature dependent forces and load effects

Applied forces in system analysis acting on the composite section

$$F_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=\varepsilon\cdot E_s\cdot A\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)$$

$$M_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=F_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)\cdot\left(z\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)-\left(z_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)\right)\right)$$

Forces acting in the steel

$$F_{temp\_st}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=F_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)\cdot\left(1-\left(\frac{A\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}{A_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}+\frac{A\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}{I_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}\cdot\left(z\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)-\left(z_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)\right)\right)^2\right)\right)$$

$$M_{temp\_st}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=\frac{I\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}{I_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}\cdot M_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)$$

Calculating stresses in lower flange (steel)

$$\sigma_{temp\_st1}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=\frac{-F_{temp\_st}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)}{A\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}+\frac{M_{temp\_st}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)\cdot\left(h_{tot}\left(t_{fl},h_w,t_{fu}\right)-z\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)\right)}{I\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}$$

Calculating stresses in upper flange (steel)

$$\sigma_{temp\_st2}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=\frac{-F_{temp\_st}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)}{A\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}+\frac{-M_{temp\_st}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)\cdot z\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}{I\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}$$

Forces acting in the concrete

$$F_{temp\_c}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=F_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)\cdot\left(\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{A_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}-\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{I_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}\cdot\left(z\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)-\left(z_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)\right)\right)\cdot\left(z_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)+\cdots\right.\right.$$

$$M_{temp\_c}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right):=\frac{\frac{1}{12}\cdot\frac{b_{eff}}{n_{L\_short}}\cdot h_{m.slab}{}^3}{I_{comp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl}\right)}\cdot M_{temp}\left(b_{fu},t_{fu},h_w,t_w,b_{fl},t_{fl},\varepsilon\right)$$

Calculating stresses in upper slab part of slab

$$\sigma_{temp\_c1}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}, \varepsilon\right) := \frac{F_{temp\_c}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}, \varepsilon\right)}{A_{slab.fic}} - \frac{M_{temp\_c}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}, \varepsilon\right) \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} b_{eff} \cdot h_{m.slab}^{3}}$$

Calculating stresses in lower slab part of slab

$$\sigma_{temp\_c2}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}, \varepsilon\right) := \frac{F_{temp\_c}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}, \varepsilon\right)}{A_{slab.fic}} + \frac{M_{temp\_c}\left(b_{fu}, t_{fu}, h_w, t_w, b_{fl}, t_{fl}, \varepsilon\right) \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} b_{eff} \cdot h_{m.slab}^{3}}$$

D.2.1.1 Load effects for flat web (lower flange)

$t_{w.fw} := 17\ mm$ — Used in original design

$F_{temp\_fw}(\varepsilon) := F_{temp\_st}\left(b_{fu}, t_{fu}, h_w, t_{w.fw}, b_{fl}, t_{fl}, \varepsilon\right)$ — Flat web

$M_{temp\_fw}(\varepsilon) := M_{temp\_st}\left(b_{fu}, t_{fu}, h_w, t_{w.fw}, b_{fl}, t_{fl}, \varepsilon\right)$ — Flat web

$\sigma_{temp\_fw}(\varepsilon) := \sigma_{temp\_st1}\left(b_{fu}, t_{fu}, h_w, t_{w.fw}, b_{fl}, t_{fl}, \varepsilon\right)$ — Flat web

$\sigma_{temp\_fw.u}(\varepsilon) := \sigma_{temp\_st2}\left(b_{fu}, t_{fu}, h_w, t_{w.fw}, b_{fl}, t_{fl}, \varepsilon\right)$ — Flat web, upper flange

D.2.1.2 Load effects for corrugated web (lower flange)

$F_{temp\_cor}(\varepsilon) := F_{temp\_st}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}, \varepsilon\right)$ — Corrugated web

$M_{temp\_cor}(\varepsilon) := M_{temp\_st}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}, \varepsilon\right)$ — Corrugated web

$\sigma_{temp\_cor}(\varepsilon) := \sigma_{temp\_st1}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}, \varepsilon\right)$ — Corrugated web

$\sigma_{temp\_cor.u}(\varepsilon) := \sigma_{temp\_st2}\left(b_{fu}, t_{fu}, h_w, 0\ mm, b_{fl}, t_{fl}, \varepsilon\right)$ — Corrugated web, upper flange

D.3 Stress distribution over the height of the cross-section

D.3.1 Comparison for flat- and corrugated web + stainless- or carbon steel

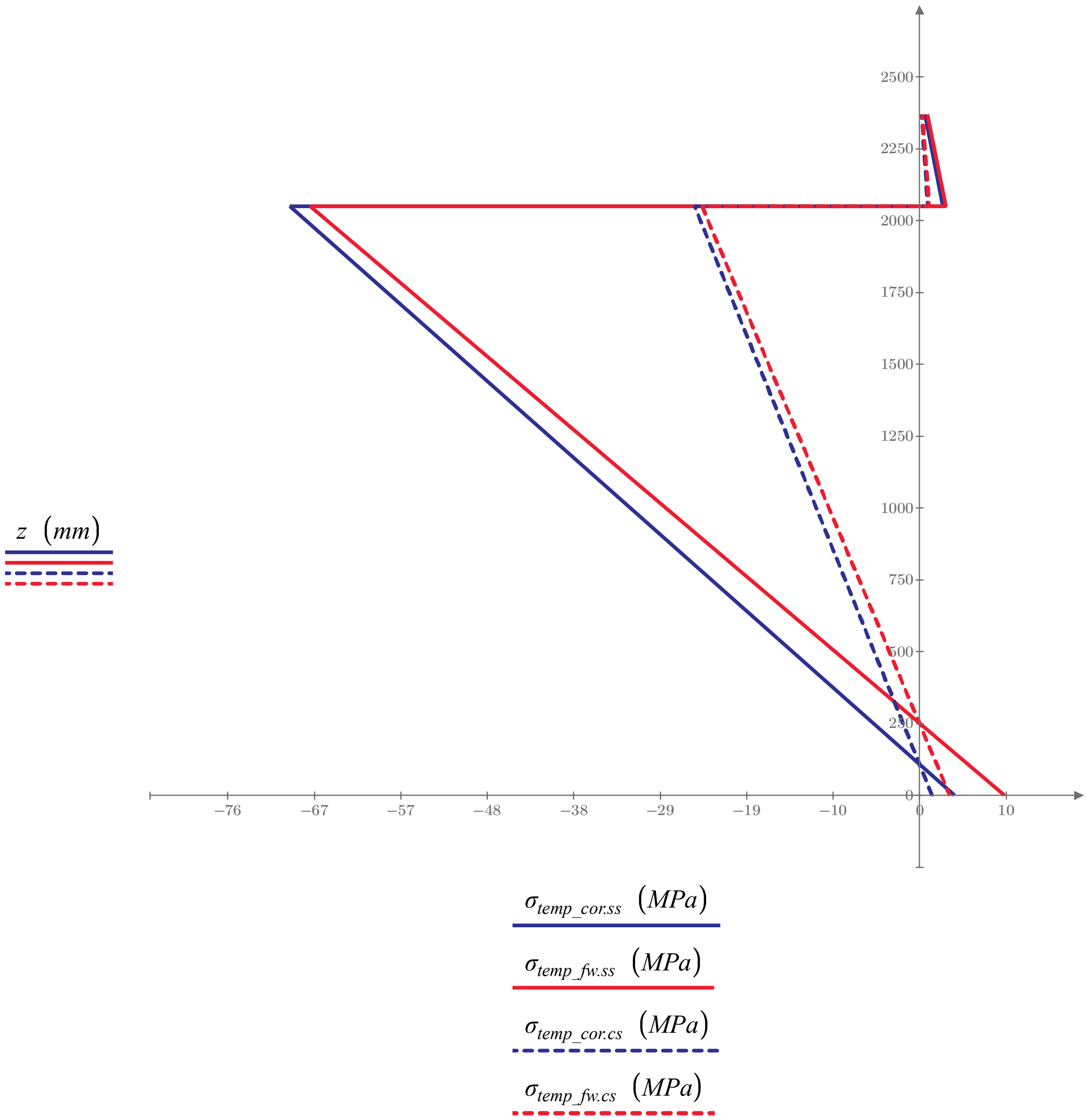
Red line = **flat** web

Blue line = **corrugated** web

______ = Stainless steel

- - - - - - = Carbon steel

The below plot displays the stresses over the height of the cross-section whereas the top coordinates represents the stresses in the concrete and the bottom coordinates represents the stresses in the steel. The dotted line shows the stresses for if instead carbon steel girders are used.



D.3.2 Comparison for flat- and corrugated web when increasing the web height

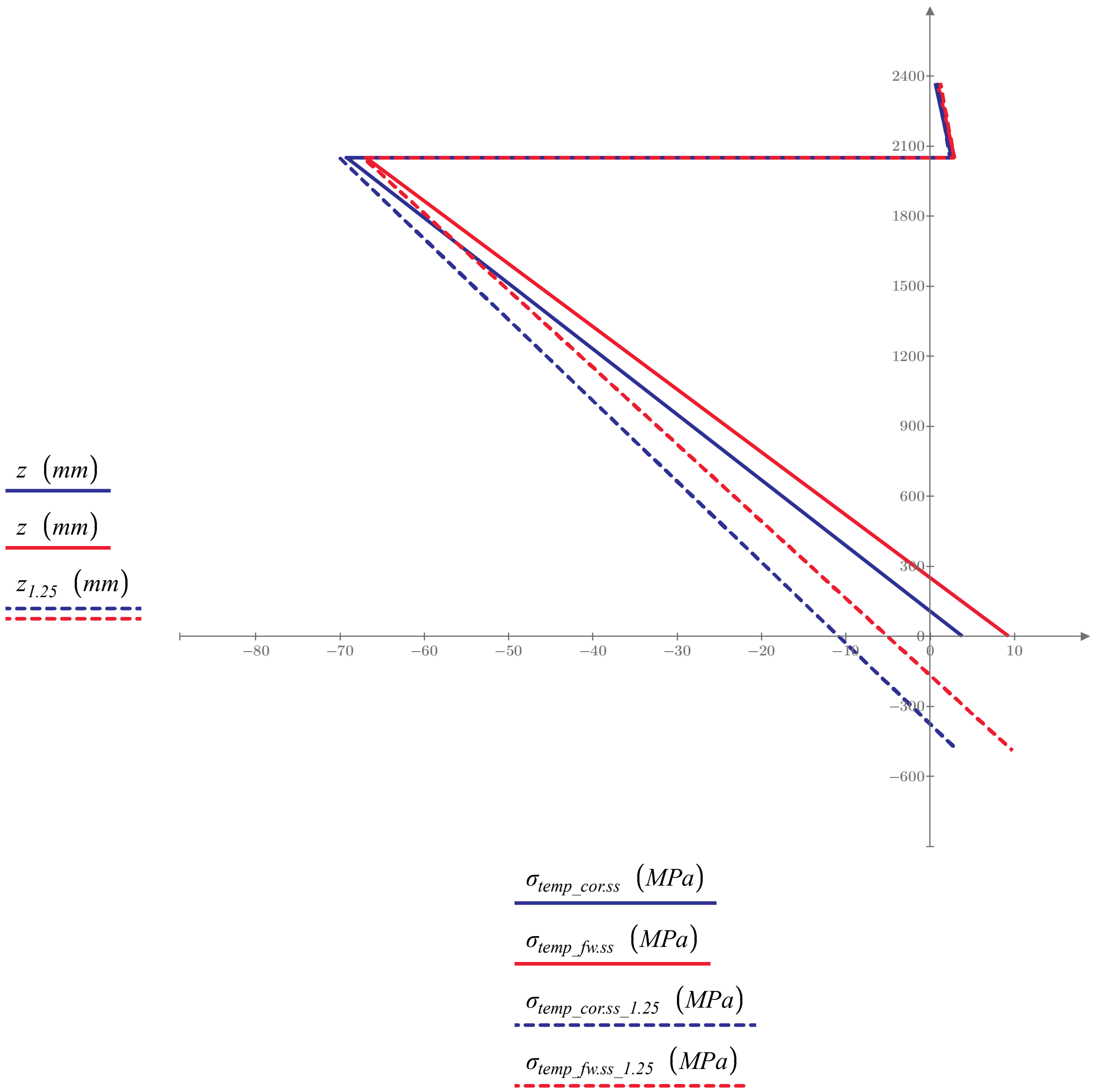
Red line = **flat** web

Blue line = **corrugated** web

______ = Original web height

\- - - - - - = Increased web height of 25%

The below plot displays the stresses over the height of the cross-section whereas the top coordinates represents the stresses in the concrete and the bottom coordinates represents the stresses in the steel. The dotted line shows the stresses for if the web height is increased with 25%.



D.3.3 Comparison for stainless- and carbon steel when increasing the web height

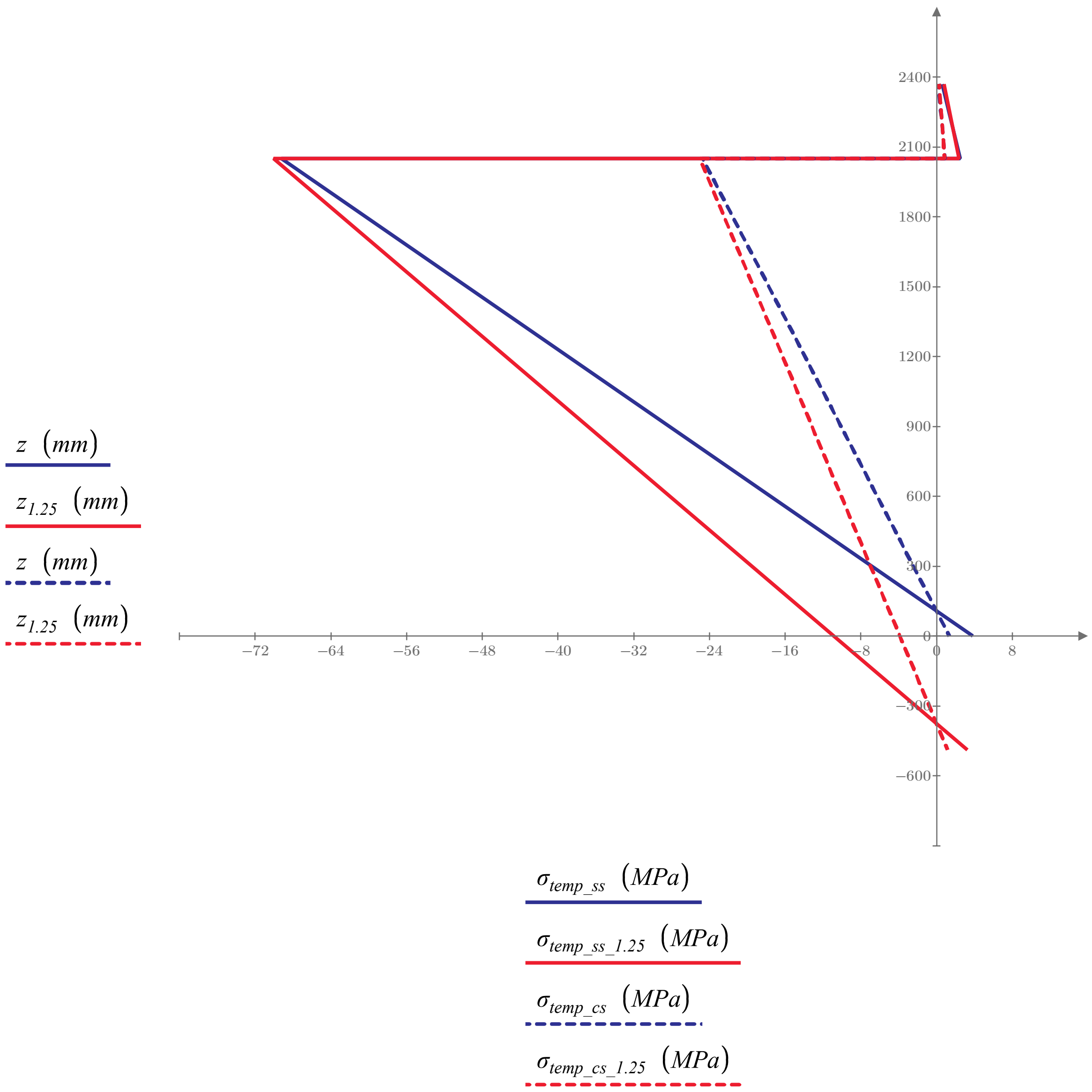
Red line = **increased** web

Blue line = **original** web

______ = Stainless steel

\- - - - - - = Carbon steel

The below plot displays the stresses over the height of the cross-section whereas the top coordinates represents the stresses in the concrete and the bottom coordinates represents the stresses in the steel. The dotted lines shows the stresses for if a girder in carbon steel is used.



D.4 Resulting stresses in the upper- and lower flange for varying temperature

D.4.1 Resulting stress in lower flange (tension)

Red line = cross-section with **flat web**

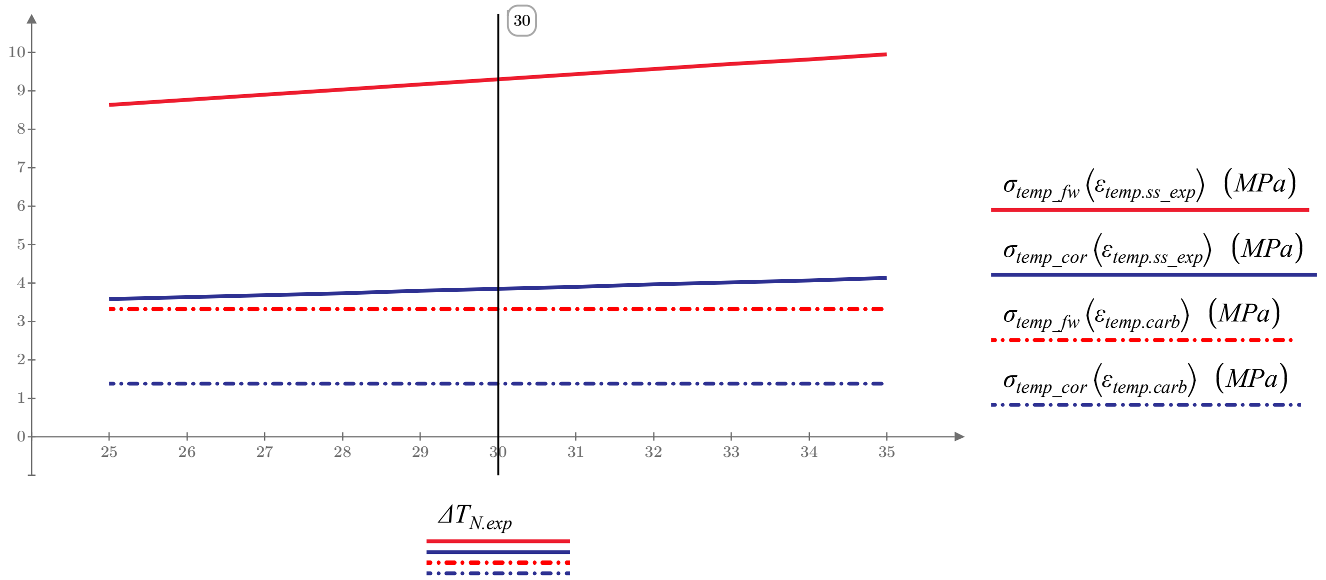
Blue line = cross-section with **corrugated web**

$\sigma_{temp\_fw}\left(\varepsilon_{temp.ss\_exp}\right)$ Resulting stress - flat web, stainless steel

$\sigma_{temp\_cor}\left(\varepsilon_{temp.ss\_exp}\right)$ Resulting stress - corrugated web, stainless steel

$\sigma_{temp\_fw}\left(\varepsilon_{temp.carb}\right)$ Resulting stress - flat web, carbon steel ($\alpha_{cs} = \alpha_c$)

$\sigma_{temp\_cor}\left(\varepsilon_{temp.carb}\right)$ Resulting stress - corrugated web, carbon steel ($\alpha_{cs} = \alpha_c$)



$\sigma_{m.cor\_ss} := mean\left(\sigma_{temp\_cor}\left(\varepsilon_{temp.ss\_exp}\right)\right) = 3.84\ MPa$

Mean stress in lower flange due to expansion - corrugated web, stainless steel (prospective bridge design)

$$\eta_{m.cor\_ss} := \frac{\sigma_{m.cor\_ss}}{f_{ylf}} = 1\%$$

Utilization ratio for lower flange due to thermal expansion in comparison to total strength

$\sigma_{m.cor\_cs} := mean\left(\sigma_{temp\_cor}\left(\varepsilon_{temp.carb}\right)\right) = 1.37\ MPa$

Mean stress in lower flange due to expansion - corrugated web, carbon steel ($\alpha_{cs} = \alpha_c$, according to guidelines)

$$\eta_{m.cor\_cs} := \frac{\sigma_{m.cor\_cs}}{f_{y\_cs}} = 0.41\%$$

Utilization ratio for lower flange due to thermal expansion in comparison to total strength

$$mean\left(\frac{\sigma_{temp\_cor}\left(\varepsilon_{temp.ss\_exp}\right)}{\sigma_{temp\_cor}\left(\varepsilon_{temp.carb}\right)}\right) = 2.8$$

Mean difference between:
- girder with stainless steel and corrugated web
- girder with carbon steel and flat web

$$mean\left(\frac{\sigma_{temp\_cor}\left(\varepsilon_{temp.ss\_exp}\right)}{\sigma_{temp\_fw}\left(\varepsilon_{temp.ss\_exp}\right)}\right) = 0.4$$

D.4.2 Resulting stress in upper flange (compression)

Red line = cross-section with **flat web**

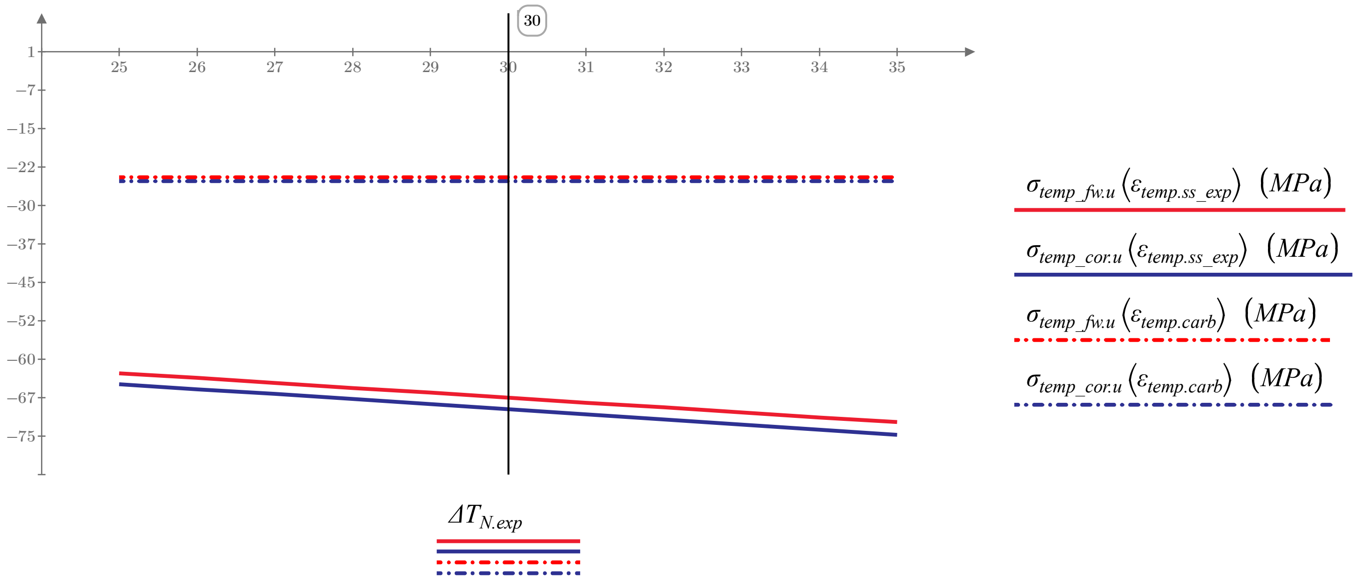
Blue line = cross-section with **corrugated web**

$\sigma_{temp\_fw.u}\left(\varepsilon_{temp.ss\_exp}\right)$ Resulting stress - flat web, stainless steel

$\sigma_{temp\_cor.u}\left(\varepsilon_{temp.ss\_exp}\right)$ Resulting stress - corrugated web, stainless steel

$\sigma_{temp\_fw.u}\left(\varepsilon_{temp.carb}\right)$ Resulting stress - flat web, carbon steel ($\alpha_{cs} = \alpha_c$)

$\sigma_{temp\_cor.u}\left(\varepsilon_{temp.carb}\right)$ Resulting stress - corrugated web, carbon steel ($\alpha_{cs} = \alpha_c$)



$\sigma_{m.cor\_ss} := mean\left(\sigma_{temp\_cor.u}\left(\varepsilon_{temp.ss\_exp}\right)\right) = -69\ MPa$

Mean stress in upper flange due to expansion - corrugated web, stainless steel (prospective bridge design)

$$\eta_{m.cor\_ss} := \frac{\left|\sigma_{m.cor\_ss}\right|}{f_{ylf}} = 15\%$$

Utilization ratio for upper flange due to thermal expansion in comparison to total strength

$\sigma_{m.cor\_cs} := mean\left(\sigma_{temp\_cor.u}\left(\varepsilon_{temp.carb}\right)\right) = -25\ MPa$

Mean stress in upper flange due to expansion - corrugated web, carbon steel ($\alpha_{cs} = \alpha_c$, according to guidelines)

$$\eta_{m.cor\_cs} := \frac{\left|\sigma_{m.cor\_cs}\right|}{f_{y\_cs}} = 7\%$$

Utilization ratio for lower flange due to thermal expansion in comparison to total strength

$$mean\left(\frac{\sigma_{temp\_cor.u}\left(\varepsilon_{temp.ss\_exp}\right)}{\sigma_{temp\_fw.u}\left(\varepsilon_{temp.ss\_exp}\right)}\right) = 1.0$$

$$mean\left(\frac{\sigma_{temp\_cor.u}\left(\varepsilon_{temp.ss\_exp}\right)}{\sigma_{temp\_cor.u}\left(\varepsilon_{temp.carb}\right)}\right) = 2.8$$

Mean difference between:
- girder with stainless steel and corrugated web
- girder with carbon steel and flat web

D.4.3 Comparison for the stress distribution for $\Delta T_{N.exp\_max}$ and $\Delta T_{N.exp\_min}$

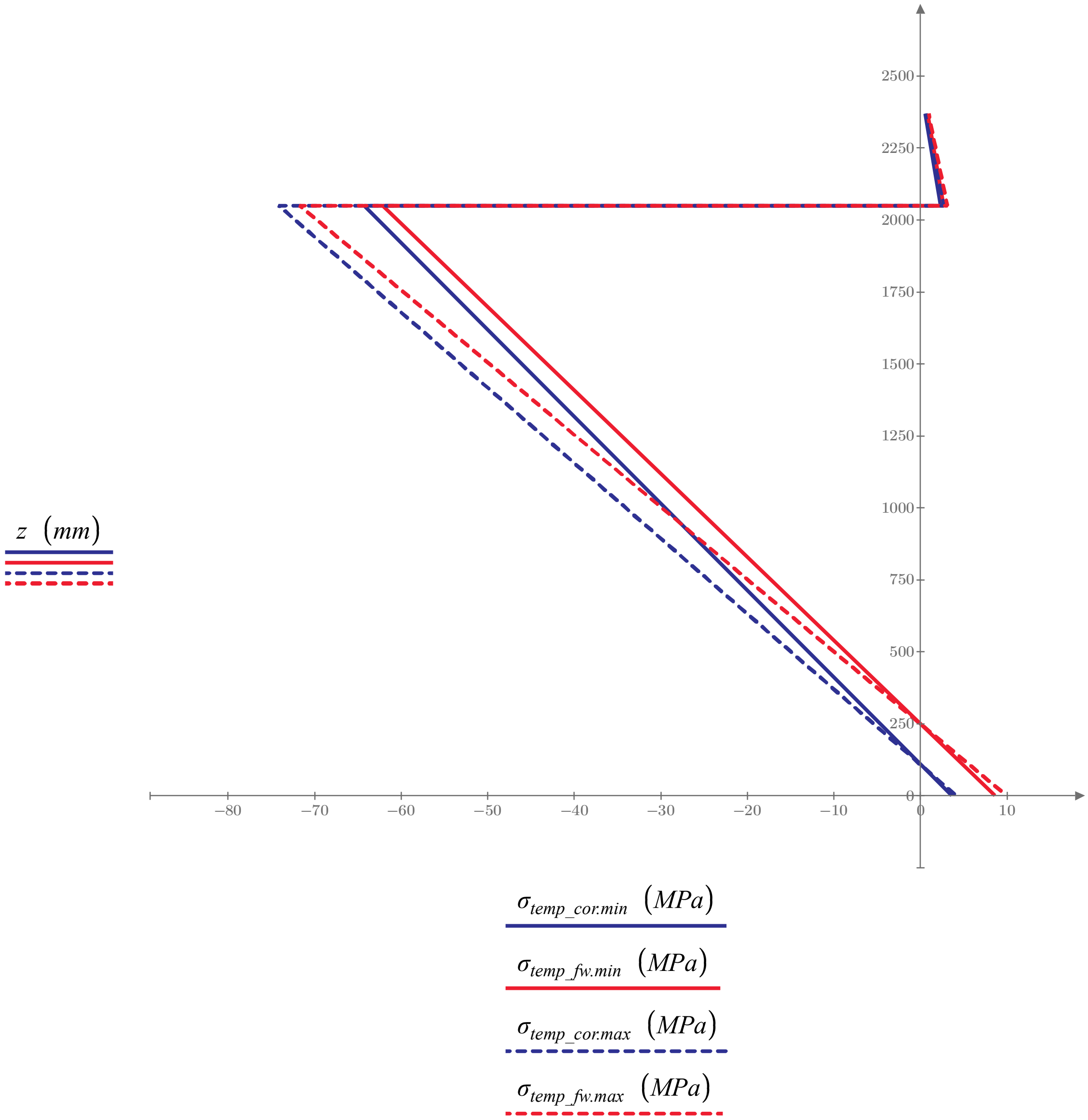
Red line = **flat** web

Blue line = **corrugated** web

______ = Stainless steel

\- - - - - - = Carbon steel

The below plot displays the difference in stress dependent for the maximum temperature during expansion ($\Delta T_{N.exp\_max}$) and the minimum temperature during expansion ($\Delta T_{N.exp\_min}$)



E

Corrugation influence on elastic bending stiffness

Appendix E Influence of corrugation on elastic bending stiffness

E.1 Geometry

$t_{uf} := 42\ mm$ Thickness of upper flange

$b_{uf} := 800\ mm$ Width of upper flange

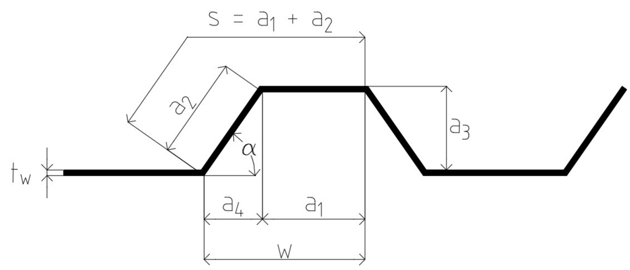
$t_{lf} := 55\ mm$ Thickness of lower flange

$b_{lf} := 1150\ mm$ Width of lower flange

$t_w := \begin{bmatrix} 10\ mm \\ 16\ mm \\ 20\ mm \end{bmatrix}$ Thickness of web

$a_{weld} := 4\ mm$ Weldthroat thickness

E.1.1 Corrugation shape



$a_{c1} := 110\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 75\ mm$ Corrugation depth

$a_{c2} := \dfrac{a_{c3}}{sin\left(\alpha_c\right)} = 128\ mm$ legnth of hypopythis

$a_{c4} := \dfrac{a_{c3}}{tan\left(\alpha_c\right)} = 103\ mm$ Length of angled part

$s_c := a_{c1} + a_{c2} = 238\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 213\ mm$ Straight length

$\dfrac{s_c}{w_c} = 1.11$ Ratio corrugation/straight length

$c_{uf}\left(t_w, b_{uf}\right) := \dfrac{b_{uf}}{2} + \dfrac{a_{c3}}{2} - \dfrac{t_w}{2} - \sqrt{2} \cdot a_{weld}$

$\varepsilon_{uf} := \sqrt{\dfrac{235}{f_{yuf}} \cdot \dfrac{E_s}{210000}} = 0.71$

E.2 Corrugated web stiffness

E.2.1 Plate buckling of compressive flange corrugated - SS-EN 1993 1-4 5.2.3

$$h_{beam}\left(h_w\right) := h_w + t_{uf} + t_{lf}$$

$$a_{bend} := a_{c1} + 2\ a_{c4}$$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$k_{\sigma u1}\left(t_w, b_{uf}\right) := 0.43 + \left(\frac{c_{uf}\left(t_w, b_{uf}\right)}{a_{bend}}\right)^2$$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$k_{\sigma u2} := 0.6$$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$k_{\sigma u}\left(t_w, b_{uf}\right) := min\left(k_{\sigma u1}\left(t_w, b_{uf}\right), k_{\sigma u2}\right)$$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$$\lambda_{pu}\left(t_w, b_{uf}\right) := \frac{\dfrac{c_{uf}\left(t_w, b_{uf}\right)}{t_{uf}}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_{\sigma u}\left(t_w, b_{uf}\right)}}$$ Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho_u\left(t_w, b_{uf}\right) := \begin{array}{l} \text{if } \lambda_{pu}\left(t_w, b_{uf}\right) \le 0.748 \\ \quad \left\| \; 1 \right. \\ \text{else} \\ \quad \left\| \; min\left(\dfrac{\lambda_{pu}\left(t_w, b_{uf}\right) - 0.188}{\lambda_{pu}\left(t_w, b_{uf}\right)^2}, 1\right) \right. \end{array}$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

$$b_{effu\_corr}\left(t_w, b_{uf}\right) := \rho_u\left(t_w, b_{uf}\right) \cdot b_{uf}$$ SS-EN 1993-1-5 Table 4.1

E.2.2 Stiffness steel beam

$$z_g(h_w, t_w, b_{uf}) := \overrightarrow{\frac{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}(h_w) - \frac{t_{lf}}{2}\right)}{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} + b_{lf} \cdot t_{lf}}}$$

$$I_y(h_w, t_w, b_{uf}) := \overrightarrow{\frac{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf}^3}{12} + b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2} - z_g(h_w, t_w, b_{uf})\right)^2 + \frac{b_{lf} \cdot t_{lf}^3}{12} + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}(h_w) - \frac{t_{lf}}{2} - z_g(h_w, t_w, b_{uf})\right)^2}$$

$$W_{gu\_corr}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{I_y(h_w, t_w, b_{uf})}{z_g(h_w, t_w, b_{uf})}}$$

$$W_{gu\_corr2}(h_w, t_w, b_{uf}) := \overrightarrow{b_{effu\_corr}(t_w, b_{uf}) \cdot t_{uf} \cdot \left(h_w + \frac{t_{uf} + t_{lf}}{2}\right)}$$

Appendix D calculation

$$W_{gl\_corr}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{I_y(h_w, t_w, b_{uf})}{h_{beam}(h_w) - z_g(h_w, t_w, b_{uf})}}$$

$$W_{gl\_corr2}(h_w) := \overrightarrow{b_{lf} \cdot t_{lf} \cdot \left(h_w + \frac{t_{uf} + t_{lf}}{2}\right)}$$

E.2.3 Stiffness composite section

$$A_s(t_w, b_{uf}) := \overrightarrow{t_{uf} \cdot b_{effu\_corr}(t_w, b_{uf}) + t_{lf} \cdot b_{lf}}$$

$$A_{slab.fic} := 1.5\ m^2$$

$$h_{m.slab} := 0.32\ m$$

$$b_{eff\_mid} := 4.85\ m$$

$$A_{sl\_short}(t_w, b_{uf}) := \overrightarrow{A_s(t_w, b_{uf}) + \frac{A_{slab.fic}}{n_{L_2}}}$$

Area of composite section for shortterm loads

$$z_{tp\_short}(h_w, t_w, b_{uf}) := \overrightarrow{\frac{\frac{A_{slab.fic}}{n_{L_2}} \cdot \frac{h_{m.slab}}{2} + t_{uf} \cdot b_{effu\_corr}(t_w, b_{uf}) \cdot \left(h_{m.slab} + \frac{t_{uf}}{2}\right) + t_{lf} \cdot b_{lf} \cdot \left(h_{m.slab} + t_{uf} + h_w + \frac{t_{lf}}{2}\right)}{\frac{A_{slab.fic}}{n_{L_2}} + t_{uf} \cdot b_{effu\_corr}(t_w, b_{uf}) + t_{lf} \cdot b_{lf}}}$$

Centre of gravity from the top

$$I_{y\_short}(h_w,t_w,b_{uf}) := \overrightarrow{I_y(h_w,t_w,b_{uf}) + A_s(t_w,b_{uf}) \cdot \left(\left(z_g(h_w,t_w,b_{uf}) + h_{m.slab}\right) - z_{tp\_short}(h_w,t_w,b_{uf})\right)^2 + \frac{h_{m.slab}{}^3 \cdot \frac{b_{eff\_mid}}{n_{L_2}}}{12} + \frac{A_{slab.fic}}{n_{L_2}} \cdot \left(\frac{h_{m.slab}}{2} - z_{tp\_short}(h_w,t_w,b_{uf})\right)^2}$$

$$W_{comp.corr.u}(h_w,t_w,b_{uf}) := \overrightarrow{\frac{I_{y\_short}(h_w,t_w,b_{uf})}{z_{tp\_short}(h_w,t_w,b_{uf})}}$$

$$W_{comp.corr.l}(h_w,t_w,b_{uf}) := \overrightarrow{\frac{I_{y\_short}(h_w,t_w,b_{uf})}{\left(h_{beam}(h_w) + h_{m.slab}\right) - z_{tp\_short}(h_w,t_w,b_{uf})}}$$

E.3 Flat web stiffness

E.3.1 Plate buckling of compressive flange flat we - SS-EN 1993 1-4 5.2.3

$$c_{uf}(t_w,b_{uf}) := \frac{b_{uf}}{2} - \frac{t_w}{2} - \sqrt{2} \cdot a_{weld}$$

$$k_{\sigma u} := 0.43$$

$$\lambda_{pu}(t_w,b_{uf}) := \frac{\frac{c_{uf}(t_w,b_{uf})}{t_{uf}}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_{\sigma u}}}$$

$$\rho_u(t_w,b_{uf}) := \begin{cases} 1 & \text{if } \lambda_{pu}(t_w,b_{uf}) \le 0.748 \\ min\left(\frac{\lambda_{pu}(t_w,b_{uf}) - 0.188}{\lambda_{pu}(t_w,b_{uf})^2}, 1\right) & \text{else} \end{cases}$$

$$b_{effu}(t_w,b_{uf}) := \rho_u(t_w,b_{uf}) \cdot b_{uf}$$

E.3.2 Stiffness steel beam flat web

$$z_{g\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2}\right) + t_w \cdot h_w \cdot \left(h_{beam}\left(h_w\right) - t_{lf} - \frac{h_w}{2}\right) + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}\left(h_w\right) - \frac{t_{lf}}{2}\right)}{b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf} + b_{lf} \cdot t_{lf} + t_w \cdot h_w}}$$

$$I_{y\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf}^{3}}{12} + b_{effu}\left(t_w, b_{uf}\right) \cdot t_{uf} \cdot \left(\frac{t_{uf}}{2} - z_{g\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2}$$
$$+ \frac{t_w \cdot h_w^{3}}{12} + t_w \cdot h_w \cdot \left(z_{g\_flat}\left(h_w, t_w, b_{uf}\right) - t_{uf} - \frac{h_w}{2}\right)^2$$
$$+ \frac{b_{lf} \cdot t_{lf}^{3}}{12} + b_{lf} \cdot t_{lf} \cdot \left(h_{beam}\left(h_w\right) - \frac{t_{lf}}{2} - z_{g\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2$$

$$W_{gu\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_flat}\left(h_w, t_w, b_{uf}\right)}{z_{g\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

$$W_{gl\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_flat}\left(h_w, t_w, b_{uf}\right)}{h_{beam}\left(h_w\right) - z_{g\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

E.3.3 Stiffness composite section

$$A_{s\_flat}\left(t_w, b_{uf}, h_w\right) := \overrightarrow{t_{uf} \cdot b_{effu}\left(t_w, b_{uf}\right) + t_{lf} \cdot b_{lf} + t_w \cdot h_w}$$

$$A_{sl\_short\_flat}\left(t_w, b_{uf}, h_w\right) := \overrightarrow{A_{s\_flat}\left(t_w, b_{uf}, h_w\right) + \frac{A_{slab.fic}}{n_{L_2}}}$$

Area of composite section for shortterm loads

$$z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{\frac{A_{slab.fic}}{n_{L_2}} \cdot \frac{h_{m.slab}}{2} + t_{uf} \cdot b_{effu}\left(t_w, b_{uf}\right) \cdot \left(h_{m.slab} + \frac{t_{uf}}{2}\right) + t_{lf} \cdot b_{lf} \cdot \left(h_{m.slab} + t_{uf} + h_w + \frac{t_{lf}}{2}\right) + h_w \cdot t_w \cdot \left(t_{uf} + \frac{h_w}{2}\right)}{\frac{A_{slab.fic}}{n_{L_2}} + t_{uf} \cdot b_{effu}\left(t_w, b_{uf}\right) + t_{lf} \cdot b_{lf} + t_w \cdot h_w}}$$

Centre of gravity from the top

$$I_{y\_short\_flat}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{I_{y\_flat}\left(h_w, t_w, b_{uf}\right)}$$
$$+ A_{s\_flat}\left(t_w, b_{uf}, h_w\right) \cdot \left(\left(z_{g\_flat}\left(h_w, t_w, b_{uf}\right) + h_{m.slab}\right) - z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2$$
$$+ \frac{h_{m.slab}^{3} \cdot \frac{b_{eff\_mid}}{n_{L_2}}}{12} + \frac{A_{slab.fic}}{n_{L_2}} \cdot \left(\frac{h_{m.slab}}{2} - z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)\right)^2$$

$$W_{comp.flat.u}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_short\_flat}\left(h_w, t_w, b_{uf}\right)}{z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

$$W_{comp.flat.l}\left(h_w, t_w, b_{uf}\right) := \overrightarrow{\frac{I_{y\_short\_flat}\left(h_w, t_w, b_{uf}\right)}{\left(h_{beam}\left(h_w\right) + h_{m.slab}\right) - z_{tp\_short\_flat}\left(h_w, t_w, b_{uf}\right)}}$$

$$W_{comp.flat}\left(t_w, b_{uf}\right) := \left\| \begin{array}{l} \text{for } i \in 1 .. length\left(h_w\right) \\ \quad \left\| out_i \leftarrow min\left(W_{comp.flat.l}\left(h_{w_i}, t_w, b_{uf}\right), W_{comp.flat.u}\left(h_{w_i}, t_w, b_{uf}\right)\right) \right. \\ out \end{array} \right.$$

$$W_{comp.flat}\left(x, t_w, b_{uf}\right) := linterp\left(h_w, W_{comp.flat}\left(t_w, b_{uf}\right), x\right)$$

E.4 Comparisson Steel cross-section

$$h_w := \left\| \begin{array}{l} \text{for } i \in 1..21 \\ \quad \left\| out_i \leftarrow 1\ m + 0.1\ m \cdot (i-1) \right. \\ out \end{array} \right.$$

$$b_{uf.plot} := \left\| \begin{array}{l} \text{for } i \in 1..81 \\ \quad \left\| out_i \leftarrow 0.5\ m + 0.01\ m \cdot (i-1) \right. \\ out \end{array} \right.$$

W_{gu_corr} Elastic bending stiffness corrugated web- W=I/z

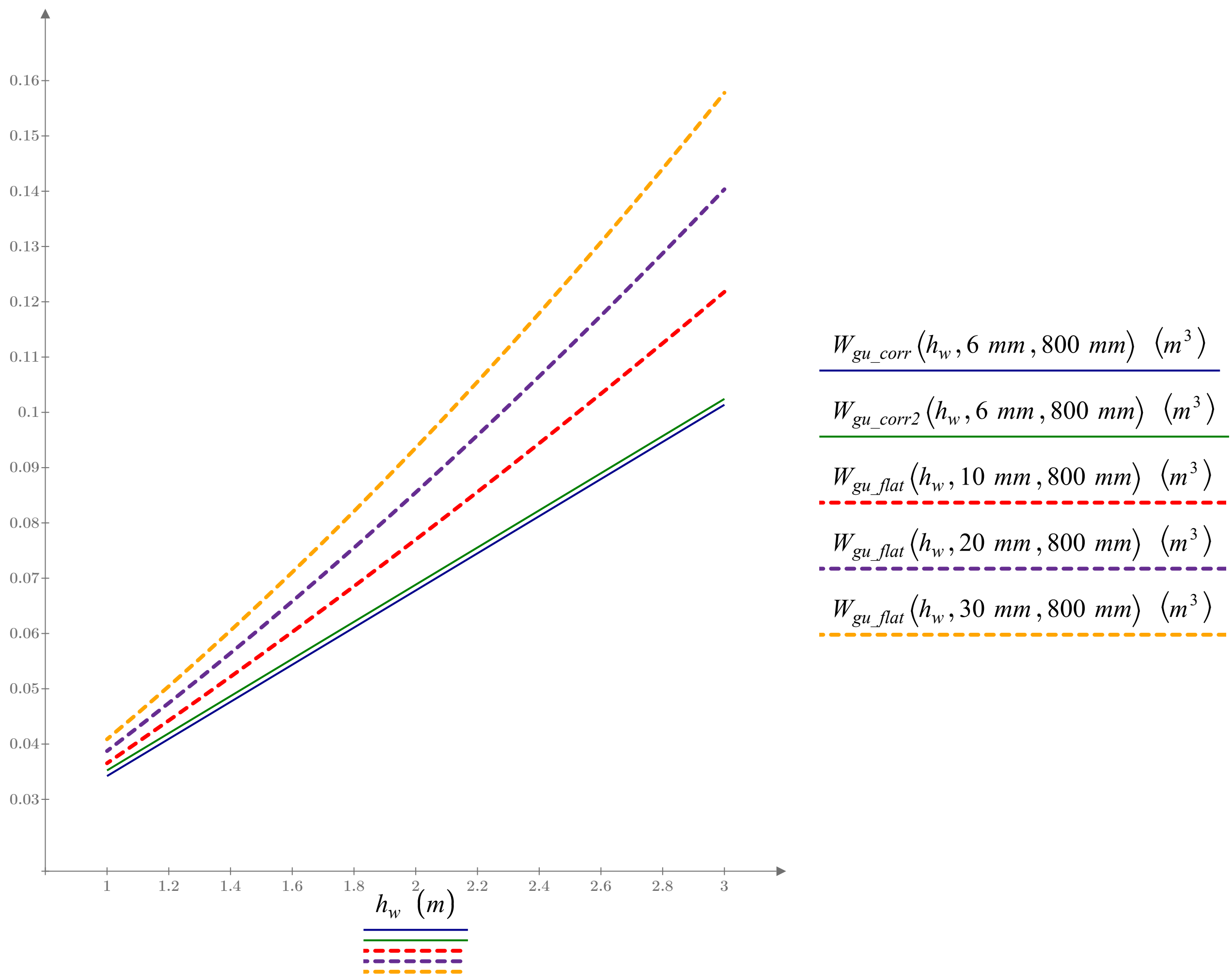
W_{gu_corr2} Elastic bending stiffness corrugated web- SS-EN 1993-1-5 Appendix D calculation

W_{gu_flat} Elastic bending stiffness flat web- W=I/z

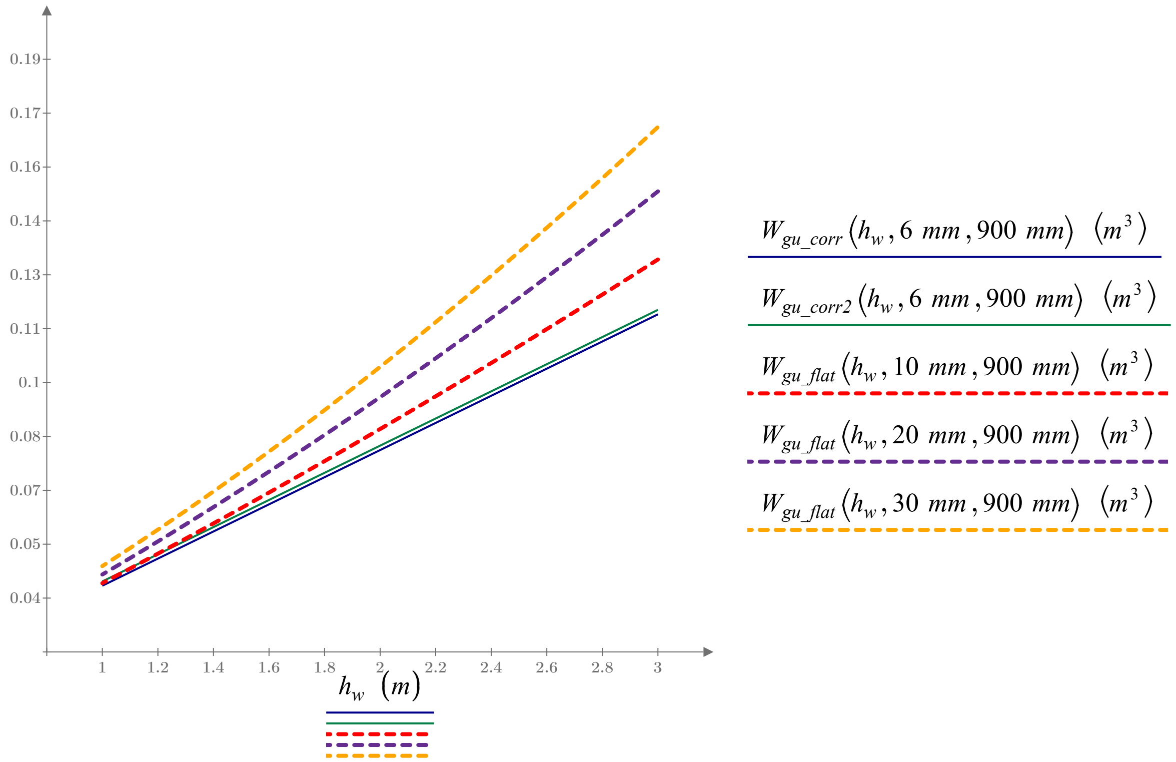
- - - - - - = Flat Web

________ = Corrugated web

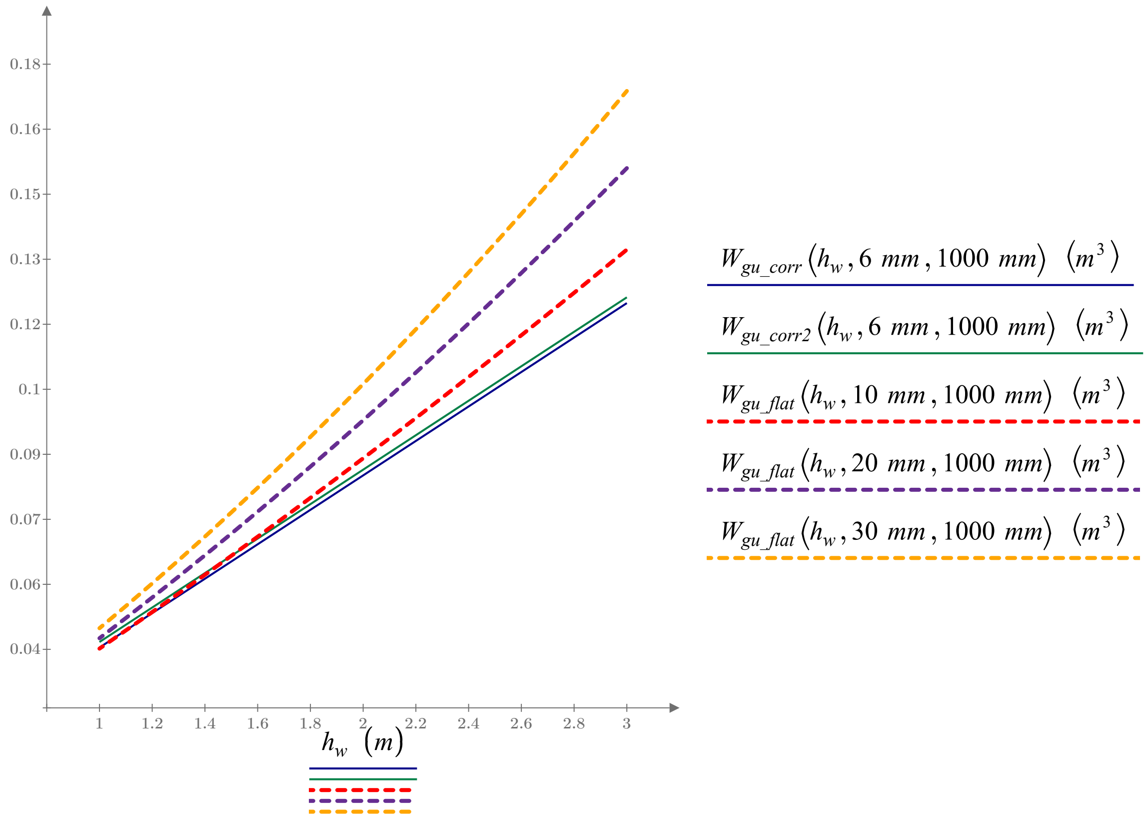
Flange width 800mm



Flange width 900mm



Flange width 1000mm



E.4.1 Comparisson Steel cross-section, deviation

Web thickness 10mm

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(1\ m, 10\ mm, 700\ mm)} = 8\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(1\ m, 10\ mm, 800\ mm)} = 6\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(1\ m, 10\ mm, 900\ mm)} = 2\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(2\ m, 10\ mm, 700\ mm)} = 14\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(2\ m, 10\ mm, 800\ mm)} = 12\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(2\ m, 10\ mm, 900\ mm)} = 7\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(3\ m, 10\ mm, 700\ mm)} = 19\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(3\ m, 10\ mm, 800\ mm)} = 17\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(3\ m, 10\ mm, 900\ mm)} = 12\%$$

Web thickness 16mm

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(1\ m, 16\ mm, 700\ mm)} = 11\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(1\ m, 16\ mm, 800\ mm)} = 10\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(1\ m, 16\ mm, 900\ mm)} = 5\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(2\ m, 16\ mm, 700\ mm)} = 20\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(2\ m, 16\ mm, 800\ mm)} = 18\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(2\ m, 16\ mm, 900\ mm)} = 13\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(3\ m, 16\ mm, 700\ mm)} = 27\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(3\ m, 16\ mm, 800\ mm)} = 24\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(3\ m, 16\ mm, 900\ mm)} = 19\%$$

---Web thickness 20mm---

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(1\ m, 20\ mm, 700\ mm)} = 14\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(1\ m, 20\ mm, 800\ mm)} = 12\%$$

$$\eta_1 := 1 - \frac{W_{gu\_corr}(1\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(1\ m, 20\ mm, 900\ mm)} = 8\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(2\ m, 20\ mm, 700\ mm)} = 24\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(2\ m, 20\ mm, 800\ mm)} = 21\%$$

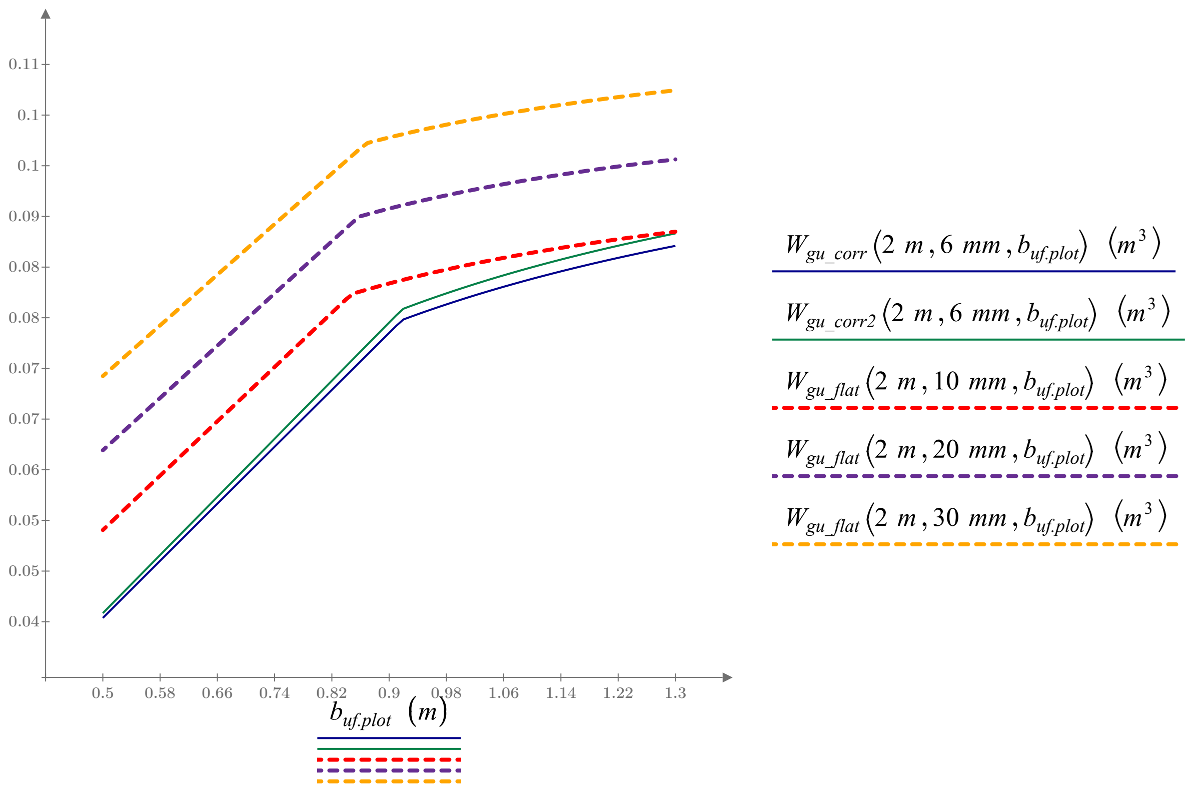
$$\eta_2 := 1 - \frac{W_{gu\_corr}(2\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(2\ m, 20\ mm, 900\ mm)} = 16\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 700\ mm)}{W_{gu\_flat}(3\ m, 20\ mm, 700\ mm)} = 31\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 800\ mm)}{W_{gu\_flat}(3\ m, 20\ mm, 800\ mm)} = 28\%$$

$$\eta_2 := 1 - \frac{W_{gu\_corr}(3\ m, 6\ mm, 900\ mm)}{W_{gu\_flat}(3\ m, 20\ mm, 900\ mm)} = 23\%$$

E.4.2 Comparisson Steel cross-section, flange width

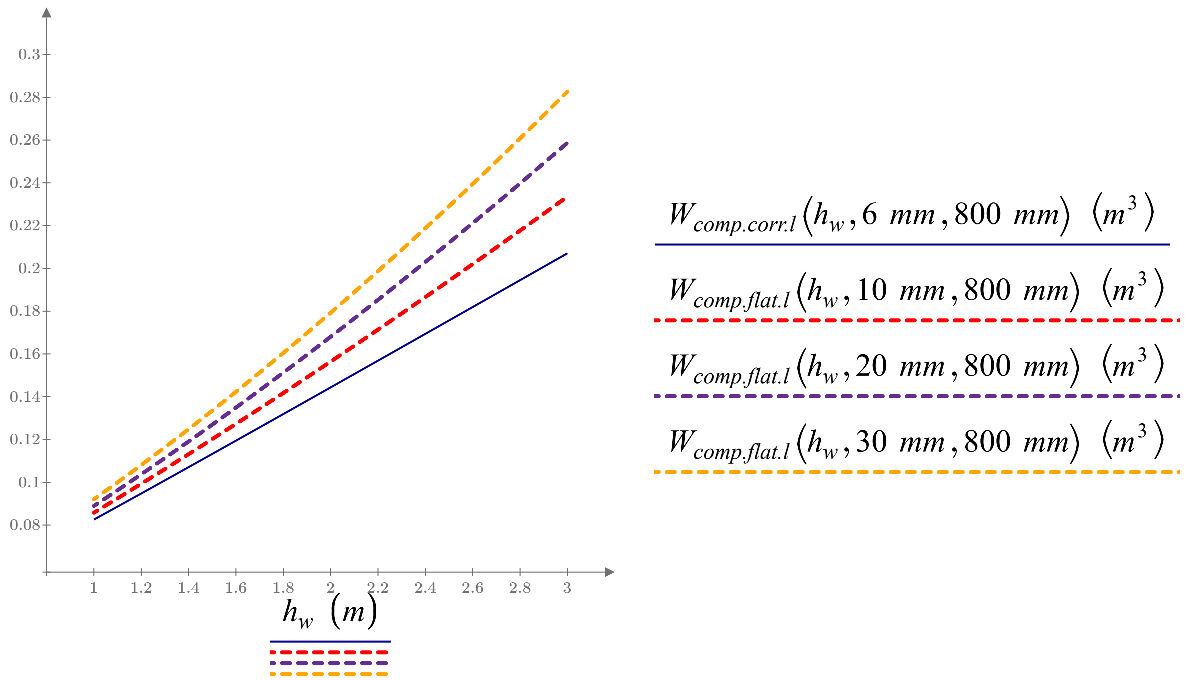


E.5 Comparisson composite cross-section

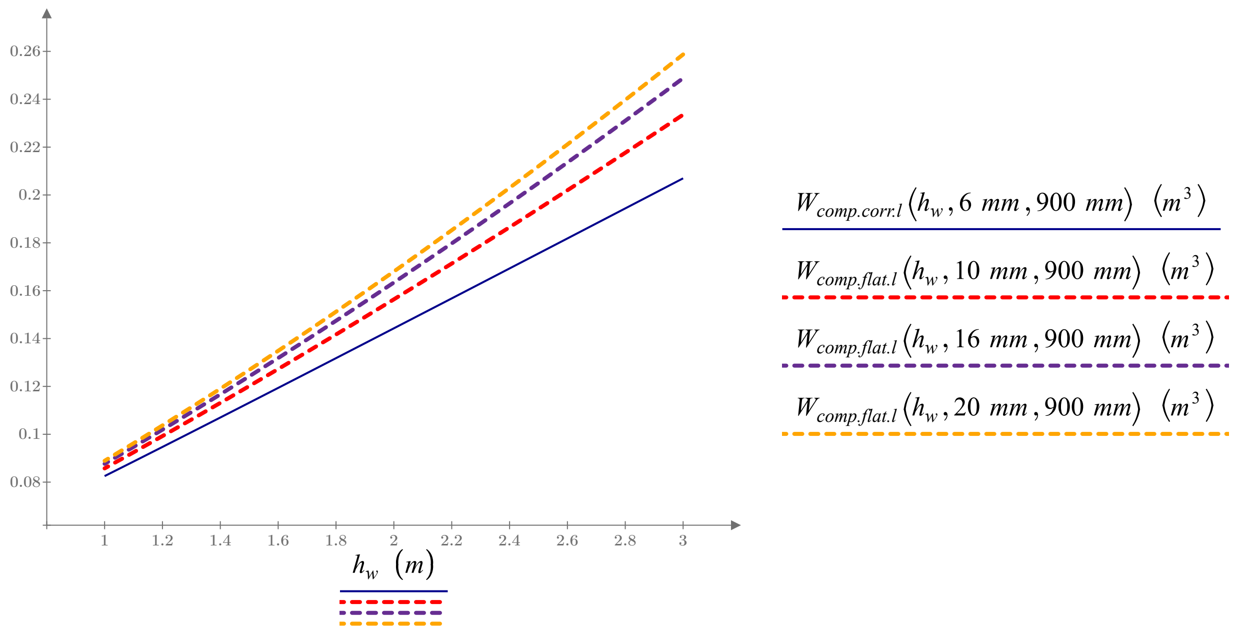
$W_{comp.corr.l}$ Elastic bending stiffness corrugated web

$W_{comp.flat.l}$ Elastic bending stiffness flat web

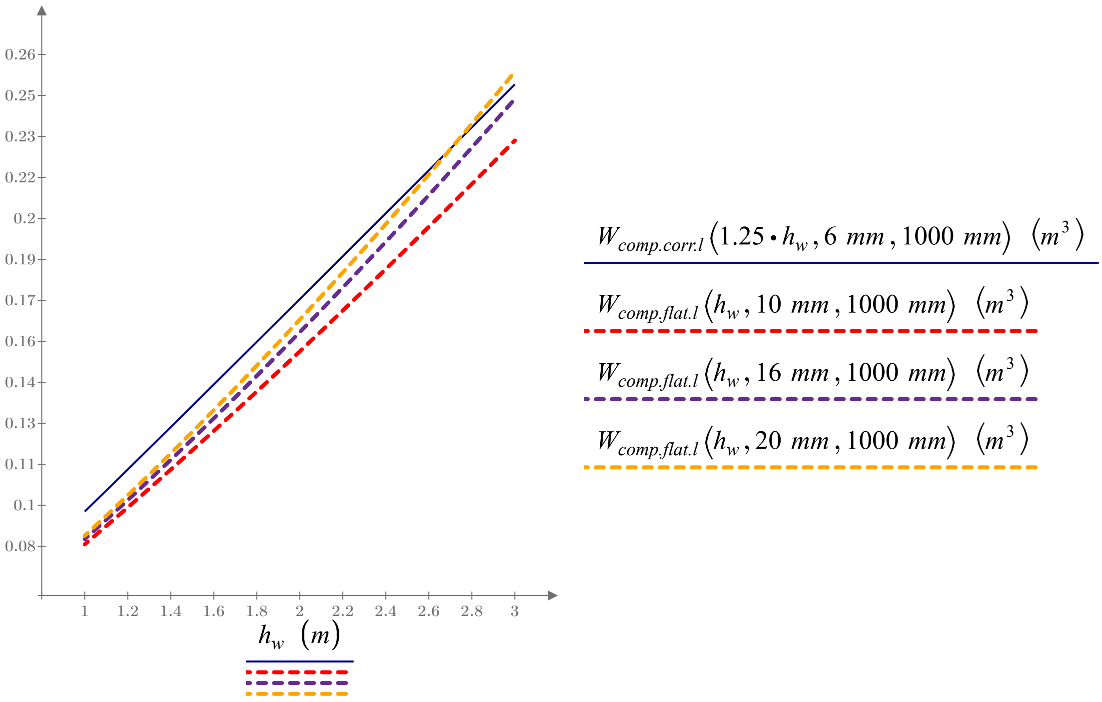
Flange width 800mm



Flange width 900mm



Flange width 1000mm



E.5.1 Comparisson composite cross-section, deviation

$$\eta_{1\_comp} := 1 - \frac{W_{comp.corr.l}(1\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(1\ m, 10\ mm, 800\ mm)} = 4\%$$

$$\eta_{2\_comp} := 1 - \frac{W_{comp.corr.l}(2\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(2\ m, 10\ mm, 800\ mm)} = 8\%$$

$$\eta_{3\_comp} := 1 - \frac{W_{comp.corr.l}(3\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(3\ m, 10\ mm, 800\ mm)} = 12\%$$

$$\eta_{1\_comp} := 1 - \frac{W_{comp.corr.l}(1\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(1\ m, 16\ mm, 800\ mm)} = 6\%$$

$$\eta_{2\_comp} := 1 - \frac{W_{comp.corr.l}(2\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(2\ m, 16\ mm, 800\ mm)} = 12\%$$

$$\eta_{3\_comp} := 1 - \frac{W_{comp.corr.l}(3\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(3\ m, 16\ mm, 800\ mm)} = 17\%$$

$$\eta_{1\_comp} := 1 - \frac{W_{comp.corr.l}(1\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(1\ m, 20\ mm, 800\ mm)} = 8\%$$

$$\eta_{2\_comp} := 1 - \frac{W_{comp.corr.l}(2\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(2\ m, 20\ mm, 800\ mm)} = 15\%$$

$$\eta_{3\_comp} := 1 - \frac{W_{comp.corr.l}(3\ m, 6\ mm, 800\ mm)}{W_{comp.flat.l}(3\ m, 20\ mm, 800\ mm)} = 20\%$$

F

Calculation report - bridge 100-262-1

1 Prerequisites

In order to show the potential of replacing conventional carbon steel for stainless steel and implementing the procedure of using corrugated web a redesign of an existing structure is carried out. The project is a part of the master thesis: *Design of composite steel-concrete brdiges using Stainless steel girders with corrugated web*.

The work is carried out at WSP's Bridge and Hydraulic Department in Gothenburg.

The studied bridge type is composite steel- concrete bridges.

1.1 Guidelines and Design codes

The governing guidlines and design codes are:

- Krav Brobyggande v3.0
- Råd Brobyggande v3.0
- TSFS 2018:57 (TRVFS 2011:12)
- SS-EN 1990
- SS-EN 1991-1-1
- SS-EN 1991-1-6
- SS-EN 1991-2
- SS-EN 1992-1-1
- SS-EN 1992-2
- SS-EN 1993-1-1
- SS-EN 1993-1-4
- SS-EN 1993-1-5
- SS-EN 1993-1-8
- SS-EN 1993-1-9
- SS-EN 1993-2

1.2 Software

The design of the bridge is carried out in the following programs:

- Mathcad Prime	Design of cross-section
- Stripstep	System analysis

1.3 Safety class and partial safety factor

The bridge is based on the risk for physical, economical or social loss in case of a failiure placed in a saftety class.

$safety\_class := 3$ Safety class of the bridge, TSFS 2018:57

$\gamma_d(safety\_class) = 1.00$ Partial safety factor TSFS 2018:57

2 Material

2.1 Stainless steel girder

The choosen steel grade is duplex stainless steel 1.4162. Material strengths are presented below for different thicknesses.

$f_{y\_8mm} = 530\ MPa$	Proof strength for $t \le 8\ mm$ - SS- EN 1993-1-4:2006/A1:2015
$f_{u\_8mm} = 700\ MPa$	Ultimate strength for $t \le 8\ mm$ - SS- EN 1993-1-4:2006/A1:2015
$f_{y\_13.5mm} = 480\ MPa$	Proof strength for $t \le 13.5\ mm$ - SS- EN 1993-1-4:2006/A1:2015
$f_{u\_13.5mm} = 680\ MPa$	Ultimate strength for $t \le 13.5\ mm$ - SS- EN 1993-1-4:2006/A1:2015
$f_{y\_75mm} = 450\ MPa$	Proof strength for $t \le 75\ mm$ - SS- EN 1993-1-4:2006/A1:2015
$f_{u\_75mm} = 650\ MPa$	Ultimate strength for $t \le 75\ mm$ - SS- EN 1993-1-4:2006/A1:2015

For duplex steel 1.4162 the limits for the proof- and ultimate strength is differ from the other grades. Below yield strengths should be chosen for thin plates.

$f_{y\_6.4mm} := 530\ MPa$	Proof strength for $t \le 6.4\ mm$ - SS- EN 1993-1-4:2006/A1:2015
$f_{y\_10mm} := 480\ MPa$	Proof strength for $t \le 10\ mm$ - SS- EN 1993-1-4:2006/A1:2015

$f_{yuf} := f_{y\_75mm} = 450\ MPa$	$f_{uuf} := f_{u\_75mm} = 650\ MPa$	Choosen strength in upper flange
$f_{ylf} := f_{y\_75mm} = 450\ MPa$	$f_{ulf} := f_{u\_75mm} = 650\ MPa$	Choosen strength in lower flange

The yield strength for the web is chosen manually dependent on the thickness of the web.

$E_s = 200\ GPa$	Modulus of elasticity - SS-EN 1993-1-4 2.1.3 (1)
$\gamma_{M0} := 1.0$	Partial coefficient considering the resistance of the cross-section -TSFS 18 ch 2 §
$\gamma_{M1} := 1.0$	Partial coefficient considering the resistance of members to instability -TSFS 18 ch 2 §
$\gamma_{M2} := 1.2$	Partial coefficient considering the resistance of the cross-section in tension to fracture -TSFS 18 ch 2 §

2.1.1 Studs, screws and bolts

Dependent on the choosen steel grade the fasteners are subcategorised into specific groups. For steel grade 1.4162 the valid group is D4. The group then gives the nominal strength values dependent on a choosen property class.

$SteelGrade = \text{“1.4162”}$ — Steel grade

$Group = \text{“D4”}$ — Group

$PropertyClass = \text{“80”}$ — Choosen property class

$f_{ub} = 800\ MPa$ — Ultimate strength of fastener - Stålbyggnadsinstitutet (2017)

$f_{yb} = 600\ MPa$ — Permanent 0.2- proof stength - Stålbyggnadsinstitutet (2017)

$\gamma_{M2} := 1.2$ — Partial factor for calculation of resistance in a connection - TSFS 18 ch 2§

$\gamma_V := 1.25$ — Partial factor for calculation of resistance in shear studs - SS-EN 1994-1-1 2.4.1.2 (5)

2.1.2 Welds

<u>Characteristic values for the ultimate strength</u>

$f_{u.W.par} := min\left(f_{uuf}, f_{ulf}, f_{u\_13.5mm}\right) = 650\ MPa$ — Ultimate strength for parent material in weld (taken for annealed steel, $t > 8\ mm$) - SS- EN 1993-1-4:2006/A1:2015 "12 Modification to 6.3, Design of Welds")

$f_{u.W.fill} := max\left(f_{uuf}, f_{ulf}\right) = 650\ MPa$ — Ultimate strength of filler material for duplex steel - Stålbyggnadsinstitutet (2017)

<u>Design resistance of fillet welds</u>

The design resistance for fillet welds are calculated according to SS-EN 1993-1-8 4.5.3.

$\beta_w := 1.0$ — Correlation factor - SS-EN 1993-1-4 6.3

$f_{u.FW} := min\left(f_{u.W.par}, f_{u.W.fill}\right) = 650\ MPa$ — Ultimate strength of the weaker part joined

$$f_{FWd\_T} := min\left(\frac{f_{u.FW}}{\beta_w \cdot \gamma_{M2}}, \frac{0.9 \cdot f_{u.FW}}{\gamma_{M2}}\right) = 487.5\ MPa$$

SS-EN 1993-1-8, Equation (4.1)

$$f_{FWd\_//} := \frac{f_{u.FW}}{\beta_w \cdot \gamma_{M2} \cdot \sqrt{3}} = 312.7\ MPa$$

Design shear strength - SS- EN 1993-1-8, Equation (4.4)

Design resistance of butt welds (full penetration)

The design resistance for butt welds is taken the same as the weaker of the parts connected.

$f_{BWd} := min\left(f_{u.W.par}, f_{u.W.fill}\right) = 650\ MPa$ Design resistance for butt welds - SS- EN 1993-1-8, 4.7.1 (1)

2.2 Concrete slab

The concrete slab is not designed in this calculation report. However some parameters are necessary in order to determine factors for phenomenom such as shrinkage and creep.

2.2.1 Concrete

The choosen concrete class is C35/45.

$f_{ck}=35\ MPa$ Compressive strength - SS- EN 1992-1-1 Table 3.1

$f_{cm}=43\ MPa$ Mean compressive strength - SS- EN 1992-1-1 Table 3.1

$f_{ctm}=3.2\ MPa$ Mean tensile strength - SS- EN 1992-1-1 Table 3.1

$f_{ct0.05}=2.2\ MPa$ 5% fractile tensile strength - SS- EN 1992-1-1 Table 3.1

$E_{cm}=34\ GPa$ Youngs modulus - SS- EN 1992-1-1 Table 3.1

$\gamma_c:=1.5$ Partial safety factor concrete (Permanent and variable loads) - SS- EN 1992-1-1 Table .2.1N

$\gamma_{cE}:=1.2$ Partial safety factor concrete (Accidental loads) - SS- EN 1992-1-1 Table 2.1N

$\alpha_{cc}:=1.0$ Accounting for longterm effects on the compressive strength - TSFS 2018:57 14 ch 3 §

$\alpha_{ct}:=1.0$ Accounting for longterm effects on the tensile strength - TSFS 2018:57 14 ch 3 §

$f_{cd}:=\alpha_{cc}\cdot\frac{f_{ck}}{\gamma_c}=23.3\ MPa$ Design compressive stength

$f_{ctd}:=\alpha_{ct}\cdot\frac{f_{ct0.05}}{\gamma_c}=1.47\ MPa$ Design tensile stength

2.2.1.1 Creep function

$RH := 80$ — [%] - Relative humidity in the ambient air

t_0 — [days] - Time for first loading

$u := 2\ B + 2\ h_{m.slab} = 20.340\ m$ — Is the perimeter of the cross-section in contact with the atmosphere, see chapter 3

$A_c = 3.472\ m^2$ — Area of full slab section - chapter 3

$$h_0 := \frac{2 \cdot A_c}{u} = 341\ mm$$
SS- EN 1992-1-1 B.1 (B.6)

$$\alpha_1 := \left(\frac{35\ MPa}{f_{cm}}\right)^{0.7} = 0.87$$
SS- EN 1992-1-1 B.1 (B.8c)

$$\alpha_2 := \left(\frac{35\ MPa}{f_{cm}}\right)^{0.2} = 0.96$$
SS- EN 1992-1-1 B.1 (B.8c)

$$\varphi_{RH} := \begin{cases} \text{if } f_{cm} > 35\ MPa \\ \quad \left(1 + \dfrac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{\frac{h_0}{mm}}} \cdot \alpha_1\right) \cdot \alpha_2 \\ \text{else} \\ \quad \left(1 + \dfrac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{\frac{h_0}{mm}}}\right) \end{cases} = 1.20$$
SS- EN 1992-1-1 B.1 (B.3a,b)

$$\beta_{fcm} := \frac{16.8}{\sqrt{\frac{f_{cm}}{MPa}}} = 2.6$$
SS- EN 1992-1-1 B.1 (B.4)

$$\beta_{t0}\left(t_0\right) := \frac{1}{0.1 + \left(\frac{t_0}{day}\right)^{0.2}}$$
SS- EN 1992-1-1 B.1 (B.5)

$$\varphi_0\left(t_0\right) := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0}\left(t_0\right)$$
Nominal creep value - SS- EN 1992-1-1 B.1 (B.2)

$\varphi_{\infty\_cs} := \varphi_0(1\ day) = 2.79$ — Final creep value for shrinkage (t= 1 day) - SS-EN 1994-2 5.4.2.2 (4)

$\varphi_{\infty\_perm} := \varphi_0(7\ day) = 1.95$ — Final creep value for permanent loads (t= 7 day) - SS-EN 1994-2 5.4.2.2

2.2.2 Reinforcement

The used reinforcement is K500C-T. The modulus of elasitcity for the reinforcement is assumed to be same as the structural steel in the girders according to SS-EN 1994-2, 3.2 (2).

$f_{yk} := 500\ MPa$	SS- 212540:2014 Table C.2
$E_{rs} := E_s = 200\ GPa$	SS- EN 1994-2 3.2(2)
$\gamma_s := 1.15$	Permanent and variable loads SS- EN 1992-1-1 Table 2.1N
$\gamma_{sfat} := 1.15$	Fatigue loading SS- EN 1992-1-1 Table 2.1N
$\gamma_{sA} := 1.0$	Accidental loading SS- EN 1992-1-1 Table 2.1N
$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435\ MPa$	Design yield strength - SS-EN 1990 6.3.3

2.2.3 Modular ratios for linear elastic analysis

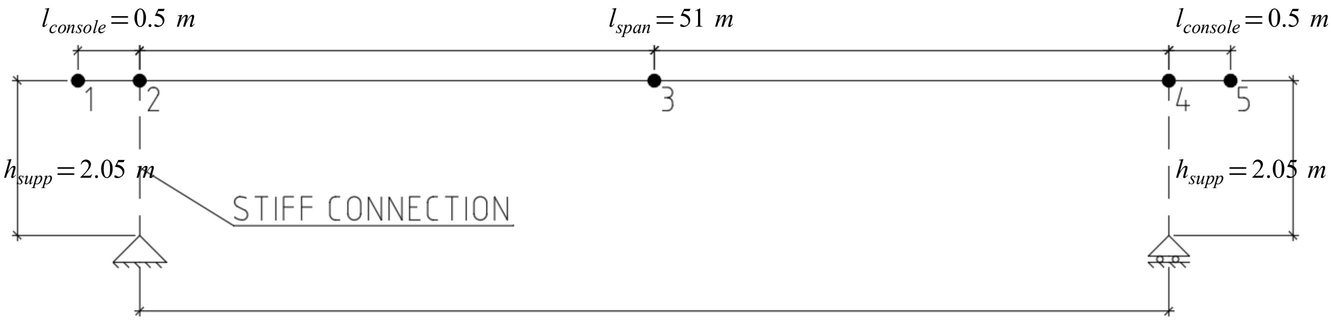
Calculated according to SS-EN 1994-2, 5.4.2.2, Equation (5.6).

$n_0 := \frac{E_s}{E_{cm}} = 5.88$	Modular ratio between stainless steel and concrete
$\varphi_{L\_perm} := 1.1$	Creep factor depending on the load durance for permanent loads - SS- EN 1994-2 5.4.2.2
$\varphi_{L\_cs} := 0.55$	Creep factor depending on the load durance for shrinkage - SS- EN 1994-2 5.4.2.2
$n_{L\_short} := n_0 \cdot (1 + 0 \cdot 0) = 5.9$	Modular ratio for short-term loads and temperature
$n_{L\_perm} := n_0 \cdot \left(1 + \varphi_{L\_perm} \cdot \varphi_{\infty\_perm}\right) = 18.5$	Modular ratio for permanent loads (excl. shrinkage)
$n_{L\_cs} := n_0 \cdot \left(1 + \varphi_{L\_cs} \cdot \varphi_{\infty\_cs}\right) = 14.9$	Modular ratio for shrinkage

3 System

In the following chapter the input data for the bridge's geometries is presented. The cross-sectional parameters that are presented is preliminary and used for the system analysis. In chapter 5 the final design is presented.

3.1 Primary system - longitudinal



The bridge is modelled as a simply supported bridge in the software StripStep-2. Due to the beams depth the supports are set offset from the neutral axis. This is modelled with a stiff connection in the system analysis.

3.1.1 Cross-section dimensions

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

__Element 1__

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	670	9	1967	48	850
500	35	670	9	1967	48	850

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	670	9	1967	48	850
10500	35	670	9	1967	48	850
10500	42	670	7	1953	55	850
11300	42	800	7	1953	55	1180
25500	42	800	7	1953	55	1180

Important! Two exactly the same values will not work with the linterp- function. Therefore 0.1 millimeter must be added for a X-value where you want to cross-sectional properties at the same time.

Element 3

S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	670	9	1967	48	850
500	35	670	9	1967	48	850

$$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=2050\ mm$$

$$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=2050\ mm$$

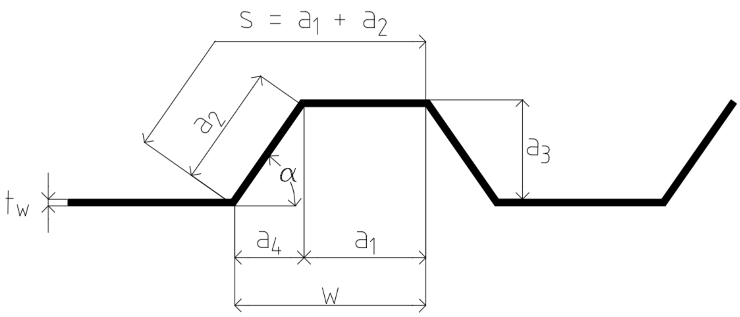
Check to see that the depth of the girder is kept constant

3.1.2 Corrugation shape

$a_{c1}:=170\ mm$ Flat-fold length

$\alpha_c:=36\ deg$ Corrugation angle

$a_{c3}:=100\ mm$ Corrugation depth



$$a_{c2}:=\frac{a_{c3}}{\sin\left(\alpha_c\right)}=170\ mm$$ Length of angled part

$$a_{c4}:=\frac{a_{c3}}{\tan\left(\alpha_c\right)}=138\ mm$$ Length of hypopythis

$s_c:=a_{c1}+a_{c2}=340\ mm$ Length of corrugation

$w_c:=a_{c1}+a_{c4}=308\ mm$ Straight length

$$r_c:=\frac{s_c}{w_c}=1.11$$ Ratio corrugation/flat-fold length

3.1.3 Effective width

3.1.3.1 Effective width, concrete slab

The effective width at mid-span for the concrete flanges is calculated according to SS-EN 1994-1-1 5.4.1.2.

$b_{eff} = b_0 + \sum b_{ei}$ Principal for calculating the effective width - SS- EN 1994-1-1 Equation (5.1)

whereas,

b_0 is the distance between centres of the outstand shear connector

b_{ei} is the value of the effective width of the concrete flange on each side of the web

The effective width at end support for the concrete flanges is calculated according to SS-EN 1994-1-1 5.4.1.2.

$b_{eff} = b_0 + \sum \beta_i b_{ei}$ Principal for calculating the effective width - SS- EN 1994-1-1 Equation (5.4)

whereas,

β_i is an reduction factor according to Equation (5.5)

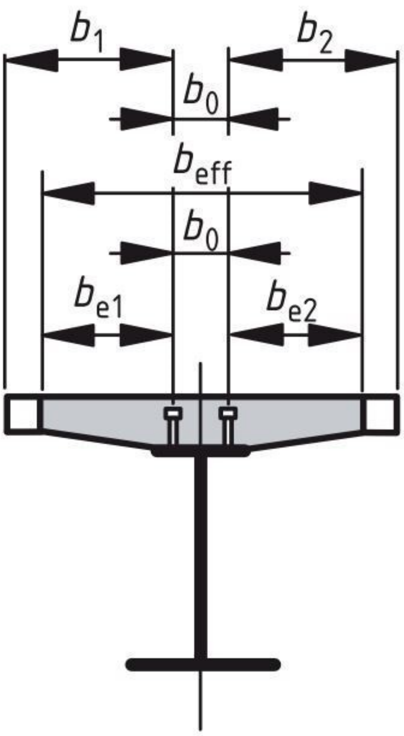
Mid-span

$b_{0\_mid} := 400\ mm$ Distance between studs

$L_e := L = 51\ m$ For simply supported bridges

$B_{out} = 2.525\ m$ Distance between edge beam and centre of web

$B_{in} = 2.800\ m$ Distance between centre line of the bridge and centre of web



$$b_{e\_mid_1} := min\left(\frac{L_e}{8}, B_{out} - \frac{b_{0\_mid}}{2}\right) = 2.325\ m$$

$$b_{e\_mid_2} := min\left(\frac{L_e}{8}, B_{in} - \frac{b_{0\_mid}}{2}\right) = 2.600\ m$$

$$b_{eff\_mid} := min\left(b_{0\_mid} + \sum b_{e\_mid}, \frac{B}{2}\right) = 4.925\ m$$

Support

$b_{0\_supp} := 330\ mm$ — Distance between studs

$L_e = 51\ m$ — For simply supported bridges

$B_{out} = 2.525\ m$ — Distance between edge beam and centre of web

$B_{in} = 2.800\ m$ — Distance between centre line of the bridge and centre of web

$$b_{e\_supp_1} := min\left(\frac{L_e}{8}, B_{out} - \frac{b_{0\_supp}}{2}\right) = 2.36\ m$$

$$b_{e\_supp_2} := min\left(\frac{L_e}{8}, B_{in} - \frac{b_{0\_supp}}{2}\right) = 2.635\ m$$

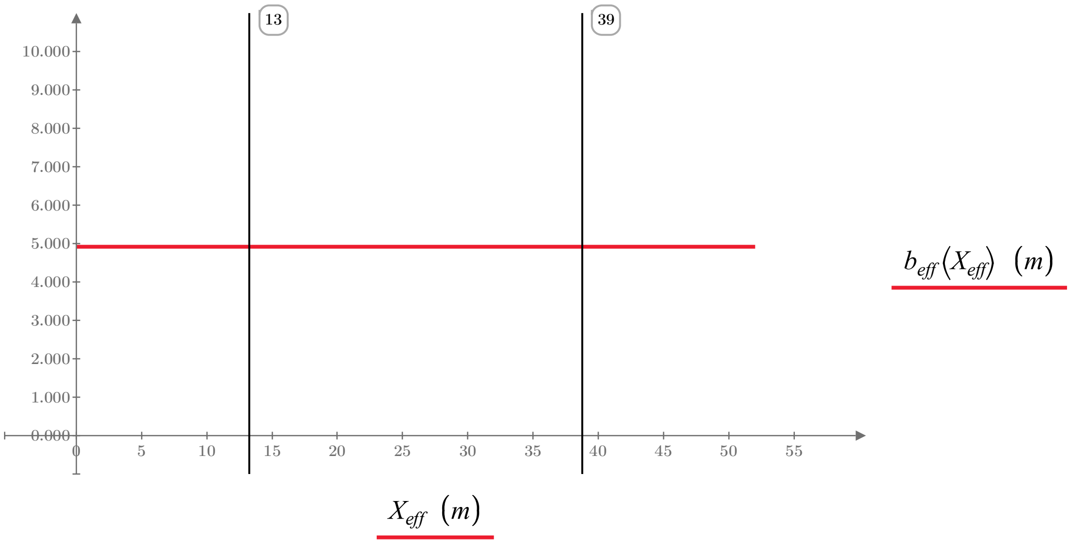
$$\beta_1 := min\left(\left(0.55 + 0.025 \cdot \frac{L_e}{b_{e\_supp_1}}\right), 1.0\right) = 1.0$$ Equation (5.5)

$$\beta_2 := min\left(\left(0.55 + 0.025 \cdot \frac{L_e}{b_{e\_supp_2}}\right), 1.0\right) = 1.0$$ Equation (5.5)

$\frac{B}{2} = 4925\ mm$ — Half the bridge width (the calculations is done for half the cross-section due to symmetry)

$b_{eff\_supp} := min\left(b_{0\_supp} + \sum \overrightarrow{\beta \cdot b_{e\_supp}}, \frac{B}{2}\right) = 4.925\ m$ — Effective width (can not be larger than the width of the bridge)

$L_{trans1} := \frac{L}{4} + max\left(S_{el1}\right) = 13.250\ m$ Transition length for effective width (end span to midspan)



3.1.3.2 Effective width, steel flanges

Calculation of the effective width in the steel flanges with regards to shear lag is calculated according to SS-EN 1993-1-5 3.2.

$$b_{eff\_f} = \beta \cdot b_0 + \frac{t_w}{2}$$

Effective width with regards to shear lag under elastic conditions - Equation (3.1)

$$X_{check\_uf} := \frac{L_{bridge}}{2} = 26.0\ m$$

$$X_{check\_lf} := 0.2\ L_{bridge} = 10.4\ m$$

<u>Upper flange</u>

$$\alpha_0 := 1.0$$

Equation given in Table 3.1. For webs without any longitudinal stiffeners $\alpha_0 = 1.0$.

$$bfu := b_{fu}\left(X_{check\_uf}\right) = 800\ mm$$

Width of upper flange - see Appendix B - Preliminary sizing

$$tw := t_w\left(X_{check\_uf}\right) = 7\ mm$$

Thickness of web - see Appendix B - Preliminary sizing

$$b0 := \frac{bfu - tw}{2} = 397\ mm$$

Figure 3.2 (Notations for shear lag)

$$\kappa fu := \frac{\alpha_0 \cdot b0}{L_{span}} = 0.008 \quad = \quad \kappa_{fu}\left(X_{check\_uf}\right) = 0.008$$

Table 3.1

$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fu \le 0.02 \\ \quad \| \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fu \le 0.70 \\ \quad \left\| \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fu^2} \right. \\ \text{else if } 0.70 < \kappa fu \\ \quad \left\| \beta \leftarrow \dfrac{1}{8.6\ \kappa fu^2} \right. \end{array} \right| = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for upper flange

$$2\ \beta_{check} \cdot b0 + tw = 800\ mm$$

$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad \| b \leftarrow bfu \\ \text{else} \\ \quad \| b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfu\right) \\ b \end{array} \right| = 0.800\ m \quad = \quad b_{eff\_fu}\left(X_{check\_uf}\right) = 0.800\ m$$

<u>Lower flange</u>

$bfl := b_{fl}\left(X_{check\_lf}\right) = 850\ mm$ Width of lower flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_lf}\right) = 9\ mm$ Thickness of web - see Appendix B - Preliminary sizing

$b0 := \dfrac{bfl - tw}{2} = 421\ mm$ Figure 3.2 (Notations for shear lag)

$\kappa fl := \dfrac{\alpha_0 \cdot b0}{L_{span}} = 0.008$ = $\kappa_{fl}\left(X_{check\_lf}\right) = 0.008$ Table 3.1

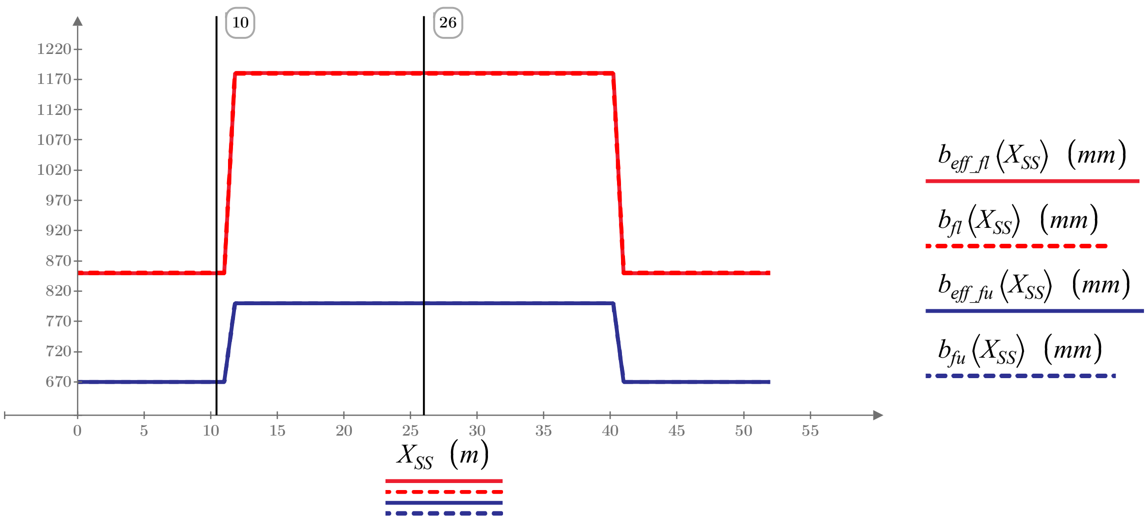
$$\beta_{check} := \begin{array}{|l} \text{if } \kappa fl \le 0.02 \\ \quad \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fl \le 0.70 \\ \quad \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fl^2} \\ \text{else if } 0.70 < \kappa fl \\ \quad \beta \leftarrow \dfrac{1}{8.6\ \kappa fl^2} \end{array} = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for lower flange

$2\ \beta_{check} \cdot b0 + tw = 850\ mm$

$$b_{ef} := \begin{array}{|l} \text{if } \beta_{check} = 1.0 \\ \quad b \leftarrow bfl \\ \text{else} \\ \quad b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfl\right) \\ b \end{array} = 0.850\ m$$ = $b_{eff\_fl}\left(X_{check\_lf}\right) = 0.850\ m$



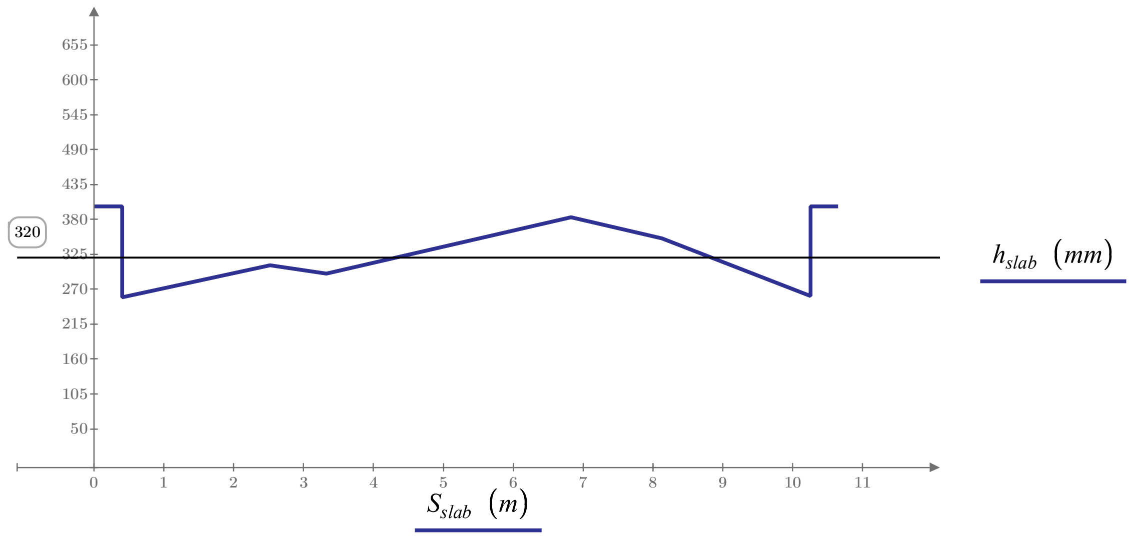
3.1.4 Cross-sectional constants for system analysis

For the system analysis an equivalent cross-section is calculated converting the concrete slab into steel.

3.1.4.1 Concrete slab

The thickness of the concrete slab varies over the width of the bridge. For calculation of the self-weight the different heights of the slab is integrated over the width, see the graph below.

h_{slab}	Height of slab over the transverse direction
S_{slab}	Transverse distance



$$A_{slab} = \sum_i \frac{\left(h_{slab_i} + h_{slab_{i+1}}\right)}{2} \cdot \left(S_{slab_{i+1}} - S_{slab_i}\right)$$ Principal for calculating the area of the slab

$A_{slab} = 3.472\ m^2$ Area of concrete slab

The stiffness of the concrete slab is calculated for the mean thickness, excluding the edge beam.

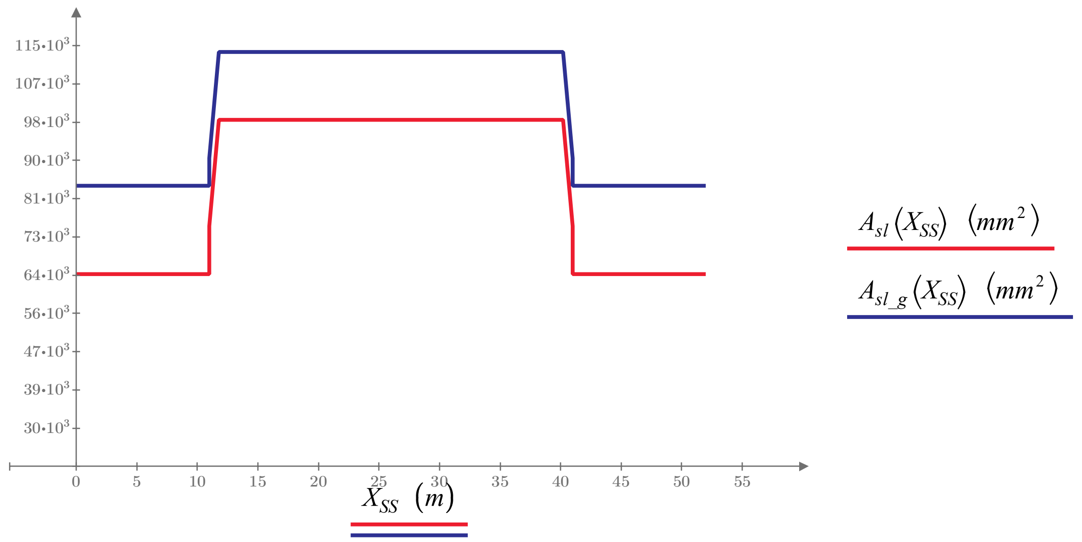
$h_{m.slab} = 320\ mm$ Mean slab height (includes height of heel)

$h_{klack} = 25\ mm$ Height of heel (for concrete slab)

3.1.4.2 Cross-sectional constants during construction

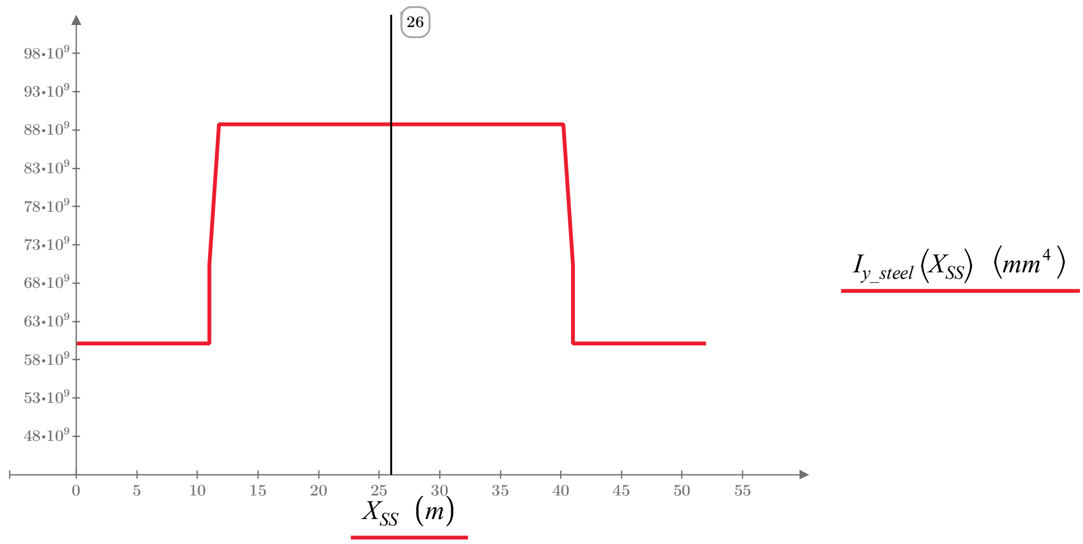
The bridge is checked so that the steel girder (alone) can withstand the loads that are imposed during construction such as the self-weight of curing concrete.

A_{sl_g} is the area of the steel including the web, for calculation of self-weight
A_{sl} is the area of the steel excluding the web, for stiffness calculations



$$A_{sl}\left(\frac{L_{bridge}}{2}\right) = 0.099\ m^2$$ Area at mid-span

I_{y_steel} is the area of the steel excluding the web, for stiffness calculations



$$I_{y\_steel}\left(\frac{L_{bridge}}{2}\right) = 0.0887\ m^4$$ Stiffness at mid-span

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [10^4m^4]	A [10^4m^2]	g [kN/m]
0.000	-1.293	2.050	600.824	643	6.330
0.500	-1.293	2.050	600.824	643	6.330

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [10^4m^4]	A [10^4m^2]	g [kN/m]
0.000	-1.293	2.050	600.824	643	6.330
10.500	-1.293	2.050	600.824	643	6.330
10.500	-1.270	2.050	703.868	749	6.796
11.300	-1.340	2.050	887.081	985	8.579
25.500	-1.340	2.050	887.081	985	8.579
39.700	-1.340	2.050	887.081	985	8.579
40.500	-1.270	2.050	703.868	749	6.796
40.500	-1.293	2.050	600.824	643	6.330
51.000	-1.293	2.050	600.824	643	6.330

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [10^4m^4]	A [10^4m^2]	g [kN/m]
0.000	-1.293	2.050	600.824	643	6.330
0.500	-1.293	2.050	600.824	643	6.330

System model used in Strip-Step2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.05	ZR	
					2
3	51.500	0.00	-2.05	YZR	
					3
4	52.000	0.00			-3

3.1.4.3 Cross-sectional constants for variable loads (short term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4m^4]$	A $[10^4m^2]$
0.000	-0.121	2.370	1717.55	3321
0.500	-0.121	2.370	1717.55	3321

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4m^4]$	A $[10^4m^2]$
0.000	-0.121	2.370	1717.55	3321
10.500	-0.121	2.370	1717.55	3321
10.500	-0.153	2.370	1924.19	3427
11.300	-0.243	2.370	2529.69	3664
25.500	-0.243	2.370	2529.69	3664
39.700	-0.243	2.370	2529.69	3664
40.500	-0.153	2.370	1924.19	3427
40.500	-0.121	2.370	1717.55	3321
51.000	-0.121	2.370	1717.55	3321

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4m^4]$	A $[10^4m^2]$
0.000	-0.121	2.370	1717.55	3321
0.500	-0.121	2.370	1717.55	3321

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.05	ZR	
					2
3	51.500	0.00	-2.05	YZR	
					3
4	52.000	0.00			-3

3.1.4.4 Cross-sectional constants for additional permanent loads (long term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.464	2.370	1381.533	1495
0.5	-0.464	2.370	1381.533	1495

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.464	2.370	1381.533	1495
10.5	-0.464	2.370	1381.533	1495
10.5	-0.509	2.370	1526.898	1602
11.3	-0.644	2.370	1922.266	1838
25.5	-0.644	2.370	1922.266	1838
39.7	-0.644	2.370	1922.266	1838
40.5	-0.509	2.370	1526.898	1602
40.5	-0.464	2.370	1381.533	1495
51	-0.464	2.370	1381.533	1495

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.464	2.370	1381.533	1495
0.5	-0.464	2.370	1381.533	1495

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.05	ZR	
					2
3	51.500	0.00	-2.05	YZR	
					3
4	52.000	0.00			-3

3.1.4.5 Cross-sectional constants for shrinkage analysis

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [10^4m^4]	A [10^4m^2]
0	-0.389	2.370	1453.391	1700
0.5	-0.389	2.370	1453.391	1700

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [10^4m^4]	A [10^4m^2]
0	-0.389	2.370	1453.391	1700
10.5	-0.389	2.370	1453.391	1700
10.5	-0.433	2.370	1609.761	1806
11.3	-0.563	2.370	2042.908	2042
25.5	-0.563	2.370	2042.908	2042
39.7	-0.563	2.370	2042.908	2042
40.5	-0.433	2.370	1609.761	1806
40.5	-0.389	2.370	1453.391	1700
51	-0.389	2.370	1453.391	1700

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [10^4m^4]	A [10^4m^2]
0	-0.389	2.370	1453.391	1700
0.5	-0.389	2.370	1453.391	1700

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.05	ZR	
					2
3	51.500	0.00	-2.05	YZR	
					3
4	52.000	0.00			-3

4 Loads and load combinations

4.1 Permanent loads

4.1.1 Self-weight

4.1.1.1 Steel

The self-weight is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: STEEL

$\rho_{st} = 75.51 \ \frac{kN}{m^3}$ Self-weight of stainless steel - SS-EN 10088-1:2014 Table E.1 or E.2

The self-weight for each element is calculated in chapter 3.

4.1.1.2 Concrete

The self-weight of the slab is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: SLAB

$Wet\_Concrete := \text{"NO"}$ "YES" or "NO" dependent if the previous designer has used the weight of wet concrete

$\rho_c := \textbf{if}\left(Wet\_Concrete = \text{"NO"}, 25 \ \frac{kN}{m^3}, 26 \ \frac{kN}{m^3}\right) = 25 \ \frac{kN}{m^3}$ Self-weight of concrete (reinforced) - SS-EN 1992-1-1 Table A.1

$A_{slab} = 3.47 \ m^2$ Area of slab, see chapter 3

$g_{slab} := \frac{A_{slab} \cdot \rho_c}{2} = 43.4 \ \frac{kN}{m}$ Self-weight of concrete slab, (half of the load goes to each girder) - applied in the casting stage

If the previous designer has considered that the hardened concrete has a smaller self-weight a reduction in self-weight is applied in the system analysis for permanent loads.

Appendix X. Name in Strip-Step2: AVSLAB

$$g_{slab.perm} := \textbf{if}\left(Wet\_Concrete = \text{"NO"}, 0 \ \frac{kN}{m}, \frac{A_{slab} \cdot (-1) \ \frac{kN}{m^3}}{2}\right) = 0.000 \ \frac{kN}{m}$$

4.1.1.3 Asphalt covering

The weight of the asphalt covering is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: ASPH

$t_{cov} = 110\ mm$ Thickness of asphalt cover

$\gamma_{cov} := 22\ \frac{kN}{m^3}$ Weight of asphalt cover

$B = 9.85\ m$ Width of bridge

$g_{cov} := \frac{t_{cov} \cdot \gamma_{cov} \cdot B}{2} = 11.92\ \frac{kN}{m}$ Self-weight of aspalt covering (half of the weight goes to each girder)

4.1.1.4 Crash barrier

The value for the self-weight of the crash barrier is assumed. Name in Strip-Step2: BARR

$g_{cb} := 0.25\ \frac{kN}{m}$ Weight per beam

4.1.1.5 Form-work

Assumes the self-weight of the form-work to be $50\ \frac{kg}{m^2}$ (bridge surface). Name in Strip-Step2: FORM

$B = 9.85\ m$ Width of bridge (inside edge beams)

$b_{eb} = 400\ mm$ Width of edge beam

$g_{fw} := \frac{(B + 2 \cdot b_{eb})}{2} \cdot 50\ \frac{kg}{m^2} \cdot g = 2.61\ \frac{kN}{m}$ Weight of edge beam

4.1.2 Shrinkage

The shrinkage force is calculated and used in Strip-Step2, Appendix X. Name in Strip-Step2: E:SHRINK

$h_0 = 341\ mm$ Equivalent thickness, calculated in chapter 2

$t = 120\ yr$

$t = 43829.1\ day$

$f_{cm} = 43\ MPa$ Mean compressive strength - see chapter 2

$t_s := 1\ day$

$RH := 80\%$ Krav Brobyggande B.3.1.5

$$\beta_{ds} := \frac{\dfrac{t - t_s}{day}}{\left(\dfrac{t - t_s}{day}\right) + 0.04 \cdot \sqrt{\left(\dfrac{h_0}{1\ mm}\right)^3}} = 0.994$$

SS-EN 1992-1-1 3.1.4 Equation 3.10

$$k_h := \begin{cases} 0.85 - \left(\dfrac{h_0}{mm} - 200\right) \cdot 0.001 & \text{if } 200\ mm \le h_0 < 300\ mm \\ 0.75 - \left(\dfrac{h_0}{mm} - 300\right) \cdot 0.0005 & \text{else if } 300\ mm \le h_0 < 500\ mm \\ 0.7 & \text{else if } h_0 \ge 500\ mm \\ 1.0 & \text{else} \end{cases} = 0.73$$

SS-EN 1992-1-1 3.1.4 table 3.3

$RH_0 := 100\%$

$\alpha_{ds1} := 4$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$\alpha_{ds2} := 0.12$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0}\right)^3\right) = 0.76$$

SS-EN 1992-1-1 Appendix B.2 equation B12

$f_{cmo} := 10\ MPa$ SS-EN 1992-1-1 Appendix B.2 (1)

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}\right)}\right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$

SS-EN 1992-1-1 Appendix B.2 equation B11

$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 1.84 \cdot 10^{-4}$ Drying shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.9

$\varepsilon_{ca0} := 2.5 \cdot \left(\frac{f_{ck} - f_{cmo}}{MPa}\right) \cdot 10^{-6} = 6.3 \cdot 10^{-5}$ SS-EN 1992-1-1 3.1.4 Equation 3.12

$\beta_{as} := 1 - e^{\left(-0.2 \cdot \sqrt{\frac{t - t_s}{day}}\right)} = 1.0$ SS-EN 1992-1-1 3.1.4 Equation 3.13

$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca0} = 6.25 \cdot 10^{-5}$ Autogenous shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.11

$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.8

4.1.2.1 Shrinkage force

The shrinkage force and corresponding moment is calculated accordingly:

$n_{L\_cs} = 14.91$ Modular ratio accounting for creep and shrinkage

$n_{L\_short} = 5.88$ Modular ratio

$E_{c.eff} := \frac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab (half of the cross-section used in system analysis)

$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5206\ kN$ Total shrinkage force

$e_{cs} := z_{tp\_cs}(0\ m) = 0.389\ m$ The shrinkage force is applied in the center of gravity for the composite section

$M_{cs} := F_{cs} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right) = 2859\ kN \cdot m$ Total bending moment

Shrinkage - anchorage of studs

Forces in the concrete

$$F_{c\_cs} := F_{cs} \cdot \left(1 - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{sl\_cs}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{y\_cs}(0\ m)} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2} \right)^2 \right) = 826\ kN$$

Force in concrete

$$b_{eff}(0\ m) = 4.925\ m$$

Effective width of flange

$$M_{c\_cs} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_cs}} \cdot h_{m.slab}{}^3}{I_{y\_cs}(0\ m)} \cdot M_{cs} = 18\ kN \cdot m$$

Moment in concrete

$$\sigma_1 := \frac{F_{c\_cs}}{A_{slab.fic}} - \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}{}^3} = 0.31\ MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{F_{c\_cs}}{A_{slab.fic}} + \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}{}^3} = 0.73\ MPa$$

Stress in concrete, lower edge of slab

$$N_{cs} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = 826\ kN \quad = \quad F_{c\_cs} = 826\ kN$$

Force imposed on studs caused by shrinkage (used in chapter 6)

4.2 Variable loads

4.2.1 Vertical trafic load

Eurocode vehicle, LM1

Dynamic amplifier is already included in the Eurocode- vehicles.

Point loads, TS

$Q_{1k} := 300\ kN$ $\qquad Q_{2k} := 200\ kN$

$\alpha_{Q1} := 0.9$ $\qquad \alpha_{Q2} := 0.9$

$Q_1 := Q_{1k} \cdot \alpha_{Q1} = 270\ kN$ $\qquad Q_2 := Q_{2k} \cdot \alpha_{Q2} = 180\ kN$

Uniformly distributed loads, UDL

$q_{1k} := 9\ \frac{kN}{m^2}$ $\qquad q_{2k} := 2.5\ \frac{kN}{m^2}$

$\alpha_{q1} := 0.8$ $\qquad \alpha_{q2} := 1.0$

$q_1 := q_{1k} \cdot \alpha_{q1} = 7.20\ \frac{kN}{m^2}$ $\qquad q_2 := q_{2k} \cdot \alpha_{q2} = 2.50\ \frac{kN}{m^2}$

Eurocode vehicle, LM2

$Q_{ak} := 400\ kN \cdot \alpha_{Q1} = 360\ kN$

Trafikverket vehicle, "typfordon"
The vehicles stated in "Bärighetsberäkning av broar" is calculated for A/B = 180/ 300 kN.

$A_{tf} := 180\ kN$ $\qquad B_{tf} := 300\ kN$

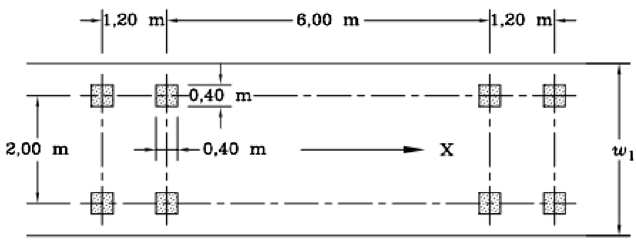
$D_{tf} := 1.20$ Dynamic amplifier - TSFS 2018:57 11 ch. 2 §

$q_{tr} := 5\ \frac{kN}{m}$

The geometries of the "Typfordon" is found in TDOK 2013:0267 10.2.

Eurocode vehicle, fatigue (number 3)
The fatigue vehicle consists of two boggie pairs with a distance of 6 meteres in between, see Figure.

$Q_{FAT3} := 120\ kN$ Weight of each axle



4.2.1.1 Divison of loads - Eurocode vehicles

$Q_{tot} := Q_1 + Q_2 = 450\ kN$ Total weight of vehicles in first and second lane

$B = 9.850\ m$ Width of bridge deck

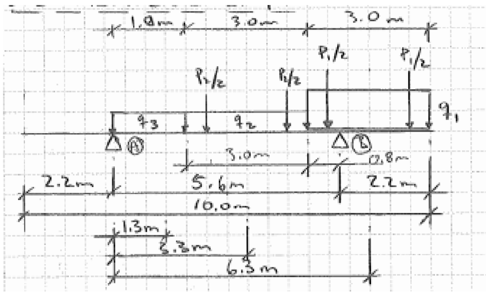
$b_{eb} = 400\ mm$ Width of edge beam

$d_{edge} := B_{out} - b_{eb} = 2.125\ m$ Distance from edge to girder

$d_{girder} := 2\ B_{in} = 5.6\ m$ Distance between steel girders

$d_q := 3\ m$ Width of loaded lane

$d_{q3} := d_{girder} - \left(3\ m - d_{edge}\right) - d_q = 1.7\ m$ Width of the third distributed load



$$F_{f.p\_lm.1} := \frac{Q_1 \cdot \left(d_{girder} + d_{edge} - \frac{d_q}{2}\right) + Q_2 \cdot \left(d_{girder} + d_{edge} - d_q - \frac{d_q}{2}\right)}{Q_{tot} \cdot d_{girder}} = 0.897$$

Load divider point load - Beam 1, EC- vehicle

$F_{f.p\_lm.2} := 1 - F_{f.p\_lm.1} = 0.103$ Load divider point load - Beam 2, EC- vehicle

$$F_{f.q\_lm.1} := \frac{q_1 \cdot d_q \cdot \left(d_{girder} + d_{edge} - \frac{d_q}{2}\right) + q_2 \cdot \left(d_q \cdot \left(d_{girder} + d_{edge} - 1.5\ d_q\right) + \frac{{d_{q3}}^2}{2}\right)}{d_{girder}} = 28.99\ \frac{kN}{m}$$

Uniformly distributed load (incl. load divider) - Beam 1, EC- vehicle

4.2.1.2 Divison of loads - 'Trafikverket' vehicles

When the load divider is calculated consideration of the dynamic amplifier is taken into account.

$$F_{f.p\_tr.1} := \frac{\left(1 \cdot \left(d_{girder} + d_{edge} - \frac{d_q}{2}\right) + 0.8 \cdot \left(d_{girder} + d_{edge} - d_q - \frac{d_q}{2}\right)\right) \cdot Q_1}{1.8 \cdot Q_1 \cdot d_{girder}} = 0.874$$

Load divider point load - Beam 1, Trafikverket- vehicle

$$F_{f.p\_tr.2} := 1 - F_{f.p\_tr.1} = 0.126$$

Load divider point load - Beam 2, Trafikverket- vehicle

$$F_{f.q\_tr.1} := \frac{(1+0.8) \cdot q_{tr}}{D_{tf}} = 7.5\ \frac{kN}{m}$$

Distributed load beam 1, choosen the same as previous designer

4.2.1.3 Divison of loads - Fatigue vehicles (Eurocode)

$$F_{f.p\_FAT.1} := \frac{Q_1 \cdot \left(d_{girder} + d_{edge} - \frac{d_q}{2}\right)}{Q_1 \cdot d_{girder}} = 1.112$$

Load divider point load - Beam 1, Fatigue- vehicle

$$F_{f.p\_FAT.2} := 1 - F_{f.p\_FAT.1} = -0.112$$

Load divider point load - Beam 2, Fatigue- vehicle

4.2.2 Horizontal trafic load

The eccentricity for the acceleration load is placed upon the asphalt covering.

$e_{br} := t_{cov} + z_{tp\_short}\left(\frac{L_{bridge}}{2}\right) + h_{m.slab} = 0.673\ m$

Eccentricity where the acceleration load is acting (top of the asphalt cover)

Eurocode vehicle

The force is used in Strip-Step2, Appendix X. Name in Strip-Step2: BRLM

$w_1 := 3\ m$

$Q_{1k\_br} := 0.6 \cdot \alpha_{Q1} \cdot 2 \cdot Q_{1k} + 0.1 \cdot \alpha_{q1} \cdot q_{1k} \cdot w_1 \cdot L_{bridge} = 436.3\ kN$

$q_{1k\_br} := \frac{Q_{1k\_br}}{2 \cdot L_{bridge}} = 4.20\ \frac{kN}{m}$

Acceleration per unit length per beam

Trafikverket vehicle

The force is used in Strip-Step2, Appendix X. Name in Strip-Step2: BRTYP

Should be calculated for the heaviest vehicle for loads in one lane.

$Q_{tr\_br} = min\left(0.35 \cdot Q_{max}, 500\ kN\right)$

$$Br := max\left(\begin{bmatrix} 0.88 \\ 1.0 \\ 1.10 \\ 1.17 \\ 1.32 \\ 0.44+1.32+0.44+1.32 \\ 0.55+1+1.32 \\ 0.44+1.10+1.10+0.66 \\ 0.44+1.32+0.44+1.32 \\ 0.55+1+1.32 \\ 0.44+1.10+1.10+0.66 \\ 0.33+1+1.32 \\ 0.55+0.55+0.55+0.33+0.12 \end{bmatrix} \cdot B_{tf}\right) = 1056\ kN$$

Weight of all B- vehicles

$Q_{tr\_br} := min\left(0.35\ Br, 500\ kN\right) = 370\ kN$

$q_{tr\_br} := \frac{Q_{tr\_br}}{2 \cdot L_{bridge}} = 3.55\ \frac{kN}{m}$

Acceleration per unit length per beam

4.2.2.1 Horizontal side trafic load

$P_{h\_side\_car} := max\left(\begin{bmatrix} 25\% \cdot Q_{1k\_br} \\ 25\% \cdot Q_{tr\_br} \end{bmatrix}\right) = 109.1\ kN$ Horisontal side force from the acceleration load (used in chapter 7)

$P_{v\_side\_car} := P_{h\_side\_car} \cdot \dfrac{h_{m.slab}}{2 \cdot B_{in}} = 6.2\ kN$ Vertical side force from the acceleration load (used in chapter 7)

4.2.3 Temperature load

Temperatures are determined according to SS-EN 1991-1-5, 6.1.3 unless otherwise stated.

Load case 1 - local temperature differences for Hudiksvall

$T_0 = 10\ °$ A.1(3)

$T_{min} = -38\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{max} = 34\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4\ ° = -34\ °$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min} = 44\ °$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4\ ° = 38\ °$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0 = 28\ °$ Expansion - Equation 6.1

$\Delta T := T_{e.max} - T_{e.min} = 72\ °$ Total temperature difference

Load case 2 - either of the components are larger than the other

$\Delta T_{c2st} := 15\ °$ Temperature difference between concrete and steel - SS-EN 1991-1-5, 6.1.6

4.2.3.1 Coefficients of thermal linear expansion

For composite bridges normally it is suggested to use the same thermal linear expansion coefficient, according to SS-EN 1991-1-5 Table C.1. However for stainless steels the thermal linear expansion coefficient is much larger than for concrete and hence a more through calculation is needed for the strain.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

4.2.3.2 Strains for the different load cases

$\Delta\varepsilon_{LC.1\_con} := -\left(\alpha_{ss} - \alpha_c\right) \cdot \frac{\Delta T_{N.con}}{\circ} = -26.4 \ 10^{-5}$ Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp} := \left(\alpha_{ss} - \alpha_c\right) \cdot \frac{\Delta T_{N.exp}}{\circ} = 16.8 \ 10^{-5}$ Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss} \cdot \frac{\Delta T_{c2st}}{\circ} = 24.0 \ 10^{-5}$ Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c \cdot \frac{\Delta T_{c2st}}{\circ} = 15.0 \ 10^{-5}$ Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st}, \Delta\varepsilon_{LC.2\_c}\right) = 24.0 \ 10^{-5}$ Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp_1} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con} = -50.4 \ 10^{-5}$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp_2} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp} = 40.8 \ 10^{-5}$ Maximum strain difference; temperature raises and the steel heatens up even higher

4.2.3.3 Temperature load - global analysis

The concrete is transformed to steel. Already defined parameters are calculated in chapter 3.

$\frac{b_{eff}(0 \ m)}{n_{L\_short}} = 0.837 \ m$ Width of transformed concrete

$\frac{A_{slab.fic}}{n_{L\_short}} = 0.268 \ m^2$ Area of transformed concrete

$A_{sl}(0 \ m) = 0.064 \ m^2$ Area of composite section

$z_{tp\_short}(0 \ m) = 121.1 \ mm$ Distance from top of concrete to center of gravity for composite section

$I_{y\_short}(0 \ m) = 0.172 \ m^4$ Moment of inertia for composite section

$F_{temp} := \varepsilon_{temp} \cdot E_s \cdot A_{sl}(0 \ m) = \begin{bmatrix} -6476 \\ 5243 \end{bmatrix} kN$ Force on composite section

$e_{temp\_F} := z_{tp\_short}(0 \ m) = 121.1 \ mm$ Level at which the force is imposed on the system

$e_{temp\_M} := z_{tp\_steel}(0 \ m) - z_{tp\_short}(0 \ m) = 1.172 \ m$ Eccentricity for the bending moment

$$M_{temp} := F_{temp} \cdot e_{temp\_M} = \begin{bmatrix} -7589 \\ 6144 \end{bmatrix} kN \cdot m$$

Bending moment - composite section

Anchorage of temperature load imposed on studs

<u>Concrete</u>

$$N_{c\_temp} := F_{temp} \cdot \left(\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{A_{sl\_short}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_short}}}{I_{y\_short}(0\ m)} \cdot \left(\left(z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) \right) \cdot \left(z_{tp\_short}(0\ m) + \frac{h_{m.slab}}{2} \right) \right) \right) = \begin{bmatrix} -1897 \\ 1536 \end{bmatrix} kN$$

$$M_{c\_temp} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_short}} \cdot h_{m.slab}^{\ 3}}{I_{y\_short}(0\ m)} \cdot M_{temp} = \begin{bmatrix} -101 \\ 82 \end{bmatrix} kN \cdot m$$

$$\sigma_1 := \frac{N_{c\_temp}}{A_{slab.fic}} - \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 2}} = \begin{bmatrix} -0.00 \\ 0.00 \end{bmatrix} MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{N_{c\_temp}}{A_{slab.fic}} + \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 2}} = \begin{bmatrix} -2.41 \\ 1.95 \end{bmatrix} MPa$$

Stress in concrete, lower edge of slab

$$N_{temp} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = \begin{bmatrix} -1897 \\ 1536 \end{bmatrix} kN$$

Force imposed on studs caused by shrinkage (used in chapter 6 when calculating the anchorage of the slab by the studs)

4.2.4 Wind load

$V_b = 23\ \frac{m}{s}$ Wind speed, Hudiksvall - TSFS 2018:57 Figure 7.1

$q_p = 0.54\ kPa$ Peak velocity pressure - TSFS 2018:57 Table 7.1

<u>Wind load on bridge only</u>

$MW = 42.20\ m$ Reference height - water surface

$LSL = 45.45\ m$ Reference height - lower edge slab

$h_{m.slab} = 320\ mm$ Mean slab height, see chapter 3

$h_{klack} = 25\ mm$ Height of heel (concrete slab)

$b := B_{tot} = 10.65\ m$ Width of bridge (total)

$c_{web} = 5.600\ m$ Center distance between girders

$d_1 := 0.6\ m$ Assume crash barrier height

$$d := h_{m.slab} + h_{klack} + h_{tot\_steel}\left(\frac{L_{bridge}}{2}\right) + 50\ mm = 2.445\ m$$

$d_{tot} := d + 2 \cdot d_1 = 3.645\ m$ SS- EN 1991-1-4,Table 8.1

$\frac{b}{d_{tot}} = 2.92$ Ratio

$$c_{fx.0}\left(b, d_{tot}\right) := linterp\left(\begin{bmatrix} 0 \\ 4 \\ 4.1 \end{bmatrix}, \begin{bmatrix} 2.4 \\ 1.3 \\ 1.3 \end{bmatrix}, \frac{b}{d_{tot}}\right)$$

SS-EN 1991-1-4, Figure 8.3

$$z_e := \left(LSL - MW - h_{tot\_steel}\left(\frac{L_{bridge}}{2}\right)\right) + \frac{d_{tot}}{2} = 3022.5\ mm$$

Eccentricity between upper steel flange and center of pressure

$$F_w := q_p \cdot c_{fx.0}\left(b, d_{tot}\right) \cdot d_{tot} = 3.14\ \frac{kN}{m}$$

Horizontal composant of wind load on bridge per meter length

$$q_{v\_b} := F_w \cdot \frac{\left(\frac{d_{tot}}{2} - h_{tot\_steel}\left(\frac{L_{bridge}}{2}\right)\right)}{c_{web}} = -0.128\ \frac{kN}{m}$$

Wind load on bridge, vertical composant per beam

Wind load on bridge and vehicle

$d_2 := 2\ m$ — Assumed height of vehicle

$d_{tot} := d + d_2 = 4.445\ m$ — Total height for the bridge and the vehicle

$z_e := \left(LSL - MW - h_{tot\_steel}\left(\frac{L_{bridge}}{2}\right)\right) + \frac{d_{tot}}{2} = 3422.5\ mm$ — Eccentricity between upper steel flange and center of pressure

$c_{fx.0}\left(b, d_{tot}\right) = 1.74$

$F_w := q_p \cdot c_{fx.0}\left(b, d_{tot}\right) \cdot d_{tot} = 4.18\ \frac{kN}{m}$ — Horizontal composant of wind load on bridge per meter length

$q_{v\_bv} := F_w \cdot \frac{\left(\frac{d_{tot}}{2} - h_{tot\_steel}\left(\frac{L_{bridge}}{2}\right)\right)}{c_{web}} = 0.129\ \frac{kN}{m}$ — Wind load on bridge and trafic, vertical composant per beam

$WIND := \text{"NO"}$ — Choosing if wind load should be included (neglectable) in the global system analysis (StripStep-2). However included for local effects in chapter 7.

$q_v := \mathbf{if}\left(WIND = \text{"YES"}, max\left(abs\left(q_{v\_b}\right), abs\left(q_{v\_bv}\right)\right), 0\ \frac{kN}{m}\right) = 0.00\ \frac{kN}{m}$ — Design wind load

4.3 Load combination

Four different system analysis is carried out using the software StripStep-2 in order to consider various stiffnesses. Thereafter stresses is calculed for each load individually before summarized. The load combination factors for each load is included in the system analysis with the below presented values.

In order to stay consequent with the previous design of the bridge, all load combination factors is the same as in the original design.

$\gamma_d = 1.00$ Partial safety factor - see chapter 1

4.3.1 Ultimate limit state

Load combination factors - Appendix X, casting

$\psi_{st} := 1.20 \cdot \gamma_d$ Load combination factor - steel

$\psi_{sl} := 1.20 \cdot \gamma_d$ Load combination factor - slab

$\psi_{fw} := 1.20 \cdot \gamma_d$ Load combination factor - form-work

Load combination factors - Appendix X, long term

$\psi_{fw} = 1.20$ Load combination factor - form-work (relaxation)

$\psi_{cov} := 1.32 \cdot \gamma_d$ Load combination factor - asphalt covering

$\psi_{cb} := 1.20 \cdot \gamma_d$ Load combination factor - crash barrier

Load combination factors - Appendix X, short term

$\psi_{temp} := 1 \cdot \gamma_d$ Load combination factor - temperature

$\psi_{multi\_tr} := 1 \cdot \gamma_d$ Load combination factor - vertical trafic load

$\psi_{multi\_br} := 0.6 \cdot \gamma_d$ Load combination factor - horisontal trafic load

$\psi_{brott\_multi} := 1.5 \cdot \gamma_d$ Load combination factor - vertical trafic load

$\psi_{brott\_temp} := 0.6 \cdot 1.5 \cdot \gamma_d$ Load combination factor - horisontal trafic load

Load combination factors - Appendix X, shrinkage

$\psi_{cs} := 1.0$ Load combination factor - shrinkage

Load combination factors (including force) - Local checks

$P_{h\_side} := P_{h\_side\_car} \cdot \psi_{multi\_br} \cdot \psi_{brott\_multi} = 98.2\ kN$

4.3.2 Servicability limit state

Allowed deflection is determined for the vertical trafic loads.

$\psi_{def} := 0.75$ Frequent load - Eq. 6.15b

$\dfrac{Q_1 + Q_2}{Q_1} = 1.667$ Scale factor considering boggi loads in two lanes

$F_{f.p\_lm.1} = 0.897$ Already accounted for

$F_{f.q\_lm.1} := 1.000$ Already accounted for

$\psi_{EC} := \psi_{def} = 0.75$

$F_{f.p\_tr.1} = 0.874$ Load divider, beam 1

$D_{tf} = 1.20$ Already accounted for

$\psi_{typ} := \psi_{def} \cdot F_{f.p\_tr.1} = 0.655$ Load coefficient, considering two lanes (1+0.8)

$\psi_{kvasi\_temp} := 0.5$

$\psi_{kvasi\_multi} := 0.75$

5 Capacity checks during construction

The capacity check that is carried out in this chapter is bending moment capacity with respect to lateral torsional buckling in the casting phase

In the casting phase the normalforces in the cross-section are neglectible.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

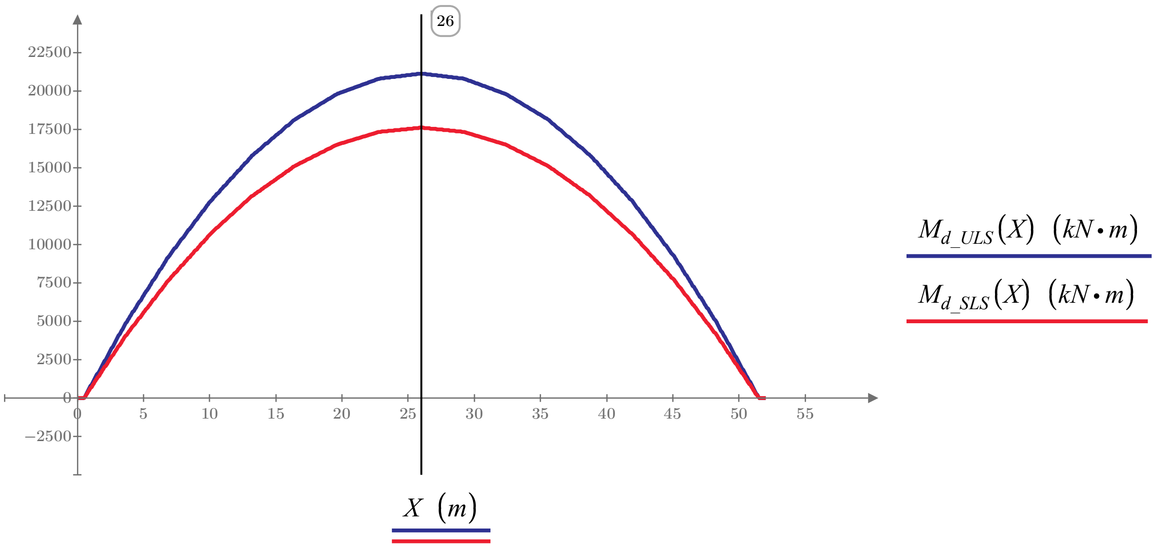
$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

5.1 Load effects

Load effects retrieved from Strip-Step2, Appendix X.

Bending moment

M_{d_ULS} Bending moment descending from permanent loads during construction in ultimate limit state
M_{d_SLS} Bending moment descending from permanent loads during construction in service limit state

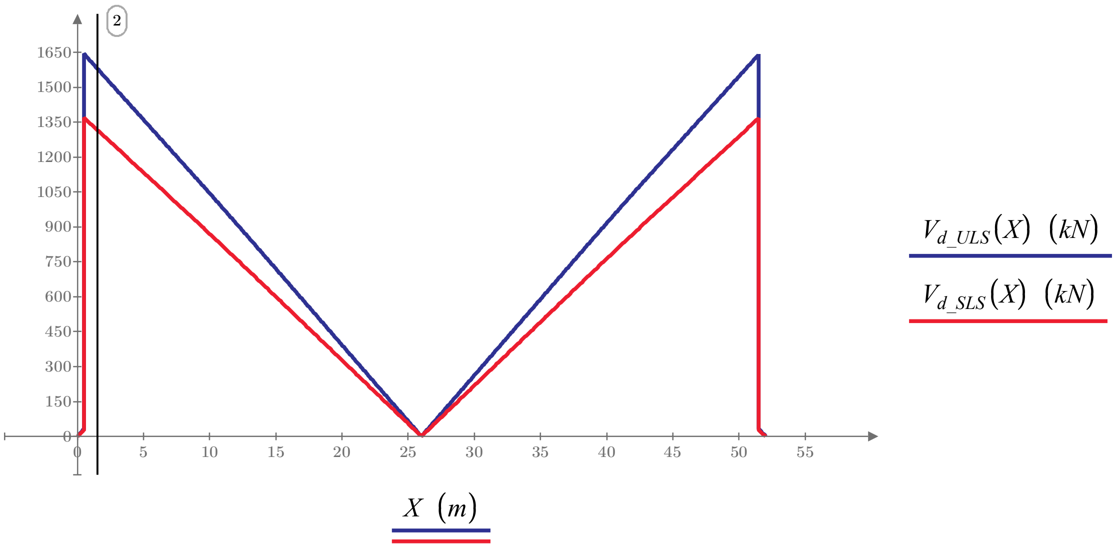


$M_{d\_ULS}(X_{check\_m}) = 21141\ kN \cdot m$ Bending moment at control point - ULS

$M_{d\_SLS}(X_{check\_m}) = 17617\ kN \cdot m$ Bending moment at control point - SLS

Shear force

V_{d_ULS} Shear force descending from permanent loads during construction in ultimate limit state
V_{d_SLS} Shear force descending from permanent loads during construction in service limit state



$V_{d\_ULS}(X_{check\_v}) = 1581\ kN$ Shear force at control point - ULS

$V_{d\_SLS}(X_{check\_v}) = 1317\ kN$ Shear force at control point - SLS

5.2 Redesign of cross-section

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange
f_{yw}	Chosen yield strength - web

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)	f_{yw_1}
0	35	765	9	1965	50	825	f_{y_10mm}
500	35	765	9	1965	50	825	f_{y_10mm}

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)	f_{yw_2}
0	35	765	9	1965	50	825	f_{y_10mm}
10500	35	765	9	1965	50	825	f_{y_10mm}
10500	45	765	6	1955	50	825	$f_{y\_6.4mm}$
11300	45	850	6	1955	50	1225	$f_{y\_6.4mm}$
25500	45	850	6	1955	50	1225	$f_{y\_6.4mm}$

Element 3

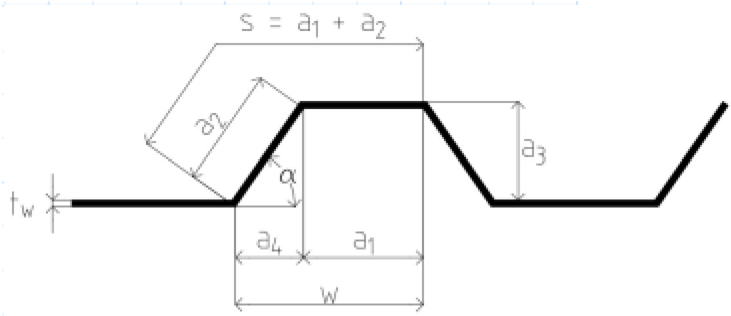
S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)	f_{yw_3}
0	35	765	9	1965	50	825	f_{y_10mm}
500	35	765	9	1965	50	825	f_{y_10mm}

$$h_{tot1} := mean\left(t_{fu\_1} + h_{w\_1} + t_{fl\_1}\right) = 2050\ mm$$

$$h_{tot2} := mean\left(t_{fu\_2} + h_{w\_2} + t_{fl\_2}\right) = 2050\ mm$$

Check to see that the girder height is kept constant

5.2.1 Shape of corrugation



$a_{c1} := 120\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 70\ mm$ Corrugation depth

$a_{weld} := 5\ mm$ Weld throat thickness

$a_{c2} := \dfrac{a_{c3}}{sin(\alpha_c)} = 119\ mm$ Legnth of hypotenuse

$a_{c4} := \dfrac{a_{c3}}{tan(\alpha_c)} = 96\ mm$ Length of corrugation depth, see figure

$s_c := a_{c1} + a_{c2} = 239\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 216\ mm$ Straight length

$r_c := \dfrac{s_c}{w_c} = 1.105$ Ratio corrugation/straight length

5.2.2 Cross-section classification

The cross-section classes is determined according to SS-EN 1993 1-4 5.2.2 with updated limits from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated and does not contribute to the axial stiffness the web is not classified.

$$c_w(x) := h_w(x) - 2 \cdot \sqrt{2} \cdot a_{weld}$$

$$c_{uf}(x) := \frac{b_{fu}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on upper flange

$$c_{lf}(x) := \frac{b_{fl}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on lower flange

<u>Cross-section class, upper flange</u>

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength of top flange

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

SS-EN 1993-1-4 5.2.2 Table 5.2

$$csc_{uf}(x) := \begin{array}{l} \text{if } \frac{c_{uf}(x)}{t_{fu}(x)} \le 9\ \varepsilon_{uf} \\ \quad \text{"csc1"} \\ \text{else if } 9\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 10\ \varepsilon_{uf} \\ \quad \text{"csc2"} \\ \text{else if } 10\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 14\ \varepsilon_{uf} \\ \quad \text{"csc3"} \\ \text{else} \\ \quad \text{"csc4"} \end{array}$$

Cross-section class upper flange

$csc_{uf}(X_{check\_m}) = \text{"csc4"}$ — Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{uf}(X_{check\_v}) = \text{"csc4"}$ — Cross-section class at $X_{check\_v} = 1.500\ m$

Cross-section class, lower flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{ylf} = 450\ MPa$ — Proof strength of top flange

$\varepsilon_{lf} := \sqrt{\frac{235}{f_{ylf}} \cdot \frac{E_s}{210000}} = 0.71$ — SS-EN 1993-1-4 5.2.2 Table 5.2

Cross-section class lower flange

$$csc_{lf}(x) := \begin{array}{l} \text{if } \frac{c_{lf}(x)}{t_{fl}(x)} \le 9\ \varepsilon_{lf} \\ \quad \text{"csc1"} \\ \text{else if } 9\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 10\ \varepsilon_{lf} \\ \quad \text{"csc2"} \\ \text{else if } 10\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 14\ \varepsilon_{lf} \\ \quad \text{"csc3"} \\ \text{else} \\ \quad \text{"csc4"} \end{array}$$

$csc_{lf}(X_{check\_m}) = \text{"csc4"}$ — Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{lf}(X_{check\_v}) = \text{"csc3"}$ — Cross-section class at $X_{check\_v} = 1.500\ m$

5.2.3 Plate buckling of compressive flange

If the compressed flange is in cross-section class four an effective width of the compressed flange is calculated according to SS-EN 1993-1-4 5.2.3. Updated values are taken from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated the buckling factor k_σ is calculated according to SS-EN 1993-1-5 D.2.1 (2)

$a_{bend} := a_{c1} + 2\ a_{c4} = 313\ mm$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$c_u := c_{uf}(X_{check\_m}) = 450\ mm$ — Width of outstand flange from weld toe to free edge

$t_u := t_{fu}(X_{check\_m}) = 45\ mm$ — Thickness of upper flange

$b_u := b_{fu}(X_{check\_m}) = 850\ mm$ — Width of upper flange

$\varepsilon_{uf} = 0.71$

$k_{\sigma 1} := 0.43 + \left(\frac{c_u}{a_{bend}}\right)^2 = 2.5$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_{\sigma 2} := 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_\sigma := min\left(k_{\sigma 1}, k_{\sigma 2}\right) = 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

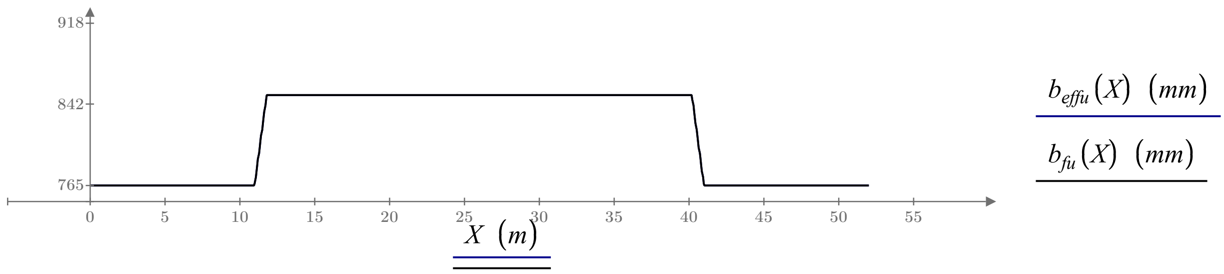
$$\lambda_p := \frac{\frac{c_u}{t_u}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_\sigma}} = 0.64$$

Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho := \mathbf{if}\left(\lambda_p \le 0.748, 1.0, \frac{\lambda_p - 0.188}{\lambda_p^{\ 2}}\right) = 1.00$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

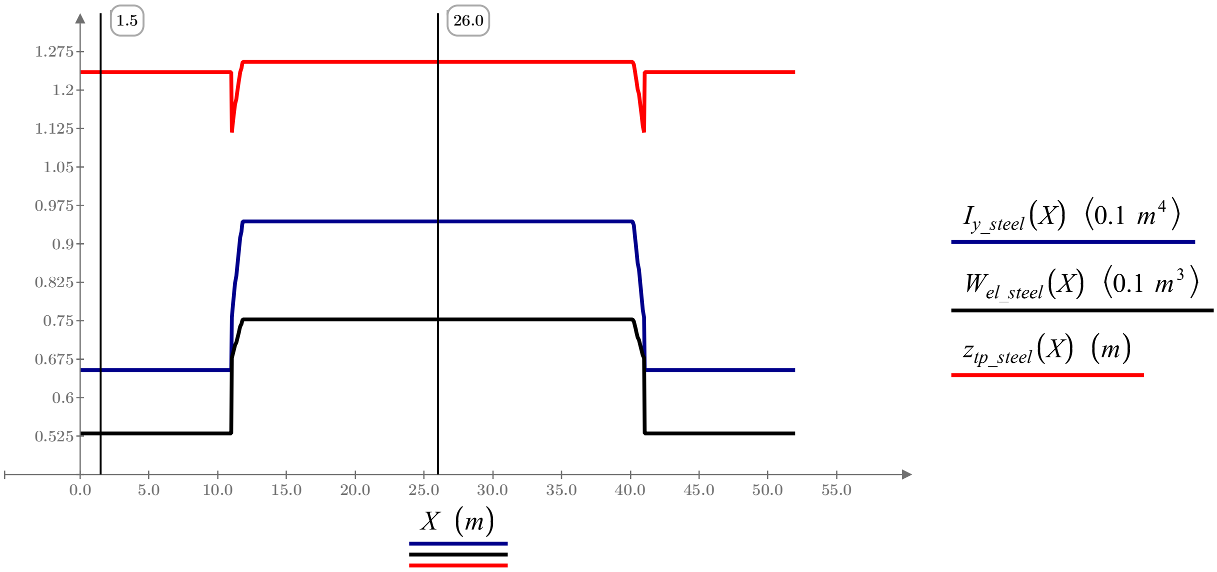
$b_{eff} := b_u \cdot \rho = 850\ mm$ = $b_{effu}(X_{check\_m}) = 850\ mm$



$b_{fu}(x) := b_{effu}(x)$ — Renaming the width of flange in order to minimize errors

5.2.6 New cross-sectional constants during casting

$I_{y\_steel}(x)$	Stiffness of steel girder alone (excluding the web)
$z_{tp\_steel}(x)$	Distance from the top of the top flange to the center of gravity for the steel section
$W_{el\_steel}(x)$	Elastic bending stiffness of steel girder alone (excluding the web)



Coordinates and cross-sectional constants for control calculations

$X_{check\_m} = 26\ m$	X- coordinate for control calculations, bending moment
$A_{sl}(X_{check\_m}) = 0.1\ m^2$	Area
$I_{y\_steel}(X_{check\_m}) = (94 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_m}) = 1.255\ m$	Center of gravity
$W_{el\_steel}(X_{check\_m}) = (75 \cdot 10^{-3})\ m^3$	Elastic bending resistance
$X_{check\_v} = 1.5\ m$	X- coordinate for control calculations, shear force
$A_{sl}(X_{check\_v}) = 0.068\ m^2$	Area
$I_{y\_steel}(X_{check\_v}) = (65 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_v}) = 1.235\ m$	Center of gravity
$W_{el\_steel}(X_{check\_v}) = (53 \cdot 10^{-3})\ m^3$	Elastic bending resistance

5.2.4 Lateral torsional buckling of compressive flange - SS-EN 1993-1-4 5.4.2.1

Simplified method only consisdering buckling of top flange according to SS-EN 1993-1-4 5.4.2.1

$\alpha_{LT} := 0.76$ — Buckling curve d, welded open cross-section

$l_{cr} := 7.29\ m$ — Distance between the cross-beams

$b_{ef} := b_{effu}\left(X_{check\_m}\right) = 850\ mm$ — Width of upper flange

$t_f := t_{fu}\left(X_{check\_m}\right) = 45\ mm$ — Thickness of upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength

$$I_{zf} := \frac{b_{ef}^{\ 3} \cdot t_f}{12} = 0.002\ m^4$$ Moment of inertia, upper flange

$$N_{crLT} := \frac{\pi^2 \cdot E_s \cdot I_{zf}}{l_{cr}^{\ 2}} = 85539\ kN$$ Critical buckling load

$$\lambda_{LT\_u} := \sqrt{\frac{b_{ef} \cdot t_f \cdot f_{yuf}}{N_{crLT}}} = 0.449$$ SS-EN 1993-1-4 5.4.2.1 Equation 5.9

$$\Phi_{LT\_}u := 0.5 \cdot \left(1 + \alpha_{LT} \cdot \left(\lambda_{LT\_u} - 0.2\right) + \lambda_{LT\_u}^{\ 2}\right) = 0.7$$ SS-EN 1993-1-4 5.4.2.1 Equation 5.7

$$\chi_{LT\_u} := min\left(\frac{1}{\Phi_{LT\_}u + \sqrt{\Phi_{LT\_}u^2 - \lambda_{LT\_u}^{\ 2}}}, 1\right) = 0.816 \quad = \quad \chi_{LT}\left(X_{check\_m}\right) = 0.816$$

SS-EN 1993-1-4 5.4.2.1 Equation 5.6

$h_{w\_u} := h_w\left(X_{check\_m}\right) = 1955\ mm$ — Height of web

$t_l := t_{fl}\left(X_{check\_m}\right) = 50\ mm$ — Thickness of lower flange

$k_{fl} := 1.1$ — Increase in capacity due to simlified method used, SS-EN 1993-1-1 6.3.2.4 (2)B

$$M_{Rd.u.LT\_u} := \frac{b_{ef} \cdot t_f \cdot k_{fl} \cdot \chi_{LT\_u} \cdot f_{yuf}}{\gamma_{M1}} \left(h_{w\_u} + \frac{t_f + t_l}{2}\right) = 30925\ kN \cdot m \quad = \quad M_{Rd.u.LT}\left(X_{check\_m}\right) = 30925\ kN \cdot m$$

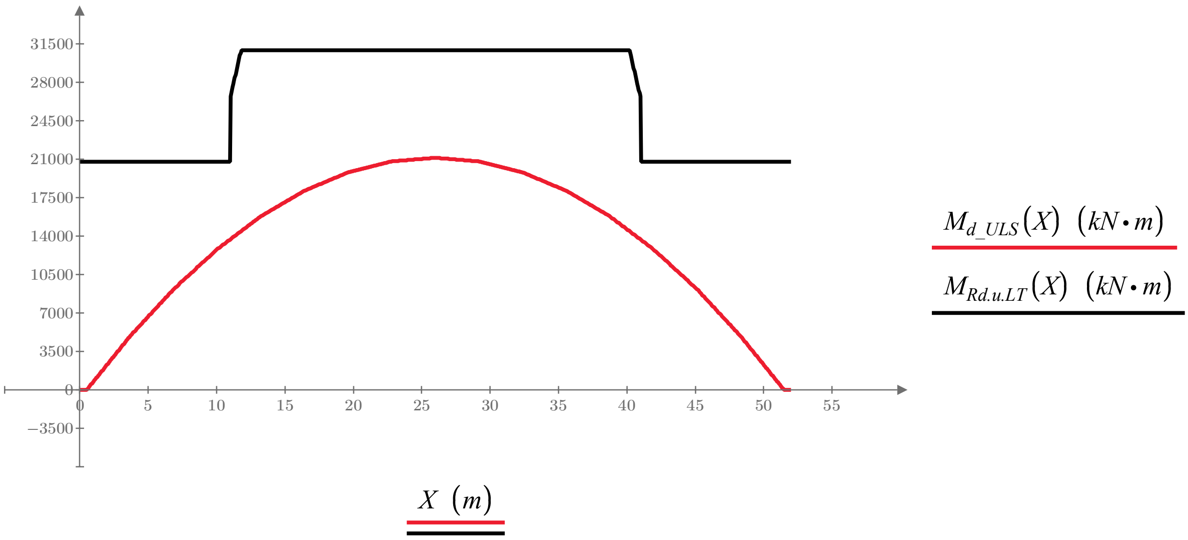
SS-EN 1993-1-5 D.2.1 Equation D.1

5.2.5 Check of lateral torsional buckling

Check of the buckling capacity of the girders is performed.

$M_{Rd.u.LT}(X)$ Capacity of the girder against LT- buckling during casting

$M_{d\_ULS}(X)$ Load effects during casting, Appendix X



$$\eta_{max} := max\left(\frac{M_{d\_ULS}(X)}{M_{Rd.u.LT}(X)}\right) = 68\%$$

Maximum utilization rate of the steel cross-section during casting

5.3 Stresses in steel cross-section

The stresses in the top and bottom flange is calculated for the loading senario to be able to superposition them with the other loadcases to determinte the ultimate capacity of the composite section.

Upper flange

Control calculation at midspan

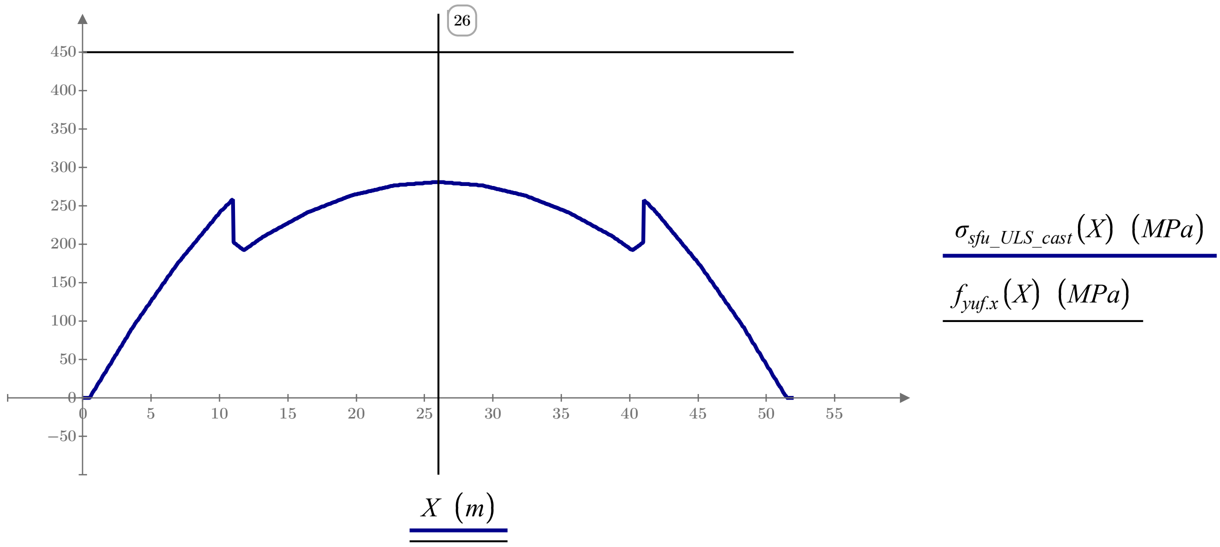
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 21141\ kN \cdot m$ Load effext at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.094\ m^4$ Moment of inertia at midspan

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 1.255\ m$ Centre of gravity at midspan maesured from the top of the beam

$\sigma := \dfrac{M}{I} \cdot z = 281\ MPa$ = $\sigma_{sfu\_ULS\_cast}\left(X_{check\_m}\right) = 281\ MPa$

$\sigma_{sfu\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$$\eta_{max} := max\left(\frac{\sigma_{sfu\_ULS\_cast}(X)}{f_{yuf}}\right) = 62\%$$

Lower flange

Check calculation at midspan

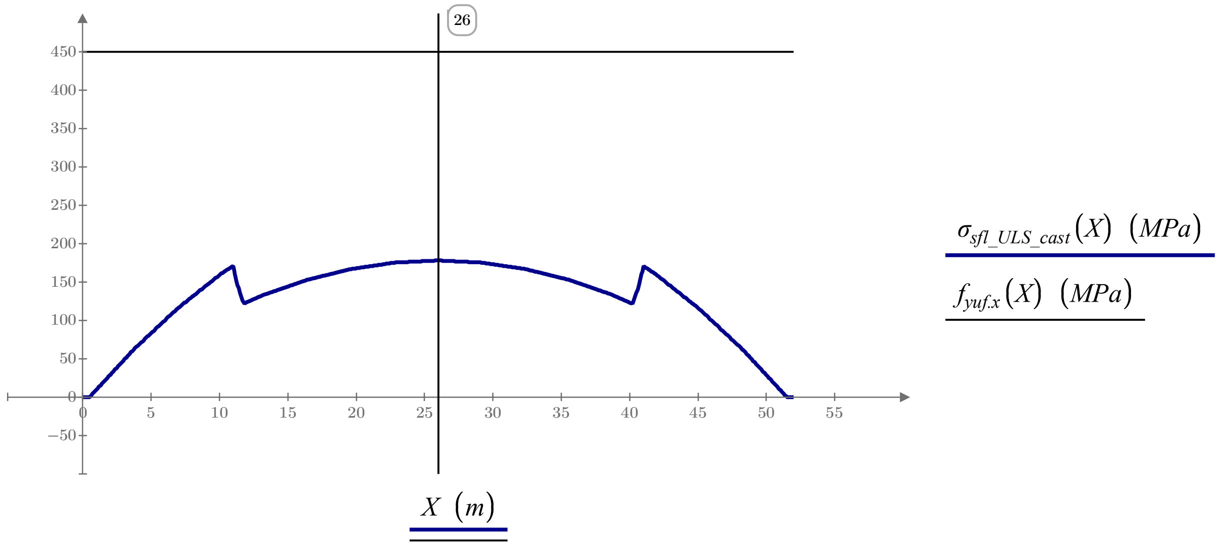
$M := M_{d\_ULS}(X_{check\_m}) = 21141\ kN \cdot m$ Load effect at midspan

$I := I_{y\_steel}(X_{check\_m}) = 0.094\ m^4$ Moment of inertia at midspan

$z := z_{tplf\_steel}(X_{check\_m}) = 0.795\ m$ Centre of gravity at midspan measured from the bottom of the beam

$\sigma := \frac{M}{I} \cdot z = 178\ MPa$ = $\sigma_{sfl\_ULS\_cast}(X_{check\_m}) = 178\ MPa$

$\sigma_{sfl\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\frac{\sigma_{sfl\_ULS\_cast}(X)}{f_{ylf}}\right) = 40\%$

6 Capacity checks - Ultimate limit state, global

The capacity checks that are to be carried out are bending moment capacity, shear capacity, web breathing and design of studs, both in ULS and due to fatigue

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 0.5\ m$ Coordinate for control calculations - shear force

6.1 Load effects in ULS

6.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

Bending moment in the ultimate limit state

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{tr}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

M_{d_ULS} Design bending moment effect from all loads

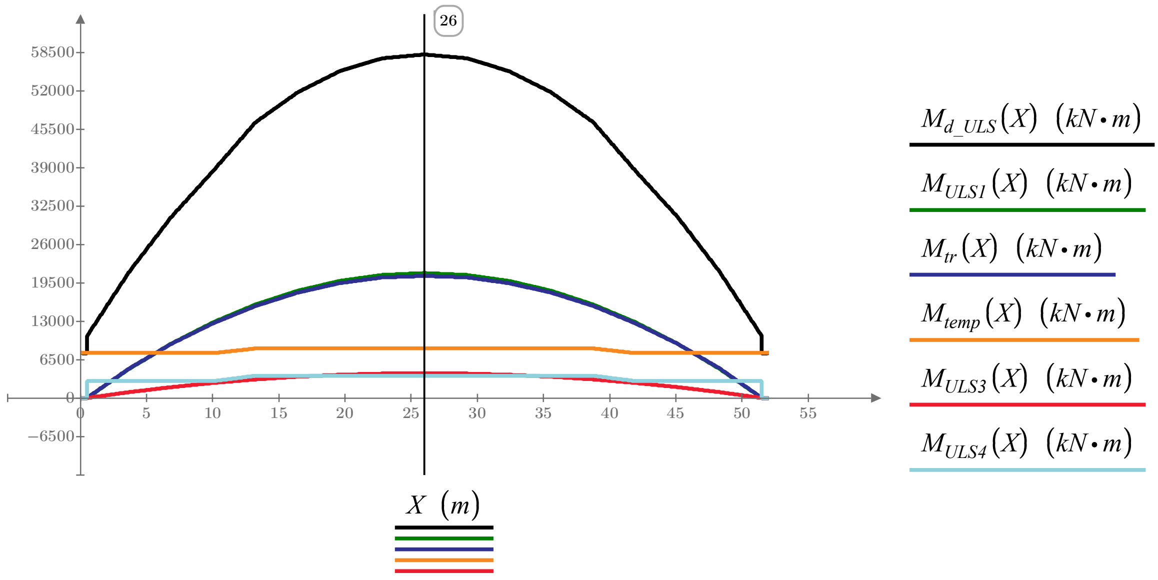
$M_{ULS1}(x)$ Bending moment during casting (Appendix X)

$M_{tr}(x)$ Bending moment from multi component loads (Appendix X)

$M_{tr}(x)$ Bending moment from temperature loads (Appendix X)

$M_{ULS3}(x)$ Bending moment from additional permanent loads after construction (Appendix X)

$M_{ULS4}(x)$ Bending moment from shrinkage (Appendix X)

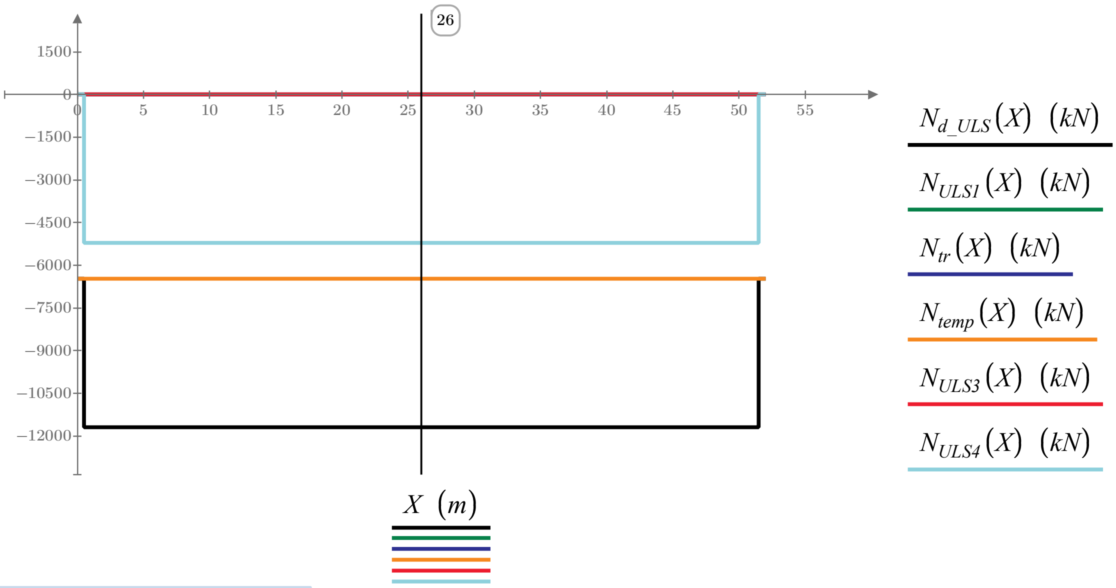


$M_{d\_ULS}(X_{check\_m}) = 58124\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{tr}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design normal force from all loads
$N_{ULS1}(x)$	Normal force during casting (Appendix X)
$N_{tr}(x)$	Normal force from multi components loads (Appendix X)
$N_{temp}(x)$	Normal force from temperature loads (Appendix X)
$N_{ULS3}(x)$	Normal force from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Normal force from shrinkage (Appendix X)

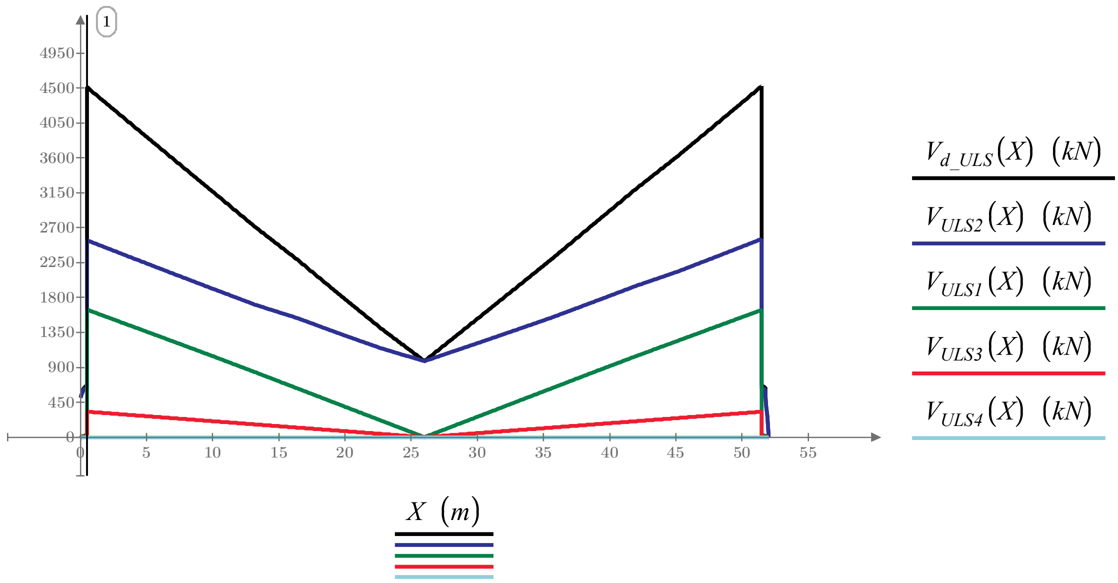


$N_{d\_ULS}(X_{check\_m}) = -11682\ kN$ Normal force at control point - ULS

6.1.2 Shear force

$$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{ULS3}(x) + V_{ULS4}(x)$$

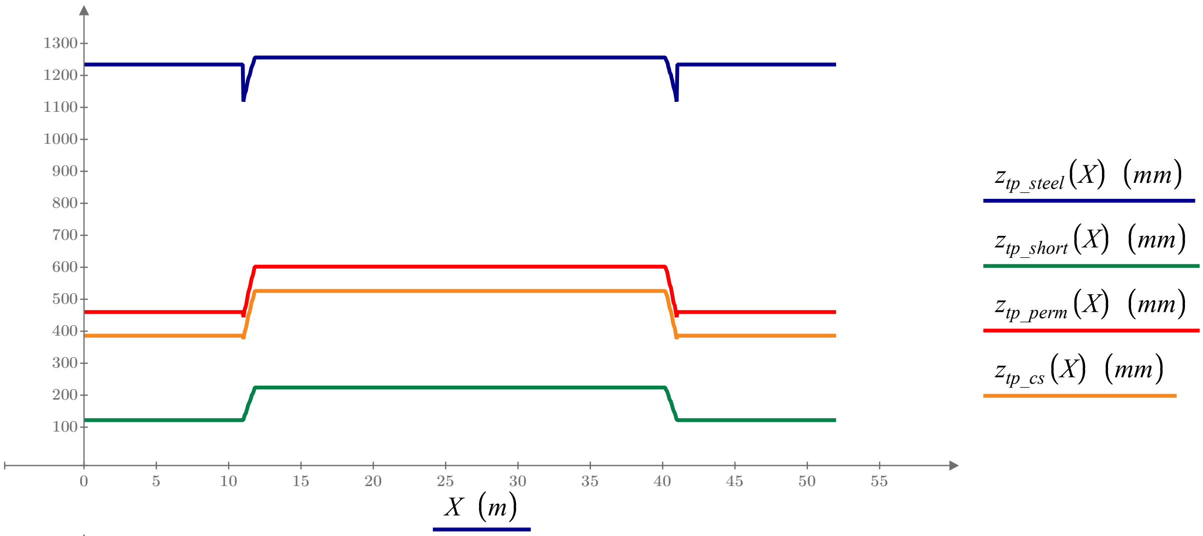
V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (both temperature and traffic) (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)

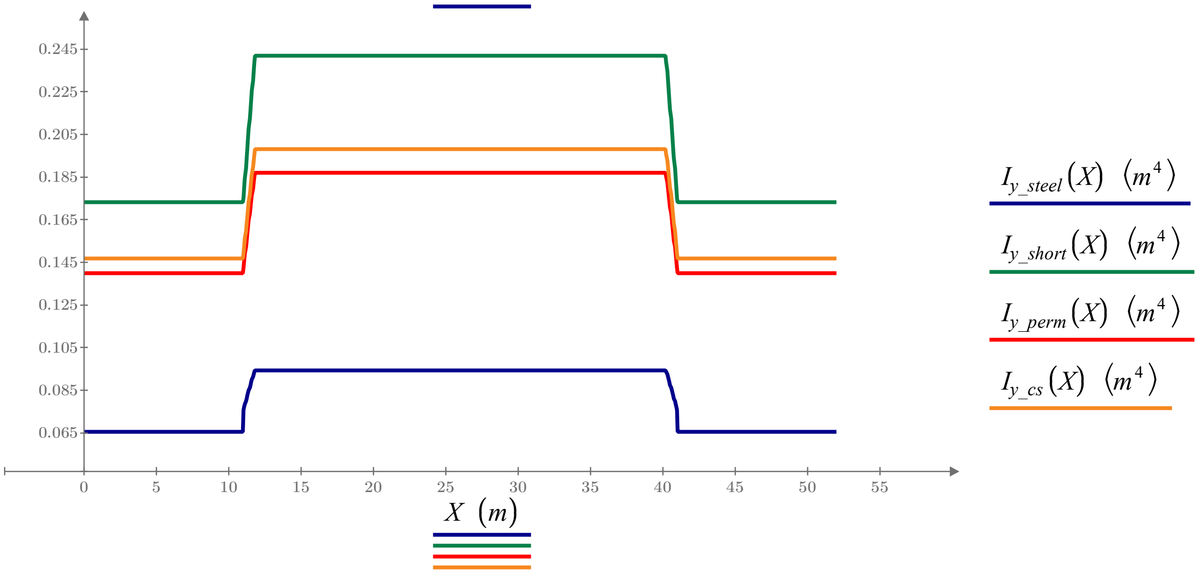


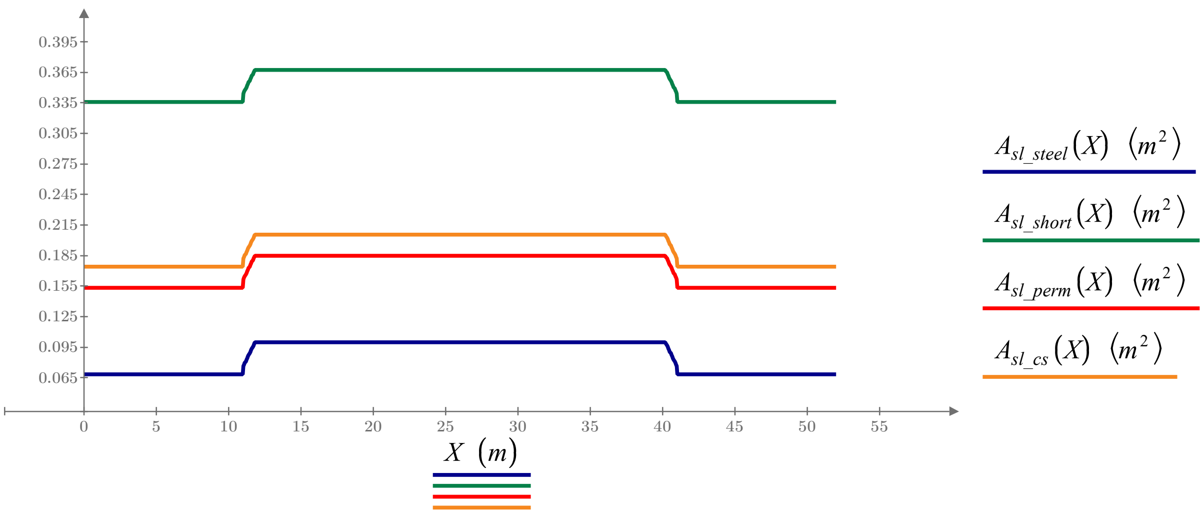
$V_{d\_ULS}(X_{check\_v}) = 4511\ kN$ Shear force at control point - ULS

6.2 Cross-sectional constants

For calculations, see chapter 5.







6.3 Stresses in steel cross-section

The stresses is calculated for each load case taking into acount load duration and creep. The stresses are then superpositioned.

6.3.1 Stresses during casting

$M := M_{ULS1}\left(X_{check\_m}\right) = 21141\ kN \cdot m$ Bending moment

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.094\ m^4$ Moment of inertia

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 1255\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2050\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot -z = -281\ MPa$ = $\sigma_{s.u.cast}\left(X_{check\_m}\right) = -281\ MPa$ Stresses in upper flange from loads durng casting

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 178\ MPa$ = $\sigma_{s.l.cast}\left(X_{check\_m}\right) = 178\ MPa$ Stresses in lower flange from loads durng casting

6.3.2 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses must be calculated for that specific section. The worst load case is determined dependent on the largest stress in each part whereas the multi-component load is the main load for the lower flange, and the temperature load is the worst load case for the upper flange.

6.3.2.1 Multi component loads (traffic)

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.367\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.242\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) = 223\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2050\ mm$ Height of girder

$\sigma_{s.u} := \frac{0.9\ N}{A} + \frac{0.9\ M}{I} \cdot (-z) = -17\ MPa$ = $\sigma_{s.u.tr}\left(X_{check\_m}\right) = -17\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{1.5\ N}{A} + \frac{1.5\ M}{I} \cdot (h - z) = 234\ MPa$ = $\sigma_{s.l.tr}\left(X_{check\_m}\right) = 234\ MPa$ Stresses in lower flange from short term loads

6.3.2.2 Temperature loads

$M := M_{temp}\left(X_{check\_m}\right) = 8379\ kN \cdot m$ Bending moment

$N := N_{temp}\left(X_{check\_m}\right) = -6476\ kN$ Normal force

$A_s := A_{sl\_steel}\left(X_{check\_m}\right) = 0.1\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.367\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}\left(X_{check\_m}\right) = 0.094\ m^4$ Moment of inertia - steel section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.242\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}\left(X_{check\_m}\right) = 1255\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 223\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}\left(X_{check\_m}\right) = 2050\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 3270\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(1 - \left(\frac{A_s}{A} + \frac{A_s}{I}\left(z_s - z\right)^2\right)\right) = -1887\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{1.5\ N_s}{A_s} + \frac{1.5\ M_s}{I_s} \cdot \left(-z_s\right) = -94\ MPa$ = $\sigma_{s.u.temp}\left(X_{check\_m}\right) = -94\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{0.9\ N_s}{A_s} + \frac{0.9\ M_s}{I_s} \cdot \left(h - z_s\right) = 8\ MPa$ = $\sigma_{s.l.temp}\left(X_{check\_m}\right) = 8\ MPa$ Stresses in lower flange from short term loads

6.3.3 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.187\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) = 602\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2050\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot (-z) = -13\ MPa$ = $\sigma_{s.u.perm}(X_{check\_m}) = -13\ MPa$ Stresses in upper flange from additional permanent loads

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 32\ MPa$ = $\sigma_{s.l.perm}(X_{check\_m}) = 32\ MPa$ Stresses in lower flange from additional permanent loads

6.3.4 Stresses due to shrinkage

In the same manner as the temperature load must be calculated for the steel section, the shrinkage which acts on the concrete section must be converted for the steel section.

$M := M_{ULS4}(X_{check\_m}) = 3765\ kN \cdot m$ Bending moment

$N := N_{ULS4}(X_{check\_m}) = -5206\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.1\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_cs}(X_{check\_m}) = 0.205\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.094\ m^4$ Moment of inertia - steel section

$I := I_{y\_cs}(X_{check\_m}) = 0.198\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 1255\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_cs}(X_{check\_m}) = 526\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 2050\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 1796\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(\frac{A_s}{A} - \frac{A_s}{I} \cdot \left(z - \frac{h_{m.slab}}{2}\right) \cdot \left(z_s + \frac{h_{m.slab}}{2} - \left(z - \frac{h_{m.slab}}{2}\right)\right)\right) = -1519\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (-z_s) = -39\ MPa$ = $\sigma_{s.u.shrink}(X_{check\_m}) = -39\ MPa$ Stresses in upper flange due to shrinkage

$\sigma_{s.l} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (h - z_s) = 0\ MPa$ = $\sigma_{s.l.shrink}(X_{check\_m}) = 0\ MPa$ Stresses in lower flange due to shrinkage

6.3.5 Summary of stresses

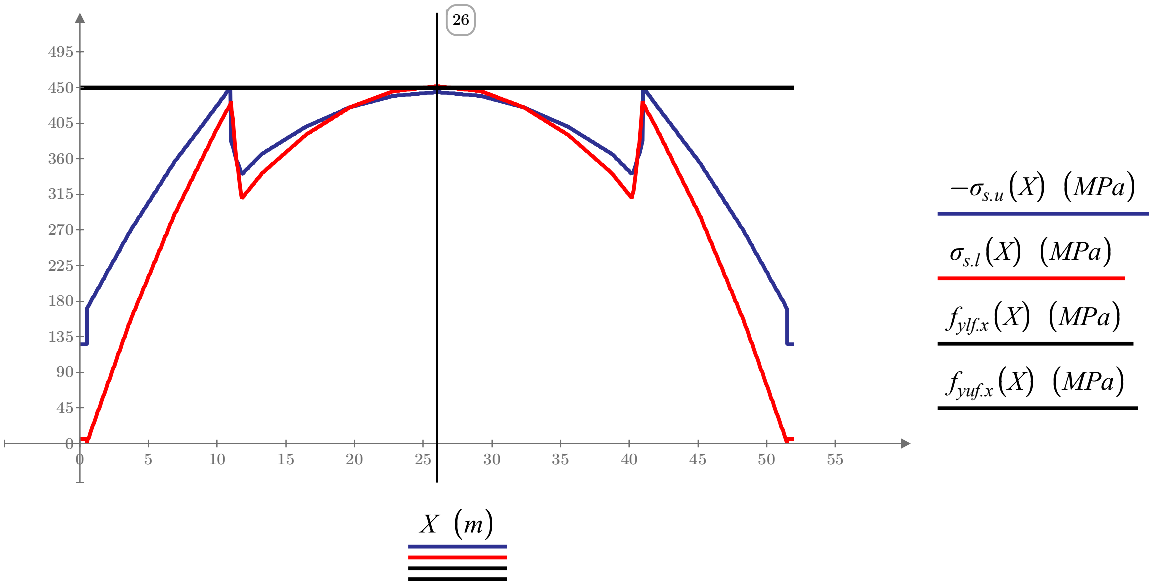
The stresses from the different phases are summarised accordingly.

$$\sigma_{s.u}(x) := \sigma_{s.u.cast}(x) + \sigma_{s.u.shrink}(x) + \sigma_{s.u.perm}(x) + \sigma_{s.u.tr}(x) + \sigma_{s.u.temp}(x)$$

Stresses in upper flange

$$\sigma_{s.l}(x) := \sigma_{s.l.cast}(x) + \sigma_{s.l.shrink}(x) + \sigma_{s.l.perm}(x) + \sigma_{s.l.tr}(x) + \sigma_{s.l.temp}(x)$$

Stresses in lower flange



$$\eta_{\sigma.u.max} := max\left(\frac{-\sigma_{s.u}(X)}{f_{yuf.x}(X)}\right) = 100\%$$

Maximum utilization rate - upper flange

$$\eta_{\sigma.l.max} := max\left(\frac{\sigma_{s.l}(X)}{f_{ylf.x}(X)}\right) = 100\%$$

Maximum utilization rate - lower flange

6.4 Calculation of stresses in concrete

The stresses in the concrete is calculated with the same principle as the steel taking creep into account.

6.4.1 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses for the concrete must be calculated separately.

6.4.1.1 Traffic and wind loads

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.367\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.242\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) + h_{m.slab} = 543\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2050\ mm$ Height of girder

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$\sigma_c := \left(\frac{N}{A} + \frac{-M \cdot z}{I}\right) \cdot \frac{1}{n_\Gamma} = -8\ MPa$ = $\sigma_{c.short}\left(X_{check\_m}\right) = -8\ MPa$ Stresses in concrete from short-term loads

6.4.1.2 Temperature load

$M := M_{temp}\left(X_{check\_m}\right) = 8379\ kN \cdot m$

$N := N_{temp}\left(X_{check\_m}\right) = -6476\ kN$

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.367\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \frac{1}{n_\Gamma} = 0.268\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.242\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}\left(X_{check\_m}\right)}{n_\Gamma} \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.002\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 223\ mm$ Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \frac{I_{c.eff}}{I} \cdot M = 79\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(\frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot \left(z_s - z\right) \cdot \left(z + \frac{h_{m.slab}}{2}\right)\right) = -1887\ kN$ Normal force imposed on concrete slab

$\sigma_c := \left(\frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c}\right) = 0.26\ MPa$ = $\sigma_{c.temp}\left(X_{check\_m}\right) = 0.26\ MPa$ Stresses in concrete from temperature loads

6.4.2 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.187\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) + h_{m.slab} = 922\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2050\ mm$ Height of girder

$n_\Gamma := n_{L_1} = 18.48$ Creep

$\sigma_c := \frac{-M \cdot z}{I} \cdot \frac{1}{n_\Gamma} = -1.1\ MPa$ = $\sigma_{c.perm}\left(X_{check\_m}\right) = -1.1\ MPa$ Stresses in concrete from permanent loads

6.4.3 Stresses due to shrinkage

$M := M_{ULS4}\left(X_{check\_m}\right) = 3765\ kN \cdot m$ — Bending moment

$N := N_{ULS4}\left(X_{check\_m}\right) = -5206\ kN$ — Normal force

$n_{\Gamma} := n_{L_3} = 14.91$ — Modular ratio considering creep

$A := A_{sl\_cs}\left(X_{check\_m}\right) = 0.205\ m^2$ — Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_{\Gamma}} = 0.106\ m^2$ — Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ — Cross-section area - concrete section

$I := I_{y\_cs}\left(X_{check\_m}\right) = 0.198\ m^4$ — Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \dfrac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ — Moment of inertia - concrete section

$I_{c.eff} := \dfrac{b_{eff}\left(X_{check\_m}\right)}{n_{\Gamma}} \cdot \dfrac{h_{m.slab}^{\ 3}}{12} = 0.001\ m^4$ — Moment of inertia - effective concrete section

$z := z_{tp\_cs}\left(X_{check\_m}\right) = 526\ mm$ — Center of gravity - composite section

$z_c := \dfrac{h_{m.slab}}{2} = 160\ mm$ — Center of gravity - concrete section

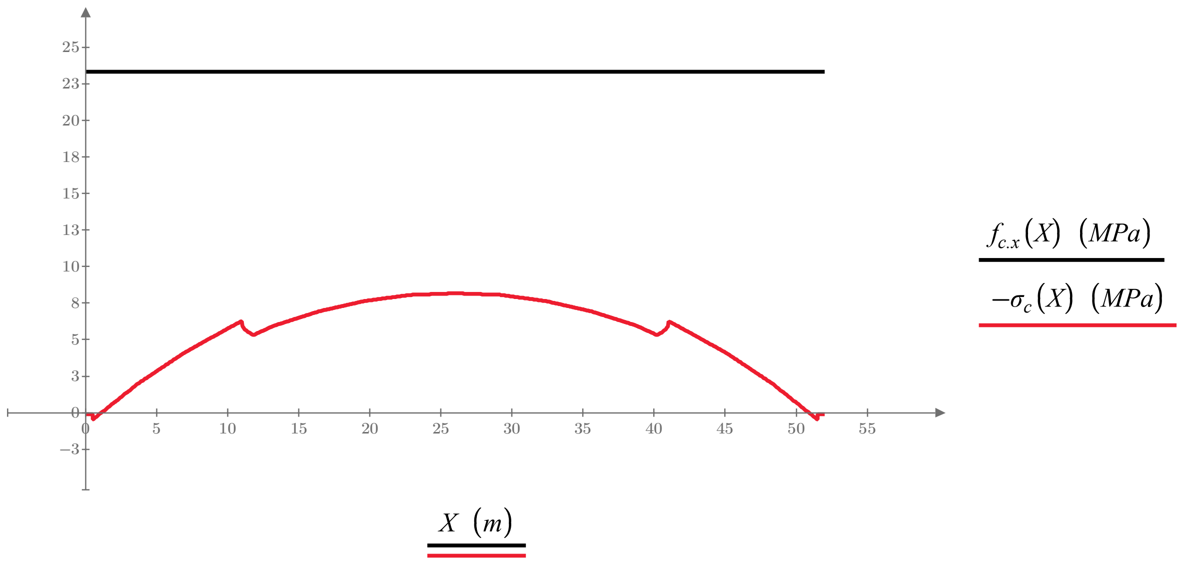
$M_c := \dfrac{I_{c.eff}}{I} \cdot M = 17\ kN \cdot m$ — Bending moment imposed on concrete slab

$N_c := N \cdot \left(1 - \dfrac{A_{c.eff}}{A} - \dfrac{A_{c.eff}}{I} \cdot \left(z + \dfrac{h_{m.slab}}{2}\right)^2\right) = -1215\ kN$ — Normal force imposed on concrete slab

$\sigma_c := \dfrac{-N_c}{A_c} + \dfrac{M_c \cdot -z_c}{I_c} = 0.6\ MPa \quad = \quad \sigma_{c.shrink}\left(X_{check\_m}\right) = 0.6\ MPa$ — Stresses in concrete from shrinkage

6.4.4 Summary of stresses

$\sigma_c(x) := \sigma_{c.short}(x) + \sigma_{c.temp}(x) + \sigma_{c.perm}(x) + \sigma_{c.shrink}(x)$ Stresses in concrete



$\eta_{\sigma.c.max} := max\left(\frac{-\sigma_c(X)}{f_{c.x}(X)}\right) = 35\%$ Maximum utilization rate - upper flange

6.5 Shear capacity

The shear capacity of the corrugated steel girder is calculated accoridng to SS-EN 1993-1-5, Appendix D.
The local- and global buckling factor are calculated according to SS-EN 1993-1-5, Appendix D.

Local buckling factor

$a_{c1} = 120\ mm$ $\quad a_{c2} = 119\ mm$ — Corrugation geometries

$a_{cmax} := max\left(a_{c1}, a_{c2}\right) = 120\ mm$ — SS-EN 1993-1-5 D.2.2.(2)

$tw := t_w\left(X_{check\_v}\right) = 9\ mm$ — Web thickness

$\tau_{cr} := 4.83 \cdot E_s \cdot \left(\dfrac{tw}{a_{cmax}}\right)^2 = 5434\ MPa$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.7

$\lambda_c := \sqrt{\dfrac{f_{yw}\left(X_{check\_v}\right)}{\tau_{cr} \cdot \sqrt{3}}} = 0.226$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.6

$\chi_l := min\left(\dfrac{1.15}{0.9 + \lambda_c}, 1\right) = 1.00 \quad = \quad \chi_{c.l}\left(X_{check\_v}\right) = 1.00$ — Reduction factor local buckling - SS-EN 1993-1-5 D.2.2.(2) Equation D.5

Global buckling factor

$D_X := \dfrac{E_s \cdot tw^3}{12 \cdot \left(1 - \nu^2\right)} \cdot \dfrac{w_c}{s_c} = 12\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$D_Z := \dfrac{E_s \cdot tw \cdot a_{c3}^{\,2}}{12} \cdot \dfrac{\left(3\ a_{c1} + a_{c2}\right)}{w_c} = 1628\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$hw := h_w\left(X_{check\_v}\right) = 1965\ mm$ — Web height

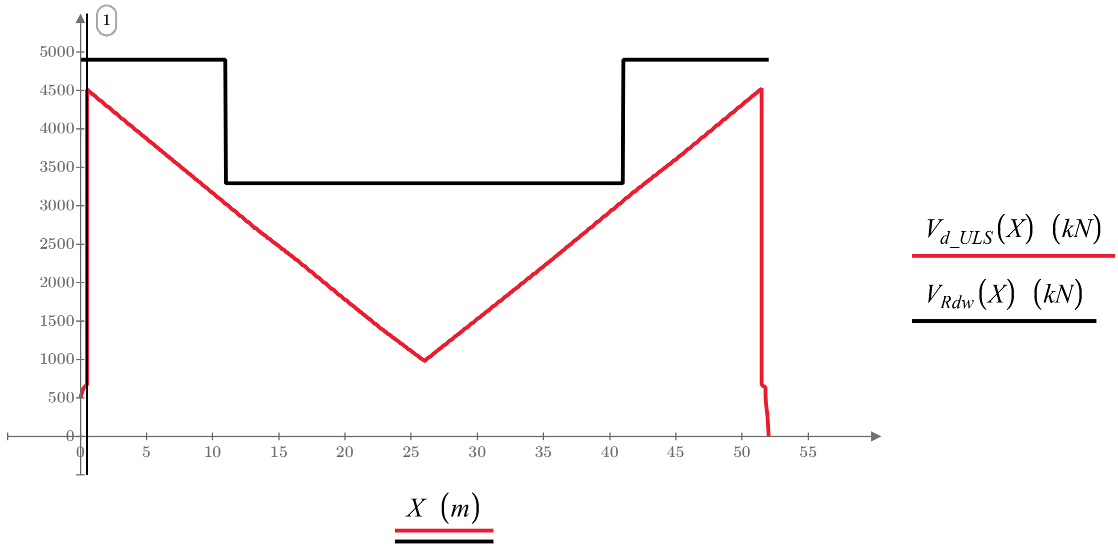
$\tau := \dfrac{32.4}{tw \cdot hw^2} \cdot \sqrt[4]{\dfrac{D_X}{N \cdot m} \cdot \dfrac{D_Z^{\,3}}{(N \cdot m)^3}}\ N \cdot m = 445\ MPa$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.10

$\lambda := \sqrt{\dfrac{f_{yw}\left(X_{check\_v}\right)}{\tau \cdot \sqrt{3}}} = 0.79$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.9

$\chi_g := min\left(\dfrac{1.5}{0.5 + \lambda^2}, 1\right) = 1.00 \quad = \quad \chi_{c.g}\left(X_{check\_v}\right) = 1.00$ — Global buckling factor SS-EN 1993-1-5 D.2.2.(3) Equation D.8

$\chi_C := min\left(\chi_g, \chi_l\right) = 1 \quad = \quad \chi_c\left(X_{check\_v}\right) = 1.00$ — SS-EN 1993-1-5 D.2.2

$V_{Rd} := \chi_C \cdot \dfrac{f_{yw}\left(X_{check\_v}\right)}{\gamma_{M1} \cdot \sqrt{3}} \cdot hw \cdot tw = 4901\ kN \quad = \quad V_{Rdw}\left(X_{check\_v}\right) = 4901\ kN$ — SS-EN 1993-1-5 D.2.2.(1) Eq.D.4



$\eta_{V.max} = 93\%$ Maximum utilization rate

6.6 Web breathing

Web breathing is checked against the largest subsection of the corrugation. The subsection is considered to be a stiffened plate. The check is performed according to SS-EN 1993-2, 7.4.

$$Breathing\_check := \left\| \begin{array}{l} \text{if } \dfrac{h_w\left(\dfrac{L_{bridge}}{2}\right)}{t_w\left(\dfrac{L_{bridge}}{2}\right)} > 30 + 4 \cdot \dfrac{L_{span}}{m} \\ \quad \| \; out \leftarrow \text{“Check breathing”} \\ \text{else} \\ \quad \| \; out \leftarrow \text{“OK!”} \end{array} \right\| = \text{“Check breathing”}$$ SS-EN 1993-2 7.4 (2) Eq: 7.5

$b_p := max\left(a_{c1}, a_{c2}\right) = 120\ mm$ "Fictive" plate, longest distance of corrugation

$t_{w\_u} := t_w\left(X_{check\_v}\right) = 9\ mm$ Thickness of web

$h_{w\_u} := h_w\left(X_{check\_v}\right) = 1965\ mm$ Height of web

$\sigma_{E\_u} := \dfrac{\pi^2 \cdot E_s \cdot {t_{w\_u}}^2}{12 \cdot \left(1 - \nu^2\right) \cdot {h_{w\_u}}^2} = 4\ MPa$ SS-EN 1993-2 7.4 (3)

$c_f := c_{uf}\left(X_{check\_v}\right) = 406\ mm$

$$k_{\tau\_u} := \left\| \begin{array}{l} \text{if } \dfrac{b_p}{h_{w\_u}} \geq 1 \\ \quad \left\| \; out \leftarrow 5.34 + 4 \cdot \left(\dfrac{h_{w\_u}}{b_p}\right)^2 \right. \\ \text{else} \\ \quad \left\| \; out \leftarrow 4 + 5.34 \cdot \left(\dfrac{h_{w\_u}}{b_p}\right)^2 \right. \end{array} \right\| = 1435.87$$ SS-EN 1993-1-5 Appendix A.3

$k_{\sigma\_u} := min\left(0.43 + \left(\dfrac{a_{c1} + 2\ a_{c4}}{c_f}\right)^2, 0.6\right) = 0.6$ SS-EN 1993-1-5 Appendix D

$\sigma_{x.ED.ser} := 0\ MPa$ Is zero due to accordion effect of web

$V_1 := V_{SLS1}\left(X_{check\_v}\right) = 1370\ kN$ Shear force from casting

$S_{u\_steel} := S_{uw\_steel}\left(X_{check\_v}\right) = \left(33 \cdot 10^{-3}\right)\ m^3$ Second moment of area, casting

$I_{y\_steel\_u} := I_{y\_steel}\left(X_{check\_v}\right) = \left(65 \cdot 10^{-3}\right)\ m^4$ Moment of inertia, casting

$\tau_{x.ED.ser\_cast\_u} := \dfrac{V_1 \cdot S_{u\_steel}}{I_{y\_steel\_u} \cdot t_{w\_u}} = 76\ MPa$ Shear stresses from casting

$V_2 := V_{SLS2}\left(X_{check\_v}\right) = 1269\ kN$ — Shear force from short term loads

$S_{u\_short} := S_{uw\_short}\left(X_{check\_v}\right) = \left(78 \cdot 10^{-3}\right)\ m^3$ — Second moment of area, short term loads

$I_{y\_short\_u} := I_{y\_short}\left(X_{check\_v}\right) = \left(173 \cdot 10^{-3}\right)\ m^4$ — Moment of inertia, short term loads

$$\tau_{x.ED.ser\_short\_u} := \frac{V_2 \cdot S_{u\_short}}{I_{y\_short\_u} \cdot t_{w\_u}} = 64\ MPa$$

Shear stresses from short term loads

$V_3 := V_{SLS3}\left(X_{check\_v}\right) = 274\ kN$ — Shear force from permanent term loads

$S_{u\_perm} := S_{uw\_perm}\left(X_{check\_v}\right) = \left(65 \cdot 10^{-3}\right)\ m^3$ — Second moment of area, permanent term loads

$I_{y\_perm\_u} := I_{y\_perm}\left(X_{check\_v}\right) = \left(140 \cdot 10^{-3}\right)\ m^4$ — Moment of inertia, permanent term loads

$$\tau_{x.ED.ser\_perm\_u} := \frac{V_3 \cdot S_{u\_perm}}{I_{y\_perm\_u} \cdot t_{w\_u}} = 14\ MPa$$

Shear stresses from additional permanent loads

$$\tau_{x.ED.ser\_u} := \tau_{x.ED.ser\_cast\_u} + \tau_{x.ED.ser\_short\_u} + \tau_{x.ED.ser\_perm\_u} = 154\ MPa \quad = \quad \tau_{x.ED.ser}\left(X_{check\_v}\right) = 154\ MPa$$

$$Breathing_{\_u} := \begin{array}{l} \text{if } \sqrt{\left(\frac{\sigma_{x.ED.ser}}{k_{\sigma\_u} \cdot \sigma_{E\_u}}\right)^2 + \left(\frac{1.1 \cdot \tau_{x.ED.ser\_u}}{k_{\tau\_u} \cdot \sigma_{E\_u}}\right)^2} \le 1.1 \\ \quad out \leftarrow \text{"OK!"} \\ \text{else} \\ \quad out \leftarrow \text{"Not OK!"} \end{array} = \text{"OK!"} \quad = \quad Breathing\left(X_{check\_v}\right) = \text{"OK!"}$$

$$\eta_{breathing} := max\left(\frac{\overrightarrow{\sqrt{\left(\frac{\sigma_{x.ED.ser}}{k_\sigma(X) \cdot \sigma_E(X)}\right)^2 + \left(\frac{1.1 \cdot \tau_{x.ED.ser}(X)}{k_\tau(X) \cdot \sigma_E(X)}\right)^2}}}{1.1}\right) = 6\%$$

Maximum utilization breathing

6.7 Studs

6.7.1 Ultimate limit state

The capacity of the studs in the ultimate limit state are calculated according to SS-EN 1994-2 6.6.3

$d_{stud} \equiv 22\ mm$ — Diameter of stud

$h_{stud} \equiv 200\ mm$ — Length of stud

$f_{ub} = 800\ MPa$ — Characteristic strength of studs

$f_{u\_stud} := min\left(f_{ub}, 500\ MPa\right) = 500\ MPa$ — Ultimate strength shear stud - SS-EN 1994-2 6.6.3.1 (1)

$\frac{h_{stud}}{d_{stud}} = 9$ — Ratio height- diameter

$$\alpha_{stud} := \left\| \begin{array}{l} \text{if } 3 \le \frac{h_{stud}}{d_{stud}} \le 4 \\ \quad \left\| a \leftarrow 0.2 \cdot \left(1 + \frac{h_{stud}}{d_{stud}}\right) \right. \\ \text{else if } 4 < \frac{h_{stud}}{d_{stud}} \\ \quad \left\| a \leftarrow 1 \right. \\ a \end{array} \right. = 1.0$$

Correction factor for length to diameter ratio shear stud SS-EN 1994-2 6.6.3.1 (1)

$$P_{rd} := min\left(\left\| \begin{array}{l} \frac{0.8 \cdot f_{u\_stud} \cdot \pi \cdot d_{stud}^2}{4 \cdot \gamma_V} \\ \frac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \end{array} \right\| \right) = 122\ kN$$

Capacity of one shear stud SS-EN 1994-2 6.6.3.1 Eq: 6.18,6.19

$S_{uf\_short}\left(X_{check\_v}\right) = \left(76 \cdot 10^{-3}\right)\ m^3$ — Second moment of area

$V_{ULS2}\left(X_{check\_v}\right) = 2538\ kN$ — Shear force

$I_{y\_short}\left(X_{check\_v}\right) = \left(173 \cdot 10^{-3}\right)\ m^4$ — Moment of inertia

$$\tau_{sh} := \frac{S_{uf\_short}\left(X_{check\_v}\right) \cdot V_{ULS2}\left(X_{check\_v}\right)}{I_{y\_short}\left(X_{check\_v}\right)} = 1108\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for short term loads

$S_{uf\_perm}\left(X_{check\_v}\right) = \left(53 \cdot 10^{-3}\right)\ m^3$ — Second moment of area

$V_{ULS3}\left(X_{check\_v}\right)=329\ kN$ Shear force

$I_{y\_perm}\left(X_{check\_v}\right)=\left(140\cdot 10^{-3}\right)\ m^4$ Moment of inertia

$$\tau_{pe}:=\frac{S_{uf\_perm}\left(X_{check\_v}\right)\cdot V_{ULS3}\left(X_{check\_v}\right)}{I_{y\_perm}\left(X_{check\_v}\right)}=124\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for permanent loads

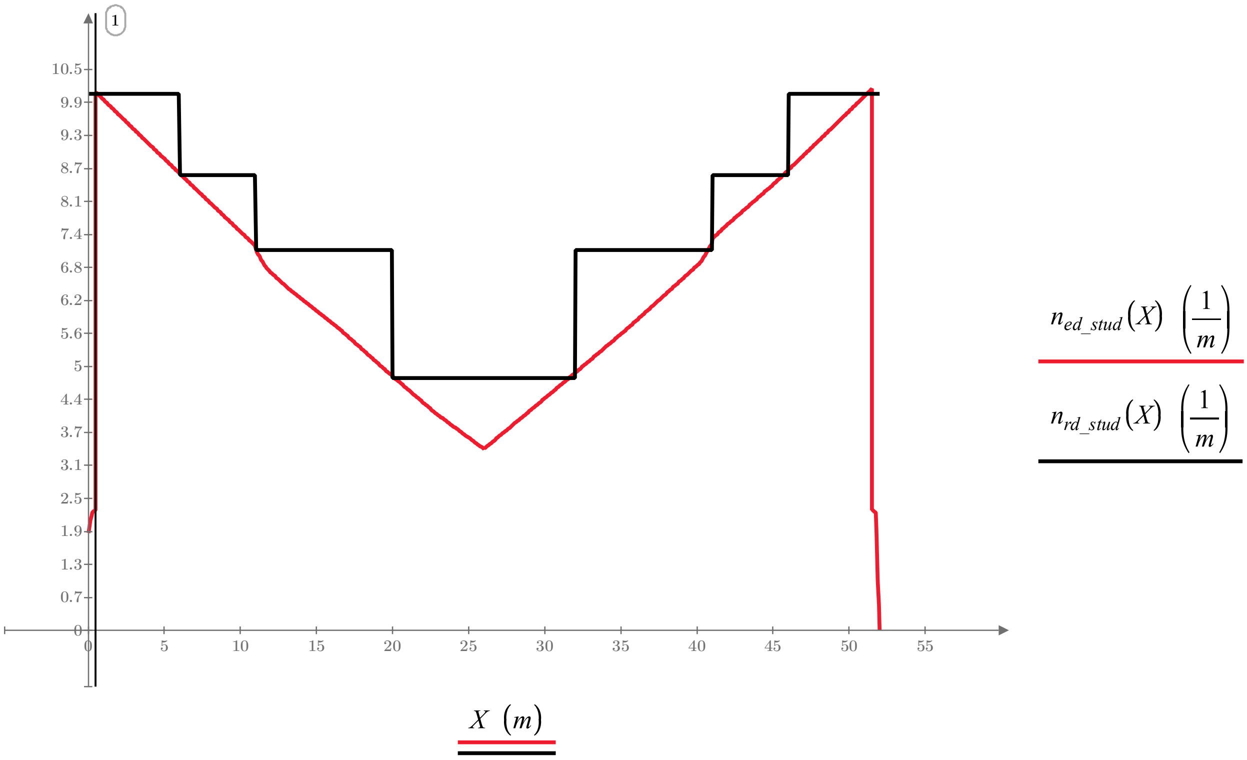
$$\tau:=\left|\tau_{sh}+\tau_{pe}\right|=1232\ \frac{kN}{m}$$

Total shear force per meter between concrete and top flange

$n_{ed} := \dfrac{\tau + 0.9\ max\left(q_{1k\_br}, q_{tr\_br}\right)}{P_{rd}} = 10.1\ \dfrac{1}{m} =$ $n_{ed\_stud}\left(X_{check\_v}\right) = 10.1\ \dfrac{1}{m}$ Number of studs needed per meter

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



6.7.1.1 Additional studs for full anchorage

Case 1: Temperature and shrinkage causes the slab to contract and therefore they are working in the opposite direction as the shear flow from ULS- loads during bending.

The anchorage length is calculated according to SS-EN 1994-2, 6.9 (3).

$B_{out} = 2.525\ m$ Distance from centre web to outer part of edge beam

$B_{in} = 2.8\ m$ Distance from web to centerline bridge

$b := max\left(B_{out}, B_{in}\right) = 2.8\ m$

$l_{anch} := 1.5 \cdot b = 4.2\ m$ Anchorage length - SS-EN 1994-2, 6.9 (3)

$N_{cs\_stud} = 826\ kN$ Shrinkage force imposed on studs - calculated in chapter 4

$\left|N_{temp\_stud_1}\right| = 1897\ kN$ Temperature force imposed on studs causing contraction - calculated in chapter 4

$$n_{ed\_cs} := \frac{1.0\ N_{cs\_stud}}{l_{anch} \cdot P_{rd}} = 1.6\ \frac{1}{m}$$

$$n_{ed\_temp} := \frac{1.5\ \left|N_{temp\_stud_1}\right|}{l_{anch} \cdot P_{rd}} = 5.5\ \frac{1}{m}$$

$$n_{ed\_anch.1} := n_{ed\_cs} + n_{ed\_temp} = 7.1\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 10.1\ \frac{1}{m}$$

$$check_1 := \textbf{if}\left(n_{ed\_anch.1} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_1 = \text{“No extra studs are needed”}$$

Case 2: Temperature causes the slab to expand and therefore working in the same direction as the shear flow from ULS- loads during bending. Shrinkage causes the slab to contract, i.e. working in the opposite direction.

$$-n_{ed\_cs} = -1.6\ \frac{1}{m}$$

$$N_{temp\_stud_2} = 1536\ kN$$

Temperature force imposed on studs causing expansion - calculated in chapter 4

$$n_{ed\_temp} := \frac{1.5 \cdot 0.6\ N_{temp\_stud_2}}{l_{anch} \cdot P_{rd}} = 2.7\ \frac{1}{m}$$

$$n_{ed\_stud}(0\ m) = 1.8\ \frac{1}{m}$$

$$n_{ed\_anch.2} := n_{ed\_stud}(0\ m) + n_{ed\_temp} - n_{ed\_cs} = 2.9\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 10.1\ \frac{1}{m}$$

$$check_2 := \textbf{if}\left(n_{ed\_anch.2} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_2 = \text{“No extra studs are needed”}$$

Extra studs needed due to shrinkage and temperature

$n_{rd\_stud.adj}(X)$ Needed amount of studs with regards to extra anchorage due to temperature and shrinkage

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section (ULS- loads)

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section

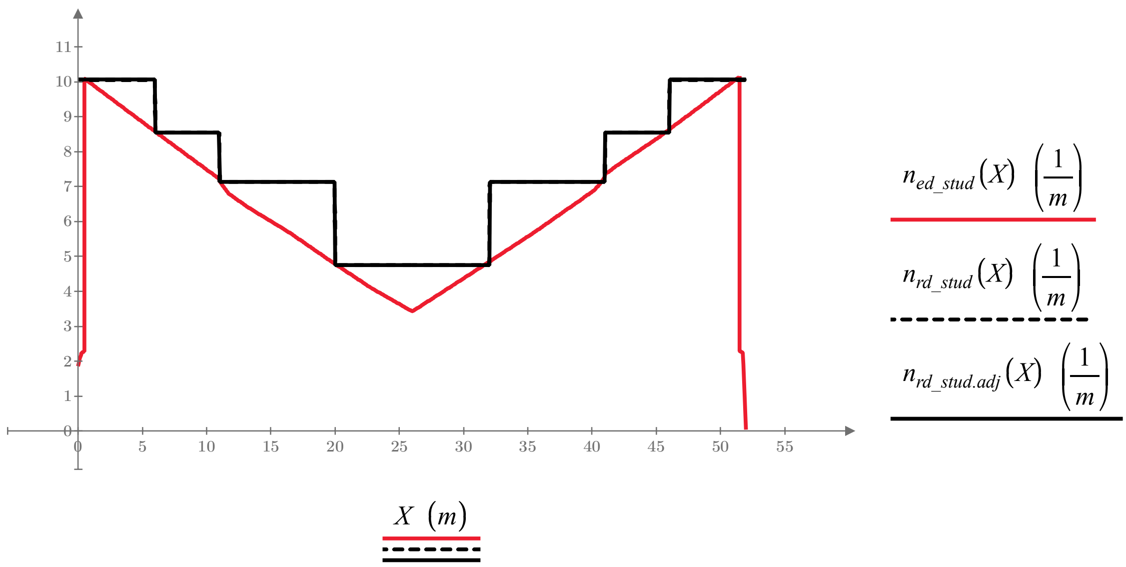


Table. Showing the adjusted need for studs near supports

X	n_{rd}	$n_{rd.adj}$
0	10.1	**10.1**
0.5	10.1	**10.1**
4.2	10.1	**10.1**
6	9.0	**9.0**
11	7.6	**7.6**
20	5.3	**5.3**

6.7.2 Fatigue limit state

6.7.2.1 Capacity

The fatigue capacity is calculated according to SS-EN 1993-1-9, Table 8.5 and SS-EN 1994-2, 6.8.3.

$\Delta\tau_c := 90\ MPa$ — SS-EN 1993-1-9 - Table 8.5 (10)

$\Delta\tau_{E2} = \lambda_v \cdot \Delta\tau_c$

$\lambda_{v.1} := 1.55$ — Bridge length less than 100m - SS-EN 1994-2 6.8.6.2 (4)

$Q_{mi} := 410\ kN$ — Mean weight of large vehicles in the slow lane

$Q_0 := 480\ kN$

$N_{obs} := 0.05 \cdot 10^6$

$N_0 := 0.5 \cdot 10^6$

$$\lambda_{v.2} := \frac{Q_{mi}}{Q_0} \cdot \left(\frac{N_{obs}}{N_0}\right)^{\frac{1}{8}} = 0.64$$

SS-EN 1994-2 6.8.6.2 (4) and SS-EN 1993-2 Eq. 9.10

$t_{Ld} := 120$ — Expected service life [years]

$$\lambda_{v.3} := \left(\frac{120}{100}\right)^{\frac{1}{8}} = 1.02$$

$\lambda_{v.4} := 1.0$ — TSFS 2018:57 - 27 ch. 3 §

$\lambda_v := \lambda_{v.1} \cdot \lambda_{v.2} \cdot \lambda_{v.3} \cdot \lambda_{v.4} = 1.02$

$\gamma_{Ff} := 1.0$ — SS-EN 1993-2, 9.3 (1)

$\gamma_{mF} := 1.0$ — SS-EN 1994-2, 2.4.1.2 (6)

$$\gamma_{Ff} \cdot \Delta\sigma_{E2} < \frac{\Delta\tau_c}{\gamma_{mF}}$$

$$F_{rd\_stud} := \Delta\tau_c \cdot \frac{\pi \cdot d_{stud}^{\ 2}}{4} = 34.2\ kN$$

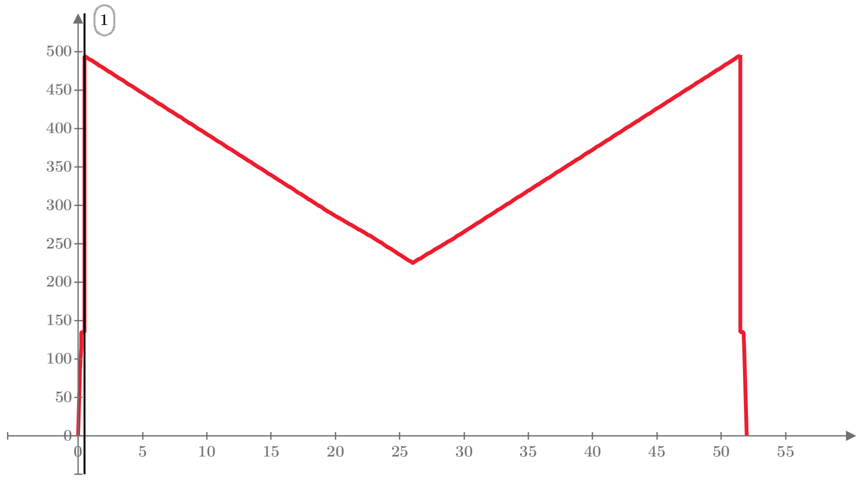
Shear fatigue capacity - one stud

$$F_{rd\_stud} := \frac{F_{rd\_stud}}{\lambda_v} = 33.7\ kN$$

Considering trafic load

6.7.2.2 Fatigue load

$V_{FAT}(X)$ Fatigue load from trafic - see Appendix X



$$\tau \cdot b = \frac{SV}{I} = \frac{stud\_capacity}{m}$$

$$S_{uf} := S_{uf\_short}\left(X_{check\_v}\right) = 0.076\ m^3$$

$$V := V_{FAT}\left(X_{check\_v}\right) = 494\ kN$$

$$I := I_{y\_short}\left(X_{check\_v}\right) = 0.173\ m^4$$

$$\Delta\tau := \frac{S_{uf} \cdot V}{I} = 216\ \frac{kN}{m}$$

$F_{rd\_stud} = 34\ kN$ Fatigue capacity - one stud

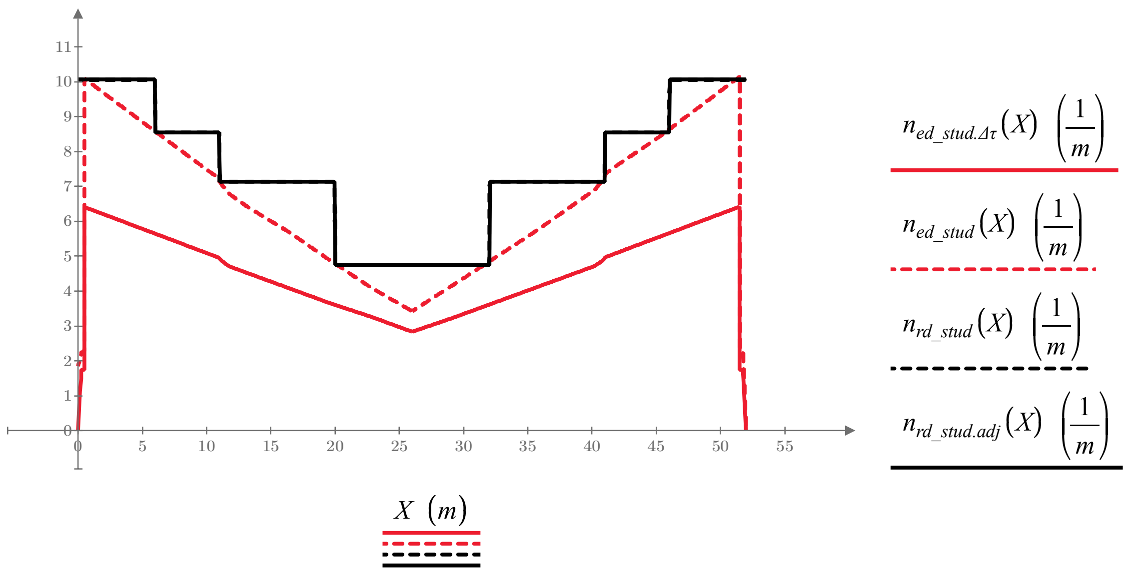
$$n_{ed\_\Delta\tau} := \frac{\Delta\tau}{F_{rd\_stud}} = 6.4\ \frac{1}{m} \quad = \quad n_{ed\_stud.\Delta\tau}\left(X_{check\_v}\right) = 6.4\ \frac{1}{m}$$

6.7.2.3 Design of studs with regards to fatigue

$n_{ed\_stud.\Delta\tau}(X)$ Needed amount of studs with regards to fatigue

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



Extra need of studs with regards to fatigue

$$Check := \mathbf{if}\left(max\left(\frac{n_{ed\_stud.\Delta\tau}(X)}{n_{rd\_stud}(X)}\right), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$Check = \text{“No extra studs are needed”}$$

6.7.3 Summary - design of studs

Table. Showing the stud design for half of the span

X	$n_{stud.used}$	$n_{stud.need}$	$n_{stud.fat}$
0	**10.1**	1.8	0.0
0.5	**10.1**	10.1	6.4
4.2	**10.1**	9.1	5.9
6	**9.0**	8.6	5.6
11	**7.6**	7.2	4.9
20	**5.3**	4.8	3.6

6.8 Utilization rates

$\eta_{\sigma.u.max} = 100\%$	Stresses in top flange
$\eta_{\sigma.l.max} = 100\%$	Stresses in lower flange
$\eta_{\sigma.c.max} = 35\%$	Stresses in concrete
$\eta_{V.max} = 93\%$	Shear capacity
$\eta_{breathing} = 6\%$	Breathing

7 Capacity checks - Ultimate limit state, local

The capacity checks that are to be carried out are to evaluate the local effects. The capacity of the capacity of stiffeners, welds and cross-beam is performed in this chapter.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

7.1 Load effects in ULS

7.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

Bending moment in the ultimate limit state

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{ULS2}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

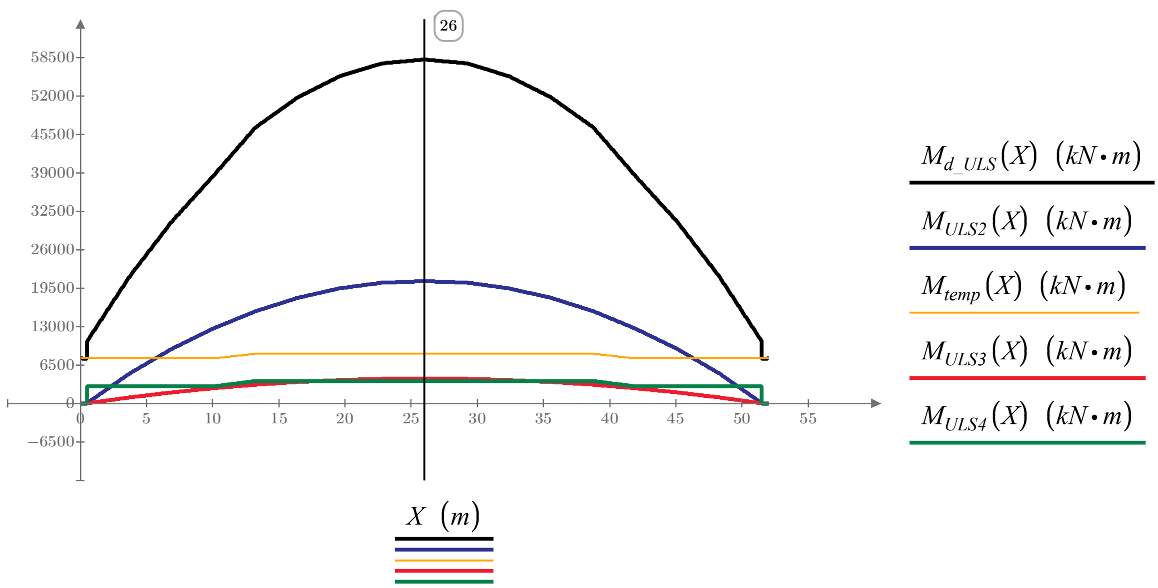
M_{d_ULS} Design bending moment effect from all loads

$M_{ULS1}(x)$ Bending moment during casting (Not used here - Appendix X)

$M_{ULS2}(x)$ Bending moment from variable loads (Appendix X)

$M_{ULS3}(x)$ Bending moment from additional permanent loads after construction (Appendix X)

$M_{ULS4}(x)$ Bending moment from shrinkage (Appendix X)

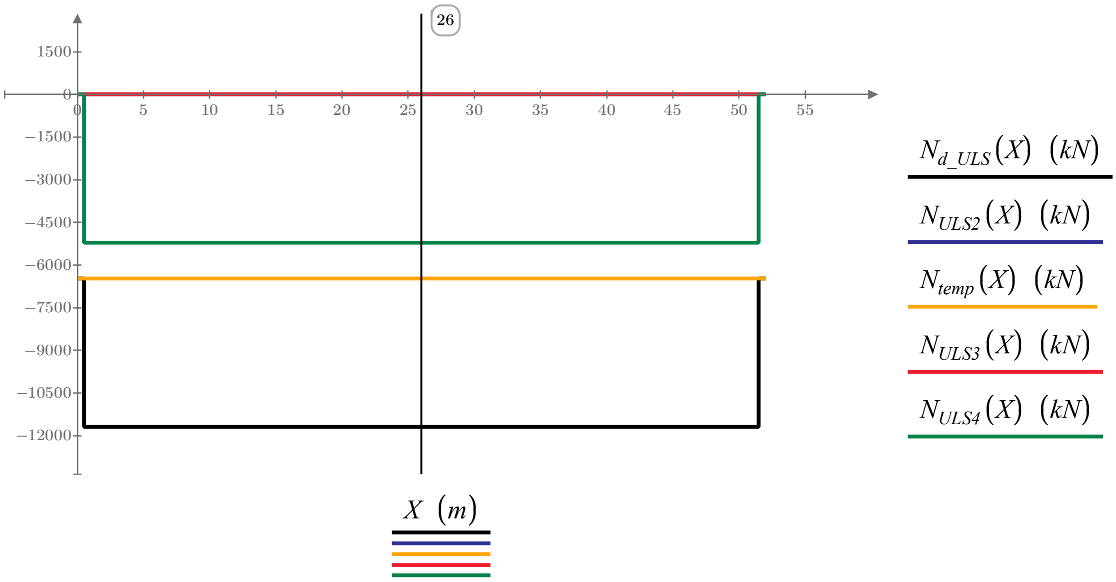


$M_{d\_ULS}(X_{check\_m}) = 58124\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{ULS2}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design bending moment effect from all loads
$N_{ULS1}(x)$	Bending moment during casting (Not used here - Appendix X)
$N_{ULS2}(x)$	Bending moment from variable loads (Appendix X)
$N_{ULS3}(x)$	Bending moment from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Bending moment from shrinkage (Appendix X)

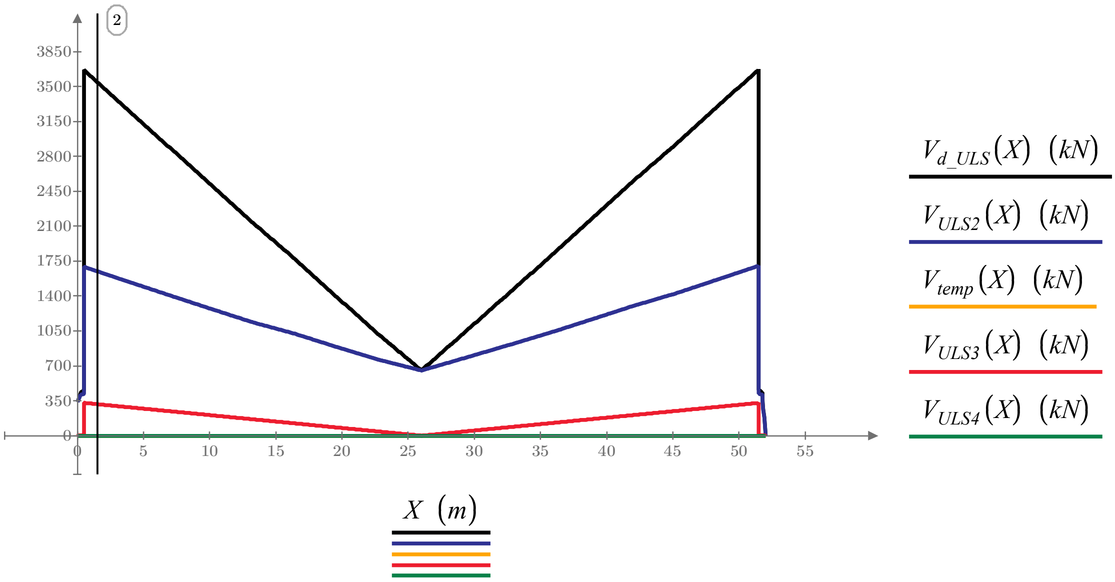


$N_{d\_ULS}(X_{check\_m}) = -11682\ kN$ Bending moment at control point - ULS

7.1.2 Shear force

$$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{temp}(x) + V_{ULS3}(x) + V_{ULS4}(x)$$

V_{d_ULS}	Shear force descending from all load effects in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)



$V_{d\_ULS}(X_{check\_v}) = 3545\ kN$ Shear force at control point - ULS

7.2 Transeverse stiffeners

Field

$x_{avst} := 11\ m$ — Location where the web changes thickness

$t_{ast\_f} := 15\ mm$ — Thickness of stiffener

$b_{ast\_f} := 360\ mm$ — Width of stiffener

Cross section class

$$CSC_{st} := \begin{array}{l} \text{if } \dfrac{t_{ast\_f}}{b_{ast\_f}} \le 9\ \varepsilon_{uf} \\ \quad \| \text{“CSC 1”} \\ \text{else if } 9\ \varepsilon_{uf} < \dfrac{t_{ast\_f}}{b_{ast\_f}} \le 10\ \varepsilon_{uf} \\ \quad \| \text{“CSC 2”} \\ \text{else if } 10\ \varepsilon_{uf} < \dfrac{t_{ast\_f}}{b_{ast\_f}} \le 14\ \varepsilon_{uf} \\ \quad \| \text{“CSC 3”} \\ \text{else} \\ \quad \| \text{“CSC 4”} \end{array} = \text{“CSC 1”}$$

The length of the web that is contributing is 11 ε t according to SS-EN 1993-1-4 5.7 (3)

$\varepsilon_{uf} = 0.71$

$t_w(x_{avst}) = 7\ mm$

$$b_{w\_ast\_f} := 11 \cdot \varepsilon_{uf} \cdot t_w(x_{avst}) = 54\ mm$$

The minimum allowed stiffness - SS-EN 1993-1-5 9.3.3 Equation 9.6

$$I_{st} := \begin{array}{l} \text{if } \dfrac{l_{cr}}{h_w(x_{avst})} \ge \sqrt{2} \\ \quad \| j \leftarrow 0.75 \cdot h_w(x_{avst}) \cdot t_w(x_{avst})^3 \\ \text{else if } \dfrac{l_{cr}}{h_w(x_{avst})} < \sqrt{2} \\ \quad \| j \leftarrow \dfrac{1.5 \cdot h_w(x_{avst})^3 \cdot t_w(x_{avst})^3}{l_{cr}^3} \\ j \end{array}$$

$$I_{xast\_f} := \frac{\left(2 \cdot b_{w\_ast\_f} + t_{ast\_f}\right) \cdot t_w(x_{avst})^3}{12} + \frac{t_{ast\_f} \cdot b_{ast\_f}^3}{12} + t_{ast\_f} \cdot b_{ast\_f} \cdot \left(\frac{b_{ast\_f} + t_w(x_{avst})}{2}\right)^2 = \left(240 \cdot 10^{-6}\right)\ m^4$$

Stiffness of stiffener

$$\begin{array}{l} \text{if } \dfrac{I_{xast\_f}}{I_{st}} \ge 1 \\ \quad \| \text{“OK!”} \\ \text{else} \\ \quad \| \text{“NOT OK”} \end{array} = \text{“OK!”}$$

Controll if stiffener satisfies stiffness requirment

Support

$x_{avst} := 0.5\ m$ Location of support

$t_{ast} := 26\ mm$ Thickness of stiffener

$b_{ast} := 440\ mm$ Width of stiffener

$$CSC_{st} := \begin{array}{l} \text{if } \dfrac{t_{ast}}{b_{ast}} \le 9\ \varepsilon_{uf} \\ \quad \text{"CSC 1"} \\ \text{else if } 9\ \varepsilon_{uf} < \dfrac{t_{ast}}{b_{ast}} \le 10\ \varepsilon_{uf} \\ \quad \text{"CSC 2"} \\ \text{else if } 10\ \varepsilon_{uf} < \dfrac{t_{ast}}{b_{ast}} \le 14\ \varepsilon_{uf} \\ \quad \text{"CSC 3"} \\ \text{else} \\ \quad \text{"CSC 4"} \end{array} = \text{"CSC 1"}$$

Cross section class

The length of the web that is contributing is 11 ε t according to SS-EN 1993-1-4 5.7 (3)

$\varepsilon_{uf} = 0.71$

$t_w(x_{avst}) = 9\ mm$

$b_{w\_ast} := 11 \cdot \varepsilon_{uf} \cdot t_w(x_{avst}) = 70\ mm$

$$I_{st} := \begin{array}{l} \text{if } \dfrac{l_{cr}}{h_w(x_{avst})} \ge \sqrt{2} \\ \quad j \leftarrow 0.75 \cdot h_w(x_{avst}) \cdot t_w(x_{avst})^3 \\ \text{else if } \dfrac{l_{cr}}{h_w(x_{avst})} < \sqrt{2} \\ \quad j \leftarrow \dfrac{1.5 \cdot h_w(x_{avst})^3 \cdot t_w(x_{avst})^3}{l_{cr}^3} \\ j \end{array}$$

The minimum allowed stiffness - SS-EN 1993-1-5 9.3.3 Equation 9.6

$A_{st\_sup} := t_{ast} \cdot b_{ast} + t_w(x_{avst}) \cdot b_{w\_ast}$

$$I_{xast} := \frac{(2 \cdot b_{w\_ast} + t_{ast}) \cdot t_w(x_{avst})^3}{12} + 2 \cdot \left(\frac{t_{ast} \cdot b_{ast}^3}{12}\right) = (369 \cdot 10^{-6})\ m^4$$

Stiffness of stiffener

$$\begin{array}{l} \text{if } \dfrac{I_{xast}}{I_{st}} \ge 1 \\ \quad \text{"OK!"} \\ \text{else} \\ \quad \text{"NOT OK"} \end{array} = \text{"OK!"}$$

Controll if stiffener satisfies stiffness requirment

Check of out of plane buckling according to SS-EN 1993-1-5 9.4 (2)

$h_w\left(x_{avst}\right)=1965\ mm$

$l_{cr\_ast}:=0.75\cdot h_w\left(x_{avst}\right)=1474\ mm$ SS-EN 1993-1-4 5.7 (2)

$A_{st\_sup}=\left(1\cdot 10^{-2}\right)\ m^2$

$$i:=\sqrt{\frac{I_{xast}}{A_{st\_sup}}}=175\ mm$$

$$\lambda_{ast}:=\frac{l_{cr\_ast}}{\pi\cdot i}\cdot\sqrt{\frac{f_{yw}}{E_s}}=0.13$$

$\alpha_{ast}:=0.49$ SS-EN 1993-1-4 5.7 (2)

$$\Phi_{ast}:=0.5\cdot\left(1+\alpha_{ast}\cdot\left(\lambda_{ast}-0.2\right)+\lambda_{ast}{}^2\right)=0.49$$

$$\chi_{ast}:=min\left(\frac{1}{\Phi_{ast}+\sqrt{\Phi_{ast}{}^2-\lambda_{ast}{}^2}},1.0\right)=1$$ Buckling factor stiffener

$\gamma_{M1}=1$

$$N_{b.rd}:=\frac{2\cdot b_{ast}\cdot t_{ast}\cdot f_{yw}\cdot\chi_{ast}}{\gamma_{M1}}=10982\ kN$$ Buckling load of stiffener

$V_{d\_ULS}\left(X_{check\_v}\right)=3545\ kN$

$$\eta_{ast}:=\frac{V_{d\_ULS}\left(X_{check\_v}\right)}{N_{b.rd}}=32.3\%$$ Utilization ratio

7.3 Welds

Material parameters

$f_{uuf} = 650\ MPa$ — Ultimate tensile stress of upper flange

$f_{ulf} = 650\ MPa$ — Ultimate tensile stress of lower flange

$f_{u.W.fill} = 650\ MPa$ — Ultimate tensile stress of filler material

$f_{u.FW} = 650\ MPa$ — Ultimate tensile stress of joint

$\beta_w := 1.0$ — SS-EN 1993-1-4 6.3

Vertical loads on web

Distributed loads on web

Concrete: $g_c := g_{slab} \cdot 1.2 = 52\ \dfrac{kN}{m}$ — Design self weight concrete -ULS

Cover: $g_{asf} := g_{cov} \cdot 1.32 = 16\ \dfrac{kN}{m}$ — Design self weight cover -ULS

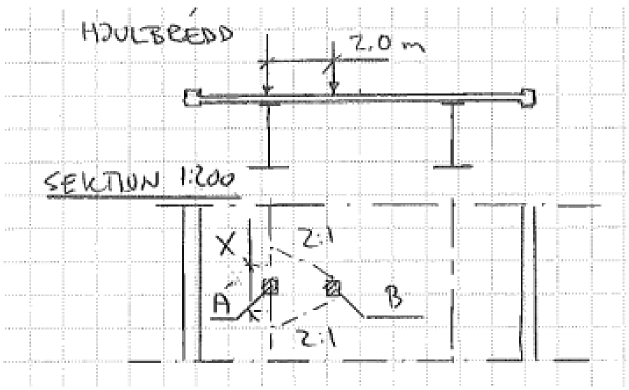
Point loads on web

The trafic load spreads though the concrete, cover and the road according to SS-EN 1991-2 Figure 4.4 --> load distributs 1:1. LM2 is the dimensioning load

$b_{wheel} := 0.2\ m$ — TSFS 2018_57 11 kap 2par

$t_{cov} = 110\ mm$ — Thickness of the cover

$$b_{load\_web} := b_{wheel} + \left(h_{m.slab} + t_{cov}\right) \cdot 2 = 1060\ mm$$



$$P_1 := \frac{1.5 \cdot Q_{ak} \cdot \alpha_{Q1}}{2 \cdot b_{load\_web}} = 229\ \frac{kN}{m}$$

$$P_2 := \frac{1.5 \cdot Q_{ak} \cdot \alpha_{Q1}}{2 \cdot \left(2\ m + b_{load\_web}\right)} = 79\ \frac{kN}{m}$$

$$P_{web} := g_c + g_{asf} + P_1 + P_2 = 377\ \frac{kN}{m}$$

Assume all load goes to one girder

Weld between web and top flange at support x=0.5m

Fillet weld

$a := 4\ mm$ Weld thickness

$X_{check\_w1} := 0.5\ m$ X-location at support

$\sigma_L(a) := \dfrac{P_{web}}{\sqrt{2} \cdot 2 \cdot a}$ Perpendicular normal stresses in weld

$\sigma_L(a) = 33\ MPa$

$\tau_L(a) := \sigma_L(a)$ Perpendicular shear stresses in weld

$\tau_{ll}(x, a) := \dfrac{S_{uw\_short}(x) \cdot V_{d\_ULS}(x)}{I_{y\_short}(x) \cdot 2 \cdot a}$ Parallell shear stresses in weld

$\sigma_{eq\_wu}(x, a) := \sqrt{\sigma_L(a)^2 + 3 \cdot \left(\tau_{ll}(x, a)^2 + \tau_L(a)^2\right)}$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$\sigma_{eq\_wu}(X_{check\_w1}, a) = 177\ MPa$

$\sigma_{Rd\_L} := \dfrac{0.9 \cdot f_{u.FW}}{\gamma_{M2}} = 488\ MPa$ SS-EN 1993-1-8 4.5.3.2 (6) EQ: 4.1

$\sigma_{Rd\_eq} := \dfrac{f_{u.FW}}{\beta_w \cdot \gamma_{M2}} = 542\ MPa$ SS-EN 1993-1-8 4.5.3.2 (6) EQ: 4.1

$\eta_{w\_uf\_eq\_1} := \dfrac{\sigma_{eq\_wu}(X_{check\_w1}, a)}{\sigma_{Rd\_eq}} = 33\%$ Utilization ratio

$\eta_{w\_uf\_L\_1} := \dfrac{\sigma_L(a)}{\sigma_{Rd\_L}} = 7\%$ Utilization ratio

Weld between web and top flange at x=11m

Fillet weld

$a = 4\ mm$ Weld thickness

$X_{check\_w2} := 11\ m$ X-location at change of web thickness

$\sigma_{eq\_wu}(X_{check\_w2}, a) = 127\ MPa$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$\sigma_L(a) = 33\ MPa$ Perpendicular normal stresses in weld

$\eta_{w\_uf\_eq\_2} := \dfrac{\sigma_{eq\_wu}(X_{check\_w2}, a)}{\sigma_{Rd\_eq}} = 24\%$ Utilization ratio

$\eta_{w\_uf\_L\_2} := \dfrac{\sigma_L(a)}{\sigma_{Rd\_L}} = 7\%$ Utilization ratio

Weld between web and lower flange at support x=0.5m

Butt weld

$$\tau_{ll.b}(x) := \frac{S_{lw\_short}(x) \cdot V_{d\_ULS}(x)}{I_{y\_short}(x) \cdot t_w(x)}$$ Parallel shear stresses in weld

$$\tau_{ll.b}(X_{check\_w1}) = 184\ MPa$$

$$b_{bearing} := 440\ mm$$ Width of bearing plate

$$P_{bearing} := 5000\ kN$$ Bearing force

$$A_{lw\_sup}(x) := t_w(x) \cdot b_{bearing} + t_{ast} \cdot (b_{bearing} - t_w(x))$$ Loaded area from bearing

$$\sigma_L(x) := \frac{P_{bearing}}{A_{lw\_sup}(x)}$$ Perenperdicular normal stresses in weld

$$\sigma_L(X_{check\_w1}) = 330\ MPa$$

$$\sigma_{eq\_wl}(x) := \sqrt{\sigma_L(x)^2 + 3\ \tau_{ll.b}(x)^2}$$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$$\sigma_{eq\_wl}(X_{check\_w1}) = 459\ MPa$$

$$\sigma_{Rd\_eq} = 542\ MPa$$ Capacity of joint - Von Mises stresses

$$\sigma_{Rd\_L} = 488\ MPa$$ Capancity of joint - stresses perpendicular to weld

$$\eta_{w\_lf\_eq\_1} := \frac{\sigma_{eq\_wl}(X_{check\_w1})}{\sigma_{Rd\_eq}} = 85\%$$ Utilization ratio

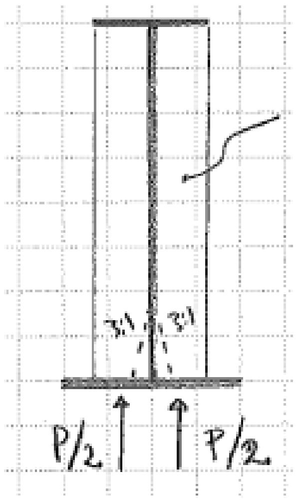
$$\eta_{w\_lf\_L\_1} := \frac{\sigma_L(X_{check\_w1})}{\sigma_{Rd\_L}} = 68\%$$ Utilization ratio

Weld between stiffener and lower flange at support x=0.5m

Butt weld

Vertical force taken by the stiffener from the bearing

$$P_{v\_ast}(x) := P_{bearing} \cdot \frac{t_{ast} \cdot (b_{bearing} - t_w(x))}{A_{lw\_sup}(x)}$$



Horizontal force as a result of 3:1 spread of load

$P_{h\_side} = 98\ kN$ Side force from acceleration

$$P_{h\_ast}(x) := \frac{P_{v\_ast}(x)}{3} + P_{h\_side}$$

$\sigma_L(X_{check\_w1}) = 330\ MPa$ See above

$\tau_{ll.s}(x) := \dfrac{P_{h\_ast}(x)}{2 \cdot t_{ast} \cdot b_{ast}}$ Parallell shear stresses in weld

$\sigma_{eq\_sl}(x) := \sqrt{\sigma_L(x)^2 + 3\ \tau_{ll.s}(x)^2}$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$\sigma_{eq\_sl}(X_{check\_w1}) = 345\ MPa$

$\sigma_{Rd\_eq} = 542\ MPa$ Capacity of joint - Von Mises stresses

$\sigma_{Rd\_L} = 488\ MPa$ Capancity of joint - stresses perpendicular to weld

$\eta_{w\_lf\_eq\_2} := \dfrac{\sigma_{eq\_sl}(X_{check\_w1})}{\sigma_{Rd\_eq}} = 64\%$ Utilization ratio

$\eta_{w\_lf\_L\_2} := \dfrac{\sigma_L(X_{check\_w1})}{\sigma_{Rd\_L}} = 68\%$ Utilization ratio

Weld between web and lower flange at x=1m

Fillet weld

$a = 4\ mm$ Weld thickness

$X_{check\_w3} := 1\ m$ X-location

$\sigma_L := 0\ MPa$ Perpendicular normal stresses in weld

$\tau_L := \sigma_L$ Perpendicular shear stresses in weld

$\tau_{ll}\left(X_{check\_w3}, a\right) = 93\ MPa$ Parallell shear stresses in weld

$\sigma_{eq\_wu}\left(X_{check\_w3}, a\right) = 174\ MPa$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$\sigma_{Rd\_eq} = 542\ MPa$ Capacity of joint - Von Mises stresses

$\eta_{w\_lf\_eq\_3} := \dfrac{\sigma_{eq\_wu}\left(X_{check\_w3}, a\right)}{\sigma_{Rd\_eq}} = 32\%$ Utilization ratio

Weld between web and lower flange x=11m

$a = 4\ mm$ Weld thickness

$X_{check\_w2} := 11\ m$ X-location

$\sigma_{eq\_wu}\left(X_{check\_w2}, a\right) = 127\ MPa$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$\sigma_{Rd\_eq} = 542\ MPa$ Capacity of joint - Von Mises stresses

$\eta_{w\_lf\_eq\_4} := \dfrac{\sigma_{eq\_wu}\left(X_{check\_w2}, a\right)}{\sigma_{Rd\_eq}} = 24\%$ Utilization ratio

7.2.1 Utilization welds and stiffeners

Stiffener

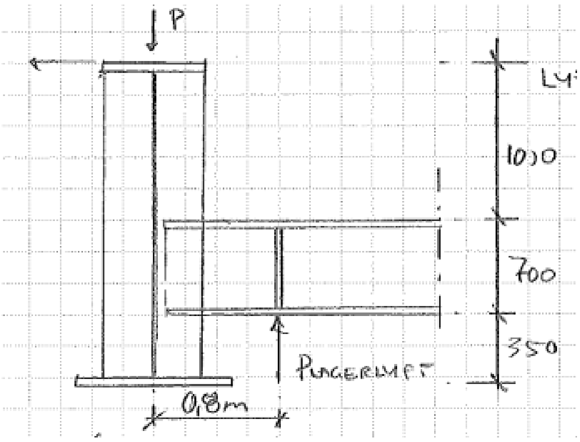
$\eta_{ast} = 32\%$	Buckling of stiffener

Welds

$\eta_{w\_uf\_eq\_1} = 33\%$	Weld at x=0.5 m upper flange equvilant stress
$\eta_{w\_uf\_L\_1} = 7\%$	Weld at x=0.5 m upper flange normal stress
$\eta_{w\_uf\_eq\_2} = 24\%$	Weld at x=11 m upper flange equvilant stress
$\eta_{w\_uf\_L\_2} = 7\%$	Weld at x=11 m upper flange normal stress
$\eta_{w\_lf\_eq\_1} = 85\%$	Weld at x=0.5 m lower flange equvilant stress
$\eta_{w\_lf\_L\_1} = 68\%$	Weld at x=0.5 m lower flange normal stress
$\eta_{w\_lf\_eq\_2} = 64\%$	Weld at x=0.5 m stiffener and lower flanger equvilant stress
$\eta_{w\_lf\_L\_2} = 68\%$	Weld at x=0.5 m stiffener and lower flanger normal stress
$\eta_{w\_lf\_eq\_3} = 32\%$	Weld at x=1 m lower flange, change of weld, equvilant stress
$\eta_{w\_lf\_eq\_4} = 24\%$	Weld at x=11 m lower flange equvilant stress

7.3 Cross-beams

7.3.1 Cross-section at support



Assume support cross-beam dimensions

$h_{w\_cbs} := 660\ mm$	Height of web -Cross-beam
$t_{w\_cbs} := 20\ mm$	Thickness of web -Cross-beam
$t_{fl\_cbs} := 20\ mm$	Thickness of lower flange -Cross-beam
$b_{fl\_cbs} := 400\ mm$	Width of lower flange -Cross-beam
$t_{fu\_cbs} := 20\ mm$	Thickness of upper flange -Cross-beam
$b_{fu\_cbs} := 400\ mm$	Width of upper flange -Cross-beam
$h_{cbs} := h_{w\_cbs} + t_{fl\_cbs} + t_{fu\_cbs} = 700\ mm$	Height of Cross-beam
$f_{ycbs} := f_{yw} = 480\ MPa$	Yield stress Cross-beam

$$CSC_{lf} := \begin{array}{l} \text{if } \dfrac{t_{fl\_cbs}}{\dfrac{b_{fl\_cbs}}{2} - t_{w\_cbs}} \le 9\ \varepsilon_{uf} \\ \quad \text{“CSC 1”} \\ \text{else if } 9\ \varepsilon_{uf} < \dfrac{t_{fl\_cbs}}{\dfrac{b_{fl\_cbs}}{2} - t_{w\_cbs}} \le 10\ \varepsilon_{uf} \\ \quad \text{“CSC 2”} \\ \text{else if } 10\ \varepsilon_{uf} < \dfrac{t_{fl\_cbs}}{\dfrac{b_{fl\_cbs}}{2} - t_{w\_cbs}} \le 14\ \varepsilon_{uf} \\ \quad \text{“CSC 3”} \\ \text{else} \\ \quad \text{“CSC 4”} \end{array} = \text{“CSC 1”}$$

Cross-section class lower and upper flange

$$CSC_{w} := \begin{array}{l} \text{if } \dfrac{t_{w\_cbs}}{h_{w\_cbs}} \le 72\ \varepsilon_{uf} \\ \quad \text{“CSC 1”} \\ \text{else if } 72\ \varepsilon_{uf} < \dfrac{t_{w\_cbs}}{h_{w\_cbs}} \le 76\ \varepsilon_{uf} \\ \quad \text{“CSC 2”} \\ \text{else if } 76\ \varepsilon_{uf} < \dfrac{t_{w\_cbs}}{h_{w\_cbs}} \le 90\ \varepsilon_{uf} \\ \quad \text{“CSC 3”} \\ \text{else} \\ \quad \text{“CSC 4”} \end{array} = \text{“CSC 1”}$$

Cross-section class web

<u>Cross-sectional constants cross-beam</u>

$$A_{cbs} := h_{w\_cbs} \cdot t_{w\_cbs} + t_{fl\_cbs} \cdot b_{fl\_cbs} + t_{fu\_cbs} \cdot b_{fu\_cbs} = \left(29 \cdot 10^{-3}\right)\ m^2$$

$$I_{y\_cbs} := \frac{h_{w\_cbs}^{\ 3} \cdot t_{w\_cbs}}{12} + \frac{t_{fl\_cbs}^{\ 3} \cdot b_{fl\_cbs}}{12} + t_{fl\_cbs} \cdot b_{fl\_cbs} \cdot \left(\frac{h_{cbs}}{2} - \frac{t_{fl\_cbs}}{2}\right)^2 + \frac{t_{fu\_cbs}^{\ 3} \cdot b_{fu\_cbs}}{12} + t_{fu\_cbs} \cdot b_{fu\_cbs} \cdot \left(\frac{h_{cbs}}{2} - \frac{t_{fu\_cbs}}{2}\right)^2 = \left(2 \cdot 10^{-3}\right)\ m^4$$

$$I_{z\_cbs} := \frac{h_{w\_cbs} \cdot t_{w\_cbs}^{\ 3}}{12} + \frac{t_{fl\_cbs} \cdot b_{fl\_cbs}^{\ 3}}{12} + \frac{t_{fu\_cbs} \cdot b_{fu\_cbs}^{\ 3}}{12} = \left(214 \cdot 10^{-6}\right)\ m^4$$

$$S_{wu\_cbs} := t_{fu\_cbs} \cdot b_{fu\_cbs} \cdot \left(\frac{h_{cbs}}{2} - \frac{t_{fu\_cbs}}{2}\right) = \left(3 \cdot 10^{-3}\right)\ m^3$$

$$S_{wl\_cbs} := S_{wu\_cbs} = \left(3 \cdot 10^{-3}\right)\ m^3$$

Forces in cross-beam

$d_{lift} := 0.8\ m$ Distance from lifting point to the centre of the main girder -Cross-beam

$X_{check\_cbs} := 0.5\ m$ Location of support cross-beam

$P_{lift} := V_{ULS1}\left(X_{check\_cbs}\right) + V_{ULS3}\left(X_{check\_cbs}\right)$ Lifting force when chaning bearing

$P_{lift} = 1973\ kN$ Lifting force when chaning bearing

$P_{wind} := 22\ kN \cdot 1.5 \cdot 0.3 = 9900\ N$ Wind force acting on the structure

$M_{ed\_cbs}(x) := P_{lift} \cdot d_{lift} + P_{wind} \cdot \left(\frac{h_w(x)}{2} + \frac{h_{cbs}}{2}\right)$ Bending moment arising when changing bearing

$M_{ed\_cbs}\left(X_{check\_cbs}\right) = 1591\ kN \cdot m$ Total moment at bearing change

7.3.1.1 Bending capacity at bearing change

$M_{rd\_cbs} := \frac{I_{y\_cbs}}{0.5 \cdot h_{cbs}} \cdot f_{yw} = 3194\ kN \cdot m$ Bending moment capacity

$M_{ed\_cbs}\left(X_{check\_cbs}\right) = 1591\ kN \cdot m$ Bending moment

$\eta_{cb\_M} := \frac{M_{ed\_cbs}\left(X_{check\_cbs}\right)}{M_{rd\_cbs}} = 50\%$ Utilization ratio

7.3.1.2 Shear capacity at bearing change

$\eta_{para\_cb} := 1.2$ SS-EN 1993-1-5 5.1 (2)

$$\varepsilon_{cb} := \sqrt{\frac{235\ MPa \cdot E_s}{f_{ycbs} \cdot 210\ GPa}} = 0.7$$ SS-EN 1993-1-4 Table 5.2

$$buckling\_check := \begin{array}{l} \text{if } \dfrac{h_{w\_cbs}}{t_{w\_cbs}} < \dfrac{56.2 \cdot \varepsilon_{cb}}{\eta_{para\_cb}} \\ \quad \text{“No plate buckling check”} \\ \text{else} \\ \quad \text{“Check plate buckling”} \end{array} = \text{“Check plate buckling”}$$ SS-EN 1993-1-5 5.1 (2)

$l_{cr\_cbs} := d_{girder} = 5.6\ m$ Buckling length of cross-beam

$a_{cbs} := l_{cr\_cbs} - 2 \cdot d_{lift} = 4\ m$

$$k_t := \begin{array}{l} \text{if } \dfrac{a_{cbs}}{h_{w\_cbs}} \ge 1 \\ \quad out \leftarrow 5.34 + 4 \cdot \left(\dfrac{h_{w\_cbs}}{a_{cbs}}\right)^2 \\ \text{else} \\ \quad out \leftarrow 4 + 5.34 \cdot \left(\dfrac{h_{w\_cbs}}{a_{cbs}}\right)^2 \end{array} = 5.45$$ SS-EN 1993-1-5 A.1 (2)

$$\sigma_{e\_cbs} := \frac{\pi^2 \cdot E_s \cdot t_{w\_cbs}^{\ 2}}{12\ (1 - \nu^2) \cdot h_{w\_cbs}^{\ 2}} = 166\ MPa$$ SS-EN 1993-1-5 A.3 (1)

$\tau_{cr\_cbs} := k_t \cdot \sigma_{e\_cbs} = 904\ MPa$ SS-EN 1993-1-5 5.3 (3) Eq: 5.4

$$\lambda_{w\_cbs} := 0.76 \cdot \sqrt{\frac{f_{ycbs}}{\tau_{cr\_cbs}}} = 0.55$$ SS-EN 1993-1-5 5.3 (3) Eq: 5.3

$$\chi_{w\_cbs} := \left\| \begin{array}{l} \text{if } buckling\_check = \text{"Check plate buckling"} \\ \quad \left\| out \leftarrow \left\| \begin{array}{l} \text{if } \lambda_{w\_cbs} \le \dfrac{0.65}{\eta_{para\_cb}} \\ \quad \| \eta_{para\_cb} \\ \text{else if } \dfrac{0.65}{\eta_{para\_cb}} < \lambda_{w\_cbs} < 0.65 \\ \quad \left\| \dfrac{0.65}{\lambda_{w\_cbs}} \right. \\ \text{else} \\ \quad \left\| \dfrac{1.56}{0.91 + \lambda_{w\_cbs}} \right. \end{array} \right. \right. \\ \text{else} \\ \quad \| out \leftarrow \eta_{para\_cb} \end{array} \right. = 1.17$$

SS-EN 1993-1-4 2006/A1 Table 5.4

$$V_{bw.rd} := \frac{\chi_{w\_cbs} \cdot f_{ycbs} \cdot h_{w\_cbs} \cdot t_{w\_cbs}}{\sqrt{3} \cdot \gamma_{M1}} = 4295\ kN$$

Shear capacity of cross-beam

$$\eta_{cb\_V} := \frac{P_{lift}}{V_{bw.rd}} = 46\%$$

Utilization ratio

7.3.1.3 Buckling at bearing change

$$l_{cr\_cbs} = 5.6\ m$$

Buckling length of cross-beam

$$i := \sqrt{\frac{I_{z\_cbs}}{A_{cbs}}} = 86\ mm$$

Radius of gyration

$$\lambda_{cbs} := \frac{l_{cr\_cbs}}{i \cdot \pi} \cdot \sqrt{\frac{f_{ycbs}}{E_s}} = 1.02$$

Slenderness of cross-beam - SS-EN 1993-1-4 5.4.2.1 Eq: 5.8

$$\alpha_{cbs} := 0.76$$

Buckling curve C - SS-EN 1993-1-4 Table 5.3

$$\Phi_{cbs} := 0.5 \cdot \left(1 + \alpha_{cbs} \cdot \left(\lambda_{cbs} - 0.2\right) + {\lambda_{cbs}}^2\right) = 1.33$$

SS-EN 1993-1-4 5.4.2.1 Eq: 5.7

$$\chi_{b\_cbs} := \frac{1}{\Phi_{cbs} + \sqrt{{\Phi_{cbs}}^2 - {\lambda_{cbs}}^2}} = 0.46$$

SS-EN 1993-1-4 5.4.2.1 Eq: 5.6

$$N_{b.rd\_cbs} := \frac{\chi_{b\_cbs} \cdot A_{cbs} \cdot f_{ycbs}}{\gamma_{M1}} = 6401.2\ kN$$

Buckling load

$$\eta_{comb\_cbs} := \frac{P_{wind}}{N_{b.rd\_cbs}} + \frac{M_{ed\_cbs}\left(X_{check\_cbs}\right)}{M_{rd\_cbs}} = 50\%$$

Utilization ratio

7.3.1.4 Weld at bearing change

Fillet weld web

$a = 4\ mm$ — Weld thickness

$\tau_{ll\_w\_cbs} := \dfrac{S_{wu\_cbs} \cdot P_{lift}}{I_{y\_cbs} \cdot 2 \cdot a} = 288\ MPa$ — Parallell shear stresses in weld

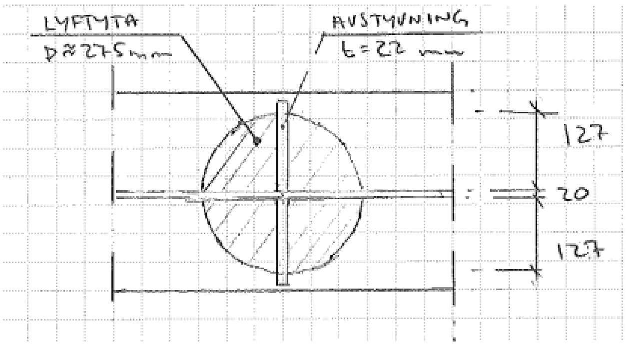
$\sigma_{eq\_cb\_1} := \sqrt{3 \cdot \tau_{ll\_w\_cbs}^{\ 2}} = 499\ MPa$ — Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$\sigma_{Rd\_eq} = 542\ MPa$ — Capacity of joint - Von Mises stresses

$\eta_{cb\_eq\_1} := \dfrac{\sigma_{eq\_cb\_1}}{\sigma_{Rd\_eq}} = 92\%$ — Utilization ratio

Butt weld at lifting point

$\tau_{ll\_l\_cbs} := \dfrac{S_{wu\_cbs} \cdot P_{lift}}{I_{y\_cbs} \cdot t_{w\_cbs}} = 115\ MPa$ — Parallell shear stresses in weld



$d_{jack} := 275\ mm$

$t_{ast\_cbs} := 22\ mm$

$b_{ast\_cbs} := 127\ mm$

$\sigma_{L\_l\_cbs} := \dfrac{P_{lift}}{t_{w\_cbs} \cdot d_{jack} + t_{ast\_cbs} \cdot d_{lift}} = 85\ MPa$ — Perpendicular normal stresses in weld

$\sigma_{eq\_cbs} := \sqrt{\sigma_{L\_l\_cbs}^{\ 2} + 3 \cdot \tau_{ll\_l\_cbs}^{\ 2}} = 217\ MPa$ — Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

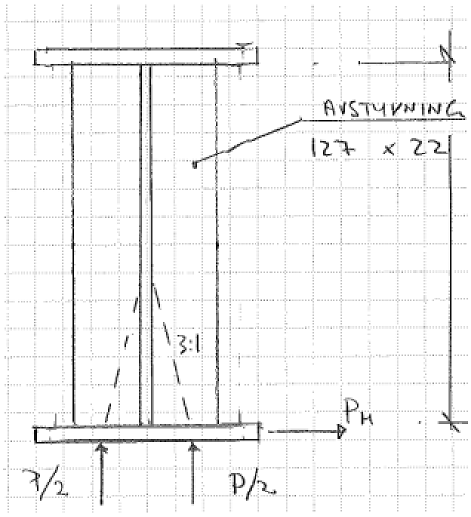
$\sigma_{Rd\_eq} = 542\ MPa$ — Capacity of joint - Von Mises stresses

$\sigma_{Rd\_L} = 488\ MPa$ — Capacity of joint - Normal stress perpendicular to weld

$\eta_{cb\_eq\_2} := \dfrac{\sigma_{eq\_cbs}}{\sigma_{Rd\_eq}} = 40\%$ — Utilization ratio

$\eta_{cb\_L\_2} := \dfrac{\sigma_{L\_l\_cbs}}{\sigma_{Rd\_L}} = 18\%$ — Utilization ratio

Butt weld at stiffener



$$P_{ast} := \frac{P_{lift} \cdot 2 \cdot b_{ast\_cbs}}{\left(d_{jack} + 2 \cdot b_{ast\_cbs}\right)} = 947\ kN$$ Load taken by stiffener

$$P_{h\_ast} := \frac{P_{ast}}{3}$$ Horisontal component of load

$$\tau_{ll\_cbs\_ast} := \frac{P_{h\_ast}}{2 \cdot b_{ast\_cbs} \cdot t_{ast\_cbs}} = 57\ MPa$$ Parallell shear stresses in weld

$$\sigma_{L\_l\_cbs} = 85\ MPa$$ see above

$$\sigma_{eq\_cbs\_s} := \sqrt{\sigma_{L\_l\_cbs}^{\ 2} + 3 \cdot \tau_{ll\_cbs\_ast}^{\ 2}} = 130\ MPa$$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$$\sigma_{Rd\_eq} = 542\ MPa$$ Capacity of joint - Von Mises stresses

$$\sigma_{Rd\_L} = 488\ MPa$$ Capacity of joint - Normal stress perpendicular to weld

$$\eta_{cb\_eq\_3} := \frac{\sigma_{eq\_cbs\_s}}{\sigma_{Rd\_eq}} = 24\%$$ Utilization ratio

$$\eta_{cb\_L\_3} := \frac{\sigma_{L\_l\_cbs}}{\sigma_{Rd\_L}} = 18\%$$ Utilization ratio

Fillet weld at stiffener

$a := 4\ mm$

Length of contributing weld

$$b_{Lw.1} := 1.2 - \frac{0.2 \cdot h_{w\_cbs}}{150 \cdot a} = 0.98$$

$$l_{weld} := b_{Lw.1} \cdot h_{w\_cbs} = 0.647\ m$$

Stresses in weld

$$\tau_{ll\_cbs\_ast} := \frac{P_{ast}}{2 \cdot 2 \cdot a \cdot l_{weld}} = 92\ MPa$$ Parallell shear stresses in weld

$$\sigma_{L\_cbs\_ast} := \frac{P_{h\_ast} \cdot \sqrt{2}}{2 \cdot 2 \cdot a \cdot l_{weld}} = 43\ MPa$$ Normal stress perpendicular to weld

$$\tau_{L\_cbs\_ast} := \sigma_{L\_cbs\_ast} = 43\ MPa$$ Perpendicular shear stress in weld

$$\sigma_{eq\_cbs\_ast} := \sqrt{\sigma_{L\_l\_cbs}^{\ 2} + 3 \cdot \left(\tau_{ll\_cbs\_ast}^{\ 2} + \tau_{L\_cbs\_ast}^{\ 2}\right)} = 195\ MPa$$ Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

Utilization ratio

$\sigma_{Rd\_eq} = 542\ MPa$ Capacity of joint - Von Mises stresses

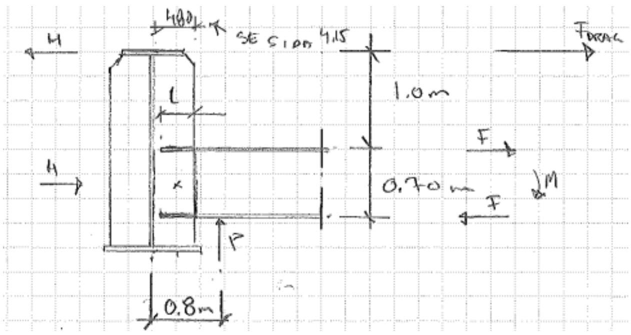
$\sigma_{Rd\_L} = 488\ MPa$ Capacity of joint - Normal stress perpendicular to weld

$$\eta_{cb\_eq\_4} := \frac{\sigma_{eq\_cbs\_ast}}{\sigma_{Rd\_eq}} = 36\%$$ Utilization ratio

$$\eta_{cb\_L\_4} := \frac{\sigma_{L\_cbs\_ast}}{\sigma_{Rd\_L}} = 9\%$$ Utilization ratio

Joint between cross-beam and main girder

Web is butt welded



$$\tau_{ll\_web\_j} := \frac{P_{lift}}{t_{w\_cbs} \cdot h_{w\_cbs}} = 149\ MPa$$

$$\sigma_{eq\_cb\_j} := \sqrt{3 \cdot \tau_{ll\_web\_j}^{\ 2}} = 259\ MPa$$

Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$$\sigma_{Rd\_eq} = 542\ MPa$$

Capacity of joint - Von Mises stresses

$$\eta_{cb\_eq\_5} := \frac{\sigma_{eq\_cb\_j}}{\sigma_{Rd\_eq}} = 48\%$$

Utilization ratio

Flanges is fillet welded

$$l_{weld\_fl} := 450\ mm$$

Lenght of weld

$$a := 6\ mm$$

Thickness of weld

$$F_M := \frac{P_{lift} \cdot d_{lift}}{h_{cbs}} = 2254\ kN$$

Force in flanges from bending moment due to excentric loading

$$\tau_{ll\_fl\_j} := \frac{F_M}{l_{weld\_fl} \cdot a \cdot 2 \cdot 2} = 209\ MPa$$

Parallell shear stresses in weld

$$\sigma_{eq\_cb\_j} := \sqrt{3 \cdot \tau_{ll\_fl\_j}^{\ 2}} = 362\ MPa$$

Von Mises stresses in weld - SS-EN 1993-1-8 4.5.3.2 EQ: 4.1

$$\sigma_{Rd\_eq} = 542\ MPa$$

Capacity of joint - Von Mises stresses

$$\eta_{cb\_eq\_6} := \frac{\sigma_{eq\_cb\_j}}{\sigma_{Rd\_eq}} = 67\%$$

Utilization ratio

7.3.1.6 Service state

A check is performed to see if the service state loads is the dimensioning case

Assumes all horisontal force goes to the cross-beam

$P_{wind} = 10\ kN$ Calculated in chapter 4

$P_{h\_side} = 98\ kN$ Calculated in chapter 4

$$M_{ed\_service}(x) := (P_{wind} + P_{h\_side}) \cdot \left(\frac{h_w(x)}{2} + \frac{h_{cbs}}{2}\right)$$

$M_{ed\_service}(X_{check\_cbs}) = 144\ kN \cdot m$ Bending moment

$M_{ed\_cbs}(X_{check\_cbs}) = 1591\ kN \cdot m$ Bending moment capacity of cross-beam

$$service\_state := \left\| \begin{array}{l} \text{if } M_{ed\_service}(X_{check\_cbs}) < M_{ed\_cbs}(X_{check\_cbs}) \\ \quad \| \text{“Not dimensioning”} \\ \text{else} \\ \quad \| \text{“Dimensioning”} \end{array} \right\| = \text{“Not dimensioning”}$$

7.3.2 Cross-beam in field

7.3.2.1 Top bar in casting phase

Assume VKR 140x140x5 mm as top bar

$h_{vkr} := 140\ mm$ — Height of top bar

$A_{cbf\_top} := 2670\ mm^2$ — Area of top bar

$I_{x\_cbf\_top} := 807 \cdot 10^4\ mm^4$ — Stiffness of top bar

$I_{xast\_f} = \left(240 \cdot 10^6\right)\ mm^4$ — Stiffness of transeverse stiffener

$\frac{I_{xast\_f}}{I_{x\_cbf\_top}} = 30$ — Stiffness ratio between stiffener and top bar

Due to the large difference in stiffness of the components little bending moment will be transferred between them

The horizontal force in the bar from the construction phase is assumed to be 1.5% of the compressive force of the top flange.

$F_{c\_top} := \frac{M_{ULS1}\left(X_{check\_m}\right)}{h_w\left(X_{check\_m}\right)} = 10814\ kN$ — Compressive force in the top flange

$P_{hor\_cbf\_top} := F_{c\_top} \cdot 1.5\% = 162\ kN$ — Compressive force in cross-beam in field

$P_{wind\_cbf} := 30\ kN$

$P_{D\_hor\_cbf\_top} := P_{hor\_cbf\_top} + P_{wind\_cbf} \cdot 0.3 \cdot 1.5 = 176\ kN$ — Total horisontal force in cross-beam

<u>**Buckling of compressed top bar**</u>

$\alpha_{cbf} := 0.49$ — Buckling curve C - SS-EN 1993-1-4 Table 5.3

$i := \sqrt{\frac{I_{x\_cbf\_top}}{A_{cbf\_top}}} = 55\ mm$ — Radius of gyration

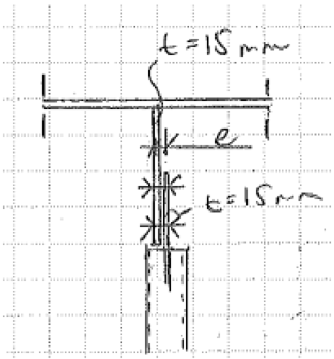
$\lambda_{cbf\_top} := \frac{l_{cr\_cbs}}{\pi \cdot i} \cdot \sqrt{\frac{f_{ycbs}}{E_s}} = 1.6$ — Slenderness of cross-beam - SS-EN 1993-1-4 5.4.2.1 Eq: 5.8

$\Phi_{cbf\_top} := 0.5 \cdot \left(1 + \alpha_{cbf} \cdot \left(\lambda_{cbf\_top} - 0.2\right) + {\lambda_{cbf\_top}}^2\right) = 2.1$ — SS-EN 1993-1-4 5.4.2.1 Eq: 5.7

$\chi_{cbf\_top} := \frac{1}{\Phi_{cbf\_top} + \sqrt{{\Phi_{cbf\_top}}^2 - {\lambda_{cbf\_top}}^2}} = 0.29$ — SS-EN 1993-1-4 5.4.2.1 Eq: 5.6

$N_{b.rd\_cbf\_top} := \dfrac{A_{cbf\_top} \cdot f_{ycbs} \cdot \chi_{cbf\_top}}{\gamma_{M2}} = 307\ kN$ Buckling load

$\eta_{cbf\_top} := \dfrac{P_{D\_hor\_cbf\_top}}{N_{b.rd\_cbf\_top}} = 57\%$ Utilization ratio



Bending moment from excentric bar fixation

$e := 2 \cdot \dfrac{15\ mm}{2} + 10\ mm = 25\ mm$

$M_{ed\_cbf\_top} := P_{D\_hor\_cbf\_top} \cdot e = 4\ kN \cdot m$

$M_{rd\_cbf\_top} := \dfrac{I_{x\_cbf\_top}}{0.5 \cdot h_{vkr}} \cdot f_{ycbs} = 55\ kN \cdot m$

$\eta_{cbf\_top\_comb} := \dfrac{P_{D\_hor\_cbf\_top}}{N_{b.rd\_cbf\_top}} + \dfrac{M_{ed\_cbf\_top}}{M_{rd\_cbf\_top}} = 65\%$ Utilization ratio

7.3.2.2 Diagonal bar in Casting phase

Assume VKR 120x120x5 mm as diagonal bar

$h_{diag\_vkr} := 120\ mm$ Height of vkr bar

$l_b := \dfrac{l_{cr\_cbs}}{2} - 0.2\ m = 2.6\ m$ Distance to centre of cross-beam

$l_a := 1.3\ m$ Distance between the top and bottom bar

$\alpha_{cb} := atan\left(\dfrac{l_a}{l_b}\right) = 27\ deg$

$l_{diag} := \dfrac{l_b}{cos\left(\alpha_{cb}\right)} = 2.91\ m$ Length of diagonal

$P_{D\_diag\_cbf} := \dfrac{P_{D\_hor\_cbf\_top}}{cos\left(\alpha_{cb}\right)} = 196\ kN$

Buckling of compressed diagonal bar

$I_{x\_cbf\_diag} := 498 \cdot 10^4 \ mm^4$ — Stiffness of diagonal bar

$A_{cbf\_diag} := 2270 \ mm^2$ — Area of diagonal bar

$i := \sqrt{\dfrac{I_{x\_cbf\_diag}}{A_{cbf\_diag}}} = 47 \ mm$ — Radius of gyration

$\lambda_{cbf\_diag} := \dfrac{l_{diag}}{\pi \cdot i} \cdot \sqrt{\dfrac{f_{ycbs}}{E_s}} = 0.97$ — Slenderness of cross-beam - SS-EN 1993-1-4 5.4.2.1 Eq: 5.8

$\Phi_{cbf\_diag} := 0.5 \cdot \left(1 + \alpha_{cbf} \cdot \left(\lambda_{cbf\_diag} - 0.2\right) + \lambda_{cbf\_diag}{}^2\right) = 1.16$ — SS-EN 1993-1-4 5.4.2.1 Eq: 5.7

$\chi_{cbf\_diag} := \dfrac{1}{\Phi_{cbf\_diag} + \sqrt{\Phi_{cbf\_diag}{}^2 - \lambda_{cbf\_diag}{}^2}} = 0.56$ — SS-EN 1993-1-4 5.4.2.1 Eq: 5.6

$N_{b.rd\_cbf\_diag} := \dfrac{A_{cbf\_diag} \cdot f_{ycbs} \cdot \chi_{cbf\_diag}}{\gamma_{M2}} = 507 \ kN$ — Buckling load

$M_{ed\_cbf\_diag} := P_{D\_diag\_cbf} \cdot e = 5 \ kN \cdot m$

$M_{rd\_cbf\_diag} := \dfrac{I_{x\_cbf\_diag}}{0.5 \cdot h_{diag\_vkr}} \cdot f_{ycbs} = 40 \ kN \cdot m$

$\eta_{cbf\_diag} := \dfrac{P_{D\_diag\_cbf}}{N_{b.rd\_cbf\_diag}} + \dfrac{M_{ed\_cbf\_diag}}{M_{rd\_cbf\_diag}} = 51\%$ — Utilization ratio

Bottom bar

Same conditions as top bar, therefore no further check needed

7.3.2.3 Joint between cross beam and main girder

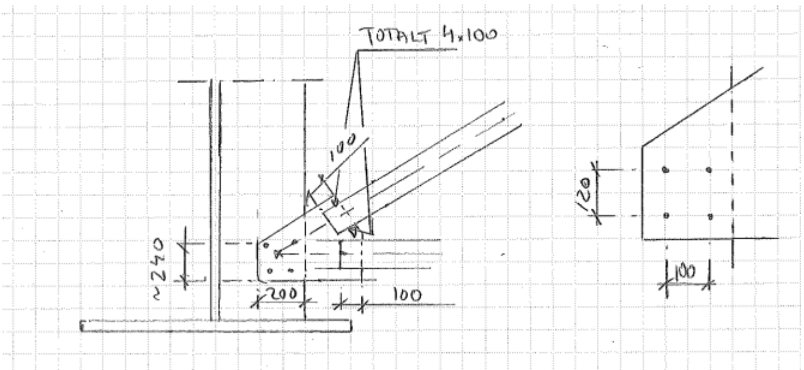
Weld of top bar

$$F_{ll} := max\left(0.7\ \frac{N_{b.rd\_cbf\_top}}{2}, 0.7 \cdot \frac{M_{rd\_cbf\_top}}{0.14\ m}, P_{D\_hor\_cbf\_top}\right) = 277\ kN$$

$$a := 4\ mm$$

$$\tau_{ll\_cbf\_top} := \frac{F_{ll}}{2 \cdot 2 \cdot 0.1\ m \cdot a} = 173\ MPa$$

Bolted connection bottom and diagonal bar



$$x_{bolt} := \frac{100\ mm}{2} = 50\ mm$$ Distance in x from rotational-centre to bolt

$$y_{bolt} := \frac{120\ mm}{2} = 60\ mm$$ Distance in y from rotational-centre to bolt

$$I_{p\_cbf} := 4 \cdot x_{bolt}^{\ 2} + 4 \cdot y_{bolt}^{\ 2} = 0.0244\ m^2$$ Rotational stiffness

Three load cases will be considered according to Råd Brobyggande
1. "real load"
2. 70% of normal force capacity
3. 70% of moment capacity

Load case 1

$$P_1 := \frac{P_{D\_diag\_cbf}}{4} = 49\ kN$$ Real load case

Load case 2

$$P_2 := 0.7 \cdot \frac{N_{b.rd\_cbf\_diag}}{4} = 89\ kN$$ 70% of normal force capacity

Load case 3

$$H_3 := \frac{0.7 \cdot M_{rd\_cbf\_diag} \cdot x_{bolt}}{I_{p\_cbf}} = 57\ kN$$ 70% of moment capacity

$$V_3 := \frac{0.7 \cdot M_{rd\_cbf\_diag} \cdot y_{bolt}}{I_{p\_cbf}} = 69\ kN$$ 70% of moment capacity

$P_3 := \sqrt{V_3^{\,2} + H_3^{\,2}} = 89\ kN$ Load composant

$P_{max} := max\left(P_1, P_2, P_3\right) = 89\ kN$ Dimensioning load

Capacity bolts

Choose diameter 20 mm 10.9 bolts

$d_{bolt} := 20\ mm$

$f_{ub} = 800\ MPa$

$$A_{bolt} := \frac{d_{bolt}^{\,2} \cdot \pi}{4} = 314\ mm^2$$

$\alpha_v := 0.6$ SS-EN 1993-1-8 Table 3.4

$$F_{v.rd} := \frac{\alpha_v \cdot f_{ub} \cdot A_{bolt}}{\gamma_{M2}} = 126\ kN$$ SS-EN 1993-1-8 Table 3.4

Bearing resistance base material

$$\alpha_b := min\left(\frac{x_{bolt}}{3 \cdot d_{bolt}}, \frac{f_{ub}}{f_{ulf}}, 1\right) = 0.8$$ SS-EN 1993-1-8 Table 3.4

$$k_1 := min\left(2.8 \cdot \frac{y_{bolt}}{d_{bolt}} - 1.7, 2.5\right) = 2.5$$ SS-EN 1993-1-8 Table 3.4

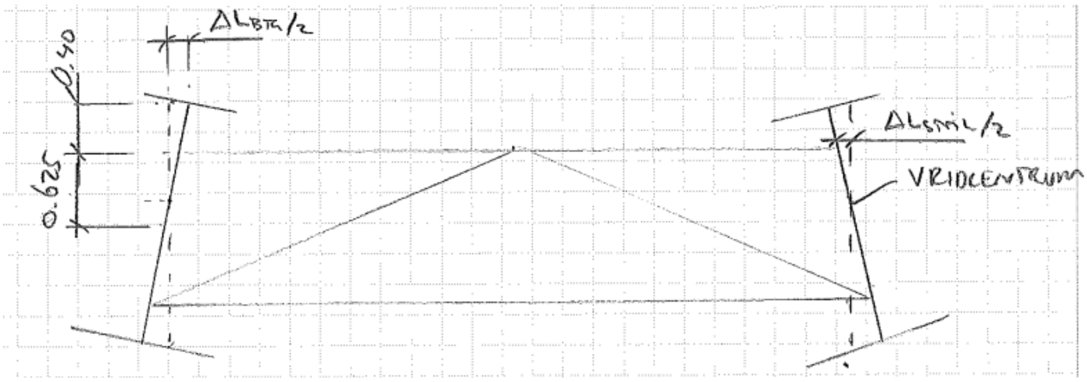
$$F_{b.rd} := \frac{k_1 \cdot \alpha_b \cdot f_{ulf} \cdot d_{bolt} \cdot t_{ast\_f}}{\gamma_{M2}} = 339\ kN$$ SS-EN 1993-1-8 Table 3.4

$$\eta_{bolt} := \frac{P_{max}}{F_{b.rd}} = 26\%$$ Utilization ratio

$$\eta_{bolt\_v} := \frac{P_{max}}{F_{v.rd}} = 71\%$$ Utilization ratio

7.3.2.4 Service state

Loads comes from shrinkage and temperature



$\varepsilon_{cs} = 246 \cdot 10^{-6}$ Shrinkage strain

$\varepsilon_{temp} = -504 \cdot 10^{-6}$ Temperature strain

$\varepsilon_{tot} := \varepsilon_{cs} \cdot 1 + \varepsilon_{temp} \cdot 1.5 = -0.00051$ Total strain ULS

$d_{top} := 400\ mm$ Distance from top flange to top bar

$d_{centre} := 650\ mm$ Distance from top bar to centre of main girder

$$\varepsilon_{cbf} := \varepsilon_{tot} \cdot \frac{d_{centre}}{d_{top} + d_{centre}} = -316 \cdot 10^{-6}$$

$F_{cbf\_top\_cs} := A_{cbf\_top} \cdot \varepsilon_{cbf} \cdot E_s = -169\ kN$ Force in top bar from shrinkage

$\eta_{cbf\_top\_cs} := \left| \frac{F_{cbf\_top\_cs}}{N_{b.rd\_cbf\_top}} \right| = 55\%$ Utilization ratio

7.3.3 Utilization cross-beams

Support

$\eta_{cb\_M} = 50\%$	Bending moment at bearing change
$\eta_{cb\_V} = 54\%$	Shear forces at bearing change
$\eta_{comb\_cbs} = 63\%$	Buckling at bearing change

Welds

$\eta_{cb\_eq\_1} = 92\%$	Fillet weld web
$\eta_{cb\_eq\_2} = 40\%$	Butt weld at lifting point
$\eta_{cb\_L\_2} = 18\%$	Butt weld at lifting point
$\eta_{cb\_eq\_3} = 24\%$	Butt weld at stiffener lower
$\eta_{cb\_L\_3} = 18\%$	Butt weld at stiffener lower
$\eta_{cb\_eq\_4} = 36\%$	Fillet weld at stiffener to web
$\eta_{cb\_L\_4} = 9\%$	Fillet weld at stiffener to web
$\eta_{cb\_eq\_5} = 48\%$	Joint to main girder web
$\eta_{cb\_eq\_6} = 67\%$	Joint to main girder flange
$service_state = $ “Not dimensioning”	Loads from service state

Field

$\eta_{cbf\_top} = 57\%$	Buckling of top bar
$\eta_{cbf\_top\_cs} = 55\%$	Buckling of top bar service state loads
$\eta_{cbf\_top\_comb} = 65\%$	Combined bending moment and normal force top bar
$\eta_{cbf\_diag} = 51\%$	Buckling of diagonal bar

Joints

$\eta_{bolt} = 26\%$	Bolt capacity
$\eta_{bolt\_v} = 71\%$	Base material capacity

7.4 Fatigue

The check for fatigue of welds is carried out according to SS-EN 1993-2 and SS-EN 1993-1-9 using the lambda method

7.4.1 λ-method

$$\gamma_{Ff} \cdot \Delta\sigma_E < \frac{\Delta\sigma_c}{\gamma_{mf}}$$

$$\Delta\sigma_E = \lambda_f \cdot \phi \cdot \Delta\sigma$$

$\gamma_{Ff} := 1.0$	SS-EN 1993-2 9.3 (1)
$\gamma_{Mf} := 1.35$	SS-EN 1993-1-9 Table 3.1
$l_{span} = 51\ m$	Span length of bridge
$Q_{m1} := 445\ kN$	Krav Brobyggande E.3.1 (f)
$Q_{m1tdok} := 410\ kN$	Krav Brobyggande E.3.1 (f)
$Q_0 := 480\ kN$	SS-EN 1993-2 9.5.2 (3)
$N_{obs} := 0.05 \cdot 10^6$	SS-EN 1991-2 4.6.1 Table 4.5
$N_0 := 0.5 \cdot 10^6$	SS-EN 1993-2 9.5.2 (3)
$\lambda_{1M} := 2.55 - 0.7 \cdot \left(\frac{l_{span} - 10\ m}{70\ m}\right) = 2.14$	Moment SS-EN 1993-2 9.5.2 Figure 9.5
$\lambda_{1Vf} := 2.55 - 0.7 \cdot \left(\frac{0.4 \cdot l_{span} - 10\ m}{70\ m}\right) = 2.45$	Shear force field SS-EN 1993-2 9.5.2 Figure 9.5
$\lambda_{1Vs} := 1.7 - 0.5 \cdot \left(\frac{l_{span} - 30\ m}{50\ m}\right) = 1.49$	Shear force support SS-EN 1993-2 9.5.2 Figure 9.5
$\lambda_2 := \frac{Q_{m1}}{Q_0} \cdot \left(\frac{N_{obs}}{N_0}\right)^{\frac{1}{5}} = 0.58$	SS-EN 1993-2 9.5.2 Equation 9.10
$\lambda_3 := \left(\frac{120}{100}\right)^{\frac{1}{5}} = 1.04$	SS-EN 1993-2 9.5.2 Equation 9.11
$\lambda_4 := 1.0$	TSFS 2018:57 27ch. $3
$\lambda_{max} := 2$	SS-EN 1993-2 9.5.2 Figure 9.6
$\lambda_m := min\left(\lambda_{1M} \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4, \lambda_{max}\right) = 1.3$	SS-EN 1993-2 9.5.2 Equation 9.9
$\lambda_{Vf} := min\left(\lambda_{1Vf} \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4, \lambda_{max}\right) = 1.48$	SS-EN 1993-2 9.5.2 Equation 9.9
$\lambda_{Vs} := min\left(\lambda_{1Vs} \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4, \lambda_{max}\right) = 0.9$	

7.4.2 Fatigue due to moment

Check midspan

$X_{check\_fat\_1} := 26\ m$ Location of joint

$k_s(x) := \left(\dfrac{25\ mm}{t_{fl}(x)}\right)^{0.2}$ Size effect SS-EN 1993-1-9 8 Figure 8.3

Check lower flange joint

$\Delta\sigma_c := 80\ MPa$ Fatigue class c80 SS-EN 1993-1-9 Table 8.3 (11)

$\Delta\sigma_{e2\_1}(x) := \dfrac{M_{FAT}(x) \cdot \lambda_m}{I_{y\_short}(x)} \cdot \left(h_{beam}(x) - z_{tp\_short}(x)\right)$ Normal stresses in flange

$\Delta\sigma_{e2\_1}\left(X_{check\_fat\_1}\right) = 48\ MPa$

$\Delta\sigma_{c.red}(x) := k_s(x) \cdot \dfrac{\Delta\sigma_c}{\gamma_{Mf}}$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$\Delta\sigma_{c.red}\left(X_{check\_fat\_1}\right) = 52\ MPa$

$\eta_{fat\_1} := \dfrac{\Delta\sigma_{e2\_1}\left(X_{check\_fat\_1}\right)}{\Delta\sigma_{c.red}\left(X_{check\_fat\_1}\right)} = 93\%$ Utilization ratio

Check upper flange at stiffener

$\Delta\sigma_c = 80\ MPa$ Fatigue class c80 SS-EN 1993-1-9 Table 8.4 (7)

$\Delta\sigma_{e2\_1}(x) := \dfrac{M_{FAT}(x) \cdot \lambda_m}{I_{y\_short}(x)} \cdot \left(h_{beam}(x) - z_{tp\_short}(x) - t_{fl}(x)\right)$ Normal stresses in flange

$\Delta\sigma_{e2\_1}\left(X_{check\_fat\_1}\right) = 46\ MPa$

$\Delta\sigma_{c.red} := \dfrac{\Delta\sigma_c}{\gamma_{Mf}} = 59\ MPa$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$\eta_{fat\_2} := \dfrac{\Delta\sigma_{e2\_1}\left(X_{check\_fat\_1}\right)}{\Delta\sigma_c} = 58\%$ Utilization ratio

Check upper flange studs

$\Delta\sigma_c = 80\ MPa$ Fatigue class c80 SS-EN 1993-1-9 Table 8.4 (9)

$$\Delta\sigma_{e2\_1}(x) := \frac{M_{FAT}(x) \cdot \lambda_m}{I_{y\_short}(x)} \cdot (z_{tp\_short}(x))$$ Normal stresses in flange

$\Delta\sigma_{e2\_1}(X_{check\_fat\_1}) = 17\ MPa$

$\Delta\sigma_{c.red} = 59\ MPa$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$$\eta_{fat\_3} := \frac{\Delta\sigma_{e2\_1}(X_{check\_fat\_1})}{\Delta\sigma_c} = 22\%$$ Utilization ratio

Check at thickness change, x=11m

$X_{check\_fat\_2} := 10.99\ m$

$$k_s(x) := \left(\frac{25\ mm}{t_{fl}(x)}\right)^{0.2}$$ Size effect SS-EN 1993-1-9 8 Table 8.3

Check lower flange joint

$\Delta\sigma_c := 80\ MPa$ Fatigue class c80 SS-EN 1993-1-9 Table 8.3 (11)

$$\Delta\sigma_{e2\_1}(x) := \frac{M_{FAT}(x) \cdot \lambda_m}{I_{y\_short}(x)} \cdot (h_{beam}(x) - z_{tp\_short}(x))$$ Normal stresses in flange

$\Delta\sigma_{e2\_1}(X_{check\_fat\_2}) = 48\ MPa$

$$\Delta\sigma_{c.red}(x) := k_s(x) \cdot \frac{\Delta\sigma_c}{\gamma_{Mf}}$$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$\Delta\sigma_{c.red}(X_{check\_fat\_2}) = 52\ MPa$

$$\eta_{fat\_4} := \frac{\Delta\sigma_{e2\_1}(X_{check\_fat\_2})}{\Delta\sigma_{c.red}(X_{check\_fat\_2})} = 93\%$$ Utilization ratio

Check at notch, x=15m

$X_{check\_fat\_3} := 15\ m$

Check lower flange joint

$\Delta\sigma_c := 71\ MPa$ Fatigue class c80 SS-EN 1993-1-9 Table 8.3 (11)

$$\Delta\sigma_{e2\_1}(x) := \frac{M_{FAT}(x) \cdot \lambda_m}{I_{y\_short}(x)} \cdot \left(h_{beam}(x) - z_{tp\_short}(x)\right)$$ Normal stresses in flange

$\Delta\sigma_{e2\_1}\left(X_{check\_fat\_3}\right) = 40.02\ MPa$

$$\Delta\sigma_{c.red}(x) := k_s(x) \cdot \frac{\Delta\sigma_c}{\gamma_{Mf}}$$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$\Delta\sigma_{c.red}\left(X_{check\_fat\_3}\right) = 46\ MPa$

$$\eta_{fat\_5} := \frac{\Delta\sigma_{e2\_1}\left(X_{check\_fat\_3}\right)}{\Delta\sigma_{c.red}\left(X_{check\_fat\_3}\right)} = 87\%$$ Utilization ratio

7.4.3 Fatigue due to shear forces

Check at support, x=0.5m

$X_{check\_fat\_4} := 0.5\ m$

$a := 4\ mm$

Check upper flange fillet weld, web

$\Delta\tau_{c\_ll\_tab} := 80\ MPa$ Fatigue class c80 m=5 SS-EN 1993-1-9 Table 8.5 (8)

$$\Delta\tau_{e\_ll}(x) := \frac{V_{FAT}(x) \cdot S_{uw\_short}(x) \cdot \lambda_{Vf}}{I_{y\_short}(x) \cdot min\left(t_w(x), 2 \cdot a\right)}$$ Shear stresses in weld

$\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right) = 18.935\ MPa$

$$\Delta\tau_{c\_ll} := \frac{\Delta\tau_{c\_ll\_tab}}{\gamma_{Mf}} = 59\ MPa$$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$$\eta_{fat\_6} := \frac{\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right)}{\Delta\tau_{c\_ll}} = 32\%$$ Utilization ratio

Vertical load on web, see section 7.1

$\Delta\tau_{c\_L} := 36\ MPa$ Fatigue class c36 m=3 SS-EN 1993-1-9 Table 8.5 (8)

$\Delta\sigma_{c\_L} := 36\ MPa$ Fatigue class c36 m=3 SS-EN 1993-1-9 Table 8.5 (8)

$Q_{fat} := 120\ kN$ Boggie load fatigue veihcle

$P := \dfrac{Q_{fat}}{2 \cdot b_{load\_web}} + \dfrac{Q_{fat}}{2\ (2\ m + b_{load\_web})} = 76\ \dfrac{kN}{m}$ Load on web per meter

$\Delta\sigma_L := \dfrac{P \cdot \sqrt{2} \cdot \lambda_{Vf}}{4 \cdot a} = 10\ MPa$ Normal stresses perpendicular to weld

$\Delta\tau_L := \Delta\sigma_L = 10\ MPa$ Shear stresses perpendicular to weld

$\Delta\sigma_{wf} := \sqrt{\Delta\sigma_L^{\ 2} + \Delta\tau_L^{\ 2}} = 14\ MPa$ SS-EN 1993-1-9 5 (6)

Control

$\eta_{fat\_7} := \left(\dfrac{\Delta\sigma_{wf}}{\Delta\sigma_{c\_L}}\right)^3 + \left(\dfrac{\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right)}{\Delta\tau_{c\_ll}}\right)^5 = 6\%$ Utilization ratio

Check change of weld lower flange fillet weld, x=1 m

$X_{check\_fat\_4} := 1\ m$

$a := 4\ mm$

$\Delta\tau_{c\_ll\_tab} := 80\ MPa$ Fatigue class c80 m=5 SS-EN 1993-1-9 Table 8.5 (8)

$\Delta\tau_{e\_ll}(x) := \dfrac{V_{FAT}(x) \cdot S_{lw\_short}(x) \cdot \lambda_{Vf}}{I_{y\_short}(x) \cdot min\left(t_w(x), 2 \cdot a\right)}$ Shear stresses in weld

$\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right) = 41.08\ MPa$

$\Delta\tau_{c\_ll} := \dfrac{\Delta\tau_{c\_ll\_tab}}{\gamma_{Mf}} = 59\ MPa$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$\eta_{fat\_8} := \dfrac{\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right)}{\Delta\tau_{c\_ll}} = 69\%$ Utilization ratio

Check at change of thickness in web, x=11 m

$X_{check\_fat\_4} := 10.99\ m$

$a := 4\ mm$

Check upper flange fillet weld, web

$\Delta\tau_{c\_ll\_tab} := 80\ MPa$ Fatigue class c80 m=5 SS-EN 1993-1-9 Table 8.5 (8)

$$\Delta\tau_{e\_ll}(x) := \frac{V_{FAT}(x) \cdot S_{uw\_short}(x) \cdot \lambda_{Vf}}{I_{y\_short}(x) \cdot min\left(t_w(x), 2 \cdot a\right)}$$ Shear stresses in weld

$\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right) = 15.621\ MPa$

$$\Delta\tau_{c\_ll} := \frac{\Delta\tau_{c\_ll\_tab}}{\gamma_{Mf}} = 59\ MPa$$ Fatigue strength SS-EN 1993-1-9 Eq: 7.1

$$\eta_{fat\_9} := \frac{\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right)}{\Delta\tau_{c\_ll}} = 26\%$$ Utilization ratio

Vertical load on web, see section 7.1

$\Delta\tau_{c\_L} := 36\ MPa$ Fatigue class c36 m=3 SS-EN 1993-1-9 Table 8.5 (8)

$\Delta\sigma_{c\_L} := 36\ MPa$ Fatigue class c36 m=3 SS-EN 1993-1-9 Table 8.5 (8)

$$\Delta\sigma_L := \frac{P \cdot \sqrt{2} \cdot \lambda_{Vf}}{4 \cdot a} = 10\ MPa$$ Normal stresses perpendicular to weld

$\Delta\tau_L := \Delta\sigma_L$ Shear stresses perpendicular to weld

$\Delta\sigma_{wf} := \sqrt{\Delta\sigma_L^{\ 2} + \Delta\tau_L^{\ 2}} = 14\ MPa$ SS-EN 1993-1-9 5 (6)

Control

$$\eta_{fat\_10} := \left(\frac{\Delta\sigma_{wf}}{\Delta\sigma_{c\_L}}\right)^3 + \left(\frac{\Delta\tau_{e\_ll}\left(X_{check\_fat\_4}\right)}{\Delta\tau_{c\_ll}}\right)^5 = 6\%$$ Utilization ratio

7.4.4 Fatigue utilization

$\eta_{fat\_1} = 93\%$	Check midspan, lower flange joint
$\eta_{fat\_2} = 58\%$	Check midspan, upper flange at stiffener
$\eta_{fat\_3} = 22\%$	Check midspan, upper flange studs
$\eta_{fat\_4} = 93\%$	Check x=11 m thickness change, lower flange joint
$\eta_{fat\_5} = 87\%$	Check x=15 m notch, lower flange joint
$\eta_{fat\_6} = 32\%$	Check shear x=0.5 m, upper flange fillet weld
$\eta_{fat\_7} = 6\%$	Check combined bending, shear x=0.5 m, upper flange fillet weld
$\eta_{fat\_8} = 69\%$	Check shear x=1 m, lower flange fillet weld
$\eta_{fat\_9} = 26\%$	Check shear x=11 m, lower flange fillet weld
$\eta_{fat\_10} = 6\%$	Check combined bending, shear x=11 m, lower flange fillet weld

8 Deflection checks - servicability limit state

8.1 Secant modulus of elasticity

As stainless steels display a non-linear behaviour below the yield limit a constant value for the modulus of elasticity can not be used. A secant modulus of elasticity is calculated dependent on the current stress.

$$E_{s.ser} = \frac{E_{s.1} + E_{s.2}}{2}$$

$E_{s.1}$ Secant modulus corresponding to the stress in the tensile flange (lower)

$E_{s.2}$ Secant modulus corresponding to the stress in the tensile flange (upper)

$$E_{s.i}\left(\sigma, f_y, n\right) := \frac{E_s}{1 + 0.002 \cdot \frac{E_s}{f_y} \cdot \left(\frac{\sigma}{f_y}\right)^n}$$

Equation for calculating the secant modulus (upper- or lower flange)

$SteelGrade = \text{“1.4162”}$

Used stainless steel

$n = 5$

Ramberg- Osgood parameter for the specific steel grade - SS-EN 1993-1-4 Table 4.1, Outokumpu - Duplex stainless steel brochure

8.1.1 Calculation of stresses and secant modulus of elasticity

Short-term loads

$$\sigma_{SLS\_tr.uf}(x) := \overrightarrow{\frac{N_{SLS\_tr}(x)}{A_{sl\_short}(x)} + \frac{M_{SLS\_tr}(x)}{I_{y\_short}(x)} \cdot z_{tp\_short}(x)}$$

$$\sigma_{SLS\_tr.lf}(x) := \overrightarrow{\frac{N_{SLS\_tr}(x)}{A_{sl\_short}(x)} + \frac{M_{SLS\_tr}(x)}{I_{y\_short}(x)} \cdot \left(z_{tp\_short}(x) - h_{beam}(x)\right)}$$

$$\sigma_{tr.max\_uf} := max\left(\sigma_{SLS\_tr.uf}(X)\right) = 34.73\ MPa \qquad \sigma_{tr.max\_lf} := -min\left(\sigma_{SLS\_tr.lf}(X)\right) = 96.34\ MPa$$

$$E_{s.defl} := \frac{E_{s.i}\left(\sigma_{tr.max\_uf}, f_{yuf}, n\right) + E_{s.i}\left(\sigma_{tr.max\_lf}, f_{ylf}, n\right)}{2} = 199.96\ GPa$$

Casting loads

$$\sigma_{SLS\_cast.uf}(x) := \overrightarrow{\frac{N_{SLS\_cast}(x)}{A_{sl}(x)} + \frac{M_{SLS\_cast}(x)}{I_{y\_steel}(x)} \cdot z_{tp\_steel}(x)}$$

$$\sigma_{SLS\_cast.lf}(x) := \overrightarrow{\frac{N_{SLS\_cast}(x)}{A_{sl}(x)} + \frac{M_{SLS\_cast}(x)}{I_{y\_steel}(x)} \cdot \left(z_{tp\_steel}(x) - h_{beam}(x)\right)}$$

$$\sigma_{cast.max\_uf} := max\left(\sigma_{SLS\_cast.uf}(X)\right) = 234.16\ MPa \qquad \sigma_{cast.max\_lf} := -min\left(\sigma_{SLS\_cast.lf}(X)\right) = 148.27\ MPa$$

$$E_{s.elev.cast} := \frac{E_{s.i}\left(\sigma_{cast.max\_uf}, f_{yuf}, n\right) + E_{s.i}\left(\sigma_{cast.max\_lf}, f_{ylf}, n\right)}{2} = 196.38\ GPa$$

Permanent loads

$$\sigma_{SLS\_per.uf}(x) := \overrightarrow{\frac{N_{SLS\_perm}(x)}{A_{sl\_perm}(x)} + \frac{M_{SLS\_perm}(x)}{I_{y\_perm}(x)} \cdot z_{tp\_perm}(x)}$$

$$\sigma_{SLS\_per.lf}(x) := \overrightarrow{\frac{N_{SLS\_perm}(x)}{A_{sl\_perm}(x)} + \frac{M_{SLS\_perm}(x)}{I_{y\_perm}(x)} \cdot \left(z_{tp\_perm}(x) - h_{beam}(x)\right)}$$

$$\sigma_{perm.max\_uf} := max\left(\sigma_{SLS\_per.uf}(X)\right) = 17.21\ MPa \qquad \sigma_{perm.max\_lf} := -min\left(\sigma_{SLS\_per.lf}(X)\right) = 21.06\ MPa$$

$$E_{s.elev.perm} := \frac{E_{s.i}\left(\sigma_{perm.max\_uf}, f_{yuf}, n\right) + E_{s.i}\left(\sigma_{perm.max\_lf}, f_{ylf}, n\right)}{2} = 200.00\ GPa$$

<u>Shrinkage</u>

$$\sigma_{SLS\_cs.uf}(x) := \overrightarrow{\frac{N_{SLS\_cs}(x)}{A_{sl\_cs}(x)} + \frac{M_{SLS\_cs}(x)}{I_{y\_cs}(x)} \cdot z_{tp\_cs}(x)}$$

$$\sigma_{SLS\_cs.lf}(x) := \overrightarrow{\frac{N_{SLS\_cs}(x)}{A_{sl\_cs}(x)} + \frac{M_{SLS\_cs}(x)}{I_{y\_cs}(x)} \cdot \left(z_{tp\_cs}(x) - h_{beam}(x)\right)}$$

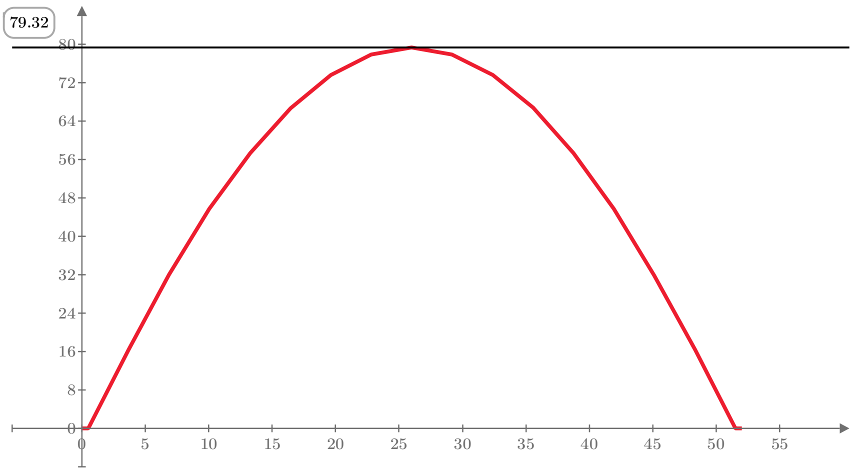
$$\sigma_{cs.max\_uf} := max\left(\sigma_{SLS\_cs.uf}(X)\right) = 43.7\ MPa$$

$$\sigma_{cs.max\_lf} := -min\left(\sigma_{SLS\_cs.lf}(X)\right) = 0\ MPa$$

$$E_{s.elev.cs} := \frac{E_{s.i}\left(\sigma_{cs.max\_uf}, f_{yuf}, n\right) + E_{s.i}\left(\sigma_{cs.max\_lf}, f_{ylf}, n\right)}{2} = 200.00\ GPa$$

8.2 Deflection

$\delta_{tr}(X)$ Calculated deflection using StripStep-2, see Appendix X



$\delta_{tr.max} := max\left(\delta_{tr}(X)\right) \cdot \dfrac{E_s}{E_{s.defl}} = 79.32\ mm$ Maximum deflection caused by trafic loads

$L_{span} = 51.0\ m$ Span length

$\delta_{tr.all} := \dfrac{L_{span}}{400} = 128\ mm$ Allowed displacement - Krav Brobyggande B.3.4.2.2

$\eta_{\delta} := \dfrac{\delta_{tr.max}}{\delta_{tr.all}} = 62\%$ Utilization rate - allowed deflection

$\delta_{tr\_end} = 2.6\ mm$ Vertical displacement at free end

$\delta_{tr\_end.all} := 5\ mm$ Allowed vertical displacement at free end - Krav Brobyggande B.3.4.2.2

$\textbf{if}\left(\delta_{tr\_end} \le \delta_{tr\_end.all}, \text{“OK”}, \text{“NOT OK”}\right) = \text{“OK”}$

8.2 Elevation of span

$\frac{L_{span}}{8} = 6.375\ m$ Dividing the span into eight parts

$$\delta_{tot}(X) = \delta_{cast}(x) \cdot \frac{E_s}{E_{s.elev.cast}} + \delta_{perm}(x) \cdot \frac{E_s}{E_{s.elev.perm}} + \delta_{cs}(x) \cdot \frac{E_s}{E_{s.elev.cs}}$$

$\delta_{choosen}(X_\delta)$ Choosen elevation for $\delta_{tot}(X)$ including weighing of secant modulus of elasticity

$\delta_{tot}(X)$ Total deflection including weighing of secant modulus of elasticity

$\delta_{cast}(X)$ Deflection from casting loads, see Appendix X

$\delta_{perm}(X)$ Deflection from permanent loads, see Appendix X

$\delta_{cs}(X)$ Deflection from shrinkage, see Appendix X

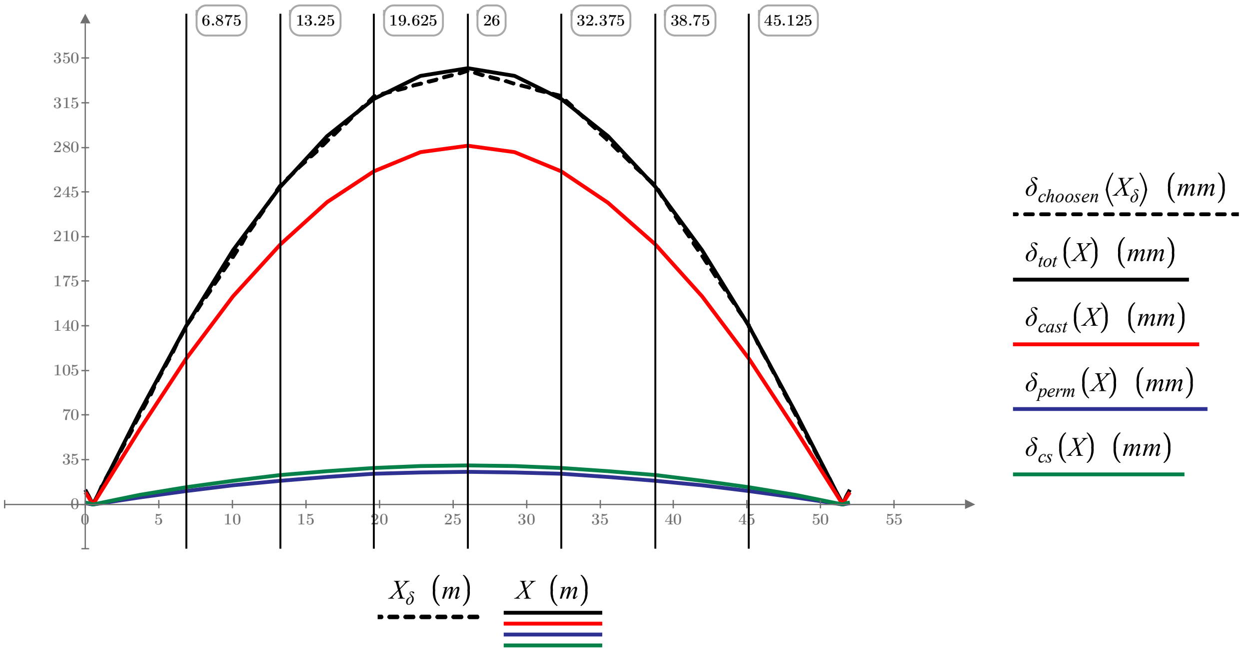


Table. Showing the choosen elevation for the span considering casting, permanent and shrinkage loads

X	δ_{tot}	$\delta_{choosen}$
0.50	0	**0**
6.88	140	**140**
13.25	249	**250**
19.63	318	**320**
26.00	342	**340**
32.38	318	**320**
38.75	249	**250**
45.13	140	**140**
51.50	0	**0**

9 Material savings

The material savings that are achieved thanks to redesigning the bridge with corrugated webs and using stainless steel is compared to the old design. However it is important to note that dependent on the utilization rates the savings may always not be comparable. For this bridge the highest utilization ratios are larger than 95% (close to 99,8%) and therefore close to comparable.

9.1 Main girder

The main girder is redesigned with a corrugated web and a slightly slimmer design. Over the full bridge length this decreases the weight of the bridge. All savings are presented for the full bridge (full width). Consideration has been taken to the extra web length arising from the corrugation.

$V_{old} = 13.25\ m^3$ Steel volume - girder with flat web; original design

$V_{new} = 10.56\ m^3$ Steel volume - girder with corrugated web; new design

$m_{old\_girder} := 7850\ \dfrac{kg}{m^3} \cdot V_{old} = 104\ 10^3 \cdot kg$ Weight of girder - flat web; original design

$m_{new\_girder} := 7551\ \dfrac{kg}{m^3} \cdot V_{new} = 80\ 10^3 \cdot kg$ Weight of girder - corrugated web; new design

$\eta_{girder} := 1 - \dfrac{m_{new\_girder}}{m_{old\_girder}} = 23\%$ Material saving [%] - steel girder

$\Delta m_{saving\_girder} := m_{old\_girder} - m_{new\_girder} = 24\ 10^3\ kg$ Material saving [kg] - steel girder

9.2 Studs

$V_{stud\_old} = 72750\ cm^3$ Volume - studs; original design

$V_{stud\_new} = 59530\ cm^3$ Volume - studs; new design

$m_{old\_stud} := 7850\ \dfrac{kg}{m^3} \cdot V_{stud\_old} = 571\ kg$ Mass - studs; original design

$m_{new\_stud} := 7551\ \dfrac{kg}{m^3} \cdot V_{stud\_new} = 450\ kg$ Mass - studs; new design

$\eta_{stud} := 1 - \dfrac{m_{new\_stud}}{m_{old\_stud}} = 21\%$ Material saving [%] - studs

$\Delta m_{saving\_stud} := m_{old\_stud} - m_{new\_stud} = 122\ kg$ Material saving [kg] - studs

9.3 Cross-beams

$V_{cb}=0.59\ m^3$ — Volume cross-beams old and new design

$m_{old\_cb}:=7850\ \frac{kg}{m^3}\cdot V_{cb}=4597\ kg$ — Mass - cross-beams; original design

$m_{new\_cb}:=7551\ \frac{kg}{m^3}\cdot V_{cb}=4422\ kg$ — Mass - cross-beams; new design

$\eta_{cb}:=1-\frac{m_{new\_cb}}{m_{old\_cb}}=4\%$ — Material saving [%] - studs

$\Delta m_{saving\_cb}:=m_{old\_cb}-m_{new\_cb}=175\ kg$ — Material saving [kg] - studs

9.4 Welds

$V_{weld\_old}=0.0051\ m^3$ — Volume - welds; original design

$V_{weld\_new}=0.0038\ m^3$ — Volume - welds; new design

$m_{old\_weld}:=7850\ \frac{kg}{m^3}\cdot V_{weld\_old}=40\ kg$ — Mass - welds; original design

$m_{new\_weld}:=7551\ \frac{kg}{m^3}\cdot V_{weld\_new}=29\ kg$ — Mass - welds; new design

$\eta_{weld}:=1-\frac{m_{new\_weld}}{m_{old\_weld}}=28\%$ — Material saving [%] - welds

$\Delta m_{saving\_weld}:=m_{old\_weld}-m_{new\_weld}=11\ kg$ — Material saving [kg] - welds

9.5 Total savings

$m_{old\_bridge}:=m_{old\_girder}+m_{old\_stud}+m_{old\_cb}=109\ 10^3\ \ kg$ — Total mass of original design

$m_{new\_bridge}:=m_{new\_girder}+m_{new\_stud}+m_{new\_cb}=85\ 10^3\ \ kg$ — Total mass of new design

$\eta_{bridge}:=1-\frac{m_{new\_bridge}}{m_{old\_bridge}}=22.5\%$ — Material saving [%] - full bridge

$\Delta m_{saving.tot}:=\Delta m_{saving\_girder}+\Delta m_{saving\_stud}+\Delta m_{saving\_cb}+\Delta m_{saving\_weld}=25\ 10^3\ \ kg$
Material saving [kg] - full bridge

Appendix 1 – Strip Step, Calculation of casting loads – bridge 100-262-1

Appendix 1 – Strip Step, System analysis – bridge 100-262-1 (cast)

```
D A T A   I N P U T        STRIP STEP 2 (PC-05-041001)        DATE: 2020-04-28   TIME: 11:50:46       PAGE  1      :
                                                                                                                    :
   CARD       0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
    NO        1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
 -------------------------------------------------------------------------------------------------------------
     1*       9000 STRIP-STEP2                           Elly Yman
     2*       9100 Project: Bro 100-262-1
     3*       9200 Composite steel- concrete bridge using corrugated web in stainless steel
     4*       9300 Appendix X - System analysis, Construction phase
     5*       2010 10000     200000              1
     6*       2020 1    BALK N
     7*       2022 0              -1.293              2.05      600.824   642.5
     8*       2022 0.5            -1.293              2.05      600.824   642.5
     9*       2020 2    BALK N
    10*       2022 0              -1.293              2.05      600.824   642.5
    11*       2022 10.5           -1.293              2.05      600.824   642.5
    12*       2022 10.5           -1.27               2.05      703.868   748.9
    13*       2022 11.3           -1.34               2.05      887.081   985
    14*       2022 25.5           -1.34               2.05      887.081   985
    15*       2022 39.7           -1.34               2.05      887.081   985
    16*       2022 40.5           -1.27               2.05      703.868   748.9
    17*       2022 40.5           -1.293              2.05      600.824   642.5
    18*       2022 51             -1.293              2.05      600.824   642.5
    19*       2020 3    BALK N
    20*       2022 0              -1.293              2.05      600.824   642.5
    21*       2022 0.5            -1.293              2.05      600.824   642.5
    22*       2040 1         0         0                                            1    2
    23*       2040 2         0.5       0                   -2.05     ZR
    24*       2040                                                                  2    3
    25*       2040 3         51.5      0                   -2.05     YZR
    26*       2040                                                                  3    4
    27*       2040 4         52        0                                            -3   3
    28*       2050 STEEL     1    2    0         S              UTBR           6.329     G/S
 BW:        6 JOINTS:   2
    29*       2050                     0.5       S              UTBR           6.329     G/S
    30*       2050           2    3    0         S              UTBR           6.329     G/S
    31*       2050                     10.5      S              UTBR           6.796     G/S
    32*       2050                     10.5      S              UTBR           6.796     G/S
    33*       2050                     11.3      S              UTBR           8.578     G/S
    34*       2050                     25.5      S              UTBR           8.578     G/S
    35*       2050                     39.7      S              UTBR           8.578     G/S
    36*       2050                     40.5      S              UTBR           6.329     G/S
    37*       2050                     40.5      S              UTBR           6.329     G/S
    38*       2050                     51        S              UTBR           6.329     G/S
    39*       2050           3    4    0         S              UTBR           6.329     G/S
    40*       2050                     0.5       S              UTBR           6.329     G/S
    41*       2050 SLAB      1    4                             UTBR           43.4      G/S
    42*       2050 FORM      1    4                             UTBR           2.611     G/S
    43*       2140 STEEL          STEEL     STAND  1.00
    44*       2140 SLAB           SLAB      STAND  1.00
    45*       2140 FORM           FORM      STAND  1.00
    46*       2140 BROTT-B        STEEL     STAND  1.20
    47*       2140                SLAB      STAND  1.20
    48*       2140                FORM      STAND  1.20
    49*       2140 KVASI          STEEL     STAND  1.00
    50*       2140                SLAB      STAND  1.00
    51*       2140                FORM      STAND  1.00
    52*       2170 1    2    2                   S    DEF
    53*       2170 2    3    16                  S    DEF
    54*       2170 3    4    2                   S    DEF
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     1     :
                 DATAKVITTO                      Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                   NR                 DATUM
                                        Elly Yman                                       20-04-28
0

0E N H E T E R :  SI-SYSTEMET

0           LANGDER                                          METER
0           VINKLAR                                          RADIAN
0           LASTER, KRAFTER                                  KILONEWTON
0           P KŽNNINGAR, ELASTICITETSMODULER                 MEGAPASCAL
0           MASSOR                                           TON
0           TEMPERATURER                                     KELVIN
0           FREKVENSER                                       HERTZ
0
0FR N DESSA GRUNDENHETER SAMT KOMBINATIONER HŽRAV G™RES
 F™LJANDE UNDANTAG:
0
            VINKEL I GEOMETRIBESKRIVNING OCH
            ROTATIONER I DEFORMATIONSRESULTAT                GRADER (360/VARV)
0           F™RSKJUTNINGAR I DEFORMATIONER                   MILLIMETER
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                               SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i       2    :
                 DATAKVITTO                      Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                  NR                 DATUM
                                        Elly Yman                                      20-04-28
0
 M A T E R I A L K O N S T A N T E R

 I    K    I    E0   I  G0/E0  I DENSI-  I OMEGA T I EPSILON I FI-KRYP I
 I         I         I         I   TET   I *100000 I *100000 I         I
 I---------I---------I---------I---------I---------I---------I---------I

      10000    200000    .500                 1.000
0
 E L E M E N T T Y P E R

 I  NR  I ARBETS I    E    I  DEN- I KRYP- I    R    I TVAR- I    F    I    D    I    H    I   K*I    I   K*A    I   K*AQ  I
 I      I BIDRAG I    G    I SITET I  IND  I         I SNITT I         I         I         I          I          I         I
 I------I--------I--------I-------I-------I---------I-------I---------I---------I---------I----------I----------I---------I

      1  N         200000                      .000         -1.293              2.050    600.824   642.500
         BALK      100000                     1.000         -1.293              2.050    600.824   642.500

      2  N         200000                      .000         -1.293              2.050    600.824   642.500
         BALK      100000                      .206         -1.293              2.050    600.824   642.500
                                               .206         -1.270              2.050    703.868   748.900
                                               .222         -1.340              2.050    887.081   985.000
                                               .500         -1.340              2.050    887.081   985.000
                                               .778         -1.340              2.050    887.081   985.000
                                               .794         -1.270              2.050    703.868   748.900
                                               .794         -1.293              2.050    600.824   642.500
                                              1.000         -1.293              2.050    600.824   642.500

      3  N         200000                      .000         -1.293              2.050    600.824   642.500
         BALK      100000                     1.000         -1.293              2.050    600.824   642.500
0
 S Y S T E M B E S K R I V N I N G

 I KNUTPT I    Y    I    Z    I DELTA Y I DELTA Z I LED-TYP I   ALFA  I ELEMENT I KNUTPT I
 I  START I         I         I         I         I  REAKT. I         I   TYP   I  SLUT  I
 I--------I---------I---------I---------I---------I---------I---------I---------I--------I

       1       .000      .000                                               1         2

       2       .500      .000             -2.050   ZR
                                                                            2         3

       3     51.500      .000             -2.050   YZR
                                                                            3         4

       4     52.000      .000                                              -3         3
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                               SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i    3     :
                 DATAKVITTO                      Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                     NR                 DATUM
                                        Elly Yman                                         20-04-28
0
 S T A N D A R D L A S T F A L L

 I LASTFALL I  KNUTPUNKT  I    R    I    W    I LAST- I    P-Y    I    P-Z    I    M    I
 I NAMN     I  START-SLUT I         I         I TYP   I           I           I         I
 I----------I-------------I-------I-------I-------I---------I---------I---------I

  STEEL                1-2    .000          UTBR                 -6.33
                             1.000          UTBR                 -6.33
                       2-3    .000          UTBR                 -6.33
                              .206          UTBR                 -6.80
                              .206          UTBR                 -6.80
                              .222          UTBR                 -8.58
                              .500          UTBR                 -8.58
                              .778          UTBR                 -8.58
                              .794          UTBR                 -6.33
                              .794          UTBR                 -6.33
                             1.000          UTBR                 -6.33
                       3-4    .000          UTBR                 -6.33
                             1.000          UTBR                 -6.33

  SLAB                 1-2    .000          UTBR                -43.40
                             1.000          UTBR                -43.40
                       2-3    .000          UTBR                -43.40
                             1.000          UTBR                -43.40
                       3-4    .000          UTBR                -43.40
                             1.000          UTBR                -43.40

  FORM                 1-2    .000          UTBR                 -2.61
                             1.000          UTBR                 -2.61
                       2-3    .000          UTBR                 -2.61
                             1.000          UTBR                 -2.61
                       3-4    .000          UTBR                 -2.61
                             1.000          UTBR                 -2.61
0
 L A S T K O M B I N A T I O N E R

 I LASTKOMBINATION I          - L A S T F A L L -        I SPANN. I K-KRYP I
 I NAMN     I ART  I NAMN      I  TYP   I FAKTOR I ALTER  I  IND   I        I
 I----------I------I----------I--------I--------I--------I--------I--------I

  STEEL             STEEL      STAND       1.000

  SLAB              SLAB       STAND       1.000

  FORM              FORM       STAND       1.000

  BROTT-B           STEEL      STAND       1.200
                    SLAB       STAND       1.200
                    FORM       STAND       1.200

  KVASI             STEEL      STAND       1.000
                    SLAB       STAND       1.000
                    FORM       STAND       1.000
1
```

Appendix 1 – Strip Step, System analysis – bridge 100-262-1 (cast)

WSP Sverige AB S T R I P STEP 2 PC-05-041001 Project: Bro 100-262-1 SIDA :
KONSTRUKTIONSTYP RAM Composite steel- concrete bridge using corrugated web i 4 :
RESULTAT Appendix X - System analysis, Construction phase
0 PROJEKT IDENT KONSTR.IDENT NR DATUM
Elly Yman 20-04-28
0
S N I T T K R A F T E R

PUNKT	STEEL M	STEEL N	STEEL Q	SLAB M	SLAB N	SLAB Q	FORM M	FORM N	FORM Q
1.000			.01	.05	.01	-.10	.01		-.02
.500	-.19		-1.57	-1.33	.01	-10.95	-.08		-.67
2.000	-.78		-3.15	-5.42	.01	-21.80	-.33		-1.33
2.000	-.78		196.49	-5.37		1106.69	-.32		66.58
.063	593.12		176.09	3301.72		968.35	198.64		58.26
.125	1121.29		155.23	6167.87		830.01	371.07		49.93
.188	1582.26		133.93	8593.06		691.68	516.97		41.61
.250	1971.60		108.99	10577.30		553.34	636.34		33.29
.313	2275.44		81.65	12120.60		415.00	729.19		24.97
.375	2492.12		54.31	13222.94		276.66	795.51		16.64
.438	2621.64		26.96	13884.33		138.33	835.30		8.32
.500	2664.01		-.38	14104.77		-.01	848.56		
.563	2619.23		-27.72	13884.26		-138.35	835.30		-8.32
.625	2487.30		-55.06	13222.80		-276.69	795.50		-16.65
.688	2268.21		-82.41	12120.39		-415.02	729.18		-24.97
.750	1961.96		-109.75	10577.03		-553.36	636.33		-33.29
.813	1570.64		-134.08	8592.71		-691.70	516.95		-41.61
.875	1111.10		-154.26	6167.45		-830.04	371.04		-49.94
.938	587.26		-174.43	3301.24		-968.37	198.61		-58.26
3.000	-.89		-194.60	-5.93		-1106.71	-.36		-66.58
3.000	-.85	-.01	3.36	-5.70	.01	22.53	-.34		1.36
.500	-.21	-.01	1.78	-1.42	.01	11.68	-.08		.70
4.000	.04	-.01	.19	.14	.01	.83	.01		.05

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      5      :
                 RESULTAT                        Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                     NR                 DATUM
                                        Elly Yman                                          20-04-28
0
  S N I T T K R A F T E R

 I           I            BROTT-B             I             KVASI             I
 I  PUNKT    I--------------------------------I--------------------------------I
 I           I    M    I     N    I     Q    I     M    I     N    I     Q    I
 I-----------I----------I----------I----------I----------I----------I----------I

        1.000        .07        .01       -.13        .06        .01       -.11
         .500      -1.92        .01     -15.83      -1.60        .01     -13.19
        2.000      -7.84        .01     -31.53      -6.53        .01     -26.28

        2.000      -7.77               1643.71      -6.48               1369.75
         .063    4912.18               1443.23    4093.48               1202.69
         .125    9192.26               1242.22    7660.22               1035.18
         .188   12830.75               1040.66   10692.29                867.22
         .250   15822.30                834.74   13185.25                695.62
         .313   18150.27                625.94   15125.23                521.62
         .375   19812.68                417.14   16510.57                347.62
         .438   20809.53                208.34   17341.28                173.61
         .500   21140.82                  -.47   17617.35                  -.39
         .563   20806.55               -209.27   17338.79               -174.39
         .625   19806.72               -418.07   16505.60               -348.39
         .688   18141.33               -626.88   15117.78               -522.40
         .750   15810.38               -835.68   13175.32               -696.40
         .813   12816.36              -1040.87   10680.30               -867.39
         .875    9179.51              -1241.07    7649.59              -1034.23
         .938    4904.52              -1441.27    4087.10              -1201.06
        3.000      -8.61              -1641.47      -7.17              -1367.90

        3.000      -8.27                 32.70      -6.89                 27.25
         .500      -2.06                 16.99      -1.72                 14.16
        4.000        .23                  1.29        .19                  1.08
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                            SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      6      :
                 RESULTAT                        Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                 NR                DATUM
                                        Elly Yman                                    20-04-28
0
  R E A K T I O N E R
```

PUNKT	STEEL R-ETA	STEEL R-ZETA	STEEL R-M	SLAB R-ETA	SLAB R-ZETA	SLAB R-M	FORM R-ETA	FORM R-ZETA	FORM R-M
2		-199.64			-1128.49			-67.91	
3	-.01	-197.96			-1129.24			-67.94	

```
0
  R E A K T I O N E R
```

PUNKT	BROTT-B R-ETA	BROTT-B R-ZETA	BROTT-B R-M	KVASI R-ETA	KVASI R-ZETA	KVASI R-M
2		-1675.24			-1396.03	
3	-.01	-1674.17			-1395.14	

1

Appendix 1 – Strip Step, System analysis – bridge 100-262-1 (cast)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      7     :
                 RESULTAT                        Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                   NR                 DATUM
                                        Elly Yman                                      20-04-28
0
  D E F O R M A T I O N E R
```

PUNKT	STEEL DELTA-Y	STEEL DELTA-Z	STEEL ROT.	SLAB DELTA-Y	SLAB DELTA-Z	SLAB ROT.	FORM DELTA-Y	FORM DELTA-Z	FORM ROT.
1.000	-1.921	1.392	-.159	-10.358	7.476	-.857	-.623	.450	-.052
.500	-1.921	.696	-.159	-10.358	3.738	-.857	-.623	.225	-.052
2.000	-1.921		-.159	-10.358		-.857	-.623		-.052
2.000	-1.921		-.159	-10.358		-.857	-.623		-.052
.063	-1.920	-8.787	-.155	-10.355	-47.192	-.831	-.623	-2.839	-.050
.125	-1.920	-17.078	-.142	-10.351	-91.634	-.759	-.623	-5.513	-.046
.188	-1.919	-24.428	-.121	-10.349	-130.900	-.646	-.623	-7.875	-.039
.250	-2.010	-30.549	-.100	-10.830	-163.497	-.532	-.652	-9.836	-.032
.313	-2.010	-35.516	-.078	-10.829	-189.899	-.415	-.651	-11.425	-.025
.375	-2.009	-39.186	-.053	-10.828	-209.389	-.284	-.651	-12.597	-.017
.438	-2.009	-41.433	-.027	-10.828	-221.333	-.144	-.651	-13.316	-.009
.500	-2.009	-42.185		-10.828	-225.355		-.651	-13.558	
.563	-2.009	-41.419	.027	-10.828	-221.333	.144	-.651	-13.316	.009
.625	-2.009	-39.157	.054	-10.828	-209.388	.284	-.651	-12.597	.017
.688	-2.010	-35.477	.078	-10.829	-189.898	.415	-.651	-11.424	.025
.750	-2.010	-30.504	.100	-10.830	-163.495	.532	-.652	-9.836	.032
.813	-2.100	-24.382	.121	-11.309	-130.898	.647	-.680	-7.875	.039
.875	-2.100	-17.040	.142	-11.312	-91.633	.759	-.681	-5.513	.046
.938	-2.101	-8.765	.155	-11.315	-47.191	.831	-.681	-2.839	.050
3.000	-2.101		.159	-11.319		.857	-.681		.052
3.000	-2.101		.159	-11.319		.857	-.681		.052
.500	-2.101	.694	.159	-11.319	3.738	.857	-.681	.225	.052
4.000	-2.101	1.388	.159	-11.319	7.476	.857	-.681	.450	.052

1

```
 WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                   SIDA     :
                  KONSTRUKTIONSTYP RAM             Composite steel- concrete bridge using corrugated web i      8      :
                  RESULTAT                         Appendix X - System analysis, Construction phase
0                 PROJEKT IDENT        KONSTR.IDENT                    NR                 DATUM
                                         Elly Yman                                        20-04-28
0
  D E F O R M A T I O N E R
```

PUNKT	BROTT-B DELTA-Y	BROTT-B DELTA-Z	BROTT-B ROT.	KVASI DELTA-Y	KVASI DELTA-Z	KVASI ROT.
1.000	-15.483	11.181	-1.281	-12.902	9.318	-1.068
.500	-15.483	5.591	-1.281	-12.902	4.659	-1.068
2.000	-15.483		-1.281	-12.902		-1.068
2.000	-15.483		-1.281	-12.902		-1.068
.063	-15.478	-70.581	-1.243	-12.898	-58.817	-1.036
.125	-15.473	-137.070	-1.136	-12.894	-114.225	-.946
.188	-15.469	-195.843	-.968	-12.891	-163.203	-.806
.250	-16.190	-244.658	-.796	-13.492	-203.882	-.663
.313	-16.188	-284.208	-.621	-13.490	-236.840	-.518
.375	-16.187	-313.407	-.425	-13.489	-261.172	-.354
.438	-16.186	-331.298	-.216	-13.489	-276.082	-.180
.500	-16.186	-337.317	.001	-13.488	-281.098	
.563	-16.186	-331.280	.217	-13.489	-276.067	.181
.625	-16.187	-313.372	.426	-13.489	-261.143	.355
.688	-16.188	-284.159	.622	-13.490	-236.799	.518
.750	-16.190	-244.602	.797	-13.492	-203.835	.664
.813	-16.907	-195.786	.968	-14.090	-163.155	.807
.875	-16.911	-137.023	1.136	-14.093	-114.186	.946
.938	-16.916	-70.554	1.243	-14.097	-58.795	1.036
3.000	-16.922		1.281	-14.101		1.067
3.000	-16.922		1.281	-14.101		1.067
.500	-16.922	5.588	1.281	-14.101	4.657	1.067
4.000	-16.922	11.177	1.281	-14.101	9.314	1.067

1

Appendix 1 – Strip Step, System analysis – bridge 100-262-1 (cast)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                    SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     -1     :
                 RESULTAT                        Appendix X - System analysis, Construction phase
0                PROJEKT IDENT        KONSTR.IDENT                  NR               DATUM
                                        Elly Yman                                    20-04-28
0
0I N N E H   L L S F ™ R T E C K N I N G
0
 DATAKVITTO                                                      SIDA
0           ENHETER                                                 1
            MATERIALKONSTANTER                                      2
            ELEMENTTYPER                                            2
            SYSTEMBESKRIVNING                                       2
            STANDARDLASTFALL                                        3
            LASTKOMBINATIONER                                       3
0
 RESULTAT

0         I LASTKOMBINATIONS I  STABI-  I  SPŽN-    I  SNITT-  I   REAK-  I  DEFOR-  I
          I NAMN             I   LITET  I  NINGAR   I  KRAFTER I  TIONER  I MATIONER I
          I------------------I----------I----------I----------I----------I----------I

            STEEL                                       4          6          7
            SLAB                                        4          6          7
            FORM                                        4          6          7
            BROTT-B                                     5          6          8
            KVASI                                       5          6          8
0
 SISTA SIDA I DENNA BERŽKNING                                      8
1
```

```
WSP Sverige AB    S T R I P   STEP 2           Project: Bro 100-262-1
                 FRAME                         Composite steel- concrete bridge using corrugated web i
                 STATISTIC                     Appendix X - System analysis, Construction phase

PROJECT NO       SYSTEM NO       NO            DATE          COMP-DATE       START-TERM.TIME      VERSION
                   Elly Yman                                 2020-04-28      11:50:46-11:50:46    PC-05-041001
0
    4 JOINTS                         I  CROSS SECTION EVAL        .00  I  MATRIX  ROWS BAND    DETERM     PIVOT    INCR
    3 ELEMENTS                       I  INPUT EVALUATION               I
    0 ARCHES                         I     GEOMETRY               .00  I   A22I      3    2  .10E+01  .10E+01     0
    2 REACTIONS                      I     LOADCASES              .06  I   SR        9    8  .11E-35  .10E-07   -28
                           LOADEL    I     PRESTRESSING           .00  I   KM1T      9    4  .14E+36  .32E+06    30
    3 STANDARD LOADCASES          9  I     RESULT REQUESTS        .00  I
    0 TEMPERATURE LOADC.          0  I  MATRIX PREPARATION             I  FILE   TRK   POS           FILE   TRK    POS
    0 SUPPORT SETTLEMENTS         0  I     GEOMETRY               .00  I
    0 PRESTR. LOADCASES           0  I     REDUCTION              .00  I    1     0     0             11     0      0
                                     I     LOADING CASES          .22  I    2     0     0             14     0      0
    0 TRACKS (MAX. 0 ELEMENTS)       I     INFLUENCE LINES        .00  I    3     0     0             19     0      0
    0 TRAFFIC GROUPS                 I     2ND ORDER/CREEP        .00  I    4     0     0             20     0      0
    0 TRAFFIC LOADCASES              I  CALCULATION PARTS              I    7     0     0             21     0      0
                                     I     RANDF. REDUCT.         .05  I    8     0     0             22     0      0
    0 CROSS SECTIONS                 I     INVERT      A22        .00  I    9     0     0             24     0      0
                                     I     SYSTEM MATRICES        .00  I   10     0     0             25     0      0
    5 COMBINATIONS                   I     INVERT       VR        .00  I
    5   ADDITIVE                     I     TRIANG        K        .00  I  ERROR WITHIN PROGRAM PART           0
    0   ENVELOPES                    I     DEFORM      URT        .00  I  NO OF CALLS TO SUBROUTINE DISC      0
    0   CREEP/SHRINKAGE              I     GROSSFORCES POO        .00  I
    0   STRESSES                     I     DEFORMATIONS           .00  I  RESULT PAGES  ECHOPRINT             3
    0   SECOND ORDER                 I     BASIC INFL.LINES       .00  I                STRESSES              0
    0   STABILITY                    I     TRAFFIC INFL.          .00  I                GROSSFORCES           2
                                     I     INFLUENCE LINES        .00  I                REACTIONS             1
   23 RESULT POINTS                  I     SECOND ORDER           .00  I                DEFORMATIONS          2
   23 DEFORMATION POINTS             I     CREEP                  .00  I                INFLUENCE LINES       0
    0 INFLUENCE LINES                I     STABILITY              .00  I                STABILITY             0
----------------------------------I  TRAFFIC EVALUATION       .00  I-----------------------------------------
                                     I  COMBINATION PART          .00  I  IBM PC
                                     I  RESULT PRINT PART              I  TOTAL ELAPSED TIME       .39 SECONDS
                                     I     STRESSES               .00  I
                                     I     FORCES / DEFORM        .06  I
                                     I     INFLUENCE LINES        .00
```

Appendix 2 – Strip Step, Calculation of shrinkage loads – bridge 100-262-1

Appendix 2 – Strip Step, System analysis – bridge 100-262-1 (shrinkage)

```
D A T A   I N P U T        STRIP STEP 2 (PC-05-041001)       DATE: 2020-04-28   TIME: 11:50:50       PAGE  1       :
                                                                                                                   :
   CARD        0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
    NO         1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
 ----------------------------------------------------------------------------------------------------------------
     1*        9000 STRIP-STEP2                            Elly Yman
     2*        9100 Project: Bro 100-262-1
     3*        9200 Composite steel- concrete bridge using corrugated web in stainless steel
     4*        9300 Appendix X - System analysis, Shrinkage
     5*        2010 10000     200000              1
     6*        2020 1    BALK N
     7*        2022 0              -0.389              2.37      1453.391  1700
     8*        2022 0.5            -0.389              2.37      1453.391  1700
     9*        2020 2    BALK N
    10*        2022 0              -0.389              2.37      1453.391  1700
    11*        2022 10.5           -0.389              2.37      1453.391  1700
    12*        2022 10.5           -0.433              2.37      1609.761  1806
    13*        2022 11.3           -0.563              2.37      2042.908  2042
    14*        2022 25.5           -0.563              2.37      2042.908  2042
    15*        2022 39.7           -0.563              2.37      2042.908  2042
    16*        2022 40.5           -0.433              2.37      1609.761  1806
    17*        2022 40.5           -0.389              2.37      1453.391  1700
    18*        2022 51             -0.389              2.37      1453.391  1700
    19*        2020 3    BALK N
    20*        2022 0              -0.389              2.37      1453.391  1700
    21*        2022 0.5            -0.389              2.37      1453.391  1700
    22*        2040 1         0         0                                            1    2
    23*        2040 2         0.5       0                   -2.05     ZR
    24*        2040                                                                  2    3
    25*        2040 3         51.5      0                   -2.05     YZR
    26*        2040                                                                  3    4
    27*        2040 4         52        0                                           -3    3
    28*        2050 E:SHRINK  2    3    0              -0.389    KONC 5206                YZ
 BW:         6 JOINTS:   2
    29*        2052 -2859
    30*        2050           3    4    0              -0.389    KONC -5206               YZ
    31*        2052 2859
    32*        2140 KRYMP          E:SHRINK  STAND  1.00
    33*        2170 1    2    2                   S    DEF
    34*        2170 2    3    16                  S    DEF
    35*        2170 3    4    2                   S    DEF
1
```

Appendix 2 – Strip Step, System analysis – bridge 100-262-1 (shrinkage)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                           SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i    1     :
                 DATAKVITTO                      Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT       KONSTR.IDENT                    NR                 DATUM
                                       Elly Yman                                        20-04-28
0

0E N H E T E R :  SI-SYSTEMET

0           LANGDER                                         METER
0           VINKLAR                                         RADIAN
0           LASTER, KRAFTER                                 KILONEWTON
0           P KŽNNINGAR, ELASTICITETSMODULER                MEGAPASCAL
0           MASSOR                                          TON
0           TEMPERATURER                                    KELVIN
0           FREKVENSER                                      HERTZ
0
0FR N DESSA GRUNDENHETER SAMT KOMBINATIONER HŽRAV G™RES
 F™LJANDE UNDANTAG:
0
            VINKEL I GEOMETRIBESKRIVNING OCH
            ROTATIONER I DEFORMATIONSRESULTAT               GRADER (360/VARV)
0           F™RSKJUTNINGAR I DEFORMATIONER                  MILLIMETER
1
```

Appendix 2 – Strip Step, System analysis – bridge 100-262-1 (shrinkage)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                              SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      2    :
                 DATAKVITTO                      Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT         KONSTR.IDENT                    NR                DATUM
                                         Elly Yman                                       20-04-28
0
 M A T E R I A L K O N S T A N T E R

 I    K    I     E0   I  G0/E0  I DENSI-  I OMEGA T I EPSILON I FI-KRYP I
 I         I          I         I   TET   I *100000 I *100000 I         I
 I---------I---------I---------I---------I---------I---------I---------I

      10000    200000    .500                1.000
0
 E L E M E N T T Y P E R

 I  NR  I ARBETS I    E    I  DEN- I KRYP- I    R    I TVAR- I    F    I    D    I    H    I    K*I    I    K*A    I   K*AQ  I
 I      I BIDRAG I    G    I SITET I  IND  I         I SNITT I         I         I         I           I           I         I
 I------I--------I--------I-------I-------I-------I-------I-------I-------I-------I---------I---------I---------I

      1   N         200000                     .000           -.389          2.370   1453.391  1700.000
          BALK      100000                    1.000           -.389          2.370   1453.391  1700.000

      2   N         200000                     .000           -.389          2.370   1453.391  1700.000
          BALK      100000                     .206           -.389          2.370   1453.391  1700.000
                                               .206           -.433          2.370   1609.761  1806.000
                                               .222           -.563          2.370   2042.908  2042.000
                                               .500           -.563          2.370   2042.908  2042.000
                                               .778           -.563          2.370   2042.908  2042.000
                                               .794           -.433          2.370   1609.761  1806.000
                                               .794           -.389          2.370   1453.391  1700.000
                                              1.000           -.389          2.370   1453.391  1700.000

      3   N         200000                     .000           -.389          2.370   1453.391  1700.000
          BALK      100000                    1.000           -.389          2.370   1453.391  1700.000
0
 S Y S T E M B E S K R I V N I N G

 I KNUTPT I     Y    I    Z    I DELTA Y I DELTA Z I LED-TYP I   ALFA  I ELEMENT I KNUTPT I
 I  START I          I         I         I         I  REAKT. I         I   TYP   I  SLUT  I
 I--------I---------I---------I---------I---------I---------I---------I---------I--------I

      1        .000      .000                                              1        2

      2        .500      .000              -2.050   ZR
                                                                           2        3

      3      51.500      .000              -2.050   YZR
                                                                           3        4

      4      52.000      .000                                             -3        3
0
 S T A N D A R D L A S T F A L L

 I LASTFALL I  KNUTPUNKT  I    R   I    W   I LAST- I    P-Y   I    P-Z   I    M     I
 I NAMN     I  START-SLUT I        I        I TYP   I          I          I          I
 I----------I-------------I-------I-------I-------I---------I---------I---------I

  E:SHRINK             2-3     .000    -.389   KONC    5206.00              -2859.00
                       3-4     .000    -.389   KONC   -5206.00               2859.00
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      3    :
                 DATAKVITTO                      Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT       KONSTR.IDENT                  NR                DATUM
                                       Elly Yman                                     20-04-28
0
 L A S T K O M B I N A T I O N E R

 I LASTKOMBINATION I          - L A S T F A L L -       I SPANN. I K-KRYP I
 I NAMN      I ART  I NAMN      I  TYP   I FAKTOR I ALTER  I  IND   I        I
 I----------I------I----------I--------I--------I--------I--------I--------I

  KRYMP             E:SHRINK   STAND       1.000
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                      SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i        4     :
                 RESULTAT                        Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT        KONSTR.IDENT                   NR                 DATUM
                                        Elly Yman                                         20-04-28
0
  S N I T T K R A F T E R

 I           I             KRYMP             I
 I  PUNKT    I-------------------------------I
 I           I     M    I      N    I     Q    I
 I-----------I----------I----------I----------I

       1.000                   .01       -.22
        .500        -.05       .01       -.22
       2.000        -.11       .01       -.22

       2.000     2858.89  -5205.99
        .063     2858.89  -5205.99
        .125     2858.89  -5205.99
        .188     2858.89  -5205.99
        .250     3764.74  -5205.99
        .313     3764.74  -5205.99
        .375     3764.74  -5205.99
        .438     3764.74  -5205.99
        .500     3764.74  -5205.99
        .563     3764.74  -5205.99
        .625     3764.74  -5205.99
        .688     3764.75  -5205.99
        .750     3764.75  -5205.99
        .813     2858.91  -5205.99
        .875     2858.91  -5205.99
        .938     2858.91  -5205.99
       3.000     2858.91  -5205.99

       3.000        -.11                  .21
        .500        -.06                  .21
       4.000        -.01                  .21
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                   SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     5     :
                 RESULTAT                        Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT        KONSTR.IDENT                    NR                  DATUM
                                        Elly Yman                                         20-04-28
0
  R E A K T I O N E R

 I           I             KRYMP             I
 I  PUNKT    I-------------------------------I
 I           I   R-ETA  I   R-ZETA I    R-M  I
 I-----------I----------I----------I---------I

       2                       -.22

       3            -.01       -.21
1
```

Appendix 2 – Strip Step, System analysis – bridge 100-262-1 (shrinkage)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                 SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      6     :
                 RESULTAT                        Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT        KONSTR.IDENT                  NR                 DATUM
                                        Elly Yman                                      20-04-28
0
  D E F O R M A T I O N E R
```

PUNKT	KRYMP DELTA-Y	KRYMP DELTA-Z	KRYMP ROT.
1.000	3.511	1.208	-.138
.500	3.511	.604	-.138
2.000	3.511		-.138
2.000	3.511		-.138
.063	3.022	-7.202	-.120
.125	2.534	-13.405	-.102
.188	2.046	-18.608	-.085
.250	1.374	-22.825	-.067
.313	.968	-26.102	-.050
.375	.561	-28.442	-.034
.438	.155	-29.847	-.017
.500	-.251	-30.315	
.563	-.658	-29.847	.017
.625	-1.064	-28.442	.034
.688	-1.470	-26.102	.051
.750	-1.877	-22.825	.067
.813	-2.550	-18.608	.085
.875	-3.038	-13.405	.103
.938	-3.526	-7.202	.121
3.000	-4.013		.138
3.000	-4.013		.138
.500	-4.013	.604	.138
4.000	-4.013	1.208	.138

1

Appendix 2 – Strip Step, System analysis – bridge 100-262-1 (shrinkage)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                   SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      -1     :
                 RESULTAT                        Appendix X - System analysis, Shrinkage
0                PROJEKT IDENT        KONSTR.IDENT                  NR                 DATUM
                                        Elly Yman                                      20-04-28
0
0I N N E H   L L S F ™ R T E C K N I N G
0
 DATAKVITTO                                                        SIDA
0           ENHETER                                                   1
            MATERIALKONSTANTER                                        2
            ELEMENTTYPER                                              2
            SYSTEMBESKRIVNING                                         2
            STANDARDLASTFALL                                          2
            LASTKOMBINATIONER                                         3
0
 RESULTAT

0         I LASTKOMBINATIONS I  STABI-  I  SPŽN-    I  SNITT-  I   REAK-   I  DEFOR-  I
          I NAMN             I   LITET  I  NINGAR   I  KRAFTER I  TIONER   I MATIONER I
          I------------------I----------I----------I----------I----------I----------I

            KRYMP                                         4          5          6
0
 SISTA SIDA I DENNA BERŽKNING                                        6
1
```

Appendix 2 – Strip Step, System analysis – bridge 100-262-1 (shrinkage)

```
 WSP Sverige AB    S T R I P   STEP 2            Project: Bro 100-262-1
                  FRAME                         Composite steel- concrete bridge using corrugated web i
                  STATISTIC                     Appendix X - System analysis, Shrinkage

 PROJECT NO       SYSTEM NO       NO           DATE          COMP-DATE     START-TERM.TIME    VERSION
                    Elly Yman                                2020-04-28    11:50:50-11:50:50  PC-05-041001
0
     4 JOINTS                          I  CROSS SECTION EVAL      .00  I  MATRIX  ROWS BAND    DETERM     PIVOT    INCR
     3 ELEMENTS                        I  INPUT EVALUATION             I
     0 ARCHES                          I     GEOMETRY             .00  I   A22I      3    2  .10E+01  .10E+01      0
     2 REACTIONS                       I     LOADCASES            .00  I   SR        9    8  .36E-35  .43E-08    -32
                            LOADEL     I     PRESTRESSING         .00  I   KM1T      9    4  .44E+36  .73E+06     33
     1 STANDARD LOADCASES         2    I     RESULT REQUESTS      .00  I
     0 TEMPERATURE LOADC.         0    I  MATRIX PREPARATION           I  FILE   TRK   POS          FILE   TRK   POS
     0 SUPPORT SETTLEMENTS        0    I     GEOMETRY             .00  I
     0 PRESTR. LOADCASES          0    I     REDUCTION            .00  I   1      0     0            11     0     0
                                       I     LOADING CASES        .17  I   2      0     0            14     0     0
     0 TRACKS (MAX. 0 ELEMENTS)        I     INFLUENCE LINES      .00  I   3      0     0            19     0     0
     0 TRAFFIC GROUPS                  I     2ND ORDER/CREEP      .00  I   4      0     0            20     0     0
     0 TRAFFIC LOADCASES               I  CALCULATION PARTS            I   7      0     0            21     0     0
                                       I     RANDF. REDUCT.       .05  I   8      0     0            22     0     0
     0 CROSS SECTIONS                  I     INVERT      A22      .00  I   9      0     0            24     0     0
                                       I     SYSTEM MATRICES      .00  I  10      0     0            25     0     0
     1 COMBINATIONS                    I     INVERT       VR      .00  I
     1   ADDITIVE                      I     TRIANG        K      .00  I  ERROR WITHIN PROGRAM PART            0
     0   ENVELOPES                     I     DEFORM      URT      .00  I  NO OF CALLS TO SUBROUTINE DISC       0
     0   CREEP/SHRINKAGE               I     GROSSFORCES POO      .00  I
     0   STRESSES                      I     DEFORMATIONS         .00  I  RESULT PAGES  ECHOPRINT              3
     0   SECOND ORDER                  I     BASIC INFL.LINES     .00  I                STRESSES               0
     0   STABILITY                     I     TRAFFIC INFL.        .00  I                GROSSFORCES            1
                                       I     INFLUENCE LINES      .00  I                REACTIONS              1
    23 RESULT POINTS                   I     SECOND ORDER         .00  I                DEFORMATIONS           1
    23 DEFORMATION POINTS              I     CREEP                .00  I                INFLUENCE LINES        0
     0 INFLUENCE LINES                 I     STABILITY            .00  I                STABILITY              0
 ----------------------------------I  TRAFFIC EVALUATION      .00  I-----------------------------------------
                                       I  COMBINATION PART        .00  I  IBM PC
                                       I  RESULT PRINT PART            I  TOTAL ELAPSED TIME       .22 SECONDS
                                       I     STRESSES             .00  I
                                       I     FORCES / DEFORM      .00  I
                                       I     INFLUENCE LINES      .00
```

Appendix 3 – Strip Step, Calculation of permanent loads – bridge 100-262-1

Appendix 3 – Strip Step, System analysis – bridge 100-262-1 (permanent)

```
D A T A   I N P U T        STRIP STEP 2 (PC-05-041001)        DATE: 2020-04-28   TIME: 11:50:53       PAGE  1      :
                                                                                                                     :
   CARD       0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
    NO        1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
 -------------------------------------------------------------------------------------------------------------
     1*       9000 STRIP-STEP2                            Elly Yman
     2*       9100 Project: Bro 100-262-1
     3*       9200 Composite steel- concrete bridge using corrugated web in stainless steel
     4*       9300 Appendix X - System analysis, Long-term loads
     5*       2010 10000     200000              1
     6*       2020 1    BALK N
     7*       2022 0              -0.464              2.37      1381.533  1495
     8*       2022 0.5            -0.464              2.37      1381.533  1495
     9*       2020 2    BALK N
    10*       2022 0              -0.464              2.37      1381.533  1495
    11*       2022 10.5           -0.464              2.37      1381.533  1495
    12*       2022 10.5           -0.509              2.37      1526.898  1602
    13*       2022 11.3           -0.644              2.37      1922.266  1838
    14*       2022 25.5           -0.644              2.37      1922.266  1838
    15*       2022 39.7           -0.644              2.37      1922.266  1838
    16*       2022 40.5           -0.509              2.37      1526.898  1602
    17*       2022 40.5           -0.464              2.37      1381.533  1495
    18*       2022 51             -0.464              2.37      1381.533  1495
    19*       2020 3    BALK N
    20*       2022 0              -0.464              2.37      1381.533  1495
    21*       2022 0.5            -0.464              2.37      1381.533  1495
    22*       2040 1         0         0                                            1    2
    23*       2040 2         0.5       0                   -2.05     ZR
    24*       2040                                                                  2    3
    25*       2040 3         51.5      0                   -2.05     YZR
    26*       2040                                                                  3    4
    27*       2040 4         52        0                                            -3   3
    28*       2050 FORM      1    4                             UTBR           -2.611    G/S
 BW:        6 JOINTS:   2
    29*       2050 AVSLAB    1    4                             UTBR           0         G/S
    30*       2050 ASPH      1    4                             UTBR           11.919    G/S
    31*       2050 BARR      1    4                             UTBR           0.25      G/S
    32*       2140 FORM           FORM      STAND  1.00
    33*       2140 ASPH           ASPH      STAND  1.00
    34*       2140 BARR           BARR      STAND  1.00
    35*       2140 AVSLAB         AVSLAB    STAND  1.00
    36*       2140 BROTT-B        FORM      STAND  1.2
    37*       2140                ASPH      STAND  1.32
    38*       2140                BARR      STAND  1.2
    39*       2140                AVSLAB    STAND  1.2
    40*       2140 KVASI          FORM      STAND  1
    41*       2140                ASPH      STAND  1.1
    42*       2140                BARR      STAND  1
    43*       2140                AVSLAB    STAND  1
    44*       2170 1    2    2                   S    DEF
    45*       2170 2    3    16                  S    DEF
    46*       2170 3    4    2                   S    DEF
1
```

Appendix 3 – Strip Step, System analysis – bridge 100-262-1 (permanent)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     1     :
                 DATAKVITTO                      Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT       KONSTR.IDENT                    NR                DATUM
                                       Elly Yman                                       20-04-28
0

0E N H E T E R :  SI-SYSTEMET

0           LANGDER                                         METER
0           VINKLAR                                         RADIAN
0           LASTER, KRAFTER                                 KILONEWTON
0           P KŽNNINGAR, ELASTICITETSMODULER                MEGAPASCAL
0           MASSOR                                          TON
0           TEMPERATURER                                    KELVIN
0           FREKVENSER                                      HERTZ
0
0FR N DESSA GRUNDENHETER SAMT KOMBINATIONER HŽRAV G™RES
 F™LJANDE UNDANTAG:
0
            VINKEL I GEOMETRIBESKRIVNING OCH
            ROTATIONER I DEFORMATIONSRESULTAT               GRADER (360/VARV)
0           F™RSKJUTNINGAR I DEFORMATIONER                  MILLIMETER
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      2     :
                 DATAKVITTO                      Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT        KONSTR.IDENT                     NR                 DATUM
                                        Elly Yman                                         20-04-28
0
 M A T E R I A L K O N S T A N T E R

 I    K    I    E0   I  G0/E0  I DENSI-  I OMEGA T I EPSILON I FI-KRYP I
 I         I         I         I   TET   I *100000 I *100000 I         I
 I---------I---------I---------I---------I---------I---------I---------I

      10000    200000    .500                1.000
0
 E L E M E N T T Y P E R

 I  NR  I ARBETS I    E    I  DEN-  I KRYP-  I    R    I TVAR-  I   F    I   D    I   H    I   K*I    I   K*A    I   K*AQ   I
 I      I BIDRAG I    G    I SITET  I  IND   I         I SNITT  I        I        I        I          I          I          I
 I------I--------I---------I--------I--------I---------I--------I--------I--------I--------I----------I----------I----------I

      1  N          200000                         .000           -.464             2.370   1381.533  1495.000
         BALK       100000                        1.000           -.464             2.370   1381.533  1495.000

      2  N          200000                         .000           -.464             2.370   1381.533  1495.000
         BALK       100000                         .206           -.464             2.370   1381.533  1495.000
                                                   .206           -.509             2.370   1526.898  1602.000
                                                   .222           -.644             2.370   1922.266  1838.000
                                                   .500           -.644             2.370   1922.266  1838.000
                                                   .778           -.644             2.370   1922.266  1838.000
                                                   .794           -.509             2.370   1526.898  1602.000
                                                   .794           -.464             2.370   1381.533  1495.000
                                                  1.000           -.464             2.370   1381.533  1495.000

      3  N          200000                         .000           -.464             2.370   1381.533  1495.000
         BALK       100000                        1.000           -.464             2.370   1381.533  1495.000
0
 S Y S T E M B E S K R I V N I N G

 I KNUTPT I    Y    I    Z    I DELTA Y I DELTA Z I LED-TYP I   ALFA  I ELEMENT I KNUTPT I
 I  START I         I         I         I         I  REAKT. I         I   TYP   I  SLUT  I
 I--------I---------I---------I---------I---------I---------I---------I---------I--------I

       1        .000      .000                                                1        2

       2        .500      .000                -2.050   ZR
                                                                              2        3

       3      51.500      .000                -2.050   YZR
                                                                              3        4

       4      52.000      .000                                               -3        3
0
 S T A N D A R D L A S T F A L L

 I LASTFALL I  KNUTPUNKT  I   R   I   W   I LAST- I   P-Y   I   P-Z   I    M    I
 I NAMN     I  START-SLUT I       I       I TYP   I         I         I         I
 I----------I-------------I-------I-------I-------I---------I---------I---------I

  FORM                1-2    .000          UTBR                 2.61
                            1.000          UTBR                 2.61
                      2-3    .000          UTBR                 2.61
                            1.000          UTBR                 2.61
                      3-4    .000          UTBR                 2.61
                            1.000          UTBR                 2.61

1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                         SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      3     :
                 DATAKVITTO                      Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                 DATUM
                                        Elly Yman                                        20-04-28
0
 S T A N D A R D L A S T F A L L   (FORTS.)

 I LASTFALL I  KNUTPUNKT  I    R   I    W   I LAST- I    P-Y   I    P-Z   I     M    I
 I NAMN     I  START-SLUT I        I        I TYP   I          I          I          I
 I----------I-------------I--------I--------I-------I----------I----------I----------I

  AVSLAB               1-2     .000            UTBR
                              1.000            UTBR
                       2-3     .000            UTBR
                              1.000            UTBR
                       3-4     .000            UTBR
                              1.000            UTBR

  ASPH                 1-2     .000            UTBR                 -11.92
                              1.000            UTBR                 -11.92
                       2-3     .000            UTBR                 -11.92
                              1.000            UTBR                 -11.92
                       3-4     .000            UTBR                 -11.92
                              1.000            UTBR                 -11.92

  BARR                 1-2     .000            UTBR                   -.25
                              1.000            UTBR                   -.25
                       2-3     .000            UTBR                   -.25
                              1.000            UTBR                   -.25
                       3-4     .000            UTBR                   -.25
                              1.000            UTBR                   -.25
0
 L A S T K O M B I N A T I O N E R

 I LASTKOMBINATION I          - L A S T F A L L -        I SPANN.  I K-KRYP I
 I NAMN       I ART  I NAMN      I   TYP    I FAKTOR I ALTER    I  IND    I        I
 I----------I------I----------I--------I--------I--------I--------I--------I

  FORM                 FORM        STAND        1.000

  ASPH                 ASPH        STAND        1.000

  BARR                 BARR        STAND        1.000

  AVSLAB               AVSLAB      STAND        1.000

  BROTT-B              FORM        STAND        1.200
                       ASPH        STAND        1.320
                       BARR        STAND        1.200
                       AVSLAB      STAND        1.200

  KVASI                FORM        STAND        1.000
                       ASPH        STAND        1.100
                       BARR        STAND        1.000
                       AVSLAB      STAND        1.000
1
```

WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1   SIDA :
KONSTRUKTIONSTYP RAM   Composite steel- concrete bridge using corrugated web i   4 :
RESULTAT   Appendix X - System analysis, Long-term loads
0   PROJEKT IDENT   KONSTR.IDENT   NR   DATUM
Elly Yman   20-04-28
0
S N I T T K R A F T E R

PUNKT	FORM M	FORM N	FORM Q	ASPH M	ASPH N	ASPH Q	BARR M	BARR N	BARR Q
1.000	-.01		.01	.01		.06			
.500	.08		.66	-.35		-2.92	-.01		-.06
2.000	.32		1.31	-1.45		-5.90	-.03		-.12
2.000	.31		-66.58	-1.43		303.93	-.03		6.37
.063	-198.65		-58.26	906.81		265.94	19.02		5.58
.125	-371.08		-49.94	1693.95		227.95	35.53		4.78
.188	-516.98		-41.61	2359.99		189.96	49.50		3.98
.250	-636.36		-33.29	2904.93		151.97	60.93		3.19
.313	-729.21		-24.97	3328.77		113.97	69.82		2.39
.375	-795.53		-16.64	3631.51		75.98	76.17		1.59
.438	-835.32		-8.32	3813.16		37.99	79.98		.80
.500	-848.58			3873.70			81.25		
.563	-835.32		8.32	3813.15		-37.99	79.98		-.80
.625	-795.52		16.65	3631.50		-75.98	76.17		-1.59
.688	-729.20		24.97	3328.75		-113.98	69.82		-2.39
.750	-636.35		33.29	2904.90		-151.97	60.93		-3.19
.813	-516.97		41.61	2359.95		-189.96	49.50		-3.98
.875	-371.07		49.94	1693.90		-227.95	35.53		-4.78
.938	-198.64		58.26	906.75		-265.94	19.02		-5.58
3.000	.33		66.58	-1.49		-303.94	-.03		-6.38
3.000	.32		-1.30	-1.48	-.01	5.91	-.03		.12
.500	.08		-.64	-.37	-.01	2.93	-.01		.06
4.000			.01	-.01	-.01	-.05			

1

Appendix 3 – Strip Step, System analysis – bridge 100-262-1 (permanent)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     5      :
                 RESULTAT                        Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                 DATUM
                                        Elly Yman                                        20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	AVSLAB M	AVSLAB N	AVSLAB Q	BROTT-B M	BROTT-B N	BROTT-B Q	KVASI M	KVASI N	KVASI Q
1.000						.09			.08
.500				-.38		-3.13	-.32		-2.61
2.000				-1.57		-6.36	-1.30		-5.30
2.000				-1.55		328.95	-1.29		274.12
.063				981.43		287.83	817.86		239.86
.125				1833.35		246.71	1527.79		205.59
.188				2554.20		205.59	2128.50		171.33
.250				3143.99		164.47	2619.99		137.06
.313				3602.71		123.35	3002.26		102.79
.375				3930.37		82.24	3275.31		68.53
.438				4126.96		41.12	3439.14		34.26
.500				4192.49			3493.74		
.563				4126.95		-41.12	3439.13		-34.27
.625				3930.35		-82.24	3275.29		-68.53
.688				3602.69		-123.36	3002.24		-102.80
.750				3143.96		-164.47	2619.96		-137.06
.813				2554.16		-205.59	2128.47		-171.33
.875				1833.30		-246.71	1527.75		-205.59
.938				981.37		-287.83	817.81		-239.86
3.000				-1.62		-328.95	-1.35		-274.12
3.000				-1.60	-.01	6.39	-1.33	-.01	5.33
.500				-.40	-.01	3.17	-.34	-.01	2.64
4.000				-.01	-.01	-.06	-.01	-.01	-.05

1

Appendix 3 – Strip Step, System analysis – bridge 100-262-1 (permanent)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                               SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      6     :
                 RESULTAT                        Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT       KONSTR.IDENT                  NR                DATUM
                                       Elly Yman                                       20-04-28
0
  R E A K T I O N E R
```

PUNKT	FORM R-ETA	FORM R-ZETA	FORM R-M	ASPH R-ETA	ASPH R-ZETA	ASPH R-M	BARR R-ETA	BARR R-ZETA	BARR R-M
2		67.89			-309.83			-6.50	
3		67.88		-.01	-309.84			-6.50	

```
0
  R E A K T I O N E R
```

PUNKT	AVSLAB R-ETA	AVSLAB R-ZETA	AVSLAB R-M	BROTT-B R-ETA	BROTT-B R-ZETA	BROTT-B R-M	KVASI R-ETA	KVASI R-ZETA	KVASI R-M
2					-335.30			-279.42	
3				-.02	-335.34		-.01	-279.45	

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   7      :
                 RESULTAT                        Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT        KONSTR.IDENT                NR               DATUM
                                        Elly Yman                                  20-04-28
0
 D E F O R M A T I O N E R
```

PUNKT	FORM DELTA-Y	FORM DELTA-Z	FORM ROT.	ASPH DELTA-Y	ASPH DELTA-Z	ASPH ROT.	BARR DELTA-Y	BARR DELTA-Z	BARR ROT.
1.000	.543	-.204	.023	-2.480	.932	-.107	-.052	.020	-.002
.500	.543	-.102	.023	-2.480	.466	-.107	-.052	.010	-.002
2.000	.543		.023	-2.480		-.107	-.052		-.002
2.000	.543		.023	-2.480		-.107	-.052		-.002
.063	.543	1.289	.023	-2.480	-5.883	-.104	-.052	-.123	-.002
.125	.543	2.506	.021	-2.480	-11.438	-.095	-.052	-.240	-.002
.188	.543	3.587	.018	-2.480	-16.374	-.082	-.052	-.343	-.002
.250	.595	4.491	.015	-2.717	-20.503	-.067	-.057	-.430	-.001
.313	.595	5.224	.012	-2.717	-23.849	-.053	-.057	-.500	-.001
.375	.595	5.766	.008	-2.717	-26.319	-.036	-.057	-.552	-.001
.438	.595	6.097	.004	-2.717	-27.833	-.018	-.057	-.584	
.500	.595	6.209		-2.717	-28.343		-.057	-.594	
.563	.595	6.097	-.004	-2.717	-27.833	.018	-.057	-.584	
.625	.595	5.766	-.008	-2.717	-26.319	.036	-.057	-.552	.001
.688	.595	5.224	-.012	-2.717	-23.849	.053	-.057	-.500	.001
.750	.595	4.491	-.015	-2.717	-20.503	.067	-.057	-.430	.001
.813	.647	3.587	-.018	-2.955	-16.374	.082	-.062	-.343	.002
.875	.647	2.506	-.021	-2.955	-11.437	.095	-.062	-.240	.002
.938	.647	1.289	-.023	-2.955	-5.883	.104	-.062	-.123	.002
3.000	.647		-.023	-2.955		.107	-.062		.002
3.000	.647		-.023	-2.955		.107	-.062		.002
.500	.647	-.102	-.023	-2.955	.466	.107	-.062	.010	.002
4.000	.647	-.204	-.023	-2.955	.932	.107	-.062	.020	.002

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                     SIDA    :
                 KONSTRUKTIONSTYP RAM             Composite steel- concrete bridge using corrugated web i      8     :
                 RESULTAT                         Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT         KONSTR.IDENT                    NR                DATUM
                                         Elly Yman                                       20-04-28
0
  D E F O R M A T I O N E R
```

PUNKT	AVSLAB DELTA-Y	AVSLAB DELTA-Z	AVSLAB ROT.	BROTT-B DELTA-Y	BROTT-B DELTA-Z	BROTT-B ROT.	KVASI DELTA-Y	KVASI DELTA-Z	KVASI ROT.
1.000				-2.684	1.008	-.116	-2.237	.840	-.096
.500				-2.684	.504	-.116	-2.237	.420	-.096
2.000				-2.684		-.116	-2.237		-.096
2.000				-2.684		-.116	-2.237		-.096
.063				-2.684	-6.367	-.112	-2.237	-5.306	-.094
.125				-2.684	-12.379	-.103	-2.237	-10.316	-.086
.188				-2.684	-17.721	-.088	-2.237	-14.768	-.074
.250				-2.941	-22.190	-.073	-2.451	-18.492	-.061
.313				-2.941	-25.812	-.057	-2.451	-21.510	-.047
.375				-2.941	-28.485	-.039	-2.451	-23.738	-.032
.438				-2.941	-30.123	-.020	-2.451	-25.103	-.016
.500				-2.941	-30.675		-2.451	-25.563	
.563				-2.941	-30.123	.020	-2.451	-25.103	.017
.625				-2.941	-28.485	.039	-2.451	-23.738	.032
.688				-2.941	-25.812	.057	-2.451	-21.510	.047
.750				-2.941	-22.190	.073	-2.451	-18.492	.061
.813				-3.198	-17.721	.088	-2.665	-14.768	.074
.875				-3.198	-12.379	.103	-2.665	-10.316	.086
.938				-3.198	-6.367	.112	-2.665	-5.306	.094
3.000				-3.198		.116	-2.665		.096
3.000				-3.198		.116	-2.665		.096
.500				-3.198	.504	.116	-2.665	.420	.096
4.000				-3.198	1.008	.116	-2.665	.840	.096

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                    SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     -1     :
                 RESULTAT                        Appendix X - System analysis, Long-term loads
0                PROJEKT IDENT        KONSTR.IDENT                   NR                 DATUM
                                        Elly Yman                                       20-04-28
0
0I N N E H   L L S F ™ R T E C K N I N G
0
 DATAKVITTO                                                         SIDA
0           ENHETER                                                    1
            MATERIALKONSTANTER                                         2
            ELEMENTTYPER                                               2
            SYSTEMBESKRIVNING                                          2
            STANDARDLASTFALL                                           2
            LASTKOMBINATIONER                                          3
0
 RESULTAT

0         I LASTKOMBINATIONS I  STABI-  I  SPŽN-   I  SNITT-  I   REAK-  I  DEFOR-  I
          I NAMN             I   LITET  I  NINGAR  I  KRAFTER I  TIONER  I MATIONER I
          I------------------I----------I----------I----------I----------I----------I

            FORM                                        4          6          7
            ASPH                                        4          6          7
            BARR                                        4          6          7
            AVSLAB                                      5          6          8
            BROTT-B                                     5          6          8

            KVASI                                       5          6          8
0
 SISTA SIDA I DENNA BERŽKNING                                          8
1
```

```
 WSP Sverige AB    S T R I P   STEP 2           Project: Bro 100-262-1
                  FRAME                        Composite steel- concrete bridge using corrugated web i
                  STATISTIC                    Appendix X - System analysis, Long-term loads

 PROJECT NO       SYSTEM NO      NO            DATE          COMP-DATE      START-TERM.TIME     VERSION
                    Elly Yman                                2020-04-28     11:50:53-11:50:54   PC-05-041001
0
     4 JOINTS                          I  CROSS SECTION EVAL      .00  I  MATRIX  ROWS BAND   DETERM     PIVOT    INCR
     3 ELEMENTS                        I  INPUT EVALUATION             I
     0 ARCHES                          I     GEOMETRY             .00  I   A22I      3    2  .10E+01  .10E+01     0
     2 REACTIONS                       I     LOADCASES            .00  I   SR        9    8  .72E-35  .45E-08   -32
                             LOADEL    I     PRESTRESSING         .00  I   KM1T      9    4  .22E+36  .65E+06    33
     4 STANDARD LOADCASES        12    I     RESULT REQUESTS      .06  I
     0 TEMPERATURE LOADC.         0    I  MATRIX PREPARATION           I  FILE   TRK   POS            FILE   TRK    POS
     0 SUPPORT SETTLEMENTS        0    I     GEOMETRY             .00  I
     0 PRESTR. LOADCASES          0    I     REDUCTION            .00  I    1     0     0              11     0      0
                                       I     LOADING CASES        .22  I    2     0     0              14     0      0
     0 TRACKS (MAX. 0 ELEMENTS)        I     INFLUENCE LINES      .00  I    3     0     0              19     0      0
     0 TRAFFIC GROUPS                  I     2ND ORDER/CREEP      .00  I    4     0     0              20     0      0
     0 TRAFFIC LOADCASES               I  CALCULATION PARTS            I    7     0     0              21     0      0
                                       I     RANDF. REDUCT.       .05  I    8     0     0              22     0      0
     0 CROSS SECTIONS                  I     INVERT       A22     .00  I    9     0     0              24     0      0
                                       I     SYSTEM MATRICES      .00  I   10     0     0              25     0      0
     6 COMBINATIONS                    I     INVERT        VR     .00  I
     6   ADDITIVE                      I     TRIANG         K     .00  I  ERROR WITHIN PROGRAM PART              0
     0   ENVELOPES                     I     DEFORM       URT     .00  I  NO OF CALLS TO SUBROUTINE DISC         0
     0   CREEP/SHRINKAGE               I     GROSSFORCES POO      .00  I
     0   STRESSES                      I     DEFORMATIONS         .00  I  RESULT PAGES  ECHOPRINT                3
     0   SECOND ORDER                  I     BASIC INFL.LINES     .00  I                STRESSES                 0
     0   STABILITY                     I     TRAFFIC INFL.        .00  I                GROSSFORCES              2
                                       I     INFLUENCE LINES      .00  I                REACTIONS                1
    23 RESULT POINTS                   I     SECOND ORDER         .00  I                DEFORMATIONS             2
    23 DEFORMATION POINTS              I     CREEP                .00  I                INFLUENCE LINES          0
     0 INFLUENCE LINES                 I     STABILITY            .00  I                STABILITY                0
 ----------------------------------I  TRAFFIC EVALUATION      .00  I------------------------------------------
                                       I  COMBINATION PART        .06  I  IBM PC
                                       I  RESULT PRINT PART            I  TOTAL ELAPSED TIME       .39 SECONDS
                                       I     STRESSES             .00  I
                                       I     FORCES / DEFORM      .00  I
                                       I     INFLUENCE LINES      .00
```

Appendix 4 – Strip Step, Calculation of short-term loads – bridge 100-262-1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
D A T A   I N P U T        STRIP STEP 2 (PC-05-041001)        DATE: 2020-04-28   TIME: 11:50:59       PAGE  1      :
                                                                                                                     :
   CARD       0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
    NO        1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
 -------------------------------------------------------------------------------------------------------------
      1*      9000 STRIP-STEP2                            Elly Yman
      2*      9100 Project: Bro 100-262-1
      3*      9200 Composite steel- concrete bridge using corrugated web in stainless steel
      4*      9300 Appendix X - System analysis, Short- term loads
      5*      2010 10000     200000              1
      6*      2020 1    BALK N
      7*      2022 0              -0.121              2.37      1717.546  3321
      8*      2022 0.5            -0.121              2.37      1717.546  3321
      9*      2020 2    BALK N
     10*      2022 0              -0.121              2.37      1717.546  3321
     11*      2022 10.5           -0.121              2.37      1717.546  3321
     12*      2022 10.5           -0.153              2.37      1924.185  3427
     13*      2022 11.3           -0.243              2.37      2529.693  3664
     14*      2022 25.5           -0.243              2.37      2529.693  3664
     15*      2022 39.7           -0.243              2.37      2529.693  3664
     16*      2022 40.5           -0.153              2.37      1924.185  3427
     17*      2022 40.5           -0.121              2.37      1717.546  3321
     18*      2022 51             -0.121              2.37      1717.546  3321
     19*      2020 3    BALK N
     20*      2022 0              -0.121              2.37      1717.546  3321
     21*      2022 0.5            -0.121              2.37      1717.546  3321
     22*      2040 1         0         0                                            1    2
     23*      2040 2         0.5       0                   -2.05     ZR
     24*      2040                                                                  2    3
     25*      2040 3         51.5      0                   -2.05     YZR
     26*      2040                                                                  3    4
     27*      2040 4         52        0                                            -3   3
     28*      2050 WIND      1    4                             UTBR           0         G/S
 BW:        6 JOINTS:   2
     29*      2050 BRLM      1    4                   0.673     UTBR 4.2                 UW
     30*      2050 BRTYP     1    4                   0.673     UTBR 3.55                UW
     31*      2050 D:TEMP1   1    2    0              -0.121    KONC 6476                YZ
     32*      2052 -7589
     33*      2050           3    4    0.5            -0.121    KONC -6476               YZ
     34*      2052 7589
     35*      2050 D:TEMP2   1    2    0              -0.121    KONC -5243               YZ
     36*      2052 6144
     37*      2050           3    4    0.5            -0.121    KONC 5243                YZ
     38*      2052 -6144
     39*      2080 FARB 1    4
     40*      2090 A-A       0.0       324
     41*      2090 B-B       0.0       237.6
     42*      2090           1.0
     43*      2090 B-C       0.0       270
     44*      2090           1.3
     45*      2090 B-D       0.0       297
     46*      2090           1.8
     47*      2090 B-E1      0.0       210.6
     48*      2090           1.0       210.6
     49*      2090           2.0
     50*      2090 B-E2      0.0       210.6
     51*      2090           1.0
     52*      2090 B-F1      0.0       237.6
     53*      2090           1.3       237.6
     54*      2090           2.6
     55*      2090 B-F2      0.0       237.6
     56*      2090           1.3
     57*      2090 B-G1V     0.0       0
     58*      2090           2.5       237.6
     59*      2090           3.8       237.6
     60*      2090           5.1       237.6
     61*      2090           7.5       237.6
     62*      2090           10.0                          7.5       0.00
     63*      2090           12.5
     64*      2090 B-G2V     0.0                           7.5       0.00
     65*      2090           2.5
     66*      2090           5.0       237.6
     67*      2090           6.3       237.6
     68*      2090           7.6       237.6
1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
D A T A   I N P U T        STRIP STEP 2 (PC-05-041001)        DATE: 2020-04-28   TIME: 11:50:59       PAGE  2

  CARD       0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
   NO        1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
-------------------------------------------------------------------------------------------------------------
   69*       2090           10.0      237.6
   70*       2090           12.5      0
   71*       2090 B-G1H     0.0       0
   72*       2090           2.5       237.6
   73*       2090           4.9       237.6
   74*       2090           6.2       237.6
   75*       2090           7.5       237.6
   76*       2090           10.0                          7.5       0.0
   77*       2090           12.5
   78*       2090 B-G2H     0.0                           7.5       0.0
   79*       2090           2.5
   80*       2090           5.0       237.6
   81*       2090           7.4       237.6
   82*       2090           8.7       237.6
   83*       2090           10.0      237.6
   84*       2090           12.5      0
   85*       2090 B-H1V     0.0       0
   86*       2090           2.5       237.6
   87*       2090           3.8       237.6
   88*       2090           5.1       237.6
   89*       2090           10.1      0
   90*       2090 B-H2V     0.0       270
   91*       2090           1.3       270
   92*       2090           4.3       297
   93*       2090           6.8       0
   94*       2090 B-H1H     0.0       237.6
   95*       2090           1.3       237.6
   96*       2090           2.6       237.6
   97*       2090           5.1       0
   98*       2090 B-H2H     0.0       0
   99*       2090           2.5       297
  100*       2090           5.5       270
  101*       2090           6.8       270
  102*       2090           11.8      0
  103*       2090 B-I1V     0.0       0
  104*       2090           2.5       180
  105*       2090           3.5       180
  106*       2090           7.1       297
  107*       2090           8.9       297
  108*       2090           12.9      0
  109*       2090 B-I2V     0.0       297
  110*       2090           1.8       297
  111*       2090           5.6       237.6
  112*       2090           8.1       0
  113*       2090 B-I2H     0.0       0
  114*       2090           2.5       237.6
  115*       2090           6.3       297
  116*       2090           8.1       297
  117*       2090           12.1      0
  118*       2090 B-I1H     0.0       297
  119*       2090           1.8       297
  120*       2090           5.4       180
  121*       2090           6.4       180
  122*       2090           8.9       0
  123*       2090 B-MH      0.000     178.2
  124*       2090           1.300     270
  125*       2090           3.100     270
  126*       2090           6.500     237.6
  127*       2090           8.300     237.6
  128*       2090           9.600     237.6
  129*       2090 B-MV      0.000     237.6
  130*       2090           1.300     237.6
  131*       2090           3.100     237.6
  132*       2090           6.500     270
  133*       2090           8.300     270
  134*       2090           9.600     178.2
  135*       2090 B-NH      0.000     297
  136*       2090           2.000     297
1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
D A T A   I N P U T       STRIP STEP 2 (PC-05-041001)       DATE: 2020-04-28    TIME: 11:50:59       PAGE  3

  CARD       0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
   NO        1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
-------------------------------------------------------------------------------------------------------------
  137*       2090           4.000     297
  138*       2090           5.500     178.2
  139*       2090           6.600     64.8
  140*       2090 B-NV      0.000     64.8
  141*       2090           1.100     178.2
  142*       2090           2.600     297
  143*       2090           4.600     297
  144*       2090           6.600     297
  145*       2090 Q                                       7.5       0.0
  146*       2090 LM1:UTB                                 29.4      0
  147*       2090 LM1       0         450
  148*       2090           1.2       450
  149*       2090 LM2       0         360
  150*       2090 UTM3      0         120
  151*       2090           1.2       120
  152*       2090           7.2       120
  153*       2090           8.4       120
  154*       2100 AXEL-A                        FARB           A-A
  155*       2100 BO-B                          FARB           B-B       FRI       B-B
  156*       2100 BO-C                          FARB           B-C       FRI       B-C
  157*       2100 BO-D                          FARB           B-D       FRI       B-D
  158*       2100 BO-E1                         FARB           B-E1      FRI       B-E2
  159*       2100 BO-E2                         FARB           B-E2      FRI       B-E1
  160*       2100 BO-F1                         FARB           B-F1      FRI       B-F2
  161*       2100 BO-F2                         FARB           B-F2      FRI       B-F1
  162*       2100 BO-G1                         FARB           Q         B-G1V     FRI
  163*       2100                                              B-G2V     Q
  164*       2100 BO-G2                         FARB           Q         B-G1H     FRI
  165*       2100                                              B-G2H     Q
  166*       2100 BO-H1                         FARB           Q         B-H1V     FRI
  167*       2100                                              B-H2V     Q
  168*       2100 BO-H2                         FARB           Q         B-H2H     FRI
  169*       2100                                              B-H1H     Q
  170*       2100 BO-I1                         FARB           Q         B-I1V     FRI
  171*       2100                                              B-I2V     Q
  172*       2100 BO-I2                         FARB           Q         B-I2H     FRI
  173*       2100                                              B-I1H     Q
  174*       2100 BO-M1                         FARB           B-MV
  175*       2100 BO-M2                         FARB           B-MH
  176*       2100 BO-N1                         FARB           B-NV
  177*       2100 BO-N2                         FARB           B-NH
  178*       2100 LM-1:U                        FARB           LM1:UTB
  179*       2100 LM-1                          FARB           LM1
  180*       2100 LM-2                          FARB           LM2
  181*       2100 UTM                           FARB           UTM3
  182*       2140 AXELLAST  M    AXEL-A    TILLF  1.2
  183*       2140           Q
  184*       2140           R
  185*       2140           DZ
  186*       2140 BOGGILAST M    BO-B      TILLF  1.2     ALTER
  187*       2140           Q    BO-C      TILLF  1.2     ALTER
  188*       2140           R    BO-D      TILLF  1.2     ALTER
  189*       2140           DZ   BO-E1     TILLF  1.2     ALTER
  190*       2140                BO-E2     TILLF  1.2     ALTER
  191*       2140                BO-F1     TILLF  1.2     ALTER
  192*       2140                BO-F2     TILLF  1.2     ALTER
  193*       2140                BO-G1     TILLF  1.2     ALTER
  194*       2140                BO-G2     TILLF  1.2     ALTER
  195*       2140                BO-H1     TILLF  1.2     ALTER
  196*       2140                BO-H2     TILLF  1.2     ALTER
  197*       2140                BO-I1     TILLF  1.2     ALTER
  198*       2140                BO-I2     TILLF  1.2     ALTER
  199*       2140                BO-M1     TILLF  1.2     ALTER
  200*       2140                BO-M2     TILLF  1.2     ALTER
  201*       2140                BO-N1     TILLF  1.2     ALTER
  202*       2140                BO-N2     TILLF  1.2     ALTER
  203*       2140 LM1       M    LM-1      TILLF  0.897
  204*       2140           Q    LM-1:U           1
1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
D A T A   I N P U T        STRIP STEP 2 (PC-05-041001)        DATE: 2020-04-28   TIME: 11:50:59      PAGE  4

  CARD         0    0    1    1    2    2    3    3    4    4    5    5    6    6    7    7
   NO          1    6    1    6    1    6    1    6    1    6    1    6    1    6    1    6
-----------------------------------------------------------------------------------------------------------------
  205*         2140           R
  206*         2140           DZ
  207*         2140 LM2       M    LM-2      TILLF  1
  208*         2140           Q              TILLF
  209*         2140           R
  210*         2140           DZ
  211*         2140 UTM3      M    UTM       TILLF  1.125
  212*         2140           Q
  213*         2140 WIND           WIND      STAND  1
  214*         2140 BRLM      M    BRLM      TILLF  1       ALTER
  215*         2140           Q    BRLM      TILLF  -1      ALTER
  216*         2140 BRTYP     M    BRTYP     TILLF  1       ALTER
  217*         2140           Q    BRTYP     TILLF  -1      ALTER
  218*         2140 TRAFIC    M    LM1       TILLF  1       ALTER
  219*         2140           Q    AXELLAST  TILLF  0.874   ALTER
  220*         2140           DZ   BOGGILAST TILLF  0.874   ALTER
  221*         2140 TEMP      M    D:TEMP1   TILLF  1       ALTER
  222*         2140           Q    D:TEMP2   TILLF  1       ALTER
  223*         2140 MULTI     M    TRAFIC    TILLF  1
  224*         2140           Q    BRLM      TILLF  0.6
  225*         2140 BROTT     M    TEMP      TILLF  0.9
  226*         2140           Q    MULTI     TILLF  1.5
  227*         2140 DEFL      DZ   LM1       TILLF  0.75    ALTER
  228*         2140           M    BOGGILAST TILLF  0.655   ALTER
  229*         2140                AXELLAST  TILLF  0.655   ALTER
  230*         2140 KVASI     M    TEMP      TILLF  0.5
  231*         2140           Q    MULTI     TILLF  0.75
  232*         2170 1    2    2              S    DEF
  233*         2170 2    3    16             S    DEF
  234*         2170 3    4    2              S    DEF
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                      SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i       1     :
                 DATAKVITTO                      Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT       KONSTR.IDENT                    NR                DATUM
                                       Elly Yman                                       20-04-28
0

0E N H E T E R :  SI-SYSTEMET

0           LANGDER                                         METER
0           VINKLAR                                         RADIAN
0           LASTER, KRAFTER                                 KILONEWTON
0           P KŽNNINGAR, ELASTICITETSMODULER                MEGAPASCAL
0           MASSOR                                          TON
0           TEMPERATURER                                    KELVIN
0           FREKVENSER                                      HERTZ
0
0FR N DESSA GRUNDENHETER SAMT KOMBINATIONER HŽRAV G™RES
 F™LJANDE UNDANTAG:
0
            VINKEL I GEOMETRIBESKRIVNING OCH
            ROTATIONER I DEFORMATIONSRESULTAT               GRADER (360/VARV)
0           F™RSKJUTNINGAR I DEFORMATIONER                  MILLIMETER
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                           SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   2     :
                 DATAKVITTO                      Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                   NR                DATUM
                                        Elly Yman                                      20-04-28
0
 M A T E R I A L K O N S T A N T E R

 I    K    I    E0   I  G0/E0  I DENSI-  I OMEGA T I EPSILON I FI-KRYP I
 I         I         I         I   TET   I *100000 I *100000 I         I
 I---------I---------I---------I---------I---------I---------I---------I

      10000    200000    .500                 1.000
0
 E L E M E N T T Y P E R

 I  NR  I ARBETS I    E    I  DEN-  I KRYP-  I    R    I TVAR-  I    F    I    D    I    H    I   K*I    I   K*A    I   K*AQ   I
 I      I BIDRAG I    G    I SITET  I  IND   I         I SNITT  I         I         I         I          I          I          I
 I------I--------I--------I--------I--------I--------I--------I--------I--------I--------I---------I---------I---------I

      1    N         200000                          .000          -.121                 2.370   1717.546  3321.000
           BALK      100000                         1.000          -.121                 2.370   1717.546  3321.000

      2    N         200000                          .000          -.121                 2.370   1717.546  3321.000
           BALK      100000                          .206          -.121                 2.370   1717.546  3321.000
                                                     .206          -.153                 2.370   1924.185  3427.000
                                                     .222          -.243                 2.370   2529.693  3664.000
                                                     .500          -.243                 2.370   2529.693  3664.000
                                                     .778          -.243                 2.370   2529.693  3664.000
                                                     .794          -.153                 2.370   1924.185  3427.000
                                                     .794          -.121                 2.370   1717.546  3321.000
                                                    1.000          -.121                 2.370   1717.546  3321.000

      3    N         200000                          .000          -.121                 2.370   1717.546  3321.000
           BALK      100000                         1.000          -.121                 2.370   1717.546  3321.000
0
 S Y S T E M B E S K R I V N I N G

 I KNUTPT I     Y     I     Z     I DELTA Y I DELTA Z I LED-TYP I   ALFA   I ELEMENT I KNUTPT I
 I  START I           I           I         I         I  REAKT. I          I   TYP   I  SLUT  I
 I--------I---------I---------I---------I---------I---------I---------I---------I--------I

      1        .000      .000                                                     1        2

      2        .500      .000              -2.050    ZR
                                                                                  2        3

      3      51.500      .000              -2.050    YZR
                                                                                  3        4

      4      52.000      .000                                                    -3        3
0
 S T A N D A R D L A S T F A L L

 I LASTFALL I  KNUTPUNKT  I    R    I    W    I LAST-  I    P-Y   I    P-Z   I    M     I
 I NAMN     I  START-SLUT I         I         I TYP    I          I          I          I
 I----------I-------------I-------I-------I-------I---------I---------I---------I

  WIND                1-2     .000            UTBR
                             1.000            UTBR
                      2-3     .000            UTBR
                             1.000            UTBR
                      3-4     .000            UTBR
                             1.000            UTBR

1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   3      :
                 DATAKVITTO                      Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                NR                 DATUM
                                        Elly Yman                                    20-04-28
0
 S T A N D A R D L A S T F A L L   (FORTS.)

 I LASTFALL I  KNUTPUNKT  I    R    I    W    I LAST- I    P-Y    I    P-Z    I     M     I
 I NAMN     I  START-SLUT I         I         I TYP   I           I           I           I
 I----------I-------------I-------I-------I-------I---------I---------I---------I

  BRLM                1-2    .000    .673   UTBR       4.20
                            1.000    .673   UTBR       4.20
                      2-3    .000    .673   UTBR       4.20
                            1.000    .673   UTBR       4.20
                      3-4    .000    .673   UTBR       4.20
                            1.000    .673   UTBR       4.20

  BRTYP               1-2    .000    .673   UTBR       3.55
                            1.000    .673   UTBR       3.55
                      2-3    .000    .673   UTBR       3.55
                            1.000    .673   UTBR       3.55
                      3-4    .000    .673   UTBR       3.55
                            1.000    .673   UTBR       3.55

  D:TEMP1             1-2    .000   -.121   KONC    6476.00              -7589.00
                      3-4   1.000   -.121   KONC   -6476.00               7589.00

  D:TEMP2             1-2    .000   -.121   KONC   -5243.00               6144.00
                      3-4   1.000   -.121   KONC    5243.00              -6144.00
0
 F A R B A N O R

 I NAMN I  KNUTPUNKT  I    H-W   I   FARBANEBREDD    I
 I      I  START-SLUT I          I  START  I   SLUT  I
 I------I-------------I---------I---------I---------I

  FARB            1-2
                  2-3
                  3-4
0
 T R A F I K L A S T G R U P P E R

 I  LAST-   I     S    I     PUNKTLAST     I    UTBREDD LAST   I    IND   I
 I  GRUPP   I          I  P-BEL  I  P-OBEL I  P-BEL  I  P-OBEL I          I
 I----------I---------I---------I---------I---------I---------I---------I

  A-A             .000    324.00    324.00

  B-B             .000    237.60    237.60
                 1.000

  B-C             .000    270.00    270.00
                 1.300

  B-D             .000    297.00    297.00
                 1.800

  B-E1            .000    210.60    210.60
                 1.000    210.60    210.60
                 2.000

  B-E2            .000    210.60    210.60
                 1.000

  B-F1            .000    237.60    237.60
                 1.300    237.60    237.60
                 2.600

1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

WSP Sverige AB S T R I P STEP 2 PC-05-041001 Project: Bro 100-262-1 SIDA :
KONSTRUKTIONSTYP RAM Composite steel- concrete bridge using corrugated web i 4 :
DATAKVITTO Appendix X - System analysis, Short- term loads
0 PROJEKT IDENT KONSTR.IDENT NR DATUM
Elly Yman 20-04-28
0
T R A F I K L A S T G R U P P E R (FORTS.)

LAST-GRUPP	S	PUNKTLAST P-BEL	PUNKTLAST P-OBEL	UTBREDD LAST P-BEL	UTBREDD LAST P-OBEL	IND
B-F2	.000	237.60	237.60			
	1.300					
B-G1V	.000	.00	.00			
	2.500	237.60	237.60			
	3.800	237.60	237.60			
	5.100	237.60	237.60			
	7.500	237.60	237.60			
	10.000			7.50	.00	
	12.500					
B-G2V	.000			7.50	.00	
	2.500					
	5.000	237.60	237.60			
	6.300	237.60	237.60			
	7.600	237.60	237.60			
	10.000	237.60	237.60			
	12.500	.00	.00			
B-G1H	.000	.00	.00			
	2.500	237.60	237.60			
	4.900	237.60	237.60			
	6.200	237.60	237.60			
	7.500	237.60	237.60			
	10.000			7.50	.00	
	12.500					
B-G2H	.000			7.50	.00	
	2.500					
	5.000	237.60	237.60			
	7.400	237.60	237.60			
	8.700	237.60	237.60			
	10.000	237.60	237.60			
	12.500	.00	.00			
B-H1V	.000	.00	.00			
	2.500	237.60	237.60			
	3.800	237.60	237.60			
	5.100	237.60	237.60			
	10.100	.00	.00			
B-H2V	.000	270.00	270.00			
	1.300	270.00	270.00			
	4.300	297.00	297.00			
	6.800	.00	.00			
B-H1H	.000	237.60	237.60			
	1.300	237.60	237.60			
	2.600	237.60	237.60			
	5.100	.00	.00			
B-H2H	.000	.00	.00			
	2.500	297.00	297.00			
	5.500	270.00	270.00			
	6.800	270.00	270.00			
	11.800	.00	.00			

1

```
 WSP Sverige AB  S T R I P  STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      5      :
                 DATAKVITTO                      Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT         KONSTR.IDENT                    NR                DATUM
                                         Elly Yman                                       20-04-28
0
 T R A F I K L A S T G R U P P E R   (FORTS.)
```

LAST-GRUPP	S	PUNKTLAST P-BEL	PUNKTLAST P-OBEL	UTBREDD LAST P-BEL	UTBREDD LAST P-OBEL	IND
B-I1V	.000	.00	.00			
	2.500	180.00	180.00			
	3.500	180.00	180.00			
	7.100	297.00	297.00			
	8.900	297.00	297.00			
	12.900	.00	.00			
B-I2V	.000	297.00	297.00			
	1.800	297.00	297.00			
	5.600	237.60	237.60			
	8.100	.00	.00			
B-I2H	.000	.00	.00			
	2.500	237.60	237.60			
	6.300	297.00	297.00			
	8.100	297.00	297.00			
	12.100	.00	.00			
B-I1H	.000	297.00	297.00			
	1.800	297.00	297.00			
	5.400	180.00	180.00			
	6.400	180.00	180.00			
	8.900	.00	.00			
B-MH	.000	178.20	178.20			
	1.300	270.00	270.00			
	3.100	270.00	270.00			
	6.500	237.60	237.60			
	8.300	237.60	237.60			
	9.600	237.60	237.60			
B-MV	.000	237.60	237.60			
	1.300	237.60	237.60			
	3.100	237.60	237.60			
	6.500	270.00	270.00			
	8.300	270.00	270.00			
	9.600	178.20	178.20			
B-NH	.000	297.00	297.00			
	2.000	297.00	297.00			
	4.000	297.00	297.00			
	5.500	178.20	178.20			
	6.600	64.80	64.80			
B-NV	.000	64.80	64.80			
	1.100	178.20	178.20			
	2.600	297.00	297.00			
	4.600	297.00	297.00			
	6.600	297.00	297.00			
Q				7.50	.00	
LM1:UTB				29.40	.00	
LM1	.000	450.00	450.00			
	1.200	450.00	450.00			
LM2	.000	360.00	360.00			

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                   SIDA    :
                 KONSTRUKTIONSTYP RAM                Composite steel- concrete bridge using corrugated web i      6     :
                 DATAKVITTO                          Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                  NR                 DATUM
                                        Elly Yman                                      20-04-28
0
 T R A F I K L A S T G R U P P E R   (FORTS.)

 I  LAST-    I     S    I      PUNKTLAST      I     UTBREDD LAST    I    IND   I
 I  GRUPP    I          I  P-BEL  I  P-OBEL I  P-BEL  I  P-OBEL I          I
 I----------I---------I---------I---------I---------I---------I---------I---------I

  UTM3           .000    120.00    120.00
                1.200    120.00    120.00
                7.200    120.00    120.00
                8.400    120.00    120.00
0
 T R A F I K L A S T F A L L

 I LASTFALL I     -BROMSKRAFT-      I FARBANE I DYNAMISKT I     L A S T G R U P P E R      I
 I NAMN     I DEL AV VERT I EXCENTR I NAMN    I TILLSKOTT I   NAMN    I   NAMN    I   NAMN    I
 I----------I-------------I---------I---------I-----------I----------I----------I----------I

  AXEL-A                            FARB                  A-A

  BO-B                              FARB                  B-B          FRI       B-B

  BO-C                              FARB                  B-C          FRI       B-C

  BO-D                              FARB                  B-D          FRI       B-D

  BO-E1                             FARB                  B-E1         FRI       B-E2

  BO-E2                             FARB                  B-E2         FRI       B-E1

  BO-F1                             FARB                  B-F1         FRI       B-F2

  BO-F2                             FARB                  B-F2         FRI       B-F1

  BO-G1                             FARB                  Q          B-G1V        FRI
                                                          B-G2V      Q

  BO-G2                             FARB                  Q          B-G1H        FRI
                                                          B-G2H      Q

  BO-H1                             FARB                  Q          B-H1V        FRI
                                                          B-H2V      Q

  BO-H2                             FARB                  Q          B-H2H        FRI
                                                          B-H1H      Q

  BO-I1                             FARB                  Q          B-I1V        FRI
                                                          B-I2V      Q

  BO-I2                             FARB                  Q          B-I2H        FRI
                                                          B-I1H      Q

  BO-M1                             FARB                  B-MV

  BO-M2                             FARB                  B-MH

  BO-N1                             FARB                  B-NV

  BO-N2                             FARB                  B-NH

  LM-1:U                            FARB                  LM1:UTB

  LM-1                              FARB                  LM1

  LM-2                              FARB                  LM2

1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                    SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      7     :
                 DATAKVITTO                      Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT       KONSTR.IDENT                    NR               DATUM
                                       Elly Yman                                      20-04-28
0
 T R A F I K L A S T F A L L   (FORTS.)

 I LASTFALL I     -BROMSKRAFT-      I FARBANE I DYNAMISKT I     L A S T G R U P P E R      I
 I NAMN     I DEL AV VERT I EXCENTR I NAMN    I TILLSKOTT I   NAMN   I   NAMN   I   NAMN   I
 I----------I-------------I---------I---------I-----------I----------I----------I----------I

  UTM                                FARB                 UTM3
0
 L A S T K O M B I N A T I O N E R

 I LASTKOMBINATION I          - L A S T F A L L -        I SPANN. I K-KRYP I
 I NAMN     I ART  I NAMN     I  TYP    I FAKTOR I ALTER  I  IND   I        I
 I----------I------I----------I--------I--------I--------I--------I--------I

  AXELLAST   MAXM   AXEL-A     TILLF       1.200
             MAXQ
             MAXRM
             MAXRY
             MAXRZ
             MAXDZ

  BOGGILAST  MAXM   BO-B       TILLF       1.200   ALTER
             MAXQ   BO-C       TILLF       1.200   ALTER
             MAXRM  BO-D       TILLF       1.200   ALTER
             MAXRY
             MAXRZ
             MAXDZ  BO-E1      TILLF       1.200   ALTER
                    BO-E2      TILLF       1.200   ALTER
                    BO-F1      TILLF       1.200   ALTER
                    BO-F2      TILLF       1.200   ALTER
                    BO-G1      TILLF       1.200   ALTER
                    BO-G2      TILLF       1.200   ALTER
                    BO-H1      TILLF       1.200   ALTER
                    BO-H2      TILLF       1.200   ALTER
                    BO-I1      TILLF       1.200   ALTER
                    BO-I2      TILLF       1.200   ALTER
                    BO-M1      TILLF       1.200   ALTER
                    BO-M2      TILLF       1.200   ALTER
                    BO-N1      TILLF       1.200   ALTER
                    BO-N2      TILLF       1.200   ALTER

  LM1        MAXM   LM-1       TILLF        .897
             MAXQ   LM-1:U     TILLF       1.000
             MAXRM
             MAXRY
             MAXRZ
             MAXDZ

  LM2        MAXM   LM-2       TILLF       1.000
             MAXQ              TILLF
             MAXRM
             MAXRY
             MAXRZ
             MAXDZ

  UTM3       MAXM   UTM        TILLF       1.125
             MAXQ

  WIND              WIND       STAND       1.000

  BRLM       MAXM   BRLM       TILLF       1.000   ALTER
             MAXQ   BRLM       TILLF      -1.000   ALTER

  BRTYP      MAXM   BRTYP      TILLF       1.000   ALTER
             MAXQ   BRTYP      TILLF      -1.000   ALTER

1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                   SIDA      :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      8      :
                 DATAKVITTO                      Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                DATUM
                                        Elly Yman                                       20-04-28
0
 L A S T K O M B I N A T I O N E R   (FORTS.)

 I LASTKOMBINATION I          - L A S T F A L L -        I SPANN. I K-KRYP I
 I NAMN      I ART  I NAMN      I  TYP   I FAKTOR I ALTER  I  IND   I        I
 I----------I------I----------I--------I--------I--------I--------I--------I

  TRAFIC      MAXM   LM1        TILLF       1.000   ALTER
              MAXQ   AXELLAST   TILLF        .874   ALTER
              MAXDZ  BOGGILAST  TILLF        .874   ALTER

  TEMP        MAXM   D:TEMP1    TILLF       1.000   ALTER
              MAXQ   D:TEMP2    TILLF       1.000   ALTER

  MULTI       MAXM   TRAFIC     TILLF       1.000
              MAXQ   BRLM       TILLF        .600

  BROTT       MAXM   TEMP       TILLF        .900
              MAXQ   MULTI      TILLF       1.500

  DEFL        MAXDZ  LM1        TILLF        .750   ALTER
              MAXM   BOGGILAST  TILLF        .655   ALTER
                     AXELLAST   TILLF        .655   ALTER

  KVASI       MAXM   TEMP       TILLF        .500
              MAXQ   MULTI      TILLF        .750
1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                      SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i    9      :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                DATUM
                                        Elly Yman                                         20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	AXELLAST MAX-M M	AXELLAST MAX-M N	AXELLAST MAX-M Q	AXELLAST MIN-M M	AXELLAST MIN-M N	AXELLAST MIN-M Q	AXELLAST MAX-Q M	AXELLAST MAX-Q N	AXELLAST MAX-Q Q
1.000	.01		.12				.01		.12
.500	.03		.12	-97.20		-388.81	.03		.12
2.000	.06		.12	-194.40		-388.81	.06		.12
2.000	.09		233.28	-194.41		3.81			388.80
.063	1161.87		364.50	-182.25		3.81	1161.87		364.50
.125	2168.82		-48.60	-170.10		3.81	2168.82		340.20
.188	3020.85		-72.90	-157.95		3.81	3020.85		315.90
.250	3717.96		291.60	-145.80		3.81	3717.96		291.60
.313	4260.15		267.30	-133.65		3.81	4260.15		267.30
.375	4647.43		-145.80	-121.50		3.81	4647.43		243.00
.438	4879.79		-170.10	-109.35		3.81	4879.79		218.70
.500	4957.24		-194.40	-97.20		3.81	4957.24		194.40
.563	4879.77		-218.70	-109.35		-3.81	4879.77		170.10
.625	4647.40		-243.00	-121.50		-3.81	4647.40		145.80
.688	4260.11		-267.30	-133.65		-3.81	4260.11		121.50
.750	3717.91		-291.60	-145.80		-3.81	3717.91		97.20
.813	3020.80		-315.90	-157.95		-3.81	3020.79		72.90
.875	2168.77		-340.20	-170.10		-3.81	2168.77		48.60
.938	1161.84		-364.50	-182.25		-3.81	1161.84		24.30
3.000				-194.40		-3.81			3.81
3.000				-194.40		388.80			388.80
.500				-97.20		388.80			388.80
4.000								-.03	.08

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

WSP Sverige AB S T R I P STEP 2 PC-05-041001 Project: Bro 100-262-1 SIDA :
KONSTRUKTIONSTYP RAM Composite steel- concrete bridge using corrugated web i 10 :
RESULTAT Appendix X - System analysis, Short- term loads
0 PROJEKT IDENT KONSTR.IDENT NR DATUM
Elly Yman 20-04-28
0
S N I T T K R A F T E R

PUNKT	AXELLAST MIN-Q			BOGGILAST MAX-M			BOGGILAST MIN-M		
	M	N	Q	M	N	Q	M	N	Q
1.000			-388.81	.03		.65			-.01
.500	-97.20		-388.81	.20		.65	-89.10		-356.41
2.000	-194.40		-388.81	.36		.65	-178.20		-356.41
2.000			-3.81	.51		1406.46	-178.20		3.49
.063	1161.87		-24.30	5703.58		1513.98	-167.07		3.49
.125	2168.82		-48.60	10482.72		1384.73	-155.93		3.49
.188	3020.85		-72.90	14485.96		970.36	-144.79		3.49
.250	3717.96		-97.20	17810.14		1180.58	-133.65		3.49
.313	4260.15		-121.50	20414.03		585.95	-122.52		3.49
.375	4647.43		-145.80	22246.50		254.60	-118.06		.70
.438	4879.79		-170.10	23338.61		16.37	-115.83		.70
.500	4957.24		-194.40	23623.45		-135.90	-113.60		.70
.563	4879.77		-218.70	23354.41		-360.67	-118.76		-1.62
.625	4647.40		-243.00	22234.33		-512.94	-123.91		-1.62
.688	4260.11		-267.30	20406.39		-589.08	-129.06		-1.62
.750	3717.91		-291.60	17822.45		-827.30	-134.21		-1.62
.813	3020.80		-315.90	14450.45		-982.93	-139.36		-1.62
.875	2168.77		-340.20	10484.25		-1397.30	-150.08		-3.36
.938	1161.84		-364.50	5712.55		-1510.39	-160.80		-3.36
3.000			-388.80			3.49	-171.52		-3.36
3.000							-171.52		356.40
.500							-82.42		356.40
4.000								-.16	.43

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                               SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   11     :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                     NR                DATUM
                                        Elly Yman                                        20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	BOGGILAST MAX-Q M	BOGGILAST MAX-Q N	BOGGILAST MAX-Q Q	BOGGILAST MIN-Q M	BOGGILAST MIN-Q N	BOGGILAST MIN-Q Q	LM1 MAX-M M	LM1 MAX-M N	LM1 MAX-M Q
1.000	.03		.65			-.01	.03		.52
.500	.20		.65	-89.10		-356.41	.16		.53
2.000	.36		.65	-178.20		-356.41	.29		.53
2.000	.36		1936.05			-3.36	.41		1222.84
.063	5654.84		1774.28	1512.23		-33.63	4609.52		1399.17
.125	10290.64		1614.32	4103.86		-91.96	8616.38		855.95
.188	13924.16		1456.19	7369.60		-177.85	12006.17		1114.94
.250	16578.39		1300.32	10275.55		-268.64	14761.07		970.27
.313	18627.34		1168.81	13219.27		-377.02	16892.45		825.61
.375	19426.52	.01	1015.80	15323.37		-480.77	18406.91		282.39
.438	19409.74		869.94	17257.47		-601.61	19320.09		137.73
.500	18617.64		730.13	18860.17		-739.66	19609.75		8.35
.563	17277.71		602.30	19626.79		-879.68	19324.60		-136.31
.625	15345.49		481.45	19535.72		-1021.54	18415.93		-280.98
.688	12982.98		370.28	18572.37		-1165.40	16883.75		-425.64
.750	10106.78		264.23	16831.38	.01	-1320.20	14742.55		-968.86
.813	7390.19		178.34	14121.01		-1476.83	11992.18		-1113.52
.875	4126.09		92.46	10322.24		-1619.36	8618.30		-1258.18
.938	1492.47		33.25	5670.87		-1779.46	4620.90		-1402.84
3.000			3.49	-.03		-1941.37			
3.000	-49.14	-.10	356.67						
.500	-74.69	-.10	356.67						
4.000		-.17	.45						

1

WSP Sverige AB S T R I P STEP 2 PC-05-041001 Project: Bro 100-262-1 SIDA :
KONSTRUKTIONSTYP RAM Composite steel- concrete bridge using corrugated web i 12 :
RESULTAT Appendix X - System analysis, Short- term loads
0 PROJEKT IDENT KONSTR.IDENT NR DATUM
Elly Yman 20-04-28
0
S N I T T K R A F T E R

PUNKT	LM1 MIN-M M	LM1 MIN-M N	LM1 MIN-M Q	LM1 MAX-Q M	LM1 MAX-Q N	LM1 MAX-Q Q	LM1 MIN-Q M	LM1 MIN-Q N	LM1 MIN-Q Q
1.000				.03		.53			-.01
.500	-101.83		-411.01	.16		.53	-101.83		-411.01
2.000	-205.50		-418.36	.29		.53	-205.50		-418.36
2.000	-205.51		4.03	-3.43		1543.91			-3.31
.063	-192.89		3.96	4466.05		1402.17	1987.01		-41.64
.125	-180.28		3.96	8069.28		1266.29	4520.09		-101.37
.188	-167.66		3.96	10911.00		1141.37	6915.04		-166.97
.250	-155.05		3.96	12966.03		1017.20	9115.84		-238.42
.313	-142.43		3.96	14322.85		898.90	11066.50		-315.73
.375	-129.82		3.96	15037.47		786.45	12548.60		-393.80
.438	-117.20		3.96	15165.91		679.86	13847.19		-482.82
.500	-104.59		3.96	14764.17		579.12	14727.62		-577.70
.563	-96.63		-3.24	13888.28		484.24	15133.87		-678.43
.625	-106.95		-3.24	12594.23		395.22	15009.95		-785.03
.688	-117.28		-3.24	10938.03		312.06	14299.84		-897.48
.750	-127.61		-3.24	8975.69		234.75	12947.54		-1015.78
.813	-137.94		-3.24	6763.22		163.30	10897.04		-1139.95
.875	-148.26		-3.24	4584.01		102.80	8092.34		-1269.97
.938	-158.59		-3.24	2055.55		43.07	4477.43		-1405.85
3.000	-168.94		-736.15			4.03	-3.70		-1547.58
3.000	-169.02	-.07	418.55	-39.10	-.08	418.55			
.500	-65.30	-.07	411.20	-65.30	-.07	411.20			
4.000					-.14	.37			

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                        SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   13  :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT         KONSTR.IDENT                   NR               DATUM
                                         Elly Yman                                     20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	LM2 MAX-M M	LM2 MAX-M N	LM2 MAX-M Q	LM2 MIN-M M	LM2 MIN-M N	LM2 MIN-M Q	LM2 MAX-Q M	LM2 MAX-Q N	LM2 MAX-Q Q
1.000	.01		.11				.01		.11
.500	.03		.11	-90.00		-360.01	.03		.11
2.000	.06		.11	-180.00		-360.01	.06		.11
2.000	.08		216.00	-180.00		3.53			360.00
.063	1075.81		337.50	-168.75		3.53	1075.81		337.50
.125	2008.17		-45.00	-157.50		3.53	2008.17		315.00
.188	2797.08		-67.50	-146.25		3.53	2797.08		292.50
.250	3442.56		270.00	-135.00		3.53	3442.56		270.00
.313	3944.58		247.50	-123.75		3.53	3944.58		247.50
.375	4303.17		-135.00	-112.50		3.53	4303.17		225.00
.438	4518.33		-157.50	-101.25		3.53	4518.32		202.50
.500	4590.04		-180.00	-90.00		3.53	4590.04		180.00
.563	4518.31		-202.50	-101.25		-3.53	4518.31		157.50
.625	4303.15		-225.00	-112.50		-3.53	4303.15		135.00
.688	3944.55		-247.50	-123.75		-3.53	3944.54		112.50
.750	3442.51		-270.00	-135.00		-3.53	3442.51		90.00
.813	2797.03		-292.50	-146.25		-3.53	2797.03		67.50
.875	2008.12		-315.00	-157.50		-3.53	2008.12		45.00
.938	1075.78		-337.50	-168.75		-3.53	1075.78		22.50
3.000				-180.00		-3.53			3.53
3.000				-180.00		360.00			360.00
.500				-90.00		360.00			360.00
4.000								-.03	.07

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                      SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   14    :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT         KONSTR.IDENT                    NR                DATUM
                                         Elly Yman                                       20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	LM2 MIN-Q M	LM2 MIN-Q N	LM2 MIN-Q Q	UTM3 MAX-M M	UTM3 MAX-M N	UTM3 MAX-M Q	UTM3 MIN-M M	UTM3 MIN-M N	UTM3 MIN-M Q
1.000			-360.01	.01		.16			
.500	-90.00		-360.01	.05		.16	-33.75		-135.00
2.000	-180.00		-360.01	.09		.16	-67.50		-135.00
2.000			-3.53	.12		317.12	-67.50		1.32
.063	1075.81		-22.50	1467.14		460.26	-63.28		1.32
.125	2008.17		-45.00	2718.86		294.58	-59.06		1.32
.188	2797.08		-67.50	3766.48		260.50	-54.85		1.32
.250	3442.55		-90.00	4596.84		226.42	-50.63		1.32
.313	3944.58		-112.50	5242.51		202.56	-46.41		1.32
.375	4303.17		-135.00	5677.88		36.89	-42.19		1.32
.438	4518.33		-157.50	5904.10		2.81	-37.97		1.32
.500	4590.04		-180.00	5913.05		26.67	-33.75		1.32
.563	4518.31		-202.50	5900.28		-4.01	-29.53		1.32
.625	4303.15		-225.00	5678.47		-169.68	-25.31		1.32
.688	3944.55		-247.50	5246.25		-203.76	-26.97		-.77
.750	3442.51		-270.00	4596.77		-227.62	-29.43		-.77
.813	2797.03		-292.50	3762.60		-261.70	-31.88		-.77
.875	2008.12		-315.00	2724.47		-427.37	-34.33		-.77
.938	1075.78		-337.50	1470.86		-461.45	-36.78		-.77
3.000			-360.00				-39.23		-.77
3.000							-39.23		135.00
.500							-5.48		135.00
4.000									

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                        SIDA      :
                 KONSTRUKTIONSTYP RAM           Composite steel- concrete bridge using corrugated web i   15      :
                 RESULTAT                       Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                 NR                DATUM
                                        Elly Yman                                    20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	UTM3 MAX-Q M	UTM3 MAX-Q N	UTM3 MAX-Q Q	UTM3 MIN-Q M	UTM3 MIN-Q N	UTM3 MIN-Q Q	WIND M	WIND N	WIND Q
1.000	.01		.16						
.500	.05		.16	-33.75		-135.00			
2.000	.09		.16	-67.50		-135.00			
2.000	.05		494.34			-.77			
.063	1467.14		460.26	617.80		-12.92			
.125	2716.96		426.17	1337.06		-29.96			
.188	3749.50		392.09	2315.82		-55.89			
.250	4564.77		358.01	3441.30		-89.97			
.313	5162.78		323.93	4349.49		-124.05			
.375	5543.51		289.85	4931.79		-154.72			
.438	5783.02		259.18	5416.31		-188.80			
.500	5740.08		225.10	5683.56		-222.88			
.563	5479.88		191.02	5733.54		-256.97			
.625	5002.41		156.94	5566.25		-291.05			
.688	4307.68		122.86	5181.71		-325.13			
.750	3395.68		88.77	4579.89		-359.21			
.813	2266.42		54.69	3760.81		-393.29			
.875	1310.45		29.37	2724.47		-427.37			
.938	589.31		12.33	1470.86		-461.45			
3.000			1.32			-495.53			
3.000	-11.83	-.01	135.03						
.500	-5.48		135.00						
4.000		-.04	.11						

1

WSP Sverige AB S T R I P STEP 2 PC-05-041001 Project: Bro 100-262-1 SIDA :
KONSTRUKTIONSTYP RAM Composite steel- concrete bridge using corrugated web i 16 :
RESULTAT Appendix X - System analysis, Short- term loads
0 PROJEKT IDENT KONSTR.IDENT NR DATUM
Elly Yman 20-04-28
0
S N I T T K R A F T E R

PUNKT	BRLM MAX-M M	BRLM MAX-M N	BRLM MAX-M Q	BRLM MIN-M M	BRLM MIN-M N	BRLM MIN-M Q	BRLM MAX-Q M	BRLM MAX-Q N	BRLM MAX-Q Q
1.000			-.01			.01			.01
.500	.83	-1.05	-.01	-.83	1.05	.01	-.83	1.05	.01
2.000	1.66	-2.10	-.01	-1.66	2.10	.01	-1.66	2.10	.01
2.000	1.66	-2.10	-11.66	-1.66	2.10	11.66	-1.66	2.10	11.66
.063	24.88	15.49	11.66	-24.88	-15.49	-11.66	24.88	15.49	11.66
.125	51.42	28.88	11.66	-51.42	-28.88	-11.66	51.42	28.88	11.66
.188	77.95	42.26	11.66	-77.95	-42.26	-11.66	77.95	42.26	11.66
.250	97.70	55.65	11.66	-97.70	-55.65	-11.66	97.70	55.65	11.66
.313	122.61	69.04	11.66	-122.61	-69.04	-11.66	122.61	69.04	11.66
.375	147.52	82.43	11.66	-147.52	-82.43	-11.66	147.52	82.43	11.66
.438	172.42	95.81	11.66	-172.42	-95.81	-11.66	172.42	95.81	11.66
.500	197.33	109.20	11.66	-197.33	-109.20	-11.66	197.33	109.20	11.66
.563	222.23	122.59	11.66	-222.23	-122.59	-11.66	222.23	122.59	11.66
.625	247.14	135.98	11.66	-247.14	-135.98	-11.66	247.14	135.98	11.66
.688	272.04	149.36	11.66	-272.04	-149.36	-11.66	272.04	149.36	11.66
.750	296.95	162.75	11.66	-296.95	-162.75	-11.66	296.95	162.75	11.66
.813	343.34	176.14	11.66	-343.34	-176.14	-11.66	343.34	176.14	11.66
.875	369.88	189.53	11.66	-369.88	-189.53	-11.66	369.88	189.53	11.66
.938	396.42	202.91	11.66	-396.42	-202.91	-11.66	396.42	202.91	11.66
3.000	422.96	216.30	11.66	-422.96	-216.30	-11.66	422.96	216.30	11.66
3.000	1.66	-2.10	.01	-1.66	2.10	-.01	1.66	-2.10	.01
.500	.83	-1.05	.01	-.83	1.05	-.01	.83	-1.05	.01
4.000			.01			-.01			.01

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                        SIDA     :
                  KONSTRUKTIONSTYP RAM              Composite steel- concrete bridge using corrugated web i    17     :
                  RESULTAT                          Appendix X - System analysis, Short- term loads
0                 PROJEKT IDENT         KONSTR.IDENT                  NR                DATUM
                                          Elly Yman                                     20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	BRLM MIN-Q M	BRLM MIN-Q N	BRLM MIN-Q Q	BRTYP MAX-M M	BRTYP MAX-M N	BRTYP MAX-M Q	BRTYP MIN-M M	BRTYP MIN-M N	BRTYP MIN-M Q
1.000			-.01			-.01			.01
.500	.83	-1.05	-.01	.70	-.89	-.01	-.70	.89	.01
2.000	1.66	-2.10	-.01	1.41	-1.77	-.01	-1.41	1.77	.01
2.000	1.66	-2.10	-11.66	1.41	-1.78	-9.86	-1.41	1.78	9.86
.063	-24.88	-15.49	-11.66	21.03	13.09	9.86	-21.03	-13.09	-9.86
.125	-51.42	-28.88	-11.66	43.46	24.41	9.86	-43.46	-24.41	-9.86
.188	-77.95	-42.26	-11.66	65.89	35.72	9.86	-65.89	-35.72	-9.86
.250	-97.70	-55.65	-11.66	82.58	47.04	9.86	-82.58	-47.04	-9.86
.313	-122.61	-69.04	-11.66	103.63	58.35	9.86	-103.63	-58.35	-9.86
.375	-147.52	-82.43	-11.66	124.69	69.67	9.86	-124.69	-69.67	-9.86
.438	-172.42	-95.81	-11.66	145.74	80.98	9.86	-145.74	-80.98	-9.86
.500	-197.33	-109.20	-11.66	166.79	92.30	9.86	-166.79	-92.30	-9.86
.563	-222.23	-122.59	-11.66	187.84	103.62	9.86	-187.84	-103.62	-9.86
.625	-247.14	-135.98	-11.66	208.89	114.93	9.86	-208.89	-114.93	-9.86
.688	-272.04	-149.36	-11.66	229.94	126.25	9.86	-229.94	-126.25	-9.86
.750	-296.95	-162.75	-11.66	250.99	137.56	9.86	-250.99	-137.56	-9.86
.813	-343.34	-176.14	-11.66	290.21	148.88	9.86	-290.21	-148.88	-9.86
.875	-369.88	-189.53	-11.66	312.64	160.19	9.86	-312.64	-160.19	-9.86
.938	-396.42	-202.91	-11.66	335.07	171.51	9.86	-335.07	-171.51	-9.86
3.000	-422.96	-216.30	-11.66	357.50	182.83	9.86	-357.50	-182.83	-9.86
3.000	-1.66	2.10	-.01	1.41	-1.78		-1.41	1.78	
.500	-.83	1.05	-.01	.70	-.89		-.70	.89	
4.000			-.01						

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                       SIDA    :
                 KONSTRUKTIONSTYP RAM             Composite steel- concrete bridge using corrugated web i    18    :
                 RESULTAT                         Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                 DATUM
                                        Elly Yman                                          20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	BRTYP MAX-Q M	N	Q	BRTYP MIN-Q M	N	Q	TRAFIC MAX-M M	N	Q
1.000			.01			-.01	.03		.57
.500	-.70	.89	.01	.70	-.89	-.01	.17		.57
2.000	-1.41	1.77	.01	1.41	-1.77	-.01	.32		.57
2.000	-1.41	1.78	9.86	1.41	-1.78	-9.86	.44		1229.24
.063	21.03	13.09	9.86	-21.03	-13.09	-9.86	4984.93		1323.22
.125	43.46	24.41	9.86	-43.46	-24.41	-9.86	9161.90		1210.25
.188	65.89	35.72	9.86	-65.89	-35.72	-9.86	12660.73		848.09
.250	82.58	47.04	9.86	-82.58	-47.04	-9.86	15566.07		1031.83
.313	103.63	58.35	9.86	-103.63	-58.35	-9.86	17841.86		512.12
.375	124.69	69.67	9.86	-124.69	-69.67	-9.86	19443.44		222.52
.438	145.74	80.98	9.86	-145.74	-80.98	-9.86	20397.95		14.31
.500	166.79	92.30	9.86	-166.79	-92.30	-9.86	20646.90		-118.78
.563	187.84	103.62	9.86	-187.84	-103.62	-9.86	20411.76		-315.22
.625	208.89	114.93	9.86	-208.89	-114.93	-9.86	19432.81		-448.31
.688	229.94	126.25	9.86	-229.94	-126.25	-9.86	17835.19		-514.85
.750	250.99	137.56	9.86	-250.99	-137.56	-9.86	15576.82		-723.06
.813	290.21	148.88	9.86	-290.21	-148.88	-9.86	12629.69		-859.08
.875	312.64	160.19	9.86	-312.64	-160.19	-9.86	9163.23		-1221.24
.938	335.07	171.51	9.86	-335.07	-171.51	-9.86	4992.77		-1320.08
3.000	357.50	182.83	9.86	-357.50	-182.83	-9.86			3.05
3.000	1.41	-1.78		-1.41	1.78				
.500	.70	-.89		-.70	.89				
4.000									

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

WSP Sverige AB S T R I P STEP 2 PC-05-041001 Project: Bro 100-262-1 SIDA :
KONSTRUKTIONSTYP RAM Composite steel- concrete bridge using corrugated web i 19 :
RESULTAT Appendix X - System analysis, Short- term loads
0 PROJEKT IDENT KONSTR.IDENT NR DATUM
Elly Yman 20-04-28
0
S N I T T K R A F T E R

PUNKT	TRAFIC MIN-M M	TRAFIC MIN-M N	TRAFIC MIN-M Q	TRAFIC MAX-Q M	TRAFIC MAX-Q N	TRAFIC MAX-Q Q	TRAFIC MIN-Q M	TRAFIC MIN-Q N	TRAFIC MIN-Q Q
1.000			-.01	.03		.57			-339.82
.500	-101.83		-411.01	.17		.57	-101.83		-411.01
2.000	-205.50		-418.36	.32		.57	-205.50		-418.36
2.000	-205.51		4.03	.32		1692.11			-3.33
.063	-192.89		3.96	4942.33		1550.72	1987.01		-41.64
.125	-180.28		3.96	8994.02		1410.92	4520.09		-101.37
.188	-167.66		3.96	12169.71		1272.71	6915.04		-166.97
.250	-155.05		3.96	14489.51		1136.48	9115.84		-238.42
.313	-142.43		3.96	16280.29		1021.54	11553.64		-329.52
.375	-129.82		3.96	16978.78		887.81	13392.62		-420.19
.438	-117.20		3.96	16964.11		760.32	15083.03		-525.81
.500	-104.59		3.96	16271.82		638.14	16483.79		-646.46
.563	-103.79		-1.41	15100.72		526.41	17153.82		-768.84
.625	-108.29		-1.41	13411.96		420.79	17074.22		-892.82
.688	-117.28		-3.24	11347.13		323.62	16232.25		-1018.56
.750	-127.61		-3.24	8975.69		234.75	14710.62		-1153.85
.813	-138.05		-3.33	6763.22		163.30	12341.76		-1290.75
.875	-148.67		-3.33	4584.01		102.80	9021.64		-1415.32
.938	-159.29		-3.33	2055.55		43.07	4956.34		-1555.25
3.000	-169.91		-3.33			4.03	-.03		-1696.76
3.000	-169.90		339.81	-39.10	-.08	418.55			
.500	-84.95		339.81	-65.30	-.07	411.20			
4.000		-.14	.38		-.15	.40			

1

```
 WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA     :
                  KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     20     :
                  RESULTAT                        Appendix X - System analysis, Short- term loads
0                 PROJEKT IDENT         KONSTR.IDENT                    NR                 DATUM
                                          Elly Yman                                        20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	TEMP MAX-M M	TEMP MAX-M N	TEMP MAX-M Q	TEMP MIN-M M	TEMP MIN-M N	TEMP MIN-M Q	TEMP MAX-Q M	TEMP MAX-Q N	TEMP MAX-Q Q
1.000	7589.09	-6476.00	.17	-6144.01	5243.00	-.32	7589.09	-6476.00	.17
.500	7589.13	-6476.00	.17	-6144.09	5243.00	-.32	7589.13	-6476.00	.17
2.000	7589.17	-6476.00	.17	-6144.17	5243.00	-.32	7589.17	-6476.00	.17
2.000	7589.31	-6476.00	-.01	-6144.25	5243.00	.01	-6144.25	5243.00	.01
.063	7589.29	-6476.00	-.01	-6144.24	5243.00	.01	-6144.24	5243.00	.01
.125	7589.27	-6476.00	-.01	-6144.22	5243.00	.01	-6144.22	5243.00	.01
.188	7589.25	-6476.00	-.01	-6144.20	5243.00	.01	-6144.20	5243.00	.01
.250	8379.30	-6476.00	-.01	-6783.83	5243.00	.01	-6783.83	5243.00	.01
.313	8379.28	-6476.00	-.01	-6783.81	5243.00	.01	-6783.81	5243.00	.01
.375	8379.25	-6476.00	-.01	-6783.79	5243.00	.01	-6783.79	5243.00	.01
.438	8379.23	-6476.00	-.01	-6783.78	5243.00	.01	-6783.78	5243.00	.01
.500	8379.21	-6476.00	-.01	-6783.76	5243.00	.01	-6783.76	5243.00	.01
.563	8379.19	-6476.00	-.01	-6783.74	5243.00	.01	-6783.74	5243.00	.01
.625	8379.17	-6476.00	-.01	-6783.72	5243.00	.01	-6783.72	5243.00	.01
.688	8379.15	-6476.00	-.01	-6783.71	5243.00	.01	-6783.71	5243.00	.01
.750	8379.12	-6476.00	-.01	-6783.69	5243.00	.01	-6783.69	5243.00	.01
.813	7589.03	-6476.00	-.01	-6144.03	5243.00	.01	-6144.03	5243.00	.01
.875	7589.01	-6476.00	-.01	-6144.01	5243.00	.01	-6144.01	5243.00	.01
.938	7588.99	-6476.00	-.01	-6143.99	5243.00	.01	-6143.99	5243.00	.01
3.000	7588.97	-6476.00	-.01	-6143.97	5243.00	.01	-6143.97	5243.00	.01
3.000	7588.97	-6476.12	.05	-6143.89	5243.07	-.24	7588.97	-6476.12	.05
.500	7588.98	-6476.12	.05	-6143.95	5243.07	-.24	7588.98	-6476.12	.05
4.000	7588.99	-6476.12	.05	-6144.01	5243.07	-.24	7588.99	-6476.12	.05

1

```
WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA    :
                KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   21    :
                RESULTAT                        Appendix X - System analysis, Short- term loads
0               PROJEKT IDENT         KONSTR.IDENT                  NR                 DATUM
                                        Elly Yman                                      20-04-28
0
 S N I T T K R A F T E R
```

PUNKT	TEMP MIN-Q M	TEMP MIN-Q N	TEMP MIN-Q Q	MULTI MAX-M M	MULTI MAX-M N	MULTI MAX-M Q	MULTI MIN-M M	MULTI MIN-M N	MULTI MIN-M Q
1.000	-6144.01	5243.00	-.32	.03		.57			
.500	-6144.09	5243.00	-.32	.67	-.63	.57	-101.83		-411.01
2.000	-6144.17	5243.00	-.32	1.31	-1.26	.57	-205.50		-418.36
2.000	7589.31	-6476.00	-.01	1.44	-1.26	1222.25	-205.51		4.03
.063	7589.29	-6476.00	-.01	4984.93		1323.22	-207.82	-9.29	-3.04
.125	7589.27	-6476.00	-.01	9161.90		1210.25	-211.13	-17.33	-3.04
.188	7589.25	-6476.00	-.01	12660.73		848.09	-214.43	-25.36	-3.04
.250	8379.30	-6476.00	-.01	15566.07		1031.83	-213.67	-33.39	-3.04
.313	8379.28	-6476.00	-.01	17841.86		512.12	-216.00	-41.42	-3.04
.375	8379.25	-6476.00	-.01	19443.44		222.52	-218.33	-49.46	-3.04
.438	8379.23	-6476.00	-.01	20397.95		14.31	-220.66	-57.49	-3.04
.500	8379.21	-6476.00	-.01	20646.90		-118.78	-222.99	-65.52	-3.04
.563	8379.19	-6476.00	-.01	20411.76		-315.22	-237.13	-73.55	-8.41
.625	8379.17	-6476.00	-.01	19432.81		-448.31	-256.58	-81.59	-8.41
.688	8379.15	-6476.00	-.01	17835.19		-514.85	-280.51	-89.62	-10.24
.750	8379.12	-6476.00	-.01	15576.82		-723.06	-305.78	-97.65	-10.24
.813	7589.03	-6476.00	-.01	12629.69		-859.08	-344.05	-105.68	-10.33
.875	7589.01	-6476.00	-.01	9163.23		-1221.24	-370.60	-113.72	-10.33
.938	7588.99	-6476.00	-.01	4992.77		-1320.08	-397.14	-121.75	-10.33
3.000	7588.97	-6476.00	-.01				-423.68	-129.78	-10.33
3.000	-6143.89	5243.07	-.24				-170.90	1.26	339.80
.500	-6143.95	5243.07	-.24				-85.45	.63	339.80
4.000	-6144.01	5243.07	-.24						

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA     :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i  22     :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT         KONSTR.IDENT                   NR                DATUM
                                         Elly Yman                                      20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	MULTI MAX-Q M	MULTI MAX-Q N	MULTI MAX-Q Q	MULTI MIN-Q M	MULTI MIN-Q N	MULTI MIN-Q Q	BROTT MAX-M M	BROTT MAX-M N	BROTT MAX-M Q
1.000	.03		.57			-339.82	6830.22	-5828.40	1.01
.500	.17		.57	-101.83		-411.01	6831.23	-5829.35	1.00
2.000	.32		.57	-205.50		-418.36	6832.23	-5830.29	1.00
2.000	.32		1692.11	1.00	-1.26	-10.33	6832.54	-5830.28	1833.36
.063	4942.33		1550.72	1972.08	-9.29	-48.63	14307.75	-5828.40	1984.83
.125	8994.02		1410.92	4489.24	-17.32	-108.37	20573.20	-5828.39	1815.37
.188	12169.71		1272.71	6868.27	-25.36	-173.96	25821.42	-5828.39	1272.13
.250	14489.51		1136.48	9057.22	-33.39	-245.42	30890.47	-5828.39	1547.73
.313	16280.29		1021.54	11480.08	-41.42	-336.51	34304.14	-5828.39	768.18
.375	16978.78		887.81	13304.12	-49.45	-427.19	36706.49	-5828.39	333.77
.438	16964.11		760.32	14979.58	-57.48	-532.80	38138.23	-5828.39	21.46
.500	16271.82		638.14	16365.40	-65.52	-653.46	38511.64	-5828.39	-178.17
.563	15100.72		526.41	17020.48	-73.55	-775.84	38158.91	-5828.39	-472.84
.625	13411.96		420.79	16925.93	-81.58	-899.82	36690.46	-5828.39	-672.47
.688	11347.13		323.62	16069.03	-89.61	-1025.55	34294.01	-5828.39	-772.29
.750	8975.69		234.75	14532.46	-97.65	-1160.85	30906.44	-5828.39	-1084.60
.813	6763.22		163.30	12135.75	-105.68	-1297.74	25774.66	-5828.39	-1288.62
.875	4584.01		102.80	8799.71	-113.71	-1422.32	20574.96	-5828.39	-1831.86
.938	2055.55		43.07	4718.48	-121.74	-1562.24	14319.24	-5828.39	-1980.13
3.000			4.03	-253.80	-129.78	-1703.75	6830.07	-5828.40	-.01
3.000	-39.10	-.08	418.55	-1.00	1.26	-.01	6830.07	-5828.51	.04
.500	-65.30	-.07	411.20	-.50	.63	-.01	6830.08	-5828.51	.04
4.000		-.15	.40			-.01	6830.09	-5828.51	.04

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                SIDA    :
                  KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i    23     :
                  RESULTAT                        Appendix X - System analysis, Short- term loads
0                 PROJEKT IDENT         KONSTR.IDENT                 NR                DATUM
                                          Elly Yman                                    20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	BROTT MIN-M M	BROTT MIN-M N	BROTT MIN-M Q	BROTT MAX-Q M	BROTT MAX-Q N	BROTT MAX-Q Q	BROTT MIN-Q M	BROTT MIN-Q N	BROTT MIN-Q Q
1.000	-5529.61	4718.70	-.29	6830.22	-5828.40	1.01	-5529.61	4718.70	-510.02
.500	-5682.43	4718.70	-616.80	6830.48	-5828.40	1.01	-5682.43	4718.70	-616.80
2.000	-5838.01	4718.70	-627.83	6830.73	-5828.40	1.01	-5838.01	4718.70	-627.83
2.000	-5838.09	4718.70	6.05	.48		2538.16	6831.87	-5830.29	-15.50
.063	-5841.54	4704.76	-4.55	7413.49		2326.08	9788.49	-5842.34	-72.95
.125	-5846.49	4692.71	-4.55	13491.04		2116.37	13564.21	-5854.39	-162.56
.188	-5851.43	4680.66	-4.55	18254.57	.01	1909.06	17132.72	-5866.43	-260.95
.250	-6425.95	4668.61	-4.55	21734.27	.01	1704.71	21127.20	-5878.48	-368.13
.313	-6429.43	4656.56	-4.55	24420.44	.01	1532.32	24761.46	-5890.53	-504.77
.375	-6432.91	4644.52	-4.55	25468.17	.01	1331.72	27497.50	-5902.58	-640.79
.438	-6436.39	4632.47	-4.55	25446.17	.01	1140.49	30010.68	-5914.63	-799.21
.500	-6439.86	4620.42	-4.55	24407.73	.01	957.20	32089.39	-5926.67	-980.20
.563	-6461.07	4608.37	-12.61	22651.08	.01	789.62	33071.98	-5938.72	-1163.77
.625	-6490.22	4596.32	-12.61	20117.94	.01	631.19	32930.15	-5950.77	-1349.74
.688	-6526.10	4584.27	-15.35	17020.69		485.43	31644.77	-5962.82	-1538.34
.750	-6563.99	4572.22	-15.35	13463.54		352.13	29339.89	-5974.87	-1741.28
.813	-6045.71	4560.17	-15.49	10144.84		244.95	25033.76	-5986.92	-1946.62
.875	-6085.50	4548.13	-15.49	6876.01		154.21	20029.67	-5998.97	-2133.48
.938	-6125.30	4536.08	-15.49	3083.32		64.60	13907.82	-6011.01	-2343.37
3.000	-6165.10	4524.03	-15.49			6.04	6449.36	-6023.06	-2555.64
3.000	-5785.85	4720.66	509.48	6771.42	-5828.62	627.87	-5531.00	4720.66	-.23
.500	-5657.73	4719.72	509.48	6732.13	-5828.62	616.84	-5530.30	4719.71	-.23
4.000	-5529.61	4718.77	-.22	6830.09	-5828.73	.64	-5529.61	4718.77	-.23

1

WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1   SIDA :
KONSTRUKTIONSTYP RAM   Composite steel- concrete bridge using corrugated web i   24 :
RESULTAT   Appendix X - System analysis, Short- term loads
0   PROJEKT IDENT   KONSTR.IDENT   NR   DATUM
Elly Yman   20-04-28
0
S N I T T K R A F T E R

PUNKT	DEFL MAX-M M	DEFL MAX-M N	DEFL MAX-M Q	DEFL MIN-M M	DEFL MIN-M N	DEFL MIN-M Q	KVASI MAX-M M	KVASI MAX-M N	KVASI MAX-M Q
1.000	.02		.43				3794.57	-3238.00	.51
.500	.13		.43	-76.38		-308.26	3795.07	-3238.47	.51
2.000	.24		.43	-154.13		-313.77	3795.57	-3238.94	.51
2.000	.33		921.23	-154.13		3.02	3795.74	-3238.94	916.68
.063	3735.84		991.66	-144.67		2.97	7533.34	-3238.00	992.41
.125	6866.19		907.00	-135.21		2.97	10666.06	-3238.00	907.69
.188	9488.30		635.58	-125.75		2.97	13290.17	-3238.00	636.07
.250	11665.65		773.28	-116.29		2.97	15864.20	-3238.00	773.87
.313	13371.19		383.80	-106.83		2.97	17571.03	-3238.00	384.09
.375	14571.46		166.76	-97.36		2.97	18772.21	-3238.00	166.88
.438	15286.79		10.72	-87.90		2.97	19488.08	-3238.00	10.73
.500	15473.36		-89.02	-78.44		2.97	19674.78	-3238.00	-89.09
.563	15297.14		-236.24	-77.79		-1.06	19498.41	-3238.00	-236.42
.625	14563.49		-335.98	-81.16		-1.06	18764.19	-3238.00	-336.24
.688	13366.19		-385.85	-87.96		-2.43	17565.96	-3238.00	-386.14
.750	11673.71		-541.88	-95.71		-2.43	15872.18	-3238.00	-542.30
.813	9465.04		-643.82	-103.46		-2.50	13266.78	-3238.00	-644.31
.875	6867.18		-915.23	-111.42		-2.50	10666.93	-3238.00	-915.93
.938	3741.72		-989.31	-119.37		-2.50	7539.07	-3238.00	-990.07
3.000			2.29	-127.33		-2.50	3794.48	-3238.00	
3.000				-127.33		254.66	3794.48	-3238.06	.02
.500				-63.67		254.66	3794.49	-3238.06	.02
4.000					-.11	.28	3794.50	-3238.06	.02

1

```
 WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                            SIDA    :
                  KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i   25    :
                  RESULTAT                        Appendix X - System analysis, Short- term loads
0                 PROJEKT IDENT        KONSTR.IDENT                   NR                 DATUM
                                         Elly Yman                                       20-04-28
0
  S N I T T K R A F T E R
```

PUNKT	KVASI MIN-M M	N	Q	KVASI MAX-Q M	N	Q	KVASI MIN-Q M	N	Q
1.000	-3072.00	2621.50	-.16	3794.57	-3238.00	.51	-3072.00	2621.50	-255.02
.500	-3148.42	2621.50	-308.42	3794.70	-3238.00	.51	-3148.42	2621.50	-308.42
2.000	-3226.21	2621.50	-313.93	3794.82	-3238.00	.51	-3226.21	2621.50	-313.93
2.000	-3226.26	2621.50	3.02	.24		1269.08	.75	-.95	-7.75
.063	-3227.98	2614.53	-2.28	3706.75		1163.04	1479.06	-6.97	-36.47
.125	-3230.45	2608.51	-2.28	6745.52		1058.19	3366.93	-12.99	-81.28
.188	-3232.93	2602.48	-2.28	9127.29		954.53	5151.20	-19.02	-130.47
.250	-3552.17	2596.46	-2.28	10867.13		852.36	6792.91	-25.04	-184.06
.313	-3553.91	2590.43	-2.28	12210.22		766.16	8610.06	-31.07	-252.38
.375	-3555.64	2584.41	-2.28	12734.09		665.86	9978.09	-37.09	-320.39
.438	-3557.38	2578.38	-2.28	12723.08		570.24	11234.68	-43.11	-399.60
.500	-3559.12	2572.36	-2.28	12203.87		478.60	12274.05	-49.14	-490.09
.563	-3569.72	2566.33	-6.30	11325.54		394.81	12765.36	-55.16	-581.88
.625	-3584.29	2560.31	-6.30	10058.97		315.59	12694.45	-61.19	-674.86
.688	-3602.23	2554.29	-7.67	8510.35		242.72	12051.77	-67.21	-769.17
.750	-3621.18	2548.26	-7.67	6731.77		176.06	10899.34	-73.23	-870.64
.813	-3330.05	2542.24	-7.74	5072.42		122.48	9101.82	-79.26	-973.31
.875	-3349.95	2536.21	-7.74	3438.00		77.10	6599.78	-85.28	-1066.74
.938	-3369.85	2530.19	-7.74	1541.66		32.30	3538.86	-91.31	-1171.68
3.000	-3389.75	2524.16	-7.74			3.02	-190.35	-97.33	-1277.81
3.000	-3200.12	2622.48	254.73	3765.16	-3238.12	313.94	-3072.69	2622.48	-.13
.500	-3136.06	2622.01	254.73	3745.51	-3238.12	308.42	-3072.35	2622.01	-.13
4.000	-3072.00	2621.54	-.12	3794.50	-3238.17	.32	-3072.00	2621.54	-.13

1

```
WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                            SIDA    :
                KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i    26     :
                RESULTAT                        Appendix X - System analysis, Short- term loads
0               PROJEKT IDENT        KONSTR.IDENT                    NR                DATUM
                                       Elly Yman                                       20-04-28
0
```

R E A K T I O N E R

PUNKT	AXELLAST MAX-RM			AXELLAST MIN-RM			AXELLAST MAX-RETA		
	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M
2									
3									

0

R E A K T I O N E R

PUNKT	AXELLAST MIN-RETA			AXELLAST MAX-RZETA			AXELLAST MIN-RZETA		
	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M
2					3.81			-392.62	
3	-.01	-233.51			3.82			-392.60	

0

R E A K T I O N E R

PUNKT	BOGGILAST MAX-RM			BOGGILAST MIN-RM			BOGGILAST MAX-RETA		
	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M
2									
3								-358.00	

0

R E A K T I O N E R

PUNKT	BOGGILAST MIN-RETA			BOGGILAST MAX-RZETA			BOGGILAST MIN-RZETA		
	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M	R-ETA	R-ZETA	R-M
2					3.36			-1968.16	
3	-.08	-1415.75			3.50		-.06	-1974.84	

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                     SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     27     :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT       KONSTR.IDENT                    NR                DATUM
                                       Elly Yman                                       20-04-28
0
  R E A K T I O N E R

 I           I          LM1  MAX-RM           I          LM1  MIN-RM           I         LM1  MAX-RETA          I
 I  PUNKT    I--------------------------------I--------------------------------I--------------------------------I
 I           I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I
 I-----------I----------I----------I----------I----------I----------I----------I----------I----------I----------I

        2

        3
0
  R E A K T I O N E R

 I           I         LM1  MIN-RETA          I         LM1  MAX-RZETA         I         LM1  MIN-RZETA         I
 I  PUNKT    I--------------------------------I--------------------------------I--------------------------------I
 I           I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I
 I-----------I----------I----------I----------I----------I----------I----------I----------I----------I----------I

        2                                                       3.31                         -1568.49

        3           -.07   -1220.04                             4.04                  -.04   -1567.95
0
  R E A K T I O N E R

 I           I          LM2  MAX-RM           I          LM2  MIN-RM           I         LM2  MAX-RETA          I
 I  PUNKT    I--------------------------------I--------------------------------I--------------------------------I
 I           I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I
 I-----------I----------I----------I----------I----------I----------I----------I----------I----------I----------I

        2

        3
0
  R E A K T I O N E R

 I           I         LM2  MIN-RETA          I         LM2  MAX-RZETA         I         LM2  MIN-RZETA         I
 I  PUNKT    I--------------------------------I--------------------------------I--------------------------------I
 I           I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I   R-ETA  I   R-ZETA I    R-M   I
 I-----------I----------I----------I----------I----------I----------I----------I----------I----------I----------I

        2                                                       3.53                          -363.54

        3           -.01    -216.21                             3.53                          -363.52
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i      28     :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT       KONSTR.IDENT                    NR                 DATUM
                                       Elly Yman                                        20-04-28
0
  R E A K T I O N E R

 I           I              WIND              I
 I  PUNKT    I--------------------------------I
 I           I   R-ETA  I   R-ZETA I    R-M   I
 I-----------I----------I----------I----------I

        2

        3
1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                  SIDA     :
                 KONSTRUKTIONSTYP RAM           Composite steel- concrete bridge using corrugated web i    29     :
                 RESULTAT                       Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT         KONSTR.IDENT                    NR               DATUM
                                         Elly Yman                                      20-04-28
0
  D E F O R M A T I O N E R
```

PUNKT	AXELLAST MAX-DZ DELTA-Y	AXELLAST MAX-DZ DELTA-Z	AXELLAST MAX-DZ ROT.	AXELLAST MIN-DZ DELTA-Y	AXELLAST MIN-DZ DELTA-Z	AXELLAST MIN-DZ ROT.	BOGGILAST MAX-DZ DELTA-Y	BOGGILAST MAX-DZ DELTA-Z	BOGGILAST MAX-DZ ROT.
1.000	-2.094	.700	.080	.063	-.041	-.005	-12.991	3.963	.454
.500	-1.924	.350	.080	.053	-.020	-.005	-12.032	1.982	.454
2.000							.058		-.004
2.000							.116		-.004
.063	.055	.231	-.004	-1.929	-4.423	.078	.109	.213	-.003
.125	.057	.408	-.003	-1.941	-8.626	.072	.110	.387	-.003
.188	.059	.535	-.002	-1.962	-12.390	.062	.112	.524	-.002
.250	.061	.618	-.001	-2.146	-15.623	.053	.114	.627	-.002
.313	.063	.670	-.001	-2.193	-18.296	.042	.115	.704	-.001
.375	.065	.696		-2.250	-20.284	.029	.117	.756	-.001
.438	.067	.698		-2.415	-21.512	.017	.119	.785	
.500	.069	.677	.001	-2.486	-21.989		.121	.791	
.563	.153	.698		-2.558	-21.512	-.017	.148	.793	
.625	.155	.696		-2.678	-20.284	-.029	.150	.770	.001
.688	.156	.670	.001	-2.736	-18.296	-.042	.151	.723	.001
.750	.159	.618	.001	-2.782	-15.623	-.053	.154	.650	.002
.813	.161	.535	.002	-2.831	-12.390	-.062	.155	.549	.002
.875	.163	.408	.003	-2.852	-8.626	-.072	.157	.409	.003
.938	.165	.231	.004	-2.864	-4.423	-.078	.159	.227	.004
3.000							-14.618		-.434
3.000									
.500	-2.868	.350	-.080	.167	-.020	.005	-16.245	1.981	-.454
4.000	-2.699	.700	-.080	.157	-.041	.005	-15.286	3.962	-.454

1

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

0
D E F O R M A T I O N E R

PUNKT	BOGGILAST MIN-DZ			LM1 MAX-DZ			LM1 MIN-DZ		
	DELTA-Y	DELTA-Z	ROT.	DELTA-Y	DELTA-Z	ROT.	DELTA-Y	DELTA-Z	ROT.
1.000	.058	-.038	-.004	-10.680	3.225	.370	.065	-.043	-.005
.500	.049	-.019	-.004	-9.899	1.613	.370	.055	-.021	-.005
2.000	-12.285		.434						
2.000	-12.989		.425						
.063	-12.057	-25.045	.442	.061	.246	-.004	-9.920	-20.373	.360
.125	-12.131	-48.781	.407	.063	.436	-.003	-9.982	-39.650	.330
.188	-12.254	-69.923	.349	.065	.572	-.002	-10.081	-56.809	.284
.250	-12.604	-87.587	.288	.067	.661	-.001	-10.398	-71.225	.236
.313	-13.022	-101.887	.224	.070	.718	-.001	-10.616	-82.984	.185
.375	-13.578	-112.478	.155	.072	.747		-10.994	-91.772	.128
.438	-13.909	-118.941	.077	.074	.750		-11.348	-97.188	.067
.500	-14.329	-121.076		.075	.729	.001	-11.685	-99.025	-.001
.563	-14.664	-118.895	-.079	.077	.687	.001	-12.035	-97.192	-.068
.625	-15.069	-112.448	-.154	.078	.626	.001	-12.367	-91.773	-.128
.688	-15.498	-101.907	-.223	.137	.592	.001	-12.643	-82.983	-.185
.750	-15.778	-87.593	-.289	.139	.545	.001	-12.860	-71.218	-.236
.813	-16.012	-69.913	-.350	.141	.471	.002	-13.040	-56.808	-.284
.875	-16.145	-48.756	-.407	.143	.359	.002	-13.138	-39.647	-.330
.938	-16.219	-25.038	-.442	.144	.203	.003	-13.202	-20.373	-.360
3.000	.151		.004						
3.000									
.500	.160	-.020	.005	-13.223	1.613	-.370	.142	-.017	.004
4.000	.151	-.039	.005	-12.442	3.225	-.370	.133	-.035	.004

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                         SIDA     :
                 KONSTRUKTIONSTYP RAM               Composite steel- concrete bridge using corrugated web i      31      :
                 RESULTAT                           Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                 DATUM
                                        Elly Yman                                        20-04-28
0
  D E F O R M A T I O N E R

 I           I          LM2  MAX-DZ           I          LM2  MIN-DZ           I              WIND             I
 I  PUNKT    I--------------------------------I--------------------------------I--------------------------------I
 I           I  DELTA-Y I  DELTA-Z I    ROT.  I  DELTA-Y I  DELTA-Z I    ROT.  I  DELTA-Y I  DELTA-Z I    ROT.  I
 I-----------I----------I----------I----------I----------I----------I----------I----------I----------I----------I

       1.000    -1.939       .648       .074       .058      -.038      -.004
        .500    -1.782       .324       .074       .049      -.019      -.004
       2.000

       2.000
        .063      .051       .214      -.003     -1.786     -4.096       .072
        .125      .053       .378      -.003     -1.797     -7.987       .067
        .188      .055       .495      -.002     -1.817    -11.472       .058
        .250      .057       .572      -.001     -1.987    -14.466       .049
        .313      .059       .621      -.001     -2.030    -16.941       .039
        .375      .060       .645                -2.083    -18.781       .027
        .438      .062       .646                -2.237    -19.918       .016
        .500      .063       .627       .001     -2.302    -20.360
        .563      .141       .646                -2.368    -19.918      -.016
        .625      .143       .645                -2.480    -18.781      -.027
        .688      .145       .621       .001     -2.533    -16.941      -.039
        .750      .147       .572       .001     -2.576    -14.466      -.049
        .813      .149       .495       .002     -2.621    -11.472      -.058
        .875      .151       .378       .003     -2.640     -7.987      -.067
        .938      .152       .214       .003     -2.652     -4.096      -.072
       3.000

       3.000
        .500    -2.656       .324      -.074       .154      -.019       .004
       4.000    -2.499       .648      -.074       .145      -.038       .004
1
```

```
 WSP Sverige AB   S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                          SIDA    :
                  KONSTRUKTIONSTYP RAM              Composite steel- concrete bridge using corrugated web i     32     :
                  RESULTAT                          Appendix X - System analysis, Short- term loads
0                 PROJEKT IDENT         KONSTR.IDENT                   NR                DATUM
                                          Elly Yman                                        20-04-28
0
```

D E F O R M A T I O N E R

PUNKT	TRAFIC MAX-DZ DELTA-Y	TRAFIC MAX-DZ DELTA-Z	TRAFIC MAX-DZ ROT.	TRAFIC MIN-DZ DELTA-Y	TRAFIC MIN-DZ DELTA-Z	TRAFIC MIN-DZ ROT.	DEFL MAX-DZ DELTA-Y	DEFL MAX-DZ DELTA-Z	DEFL MAX-DZ ROT.
1.000	-11.354	3.464	.397	.065	-.043	-.005	-8.509	2.596	.297
.500	-10.516	1.732	.397	.055	-.021	-.005	-7.881	1.298	.297
2.000	.050		-.004	-10.737		.380	.038		-.003
2.000	.102		-.004	-11.352		.371	.076		-.003
.063	.061	.246	-.004	-10.538	-21.890	.387	.046	.185	-.003
.125	.063	.436	-.003	-10.603	-42.635	.356	.047	.327	-.002
.188	.065	.572	-.002	-10.710	-61.113	.305	.049	.429	-.001
.250	.067	.661	-.001	-11.016	-76.551	.252	.051	.496	-.001
.313	.070	.718	-.001	-11.381	-89.049	.196	.052	.539	-.001
.375	.072	.747		-11.867	-98.305	.135	.054	.560	
.438	.074	.750		-12.156	-103.954	.068	.055	.562	
.500	.075	.729	.001	-12.524	-105.820		.056	.546	
.563	.129	.693		-12.816	-103.914	-.069	.097	.519	
.625	.131	.673	.001	-13.170	-98.280	-.135	.098	.505	
.688	.132	.632	.001	-13.545	-89.066	-.195	.099	.474	.001
.750	.134	.568	.001	-13.790	-76.557	-.253	.101	.426	.001
.813	.136	.480	.002	-13.995	-61.104	-.306	.102	.359	.001
.875	.143	.359	.002	-14.110	-42.613	-.355	.107	.269	.002
.938	.144	.203	.003	-14.176	-21.883	-.386	.108	.152	.002
3.000	-12.776		-.379	.132		.004	-9.575		-.284
3.000									
.500	-14.198	1.731	-.397	.146	-.018	.004	-10.640	1.298	-.297
4.000	-13.360	3.463	-.397	.137	-.036	.004	-10.012	2.595	-.297

1

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                 SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i     33     :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                    NR                 DATUM
                                        Elly Yman                                        20-04-28
0
  D E F O R M A T I O N E R

 I           I          DEFL  MIN-DZ          I
 I  PUNKT    I--------------------------------I
 I           I  DELTA-Y I  DELTA-Z I    ROT.  I
 I-----------I----------I----------I----------I

       1.000       .049      -.032      -.004
        .500       .041      -.016      -.004
       2.000     -8.047                  .284

       2.000     -8.508                  .278
        .063     -7.897    -16.405       .290
        .125     -7.946    -31.952       .267
        .188     -8.026    -45.800       .229
        .250     -8.256    -57.369       .189
        .313     -8.529    -66.736       .147
        .375     -8.894    -73.673       .101
        .438     -9.110    -77.906       .051
        .500     -9.386    -79.305
        .563     -9.605    -77.876      -.052
        .625     -9.870    -73.654      -.101
        .688    -10.151    -66.749      -.146
        .750    -10.334    -57.374      -.189
        .813    -10.488    -45.793      -.229
        .875    -10.575    -31.935      -.266
        .938    -10.624    -16.400      -.290
       3.000       .099                  .003

       3.000
        .500       .109      -.013       .003
       4.000       .103      -.027       .003
1
```

```
 WSP Sverige AB  S T R I P   STEP 2 PC-05-041001 Project: Bro 100-262-1                                SIDA    :
                 KONSTRUKTIONSTYP RAM            Composite steel- concrete bridge using corrugated web i    -1     :
                 RESULTAT                        Appendix X - System analysis, Short- term loads
0                PROJEKT IDENT        KONSTR.IDENT                   NR                 DATUM
                                        Elly Yman                                       20-04-28
0
0I N N E H   L L S F ™ R T E C K N I N G
0
 DATAKVITTO                                                      SIDA
0           ENHETER                                                 1
            MATERIALKONSTANTER                                      2
            ELEMENTTYPER                                            2
            SYSTEMBESKRIVNING                                       2
            STANDARDLASTFALL                                        2
            FARBANOR                                                3
            TRAFIKLASTGRUPPER                                       3
            TRAFIKLASTFALL                                          6
            LASTKOMBINATIONER                                       7
0
 RESULTAT

0         I LASTKOMBINATIONS I  STABI-  I  SPŽN-   I  SNITT-  I   REAK-  I  DEFOR-  I
          I NAMN             I   LITET  I  NINGAR  I  KRAFTER I  TIONER  I MATIONER I
          I------------------I----------I----------I----------I----------I----------I

            AXELLAST                                      9         26         29
            BOGGILAST                                    10         26         29
            LM1                                          11         27         30
            LM2                                          13         27         31
            UTM3                                         14

            WIND                                         15         28         31
            BRLM                                         16
            BRTYP                                        17
            TRAFIC                                       18                    32
            TEMP                                         20

            MULTI                                        21
            BROTT                                        22
            DEFL                                         24                    32
            KVASI                                        24
0
 SISTA SIDA I DENNA BERŽKNING                                      33
1
```

Appendix 4 – Strip Step, System analysis – bridge 100-262-1 (short-term)

```
 WSP Sverige AB    S T R I P   STEP 2           Project: Bro 100-262-1
                  FRAME                        Composite steel- concrete bridge using corrugated web i
                  STATISTIC                    Appendix X - System analysis, Short- term loads

 PROJECT NO       SYSTEM NO       NO           DATE          COMP-DATE      START-TERM.TIME      VERSION
                    Elly Yman                                2020-04-28     11:50:59-11:51:09    PC-05-041001
0
     4 JOINTS                          I  CROSS SECTION EVAL      .00  I  MATRIX  ROWS BAND   DETERM     PIVOT    INCR
     3 ELEMENTS                        I  INPUT EVALUATION             I
     0 ARCHES                          I     GEOMETRY             .00  I   A22I      3    2  .10E+01  .10E+01     0
     2 REACTIONS                       I     LOADCASES            .00  I   SR        9    8  .18E-35  .36E-08   -33
                             LOADEL    I     PRESTRESSING         .00  I   KM1T      9    4  .88E+36  .14E+07    34
     5 STANDARD LOADCASES        13    I     RESULT REQUESTS      .05  I
     0 TEMPERATURE LOADC.         0    I  MATRIX PREPARATION           I  FILE   TRK   POS          FILE   TRK   POS
     0 SUPPORT SETTLEMENTS        0    I     GEOMETRY             .00  I
     0 PRESTR. LOADCASES          0    I     REDUCTION            .06  I   1      0     0            11     0     0
                                       I     LOADING CASES        .22  I   2      0     0            14     0     0
     1 TRACKS (MAX. 3 ELEMENTS)        I     INFLUENCE LINES     1.70  I   3      0     0            19     0     0
    29 TRAFFIC GROUPS                  I     2ND ORDER/CREEP      .00  I   4      0     0            20     0     0
    22 TRAFFIC LOADCASES               I  CALCULATION PARTS            I   7      0     0            21     0     0
                                       I     RANDF. REDUCT.       .00  I   8      0     0            22     0     0
     0 CROSS SECTIONS                  I     INVERT       A22     .05  I   9      0     0            24     0     0
                                       I     SYSTEM MATRICES      .00  I  10      0     0            25     0     0
    14 COMBINATIONS                    I     INVERT        VR     .00  I
     1   ADDITIVE                      I     TRIANG         K     .00  I  ERROR WITHIN PROGRAM PART          0
    43   ENVELOPES                     I     DEFORM       URT     .00  I  NO OF CALLS TO SUBROUTINE DISC     0
     0   CREEP/SHRINKAGE               I     GROSSFORCES POO      .00  I
     0   STRESSES                      I     DEFORMATIONS         .00  I  RESULT PAGES  ECHOPRINT            8
     0   SECOND ORDER                  I     BASIC INFL.LINES     .00  I                STRESSES             0
     0   STABILITY                     I     TRAFFIC INFL.        .11  I                GROSSFORCES         17
                                       I     INFLUENCE LINES      .00  I                REACTIONS            3
    23 RESULT POINTS                   I     SECOND ORDER         .00  I                DEFORMATIONS         5
    23 DEFORMATION POINTS              I     CREEP                .00  I                INFLUENCE LINES      0
     0 INFLUENCE LINES                 I     STABILITY            .00  I                STABILITY            0
 ----------------------------------I  TRAFFIC EVALUATION      .00  I-------------------------------------------
                                       I  COMBINATION PART        .05  I  IBM PC
                                       I  RESULT PRINT PART            I  TOTAL ELAPSED TIME       9.99 SECONDS
                                       I     STRESSES             .00  I
                                       I     FORCES / DEFORM      .11  I
                                       I     INFLUENCE LINES      .00
```

G

Investigation of the web height's influence on the material savings - Calculation reports

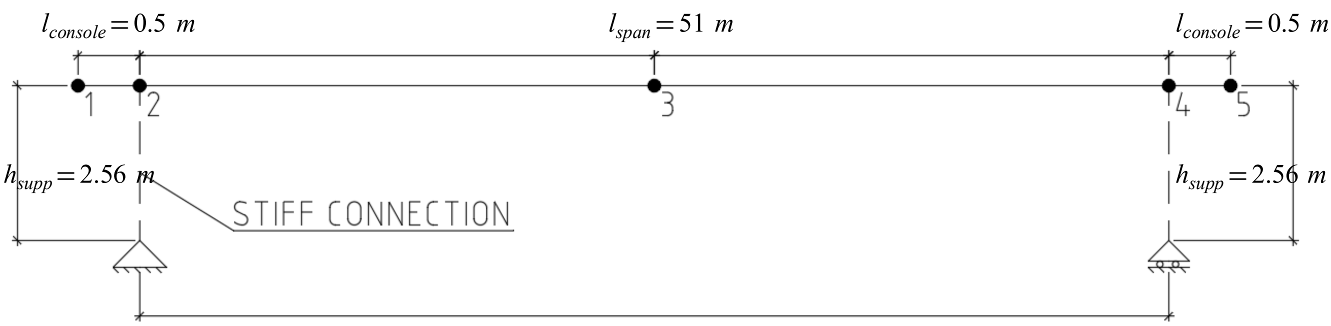
In the following chapter the calculation reports for the investigation of the web height's influence on the material savings are presented. In general, the reports consists of the same calculations as Appendix F (Calculation report for bridge 100-262-1). Only the calculations that in any way differ from the original calculation report are presented. The scope of the extracts involves increasing the girder height with the factor 1.25, 1.375, 1.5, 1.75 and 2.0.

Extracts - Calculation report 1.25h_w

3 System

In the following chapter the input data for the bridge's geometries is presented. The cross-sectional parameters that are presented is preliminary and used for the system analysis. In chapter 5 the final design is presented.

3.1 Primary system - longitudinal



The bridge is modelled as a simply supported bridge in the software StripStep-2. Due to the beams depth the supports are set offset from the neutral axis. This is modelled with a stiff connection in the system analysis.

3.1.1 Cross-section dimensions

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

New height of girder is taken as: $Round(2050 \cdot 1.25, 10) = 2560$

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	600	8	2477	48	850
500	35	600	8	2477	48	850

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	600	8	2477	48	850
10500	35	600	8	2477	48	850
10500	42	600	6	2463	55	850
11300	42	800	6	2463	55	1180
25500	42	800	6	2463	55	1180

Important! Two exactly the same values will not work with the linterp- function. Therefore 0.1 millimeter must be added for a X-value where you want to cross-sectional properties at the same time.

Element 3

S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	600	8	2477	48	850
500	35	600	8	2477	48	850

$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=2560\ mm$

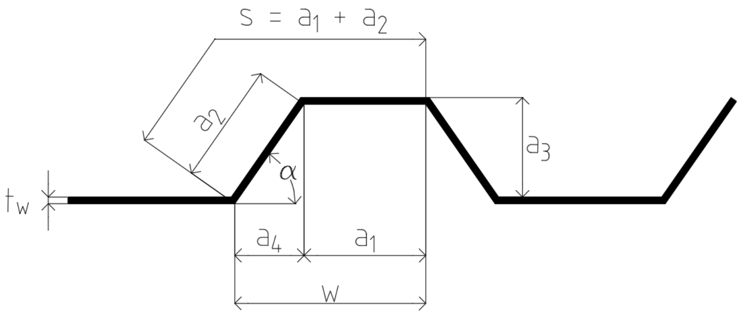
$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=2560\ mm$

3.1.2 Corrugation shape

$a_{c1}:=150\ mm$ Flat-fold length

$\alpha_c:=36\ deg$ Corrugation angle

$a_{c3}:=80\ mm$ Corrugation depth



$a_{c2}:=\dfrac{a_{c3}}{sin\left(\alpha_c\right)}=136\ mm$ Length of angled part

$a_{c4}:=\dfrac{a_{c3}}{tan\left(\alpha_c\right)}=110\ mm$ Length of hypopythis

$s_c:=a_{c1}+a_{c2}=286\ mm$ Length of corrugation

$w_c:=a_{c1}+a_{c4}=260\ mm$ Straight length

$r_c:=\dfrac{s_c}{w_c}=1.10$ Ratio corrugation/flat-fold length

3.1.3.2 Effective width, steel flanges

Calculation of the effective width in the steel flanges with regards to shear lag is calculated according to SS-EN 1993-1-5 3.2.

$b_{eff\_f} = \beta \cdot b_0 + \frac{t_w}{2}$

Effective width with regards to shear lag under elastic conditions - Equation (3.1)

$X_{check\_uf} := \frac{L_{bridge}}{2} = 26.0\ m$

$X_{check\_lf} := 0.2\ L_{bridge} = 10.4\ m$

Upper flange

$\alpha_0 := 1.0$

Equation given in Table 3.1. For webs without any longitudinal stiffeners $\alpha_0 = 1.0$.

$bfu := b_{fu}\left(X_{check\_uf}\right) = 800\ mm$

Width of upper flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_uf}\right) = 6\ mm$

Thickness of web - see Appendix B - Preliminary sizing

$b0 := \frac{bfu - tw}{2} = 397\ mm$

Figure 3.2 (Notations for shear lag)

$\kappa fu := \frac{\alpha_0 \cdot b0}{L_{span}} = 0.008$ = $\kappa_{fu}\left(X_{check\_uf}\right) = 0.008$

Table 3.1

$$\beta_{check} := \begin{array}{|l} \text{if } \kappa fu \le 0.02 \\ \quad \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fu \le 0.70 \\ \quad \beta \leftarrow \frac{1}{1 + 6.4\ \kappa fu^2} \\ \text{else if } 0.70 < \kappa fu \\ \quad \beta \leftarrow \frac{1}{8.6\ \kappa fu^2} \end{array} = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for upper flange

$2\ \beta_{check} \cdot b0 + tw = 800\ mm$

$$b_{ef} := \begin{array}{|l} \text{if } \beta_{check} = 1.0 \\ \quad b \leftarrow bfu \\ \text{else} \\ \quad b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfu\right) \\ b \end{array} = 0.800\ m$$ = $b_{eff\_fu}\left(X_{check\_uf}\right) = 0.800\ m$

Lower flange

$bfl := b_{fl}\left(X_{check\_lf}\right) = 850\ mm$ Width of lower flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_lf}\right) = 8\ mm$ Thickness of web - see Appendix B - Preliminary sizing

$b0 := \dfrac{bfl - tw}{2} = 421\ mm$ Figure 3.2 (Notations for shear lag)

$\kappa fl := \dfrac{\alpha_0 \cdot b0}{L_{span}} = 0.008$ = $\kappa_{fl}\left(X_{check\_lf}\right) = 0.008$ Table 3.1

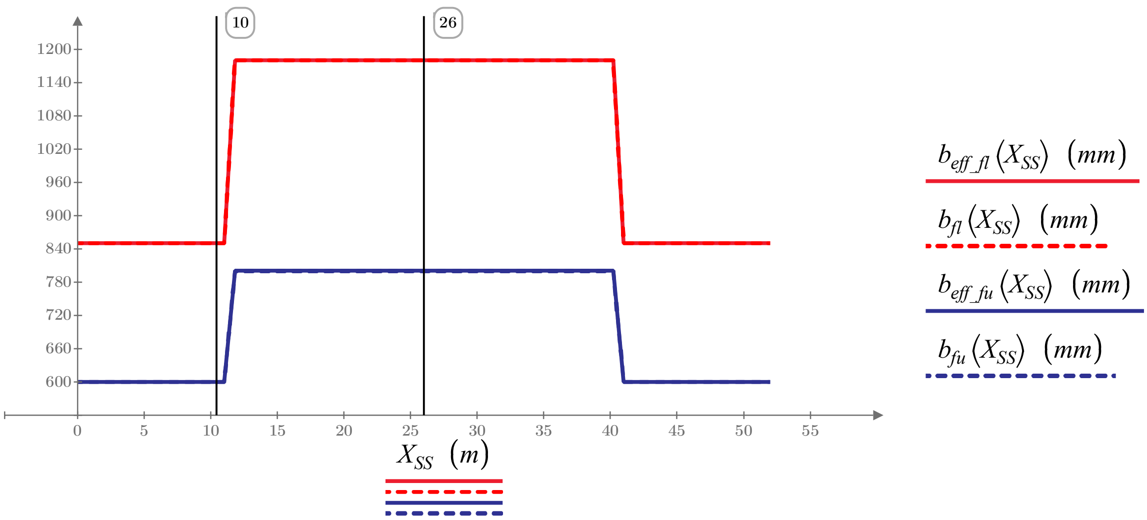
$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fl \le 0.02 \\ \quad \| \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fl \le 0.70 \\ \quad \left\| \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fl^2} \right. \\ \text{else if } 0.70 < \kappa fl \\ \quad \left\| \beta \leftarrow \dfrac{1}{8.6\ \kappa fl^2} \right. \end{array} \right. = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for lower flange

$2\ \beta_{check} \cdot b0 + tw = 850\ mm$

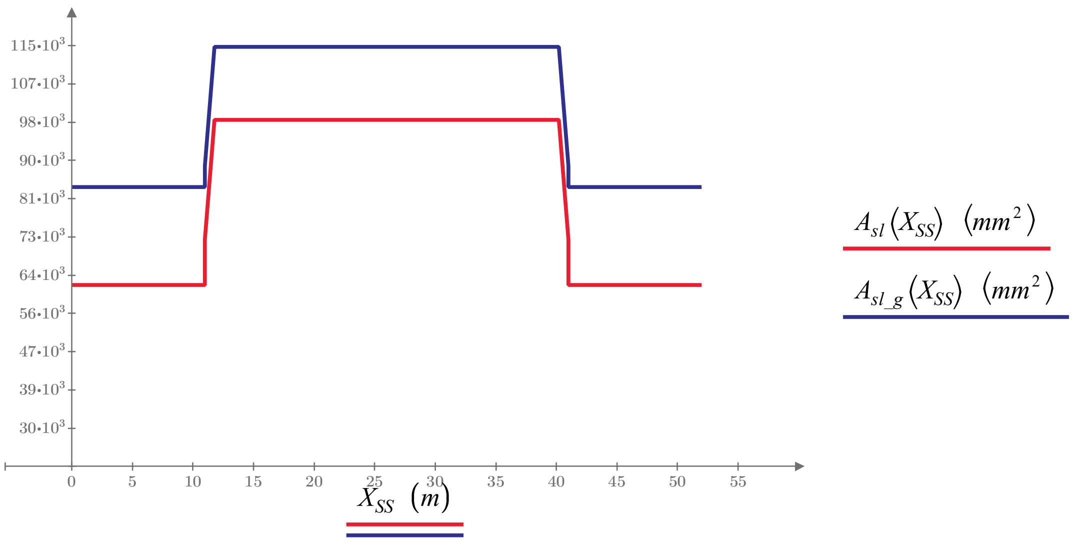
$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad \| b \leftarrow bfl \\ \text{else} \\ \quad \| b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfl\right) \\ b \end{array} \right. = 0.850\ m \quad = \quad b_{eff\_fl}\left(X_{check\_lf}\right) = 0.850\ m$$



3.1.4.2 Cross-sectional constants during construction

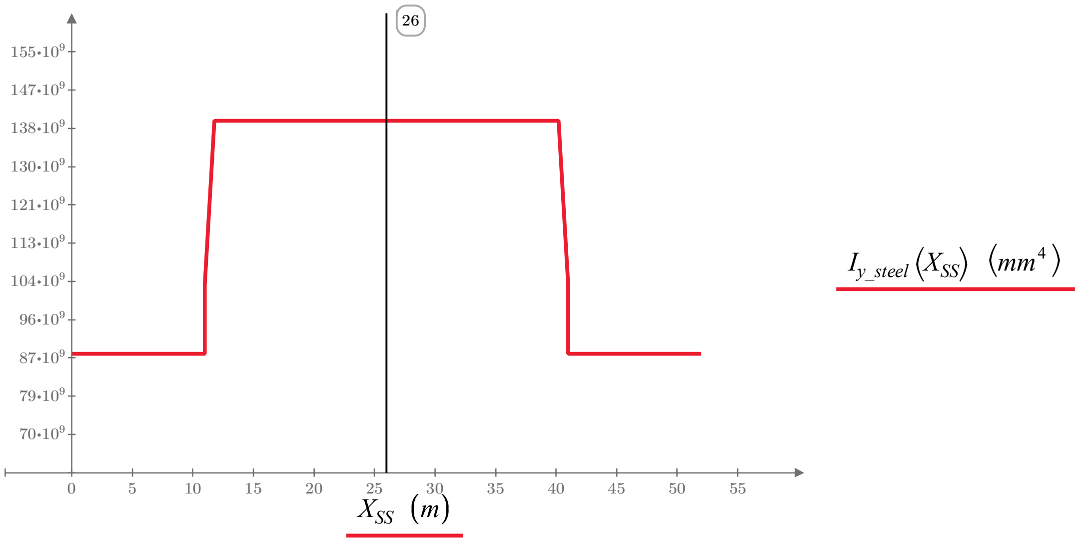
The bridge is checked so that the steel girder (alone) can withstand the loads that are imposed during construction such as the self-weight of curing concrete.

A_{sl_g} is the area of the steel including the web, for calculation of self-weight
A_{sl} is the area of the steel excluding the web, for stiffness calculations



$$A_{sl}\left(\frac{L_{bridge}}{2}\right)=0.099\ m^2$$ Area at mid-span

I_{y_steel} is the area of the steel excluding the web, for stiffness calculations



$$I_{y\_steel}\left(\frac{L_{bridge}}{2}\right)=0.1397\ m^4$$ Stiffness at mid-span

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$	[kN/m]
0.000	-1.680	2.560	879.476	618	6.312
0.500	-1.680	2.560	879.476	618	6.312

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$	[kN/m]
0.000	-1.680	2.560	879.476	618	6.312
10.500	-1.680	2.560	879.476	618	6.312
10.500	-1.653	2.560	1032.958	720	6.660
11.300	-1.676	2.560	1396.627	985	8.665
25.500	-1.676	2.560	1396.627	985	8.665
39.700	-1.676	2.560	1396.627	985	8.665
40.500	-1.653	2.560	1032.958	720	6.660
40.500	-1.680	2.560	879.476	618	6.312
51.000	-1.680	2.560	879.476	618	6.312

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$	[kN/m]
0.000	-1.680	2.560	879.476	618	6.312
0.500	-1.680	2.560	879.476	618	6.312

System model used in Strip-Step2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.56	ZR	
					2
3	51.500	0.00	-2.56	YZR	
					3
4	52.000	0.00			-3

3.1.4.3 Cross-sectional constants for variable loads (short term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.185	2.880	2602.68	3297
0.500	-0.185	2.880	2602.68	3297

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.185	2.880	2602.68	3297
10.500	-0.185	2.880	2602.68	3297
10.500	-0.224	2.880	2919.66	3398
11.300	-0.334	2.880	3846.42	3664
25.500	-0.334	2.880	3846.42	3664
39.700	-0.334	2.880	3846.42	3664
40.500	-0.224	2.880	2919.66	3398
40.500	-0.185	2.880	2602.68	3297
51.000	-0.185	2.880	2602.68	3297

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.185	2.880	2602.68	3297
0.500	-0.185	2.880	2602.68	3297

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.56	ZR	
					2
3	51.500	0.00	-2.56	YZR	
					3
4	52.000	0.00			-3

3.1.4.4 Cross-sectional constants for additional permanent loads (long term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$
0	-0.613	2.880	2100.016	1471
0.5	-0.613	2.880	2100.016	1471

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$
0	-0.613	2.880	2100.016	1471
10.5	-0.613	2.880	2100.016	1471
10.5	-0.670	2.880	2322.604	1572
11.3	-0.824	2.880	2944.051	1838
25.5	-0.824	2.880	2944.051	1838
39.7	-0.824	2.880	2944.051	1838
40.5	-0.670	2.880	2322.604	1572
40.5	-0.613	2.880	2100.016	1471
51	-0.613	2.880	2100.016	1471

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$
0	-0.613	2.880	2100.016	1471
0.5	-0.613	2.880	2100.016	1471

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.56	ZR	
					2
3	51.500	0.00	-2.56	YZR	
					3
4	52.000	0.00			-3

3.1.4.5 Cross-sectional constants for shrinkage analysis

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.519	2.880	2209.100	1675
0.5	-0.519	2.880	2209.100	1675

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.519	2.880	2209.100	1675
10.5	-0.519	2.880	2209.100	1675
10.5	-0.574	2.880	2448.889	1777
11.3	-0.726	2.880	3123.943	2042
25.5	-0.726	2.880	3123.943	2042
39.7	-0.726	2.880	3123.943	2042
40.5	-0.574	2.880	2448.889	1777
40.5	-0.519	2.880	2209.100	1675
51	-0.519	2.880	2209.100	1675

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.519	2.880	2209.100	1675
0.5	-0.519	2.880	2209.100	1675

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.56	ZR	
					2
3	51.500	0.00	-2.56	YZR	
					3
4	52.000	0.00			-3

4 Loads and load combinations

4.1 Permanent loads

4.1.1 Self-weight

4.1.1.1 Steel

The self-weight is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: STEEL

$\rho_{st} = 75.51 \ \frac{kN}{m^3}$ Self-weight of stainless steel - SS-EN 10088-1:2014 Table E.1 or E.2

The self-weight for each element is calculated in chapter 3.

4.1.1.2 Concrete

The self-weight of the slab is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: SLAB

$Wet\_Concrete := \text{“NO”}$ "YES" or "NO" dependent if the previous designer has used the weight of wet concrete

$\rho_c := \textbf{if}\left(Wet\_Concrete = \text{“NO”}, 25 \ \frac{kN}{m^3}, 26 \ \frac{kN}{m^3}\right) = 25 \ \frac{kN}{m^3}$ Self-weight of concrete (reinforced) - SS-EN 1992-1-1 Table A.1

$A_{slab} = 3.47 \ m^2$ Area of slab, see chapter 3

$g_{slab} := \frac{A_{slab} \cdot \rho_c}{2} = 43.4 \ \frac{kN}{m}$ Self-weight of concrete slab, (half of the load goes to each girder) - applied in the casting stage

If the previous designer has considered that the hardened concrete has a smaller self-weight a reduction in self-weight is applied in the system analysis for permanent loads.

Appendix X. Name in Strip-Step2: AVSLAB

$$g_{slab.perm} := \textbf{if}\left(Wet\_Concrete = \text{“NO”}, 0 \ \frac{kN}{m}, \frac{A_{slab} \cdot (-1) \ \frac{kN}{m^3}}{2}\right) = 0.000 \ \frac{kN}{m}$$

4.1.2 Shrinkage

The shrinkage force is calculated and used in Strip-Step2, Appendix X. Name in Strip-Step2: E:SHRINK

$h_0 = 341\ mm$ Equivalent thickness, calculated in chapter 2

$t = 120\ yr$

$t = 43829.1\ day$

$f_{cm} = 43\ MPa$ Mean compressive strength - see chapter 2

$t_s := 1\ day$

$RH := 80\%$ Krav Brobyggande B.3.1.5

$$\beta_{ds} := \frac{\dfrac{t - t_s}{day}}{\left(\dfrac{t - t_s}{day}\right) + 0.04 \cdot \sqrt{\left(\dfrac{h_0}{1\ mm}\right)^3}} = 0.994$$

SS-EN 1992-1-1 3.1.4 Equation 3.10

$$k_h := \begin{array}{l} \text{if } 200\ mm \le h_0 < 300\ mm \\ \quad \left\| 0.85 - \left(\dfrac{h_0}{mm} - 200\right) \cdot 0.001 \right. \\ \text{else if } 300\ mm \le h_0 < 500\ mm \\ \quad \left\| 0.75 - \left(\dfrac{h_0}{mm} - 300\right) \cdot 0.0005 \right. \\ \text{else if } h_0 \ge 500\ mm \\ \quad \| 0.7 \\ \text{else} \\ \quad \| 1.0 \end{array} = 0.73$$

SS-EN 1992-1-1 3.1.4 table 3.3

$RH_0 := 100\%$

$\alpha_{ds1} := 4$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$\alpha_{ds2} := 0.12$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0}\right)^3\right) = 0.76$$

SS-EN 1992-1-1 Appendix B.2 equation B12

$f_{cmo} := 10\ MPa$ SS-EN 1992-1-1 Appendix B.2 (1)

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}\right)}\right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$

SS-EN 1992-1-1 Appendix B.2 equation B11

$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 1.84 \cdot 10^{-4}$ Drying shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.9

$\varepsilon_{ca0} := 2.5 \cdot \left(\frac{f_{ck} - f_{cmo}}{MPa}\right) \cdot 10^{-6} = 6.3 \cdot 10^{-5}$ SS-EN 1992-1-1 3.1.4 Equation 3.12

$\beta_{as} := 1 - e^{\left(-0.2 \cdot \sqrt{\frac{t - t_s}{day}}\right)} = 1.0$ SS-EN 1992-1-1 3.1.4 Equation 3.13

$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca0} = 6.25 \cdot 10^{-5}$ Autogenous shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.11

$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.8

4.1.2.1 Shrinkage force

The shrinkage force and corresponding moment is calculated accordingly:

$n_{L\_cs} = 14.91$ Modular ratio accounting for creep and shrinkage

$n_{L\_short} = 5.88$ Modular ratio

$E_{c.eff} := \frac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab (half of the cross-section used in system analysis)

$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5206\ kN$ Total shrinkage force

$e_{cs} := z_{tp\_cs}(0\ m) = 0.519\ m$ The shrinkage force is applied in the center of gravity for the composite section

$M_{cs} := F_{cs} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right) = 3534\ kN \cdot m$ Total bending moment

Shrinkage - anchorage of studs

Forces in the concrete

$$F_{c\_cs} := F_{cs} \cdot \left(1 - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{sl\_cs}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{y\_cs}(0\ m)} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2} \right)^2 \right) = 771\ kN$$

Force in concrete

$$b_{eff}(0\ m) = 4.925\ m$$

Effective width of flange

$$M_{c\_cs} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_cs}} \cdot h_{m.slab}^{\ 3}}{I_{y\_cs}(0\ m)} \cdot M_{cs} = 14\ kN \cdot m$$

Moment in concrete

$$\sigma_1 := \frac{F_{c\_cs}}{A_{slab.fic}} - \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 3}} = 0.32\ MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{F_{c\_cs}}{A_{slab.fic}} + \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 3}} = 0.66\ MPa$$

Stress in concrete, lower edge of slab

$$N_{cs} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = 771\ kN \quad = \quad F_{c\_cs} = 771\ kN$$

Force imposed on studs caused by shrinkage (used in chapter 6)

4.2.2.1 Horizontal side trafic load

$P_{h\_side\_car} := max\left(\begin{bmatrix} 25\% \cdot Q_{1k\_br} \\ 25\% \cdot Q_{tr\_br} \end{bmatrix}\right) = 109.1\ kN$ Horisontal side force from the acceleration load (used in chapter 7)

$P_{v\_side\_car} := P_{h\_side\_car} \cdot \dfrac{h_{m.slab}}{2 \cdot B_{in}} = 6.2\ kN$ Vertical side force from the acceleration load (used in chapter 7)

4.2.3 Temperature load

Temperatures are determined according to SS-EN 1991-1-5, 6.1.3 unless otherwise stated.

Load case 1 - local temperature differences for Hudiksvall

$T_0 = 10\ °$ A.1(3)

$T_{min} = -38\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{max} = 34\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4\ ° = -34\ °$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min} = 44\ °$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4\ ° = 38\ °$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0 = 28\ °$ Expansion - Equation 6.1

$\Delta T := T_{e.max} - T_{e.min} = 72\ °$ Total temperature difference

Load case 2 - either of the components are larger than the other

$\Delta T_{c2st} := 15\ °$ Temperature difference between concrete and steel - SS-EN 1991-1-5, 6.1.6

4.2.3.1 Coefficients of thermal linear expansion

For composite bridges normally it is suggested to use the same thermal linear expansion coefficient, according to SS-EN 1991-1-5 Table C.1. However for stainless steels the thermal linear expansion coefficient is much larger than for concrete and hence a more through calculation is needed for the strain.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

4.2.3.2 Strains for the different load cases

$\Delta\varepsilon_{LC.1\_con} := -(\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.con}}{\circ} = -26.4\ 10^{-5}$ Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp} := (\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.exp}}{\circ} = 16.8\ 10^{-5}$ Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss} \cdot \frac{\Delta T_{c2st}}{\circ} = 24.0\ 10^{-5}$ Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c \cdot \frac{\Delta T_{c2st}}{\circ} = 15.0\ 10^{-5}$ Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st}, \Delta\varepsilon_{LC.2\_c}\right) = 24.0\ 10^{-5}$ Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp_1} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con} = -50.4\ 10^{-5}$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp_2} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp} = 40.8\ 10^{-5}$ Maximum strain difference; temperature raises and the steel heatens up even higher

4.2.3.3 Temperature load - global analysis

The concrete is transformed to steel. Already defined parameters are calculated in chapter 3.

$\frac{b_{eff}(0\ m)}{n_{L\_short}} = 0.837\ m$ Width of transformed concrete

$\frac{A_{slab.fic}}{n_{L\_short}} = 0.268\ m^2$ Area of transformed concrete

$A_{sl}(0\ m) = 0.062\ m^2$ Area of composite section

$z_{tp\_short}(0\ m) = 185\ mm$ Distance from top of concrete to center of gravity for composite section

$I_{y\_short}(0\ m) = 0.26\ m^4$ Moment of inertia for composite section

$F_{temp} := \varepsilon_{temp} \cdot E_s \cdot A_{sl}(0\ m) = \begin{bmatrix} -6229 \\ 5043 \end{bmatrix} kN$ Force on composite section

$e_{temp\_F} := z_{tp\_short}(0\ m) = 185\ mm$ Level at which the force is imposed on the system

$e_{temp\_M} := z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) = 1.495\ m$ Eccentricity for the bending moment

$$M_{temp} := F_{temp} \cdot e_{temp\_M} = \begin{bmatrix} -9314 \\ 7540 \end{bmatrix} kN \cdot m$$

Bending moment - composite section

Anchorage of temperature load imposed on studs

Concrete

$$N_{c\_temp} := F_{temp} \cdot \left(\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{A_{sl\_short}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_short}}}{I_{y\_short}(0\ m)} \cdot \left(\left(z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) \right) \cdot \left(z_{tp\_short}(0\ m) + \frac{h_{m.slab}}{2} \right) \right) \right) = \begin{bmatrix} -1755 \\ 1420 \end{bmatrix} kN$$

$$M_{c\_temp} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_short}} \cdot h_{m.slab}^{3}}{I_{y\_short}(0\ m)} \cdot M_{temp} = \begin{bmatrix} -82 \\ 66 \end{bmatrix} kN \cdot m$$

$$\sigma_1 := \frac{N_{c\_temp}}{A_{slab.fic}} - \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{2}} = \begin{bmatrix} -0.14 \\ 0.11 \end{bmatrix} MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{N_{c\_temp}}{A_{slab.fic}} + \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{2}} = \begin{bmatrix} -2.09 \\ 1.69 \end{bmatrix} MPa$$

Stress in concrete, lower edge of slab

$$N_{temp} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = \begin{bmatrix} -1755 \\ 1420 \end{bmatrix} kN$$

Force imposed on studs caused by shrinkage (used in chapter 6 when calculating the anchorage of the slab by the studs)

5 Capacity checks during construction

The capacity check that is carried out in this chapter is bending moment capacity with respect to lateral torsional buckling in the casting phase

In the casting phase the normalforces in the cross-section are neglectible.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

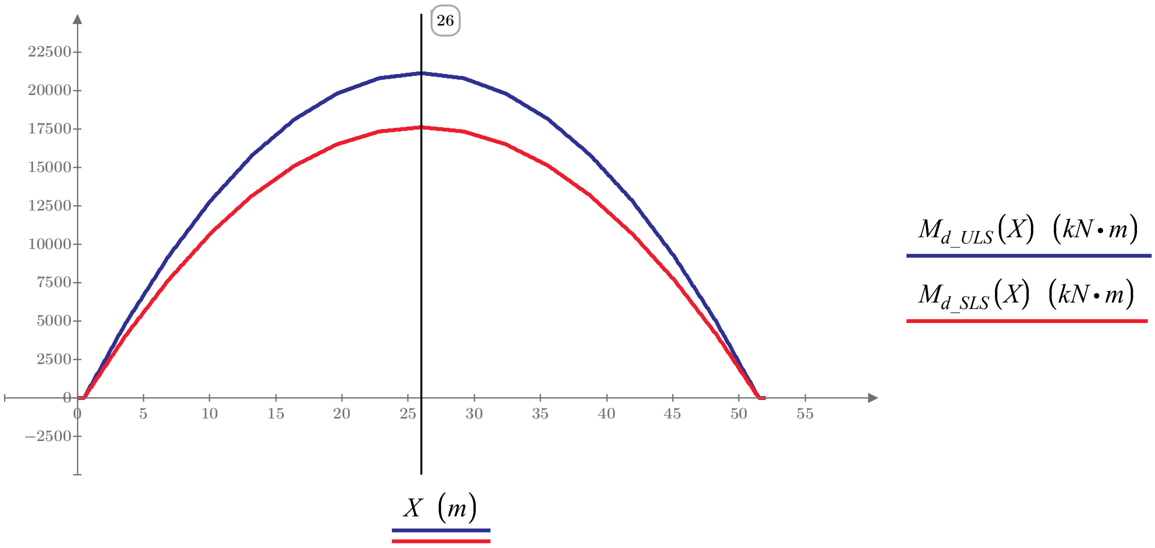
$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

5.1 Load effects

Load effects retrieved from Strip-Step2, Appendix X.

Bending moment

M_{d_ULS} Bending moment descending from permanent loads during construction in ultimate limit state
M_{d_SLS} Bending moment descending from permanent loads during construction in service limit state

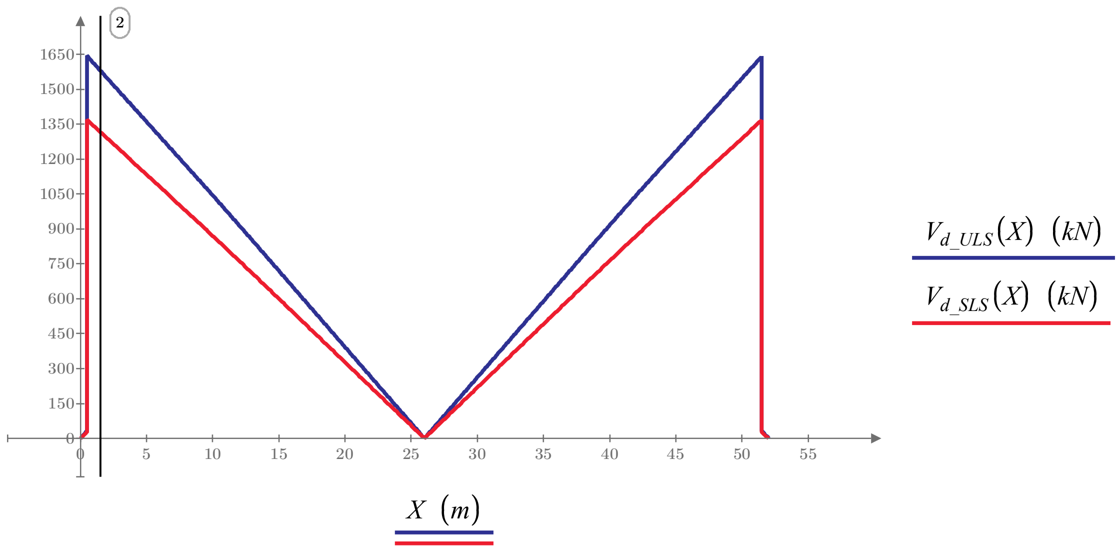


$M_{d\_ULS}(X_{check\_m}) = 21141\ kN \cdot m$ Bending moment at control point - ULS

$M_{d\_SLS}(X_{check\_m}) = 17617\ kN \cdot m$ Bending moment at control point - SLS

Shear force

V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
V_{d_SLS}	Shear force descending from permanent loads during construction in service limit state



$V_{d\_ULS}(X_{check\_v}) = 1581\ kN$ Shear force at control point - ULS

$V_{d\_SLS}(X_{check\_v}) = 1317\ kN$ Shear force at control point - SLS

5.2 Redesign of cross-section

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange
f_{yw}	Chosen yield strength - web

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)	f_{yw_1}
0	35	725	7	2475	50	700	f_{y_10mm}
500	35	725	7	2475	50	700	f_{y_10mm}

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)	f_{yw_2}	
0	35	725	7	2475	50	700	f_{y_10mm}	
11500	35	725	7	2475	50	700	f_{y_10mm}	10500
11500	45	725	5	2460	55	700	$f_{y\_6.4mm}$	10500
12300	45	725	5	2460	55	925	$f_{y\_6.4mm}$	11300
25500	45	725	5	2460	55	925	$f_{y\_6.4mm}$	

Element 3

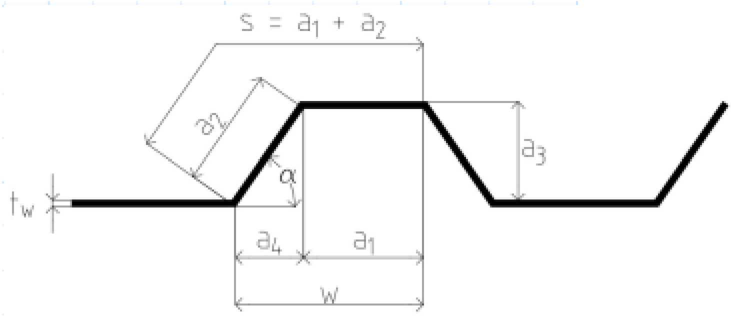
S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)	f_{yw_3}
0	35	725	7	2475	50	700	f_{y_10mm}
500	35	725	7	2475	50	700	f_{y_10mm}

$$h_{tot1} := mean\left(t_{fu\_1} + h_{w\_1} + t_{fl\_1}\right) = 2560\ mm$$

$$h_{tot2} := mean\left(t_{fu\_2} + h_{w\_2} + t_{fl\_2}\right) = 2560\ mm$$

Check to see that the girder height is kept constant

5.2.1 Shape of corrugation



$a_{c1} := 124\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 73\ mm$ Corrugation depth

$a_{weld} := 5\ mm$ Weld throat thickness

$a_{c2} := \dfrac{a_{c3}}{sin\left(\alpha_c\right)} = 124\ mm$ Legnth of hypotenuse

$a_{c4} := \dfrac{a_{c3}}{tan\left(\alpha_c\right)} = 100\ mm$ Length of corrugation depth, see figure

$s_c := a_{c1} + a_{c2} = 248\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 224\ mm$ Straight length

$r_c := \dfrac{s_c}{w_c} = 1.106$ Ratio corrugation/straight length

5.2.2 Cross-section classification

The cross-section classes is determined according to SS-EN 1993 1-4 5.2.2 with updated limits from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated and does not contribute to the axial stiffness the web is not classified.

$$c_w(x) := h_w(x) - 2 \cdot \sqrt{2} \cdot a_{weld}$$

$$c_{uf}(x) := \frac{b_{fu}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on upper flange

$$c_{lf}(x) := \frac{b_{fl}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on lower flange

Cross-section class, upper flange

$E_s = 200\ GPa$ Modulus of elasticity

$f_{yuf} = 450\ MPa$ Proof strength of top flange

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

SS-EN 1993-1-4 5.2.2 Table 5.2

$$csc_{uf}(x) := \begin{array}{l} \text{if } \dfrac{c_{uf}(x)}{t_{fu}(x)} \le 9\ \varepsilon_{uf} \\ \quad \text{“csc1”} \\ \text{else if } 9\ \varepsilon_{uf} < \dfrac{c_{uf}(x)}{t_{fu}(x)} \le 10\ \varepsilon_{uf} \\ \quad \text{“csc2”} \\ \text{else if } 10\ \varepsilon_{uf} < \dfrac{c_{uf}(x)}{t_{fu}(x)} \le 14\ \varepsilon_{uf} \\ \quad \text{“csc3”} \\ \text{else} \\ \quad \text{“csc4”} \end{array}$$

Cross-section class upper flange

$csc_{uf}(X_{check\_m}) = \text{“csc3”}$ Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{uf}(X_{check\_v}) = \text{“csc4”}$ Cross-section class at $X_{check\_v} = 1.500\ m$

Cross-section class, lower flange

$E_s = 200\ GPa$ Modulus of elasticity

$f_{ylf} = 450\ MPa$ Proof strength of top flange

$\varepsilon_{lf} := \sqrt{\dfrac{235}{f_{ylf}} \cdot \dfrac{E_s}{210000}} = 0.71$ SS-EN 1993-1-4 5.2.2 Table 5.2

Cross-section class lower flange

$$csc_{lf}(x) := \begin{array}{l} \text{if } \dfrac{c_{lf}(x)}{t_{fl}(x)} \le 9\ \varepsilon_{lf} \\ \quad \text{"csc1"} \\ \text{else if } 9\ \varepsilon_{lf} < \dfrac{c_{lf}(x)}{t_{fl}(x)} \le 10\ \varepsilon_{lf} \\ \quad \text{"csc2"} \\ \text{else if } 10\ \varepsilon_{lf} < \dfrac{c_{lf}(x)}{t_{fl}(x)} \le 14\ \varepsilon_{lf} \\ \quad \text{"csc3"} \\ \text{else} \\ \quad \text{"csc4"} \end{array}$$

$csc_{lf}(X_{check\_m}) = \text{"csc3"}$ Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{lf}(X_{check\_v}) = \text{"csc3"}$ Cross-section class at $X_{check\_v} = 1.500\ m$

5.2.3 Plate buckling of compressive flange

If the compressed flange is in cross-section class four an effective width of the compressed flange is calculated according to SS-EN 1993-1-4 5.2.3. Updated values are taken from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated the buckling factor k_σ is calculated according to SS-EN 1993-1-5 D.2.1 (2)

$a_{bend} := a_{c1} + 2\ a_{c4} = 325\ mm$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$c_u := c_{uf}\left(X_{check\_m}\right) = 389\ mm$ — Width of outstand flange from weld toe to free edge

$t_u := t_{fu}\left(X_{check\_m}\right) = 45\ mm$ — Thickness of upper flange

$b_u := b_{fu}\left(X_{check\_m}\right) = 725\ mm$ — Width of upper flange

$\varepsilon_{uf} = 0.71$

$k_{\sigma 1} := 0.43 + \left(\frac{c_u}{a_{bend}}\right)^2 = 1.9$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_{\sigma 2} := 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_\sigma := min\left(k_{\sigma 1}, k_{\sigma 2}\right) = 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

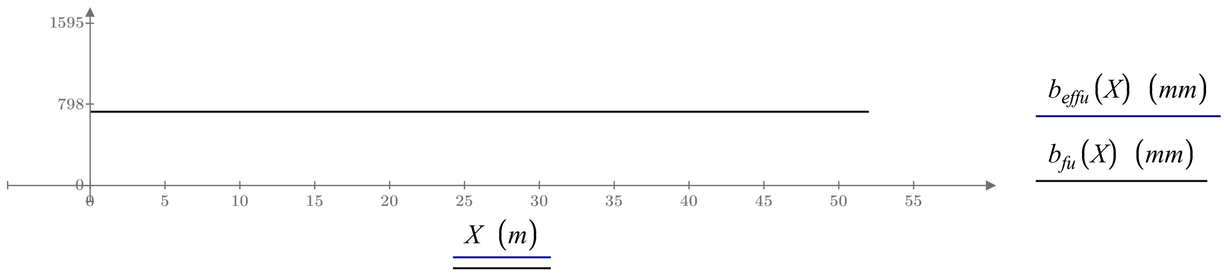
$$\lambda_p := \frac{\frac{c_u}{t_u}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_\sigma}} = 0.56$$

Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho := \mathbf{if}\left(\lambda_p \le 0.748, 1.0, \frac{\lambda_p - 0.188}{\lambda_p^{\ 2}}\right) = 1.00$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

$b_{eff} := b_u \cdot \rho = 725\ mm$ = $b_{effu}\left(X_{check\_m}\right) = 725\ mm$



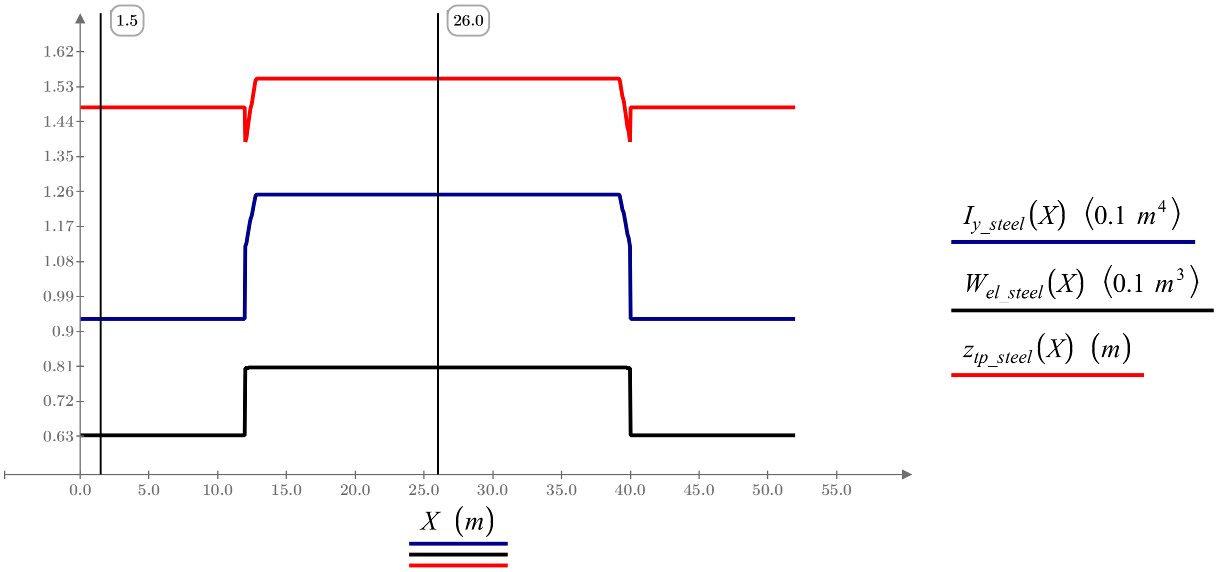
$b_{fu}(x) := b_{effu}(x)$ — Renaming the width of flange in order to minimize errors

5.2.6 New cross-sectional constants during casting

$I_{y\_steel}(x)$ Stiffness of steel girder alone (excluding the web)

$z_{tp\_steel}(x)$ Distance from the top of the top flange to the center of gravity for the steel section

$W_{el\_steel}(x)$ Elastic bending stiffness of steel girder alone (excluding the web)



Coordinates and cross-sectional constants for control calculations

$X_{check\_m} = 26\ m$ X- coordinate for control calculations, bending moment

$A_{sl}(X_{check\_m}) = 0.084\ m^2$ Area

$I_{y\_steel}(X_{check\_m}) = (125 \cdot 10^{-3})\ m^4$ Stiffness

$z_{tp\_steel}(X_{check\_m}) = 1.552\ m$ Center of gravity

$W_{el\_steel}(X_{check\_m}) = (81 \cdot 10^{-3})\ m^3$ Elastic bending resistance

$X_{check\_v} = 1.5\ m$ X- coordinate for control calculations, shear force

$A_{sl}(X_{check\_v}) = 0.06\ m^2$ Area

$I_{y\_steel}(X_{check\_v}) = (93 \cdot 10^{-3})\ m^4$ Stiffness

$z_{tp\_steel}(X_{check\_v}) = 1.477\ m$ Center of gravity

$W_{el\_steel}(X_{check\_v}) = (63 \cdot 10^{-3})\ m^3$ Elastic bending resistance

5.2.4 Lateral torsional buckling of compressive flange - SS-EN 1993-1-4 5.4.2.1

Simplified method only consisdering buckling of top flange according to SS-EN 1993-1-4 5.4.2.1

$\alpha_{LT} := 0.76$ — Buckling curve d, welded open cross-section

$l_{cr} := 7.29\ m$ — Distance between the cross-beams

$b_{ef} := b_{effu}\left(X_{check\_m}\right) = 725\ mm$ — Width of upper flange

$t_f := t_{fu}\left(X_{check\_m}\right) = 45\ mm$ — Thickness of upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength

$$I_{zf} := \frac{b_{ef}^{\ 3} \cdot t_f}{12} = 0.001\ m^4$$ Moment of inertia, upper flange

$$N_{crLT} := \frac{\pi^2 \cdot E_s \cdot I_{zf}}{l_{cr}^{\ 2}} = 53079\ kN$$ Critical buckling load

$$\lambda_{LT\_u} := \sqrt{\frac{b_{ef} \cdot t_f \cdot f_{yuf}}{N_{crLT}}} = 0.526$$ SS-EN 1993-1-4 5.4.2.1 Equation 5.9

$$\Phi_{LT\_u} := 0.5 \cdot \left(1 + \alpha_{LT} \cdot \left(\lambda_{LT\_u} - 0.2\right) + \lambda_{LT\_u}^{\ 2}\right) = 0.76$$ SS-EN 1993-1-4 5.4.2.1 Equation 5.7

$$\chi_{LT\_u} := min\left(\frac{1}{\Phi_{LT\_u} + \sqrt{\Phi_{LT\_u}^2 - \lambda_{LT\_u}^{\ 2}}}, 1\right) = 0.761 \quad = \quad \chi_{LT}\left(X_{check\_m}\right) = 0.761$$

SS-EN 1993-1-4 5.4.2.1 Equation 5.6

$h_{w\_u} := h_w\left(X_{check\_m}\right) = 2460\ mm$ — Height of web

$t_l := t_{fl}\left(X_{check\_m}\right) = 55\ mm$ — Thickness of lower flange

$k_{fl} := 1.1$ — Increase in capacity due to simlified method used, SS-EN 1993-1-1 6.3.2.4 (2)B

$$M_{Rd.u.LT\_u} := \frac{b_{ef} \cdot t_f \cdot k_{fl} \cdot \chi_{LT\_u} \cdot f_{yuf}}{\gamma_{M1}} \left(h_{w\_u} + \frac{t_f + t_l}{2}\right) = 30854\ kN \cdot m \quad = \quad M_{Rd.u.LT}\left(X_{check\_m}\right) = 30854\ kN \cdot m$$

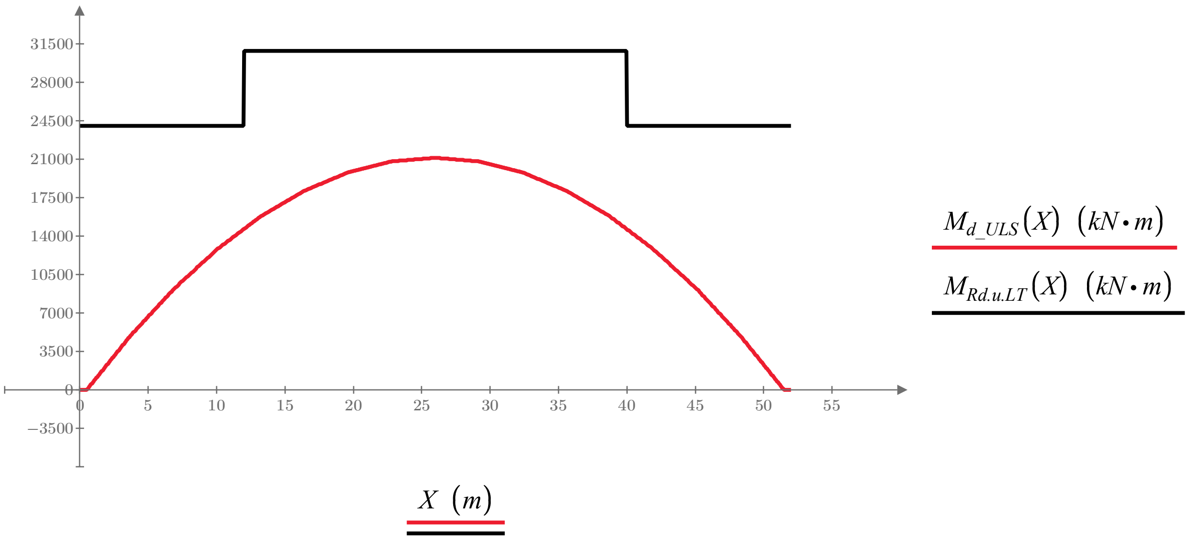
SS-EN 1993-1-5 D.2.1 Equation D.1

5.2.5 Check of lateral torsional buckling

Check of the buckling capacity of the girders is performed.

$M_{Rd.u.LT}(X)$ Capacity of the girder against LT- buckling during casting

$M_{d\_ULS}(X)$ Load effects during casting, Appendix X



$$\eta_{max} := max\left(\frac{M_{d\_ULS}(X)}{M_{Rd.u.LT}(X)}\right) = 69\%$$

Maximum utilization rate of the steel cross-section during casting

5.3 Stresses in steel cross-section

The stresses in the top and bottom flange is calculated for the loading senario to be able to superposition them with the other loadcases to determinte the ultimate capacity of the composite section.

Upper flange

Control calculation at midspan

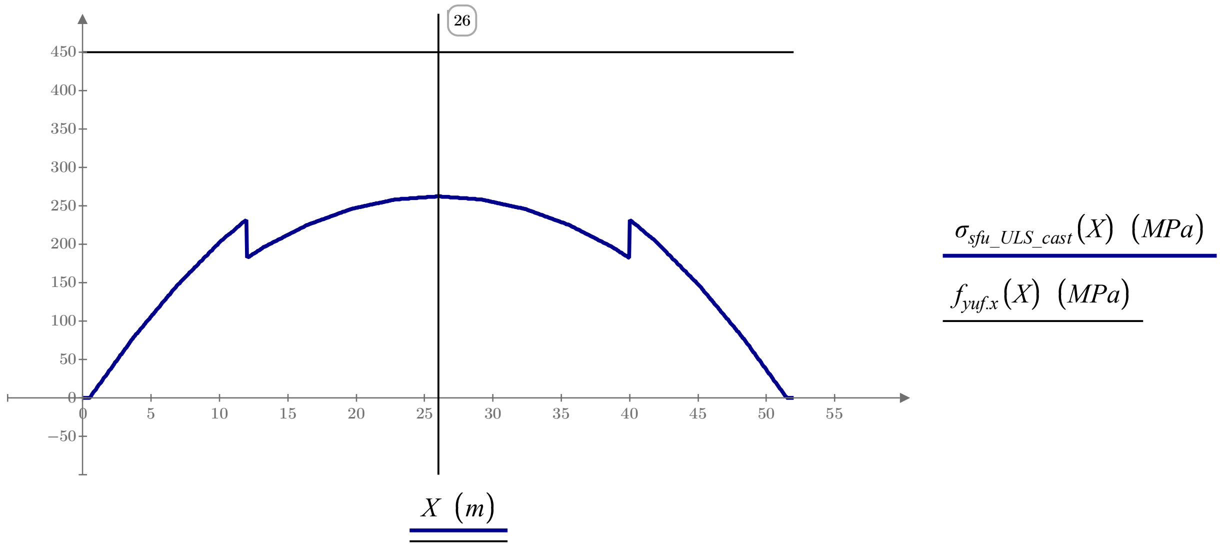
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 21141\ kN \cdot m$ Load effext at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.125\ m^4$ Moment of inertia at midspan

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 1.552\ m$ Centre of gravity at midspan maesured from the top of the beam

$\sigma := \dfrac{M}{I} \cdot z = 262\ MPa$ = $\sigma_{sfu\_ULS\_cast}\left(X_{check\_m}\right) = 262\ MPa$

$\sigma_{sfu\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\dfrac{\sigma_{sfu\_ULS\_cast}(X)}{f_{yuf}}\right) = 58\%$

Lower flange

Check calculation at midspan

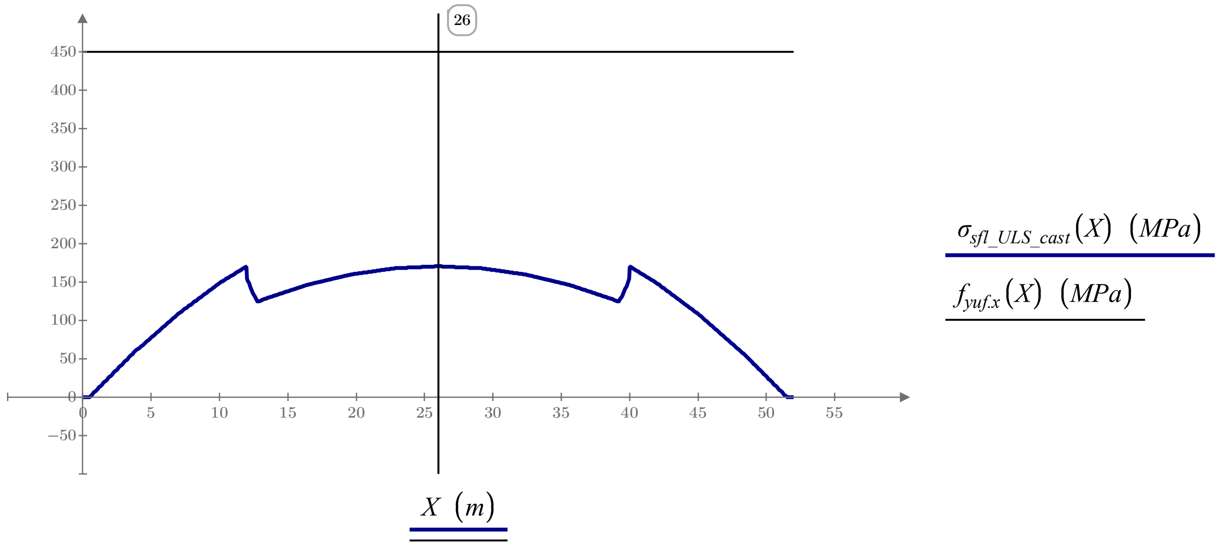
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 21141\ kN \cdot m$ Load effect at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.125\ m^4$ Moment of inertia at midspan

$z := z_{tplf\_steel}\left(X_{check\_m}\right) = 1.008\ m$ Centre of gravity at midspan measured from the bottom of the beam

$\sigma := \frac{M}{I} \cdot z = 170\ MPa$ = $\sigma_{sfl\_ULS\_cast}\left(X_{check\_m}\right) = 170\ MPa$

$\sigma_{sfl\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\frac{\sigma_{sfl\_ULS\_cast}(X)}{f_{ylf}}\right) = 38\%$

6 Capacity checks - Ultimate limit state, global

The capacity checks that are to be carried out are bending moment capacity, shear capacity, web breathing and design of studs, both in ULS and due to fatigue

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 0.5\ m$ Coordinate for control calculations - shear force

6.1 Load effects in ULS

6.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

Bending moment in the ultimate limit state

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{tr}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

M_{d_ULS} Design bending moment effect from all loads

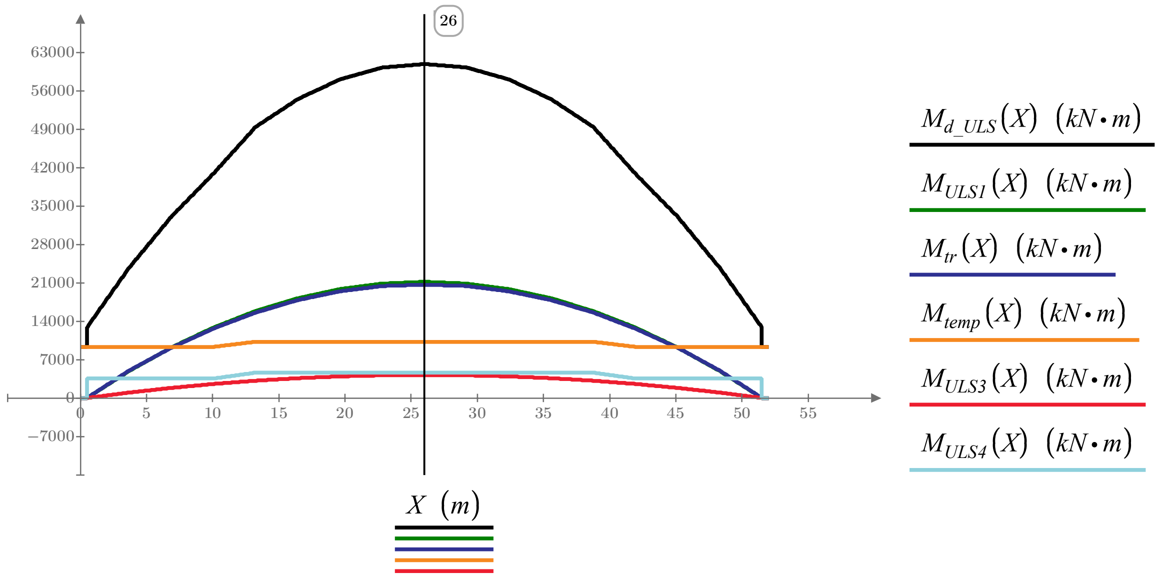
$M_{ULS1}(x)$ Bending moment during casting (Appendix X)

$M_{tr}(x)$ Bending moment from multi component loads (Appendix X)

$M_{tr}(x)$ Bending moment from temperature loads (Appendix X)

$M_{ULS3}(x)$ Bending moment from additional permanent loads after construction (Appendix X)

$M_{ULS4}(x)$ Bending moment from shrinkage (Appendix X)

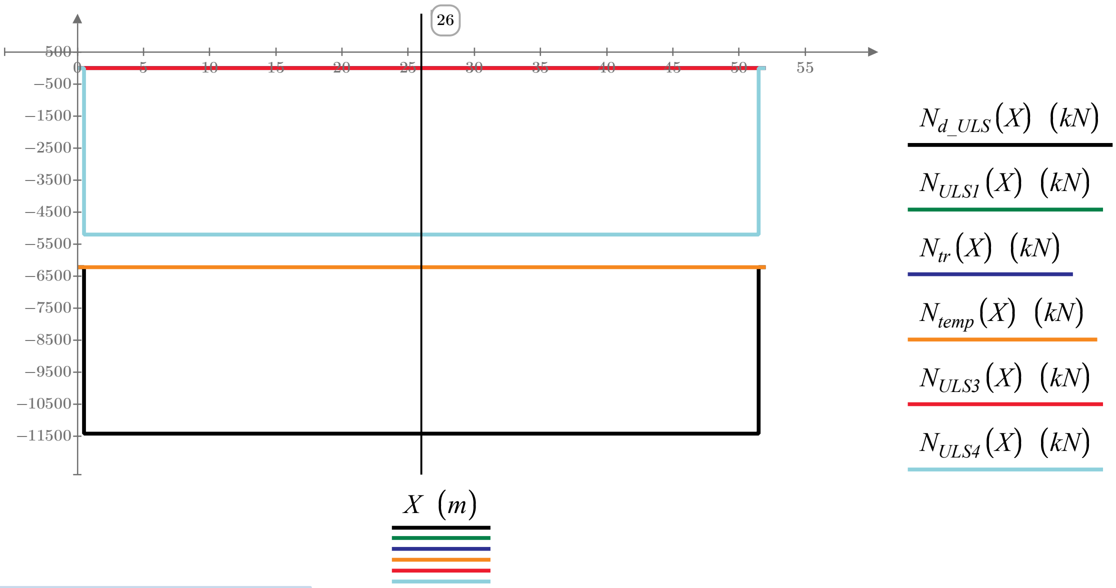


$M_{d\_ULS}(X_{check\_m}) = 60859\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{tr}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design normal force from all loads
$N_{ULS1}(x)$	Normal force during casting (Appendix X)
$N_{tr}(x)$	Normal force from multi components loads (Appendix X)
$N_{temp}(x)$	Normal force from temperature loads (Appendix X)
$N_{ULS3}(x)$	Normal force from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Normal force from shrinkage (Appendix X)

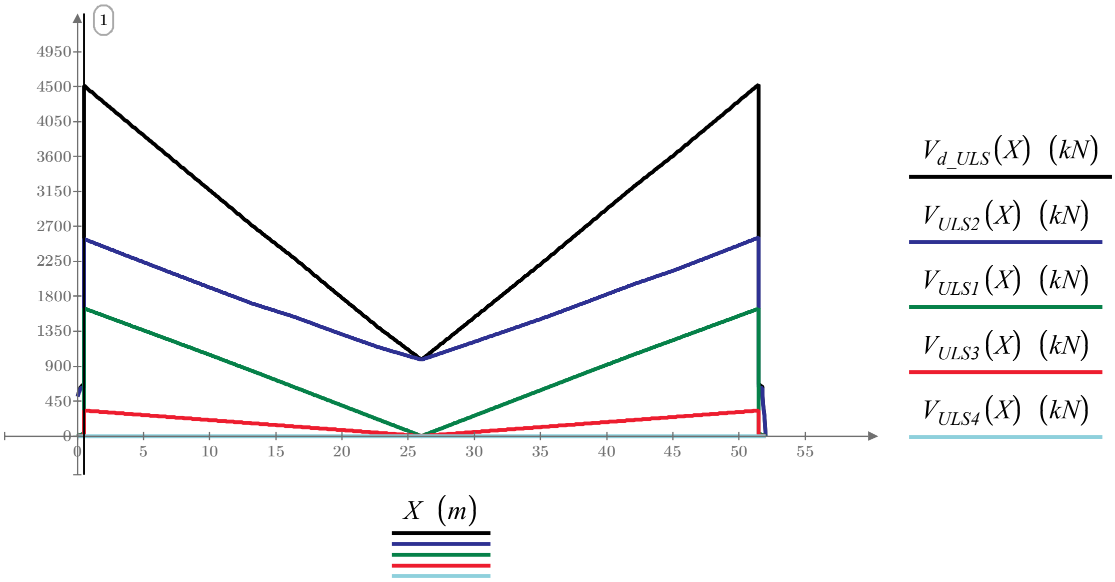


$N_{d\_ULS}(X_{check\_m}) = -11435\ kN$ Normal force at control point - ULS

6.1.2 Shear force

$$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{ULS3}(x) + V_{ULS4}(x)$$

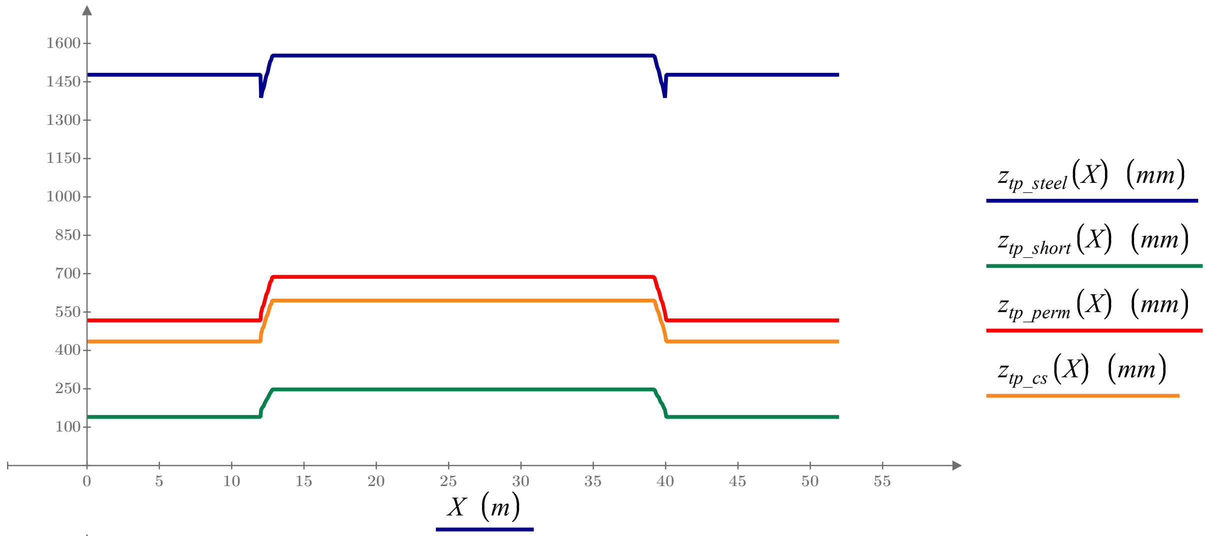
V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (both temperature and traffic) (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)

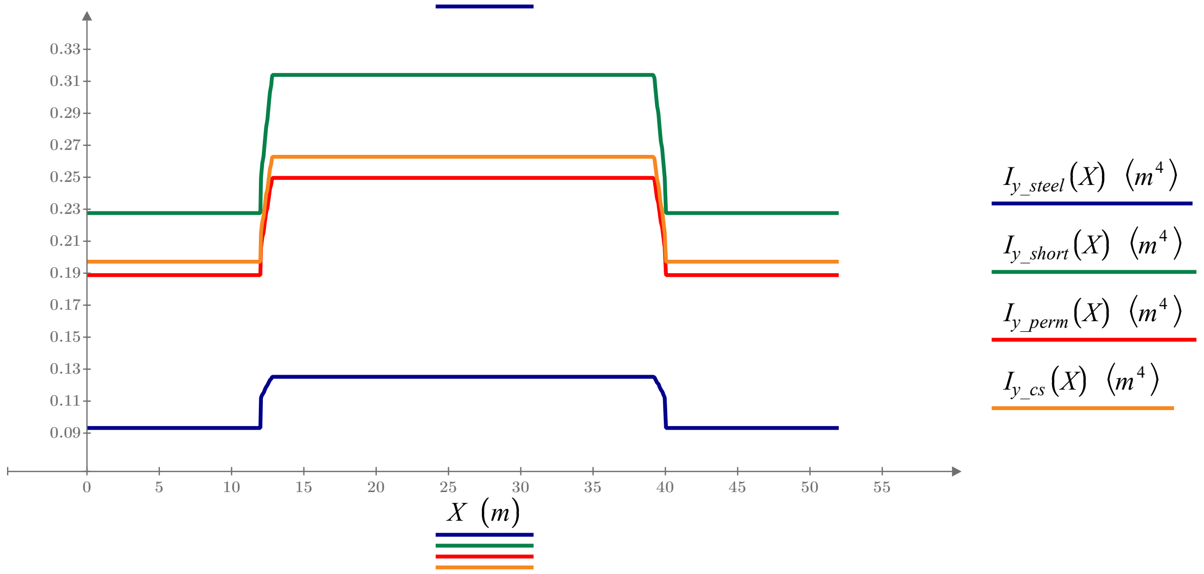


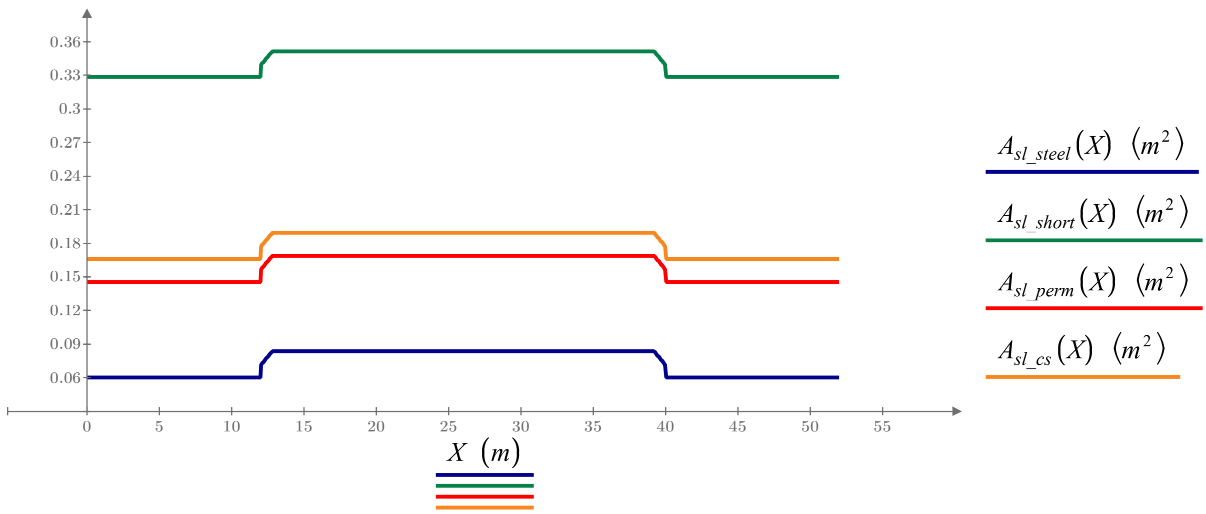
$V_{d\_ULS}(X_{check\_v}) = 4512\ kN$ Shear force at control point - ULS

6.2 Cross-sectional constants

For calculations, see chapter 5.







6.3 Stresses in steel cross-section

The stresses is calculated for each load case taking into acount load duration and creep. The stresses are then superpositioned.

6.3.1 Stresses during casting

$M := M_{ULS1}\left(X_{check\_m}\right) = 21166\ kN \cdot m$ Bending moment

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.125\ m^4$ Moment of inertia

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 1552\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2560\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot -z = -262\ MPa$ = $\sigma_{s.u.cast}\left(X_{check\_m}\right) = -262\ MPa$ Stresses in upper flange from loads durng casting

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 170\ MPa$ = $\sigma_{s.l.cast}\left(X_{check\_m}\right) = 170\ MPa$ Stresses in lower flange from loads durng casting

6.3.2 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses must be calculated for that specific section. The worst load case is determined dependent on the largest stress in each part whereas the multi-component load is the main load for the lower flange, and the temperature load is the worst load case for the upper flange.

6.3.2.1 Multi component loads (traffic)

$M := M_{tr}\left(X_{check\_m}\right) = 20646\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = -0.04\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.351\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.314\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) = 247\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2560\ mm$ Height of girder

$\sigma_{s.u} := \frac{0.9\ N}{A} + \frac{0.9\ M}{I} \cdot (-z) = -15\ MPa$ = $\sigma_{s.u.tr}\left(X_{check\_m}\right) = -15\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{1.5\ N}{A} + \frac{1.5\ M}{I} \cdot (h - z) = 228\ MPa$ = $\sigma_{s.l.tr}\left(X_{check\_m}\right) = 228\ MPa$ Stresses in lower flange from short term loads

6.3.2.2 Temperature loads

$M := M_{temp}(X_{check\_m}) = 10242\ kN \cdot m$ Bending moment

$N := N_{temp}(X_{check\_m}) = -6229\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.084\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_short}(X_{check\_m}) = 0.351\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.125\ m^4$ Moment of inertia - steel section

$I := I_{y\_short}(X_{check\_m}) = 0.314\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 1552\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_short}(X_{check\_m}) = 247\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 2560\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 4085\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(1 - \left(\frac{A_s}{A} + \frac{A_s}{I} \left(z_s - z\right)^2\right)\right) = -1928\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{1.5\ N_s}{A_s} + \frac{1.5\ M_s}{I_s} \cdot \left(-z_s\right) = -111\ MPa$ = $\sigma_{s.u.temp}(X_{check\_m}) = -111\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{0.9\ N_s}{A_s} + \frac{0.9\ M_s}{I_s} \cdot \left(h - z_s\right) = 9\ MPa$ = $\sigma_{s.l.temp}(X_{check\_m}) = 9\ MPa$ Stresses in lower flange from short term loads

6.3.3 Stresses due to additional permanent loads

$M := M_{ULS3}(X_{check\_m}) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}(X_{check\_m}) = 0.25\ m^4$ Moment of inertia

$z := z_{tp\_perm}(X_{check\_m}) = 687\ mm$ Center of gravity from top flange

$h := h_{beam}(X_{check\_m}) = 2560\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot (-z) = -12\ MPa$ = $\sigma_{s.u.perm}(X_{check\_m}) = -12\ MPa$ Stresses in upper flange from additional permanent loads

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 31\ MPa$ = $\sigma_{s.l.perm}(X_{check\_m}) = 31\ MPa$ Stresses in lower flange from additional permanent loads

6.3.4 Stresses due to shrinkage

In the same manner as the temperature load must be calculated for the steel section, the shrinkage which acts on the concrete section must be converted for the steel section.

$M := M_{ULS4}(X_{check\_m}) = 4612\ kN \cdot m$ Bending moment

$N := N_{ULS4}(X_{check\_m}) = -5206\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.084\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_cs}(X_{check\_m}) = 0.189\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.125\ m^4$ Moment of inertia - steel section

$I := I_{y\_cs}(X_{check\_m}) = 0.263\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 1552\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_cs}(X_{check\_m}) = 595\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 2560\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 2197\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(\frac{A_s}{A} - \frac{A_s}{I} \cdot \left(z - \frac{h_{m.slab}}{2} \right) \cdot \left(z_s + \frac{h_{m.slab}}{2} - \left(z - \frac{h_{m.slab}}{2} \right) \right) \right) = -1378\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (-z_s) = -44\ MPa$ = $\sigma_{s.u.shrink}(X_{check\_m}) = -44\ MPa$ Stresses in upper flange due to shrinkage

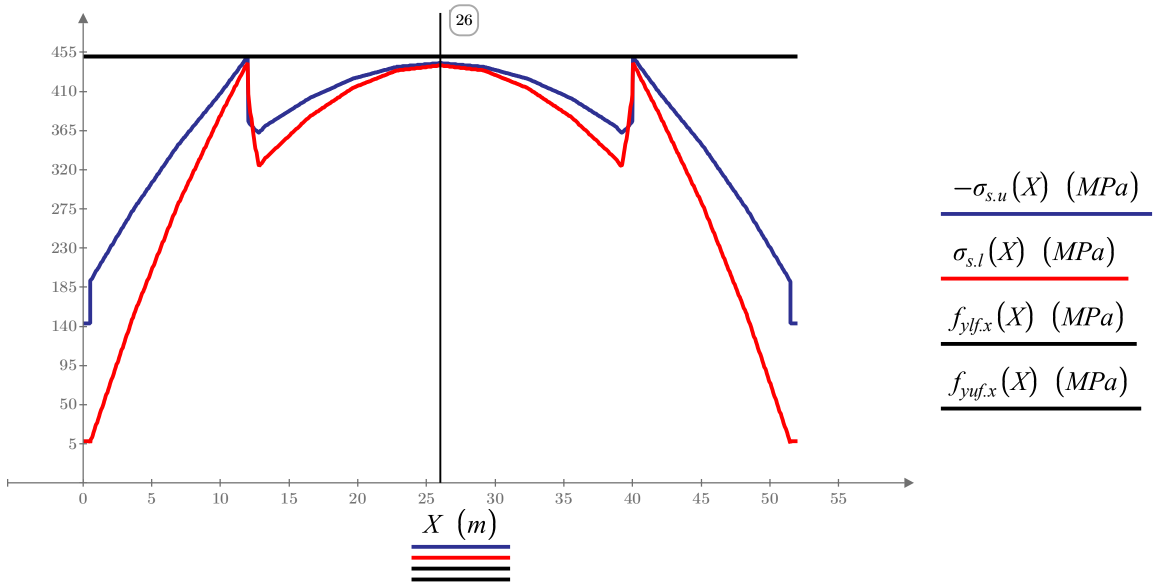
$\sigma_{s.l} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (h - z_s) = 1\ MPa$ = $\sigma_{s.l.shrink}(X_{check\_m}) = 1\ MPa$ Stresses in lower flange due to shrinkage

6.3.5 Summary of stresses

The stresses from the different phases are summarised accordingly.

$$\sigma_{s.u}(x) := \sigma_{s.u.cast}(x) + \sigma_{s.u.shrink}(x) + \sigma_{s.u.perm}(x) + \sigma_{s.u.tr}(x) + \sigma_{s.u.temp}(x)$$ Stresses in upper flange

$$\sigma_{s.l}(x) := \sigma_{s.l.cast}(x) + \sigma_{s.l.shrink}(x) + \sigma_{s.l.perm}(x) + \sigma_{s.l.tr}(x) + \sigma_{s.l.temp}(x)$$ Stresses in lower flange



$$\eta_{\sigma.u.max} := max\left(\frac{-\sigma_{s.u}(X)}{f_{yuf.x}(X)}\right) = 100\%$$ Maximum utilization rate - upper flange

$$\eta_{\sigma.l.max} := max\left(\frac{\sigma_{s.l}(X)}{f_{ylf.x}(X)}\right) = 98\%$$ Maximum utilization rate - lower flange

6.4 Calculation of stresses in concrete

The stresses in the concrete is calculated with the same principle as the steel taking creep into account.

6.4.1 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses for the concrete must be calculated separately.

6.4.1.1 Traffic and wind loads

$M := M_{tr}\left(X_{check\_m}\right) = 20646\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.351\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.314\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) + h_{m.slab} = 567\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2560\ mm$ Height of girder

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$\sigma_c := \left(\frac{N}{A} + \frac{-M \cdot z}{I}\right) \cdot \frac{1}{n_\Gamma} = -6\ MPa$ = $\sigma_{c.short}\left(X_{check\_m}\right) = -6\ MPa$ Stresses in concrete from short-term loads

6.4.1.2 Temperature load

$M := M_{temp}\left(X_{check\_m}\right) = 10242\ kN \cdot m$

$N := N_{temp}\left(X_{check\_m}\right) = -6229\ kN$

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.351\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \frac{1}{n_\Gamma} = 0.268\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.314\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}\left(X_{check\_m}\right)}{n_{\Gamma}} \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.002\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 247\ mm$ Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \frac{I_{c.eff}}{I} \cdot M = 74\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(\frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot \left(z_s - z\right) \cdot \left(z + \frac{h_{m.slab}}{2}\right)\right) = -1928\ kN$ Normal force imposed on concrete slab

$\sigma_c := \left(\frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c}\right) = 0.34\ MPa$ = $\sigma_{c.temp}\left(X_{check\_m}\right) = 0.34\ MPa$ Stresses in concrete from temperature loads

6.4.2 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.25\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) + h_{m.slab} = 1007\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2560\ mm$ Height of girder

$n_{\Gamma} := n_{L_1} = 18.48$ Creep

$\sigma_c := \frac{-M \cdot z}{I} \cdot \frac{1}{n_{\Gamma}} = -0.9\ MPa$ = $\sigma_{c.perm}\left(X_{check\_m}\right) = -0.9\ MPa$ Stresses in concrete from permanent loads

6.4.3 Stresses due to shrinkage

$M := M_{ULS4}\left(X_{check\_m}\right) = 4612\ kN \cdot m$ — Bending moment

$N := N_{ULS4}\left(X_{check\_m}\right) = -5206\ kN$ — Normal force

$n_{\Gamma} := n_{L_3} = 14.91$ — Modular ratio considering creep

$A := A_{sl\_cs}\left(X_{check\_m}\right) = 0.189\ m^2$ — Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_{\Gamma}} = 0.106\ m^2$ — Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ — Cross-section area - concrete section

$I := I_{y\_cs}\left(X_{check\_m}\right) = 0.263\ m^4$ — Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \dfrac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ — Moment of inertia - concrete section

$I_{c.eff} := \dfrac{b_{eff}\left(X_{check\_m}\right)}{n_{\Gamma}} \cdot \dfrac{h_{m.slab}^{\ 3}}{12} = 0.001\ m^4$ — Moment of inertia - effective concrete section

$z := z_{tp\_cs}\left(X_{check\_m}\right) = 595\ mm$ — Center of gravity - composite section

$z_c := \dfrac{h_{m.slab}}{2} = 160\ mm$ — Center of gravity - concrete section

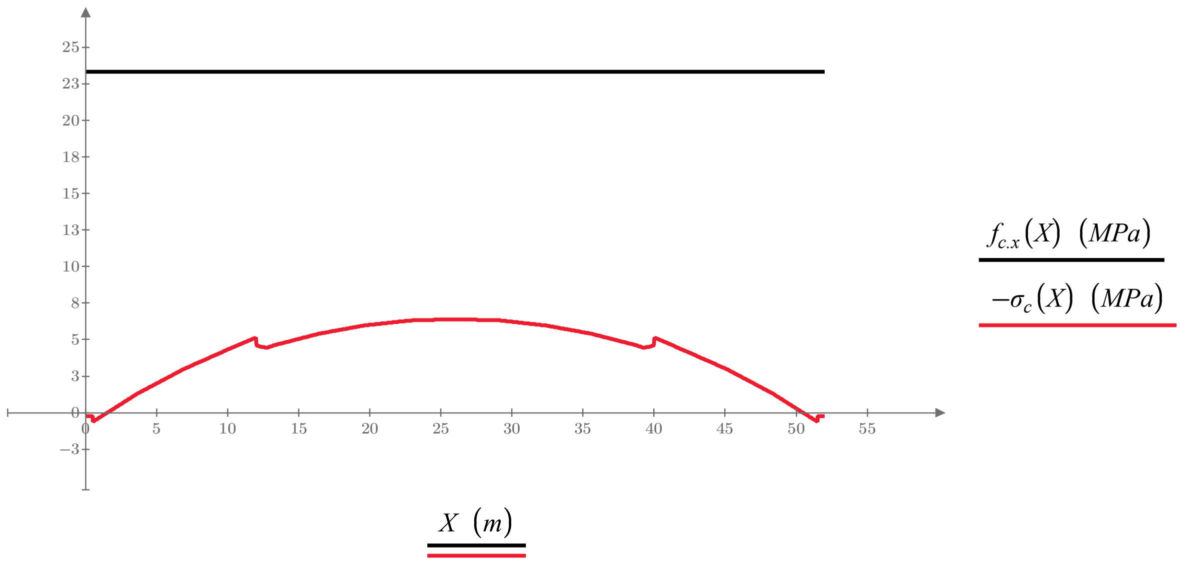
$M_c := \dfrac{I_{c.eff}}{I} \cdot M = 16\ kN \cdot m$ — Bending moment imposed on concrete slab

$N_c := N \cdot \left(1 - \dfrac{A_{c.eff}}{A} - \dfrac{A_{c.eff}}{I} \cdot \left(z + \dfrac{h_{m.slab}}{2}\right)^2\right) = -1102\ kN$ — Normal force imposed on concrete slab

$\sigma_c := \dfrac{-N_c}{A_c} + \dfrac{M_c \cdot -z_c}{I_c} = 0.5\ MPa \quad = \quad \sigma_{c.shrink}\left(X_{check\_m}\right) = 0.5\ MPa$ — Stresses in concrete from shrinkage

6.4.4 Summary of stresses

$\sigma_c(x) := \sigma_{c.short}(x) + \sigma_{c.temp}(x) + \sigma_{c.perm}(x) + \sigma_{c.shrink}(x)$ Stresses in concrete



$\eta_{\sigma.c.max} := max\left(\frac{-\sigma_c(X)}{f_{c.x}(X)}\right) = 27\%$ Maximum utilization rate - upper flange

6.5 Shear capacity

The shear capacity of the corrugated steel girder is calculated accoridng to SS-EN 1993-1-5, Appendix D.
The local- and global buckling factor are calculated according to SS-EN 1993-1-5, Appendix D.

Local buckling factor

$a_{c1} = 124\ mm \qquad a_{c2} = 124\ mm$ — Corrugation geometries

$a_{cmax} := max\left(a_{c1}, a_{c2}\right) = 124\ mm$ — SS-EN 1993-1-5 D.2.2.(2)

$tw := t_w\left(X_{check\_v}\right) = 7\ mm$ — Web thickness

$$\tau_{cr} := 4.83 \cdot E_s \cdot \left(\frac{tw}{a_{cmax}}\right)^2 = 3069\ MPa$$

SS-EN 1993-1-5 D.2.2.(2) Equation D.7

$$\lambda_c := \sqrt{\frac{f_{yw}\left(X_{check\_v}\right)}{\tau_{cr} \cdot \sqrt{3}}} = 0.301$$

SS-EN 1993-1-5 D.2.2.(2) Equation D.6

$$\chi_l := min\left(\frac{1.15}{0.9 + \lambda_c}, 1\right) = 0.96 \quad = \quad \chi_{c.l}\left(X_{check\_v}\right) = 0.96$$

Reduction factor local buckling - SS-EN 1993-1-5 D.2.2.(2) Equation D.5

Global buckling factor

$$D_X := \frac{E_s \cdot tw^3}{12 \cdot \left(1 - \nu^2\right)} \cdot \frac{w_c}{s_c} = 6\ kN \cdot m$$

SS-EN 1993-1-5 D.2.2.(3)

$$D_Z := \frac{E_s \cdot tw \cdot a_{c3}{}^2}{12} \cdot \frac{\left(3\ a_{c1} + a_{c2}\right)}{w_c} = 1374\ kN \cdot m$$

SS-EN 1993-1-5 D.2.2.(3)

$hw := h_w\left(X_{check\_v}\right) = 2475\ mm$ — Web height

$$\tau := \frac{32.4}{tw \cdot hw^2} \cdot \sqrt[4]{\frac{D_X}{N \cdot m} \cdot \frac{D_Z{}^3}{\left(N \cdot m\right)^3}}\ N \cdot m = 263\ MPa$$

SS-EN 1993-1-5 D.2.2.(3) Equation D.10

$$\lambda := \sqrt{\frac{f_{yw}\left(X_{check\_v}\right)}{\tau \cdot \sqrt{3}}} = 1.03$$

SS-EN 1993-1-5 D.2.2.(3) Equation D.9

$$\chi_g := min\left(\frac{1.5}{0.5 + \lambda^2}, 1\right) = 0.97 \quad = \quad \chi_{c.g}\left(X_{check\_v}\right) = 0.97$$

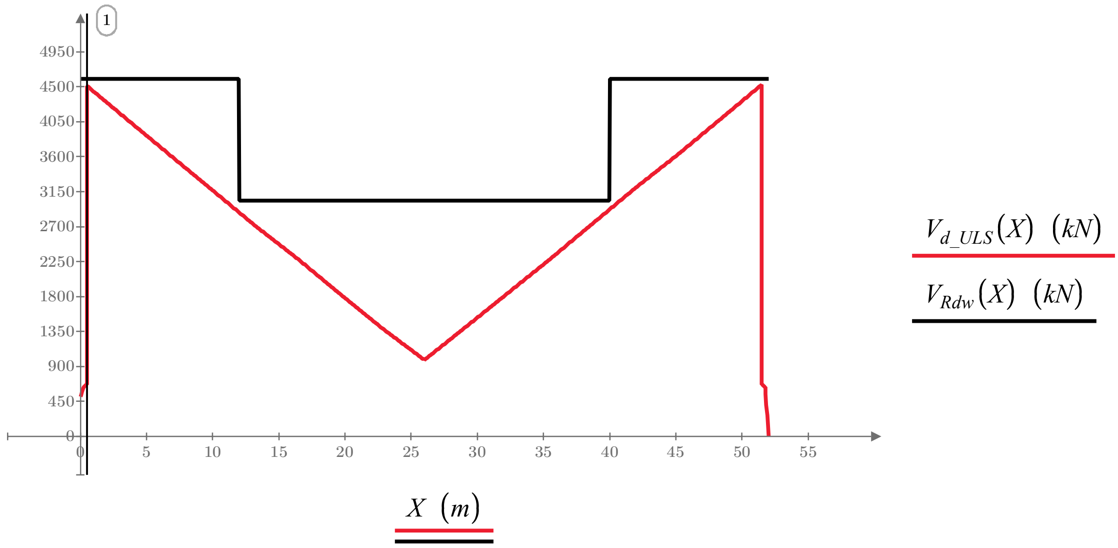
Global buckling factor SS-EN 1993-1-5 D.2.2.(3) Equation D.8

$$\chi_C := min\left(\chi_g, \chi_l\right) = 0.96 \quad = \quad \chi_c\left(X_{check\_v}\right) = 0.96$$

SS-EN 1993-1-5 D.2.2

$$V_{Rd} := \chi_C \cdot \frac{f_{yw}\left(X_{check\_v}\right)}{\gamma_{M1} \cdot \sqrt{3}} \cdot hw \cdot tw = 4599\ kN \quad = \quad V_{Rdw}\left(X_{check\_v}\right) = 4599\ kN$$

SS-EN 1993-1-5 D.2.2.(1) Eq.D.4



$\eta_{V.max} = 98\%$ Maximum utilization rate

6.7 Studs

6.7.1 Ultimate limit state

The capacity of the studs in the ultimate limit state are calculated according to SS-EN 1994-2 6.6.3

$d_{stud} \equiv 22\ mm$ — Diameter of stud

$h_{stud} \equiv 200\ mm$ — Length of stud

$f_{ub} = 800\ MPa$ — Characteristic strength of studs

$f_{u\_stud} := min\left(f_{ub}, 500\ MPa\right) = 500\ MPa$ — Ultimate strength shear stud - SS-EN 1994-2 6.6.3.1 (1)

$\dfrac{h_{stud}}{d_{stud}} = 9$ — Ratio height- diameter

$$\alpha_{stud} := \begin{array}{|l} \text{if } 3 \le \dfrac{h_{stud}}{d_{stud}} \le 4 \\ \quad \| \ a \leftarrow 0.2 \cdot \left(1 + \dfrac{h_{stud}}{d_{stud}}\right) \\ \text{else if } 4 < \dfrac{h_{stud}}{d_{stud}} \\ \quad \| \ a \leftarrow 1 \\ a \end{array} = 1.0$$

Correction factor for length to diameter ratio shear stud SS-EN 1994-2 6.6.3.1 (1)

$$P_{rd} := min\left(\begin{array}{|l} \dfrac{0.8 \cdot f_{u\_stud} \cdot \pi \cdot d_{stud}^{\ 2}}{4 \cdot \gamma_V} \\ \dfrac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^{\ 2} \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \end{array}\right) = 122\ kN$$

Capacity of one shear stud SS-EN 1994-2 6.6.3.1 Eq: 6.18,6.19

$S_{uf\_short}\left(X_{check\_v}\right) = \left(81 \cdot 10^{-3}\right)\ m^3$ — Second moment of area

$V_{ULS2}\left(X_{check\_v}\right) = 2538\ kN$ — Shear force

$I_{y\_short}\left(X_{check\_v}\right) = \left(228 \cdot 10^{-3}\right)\ m^4$ — Moment of inertia

$$\tau_{sh} := \frac{S_{uf\_short}\left(X_{check\_v}\right) \cdot V_{ULS2}\left(X_{check\_v}\right)}{I_{y\_short}\left(X_{check\_v}\right)} = 899\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for short term loads

$S_{uf\_perm}\left(X_{check\_v}\right) = \left(58 \cdot 10^{-3}\right)\ m^3$ — Second moment of area

$V_{ULS3}\left(X_{check\_v}\right)=329\ kN$ Shear force

$I_{y\_perm}\left(X_{check\_v}\right)=\left(189\cdot 10^{-3}\right)\ m^4$ Moment of inertia

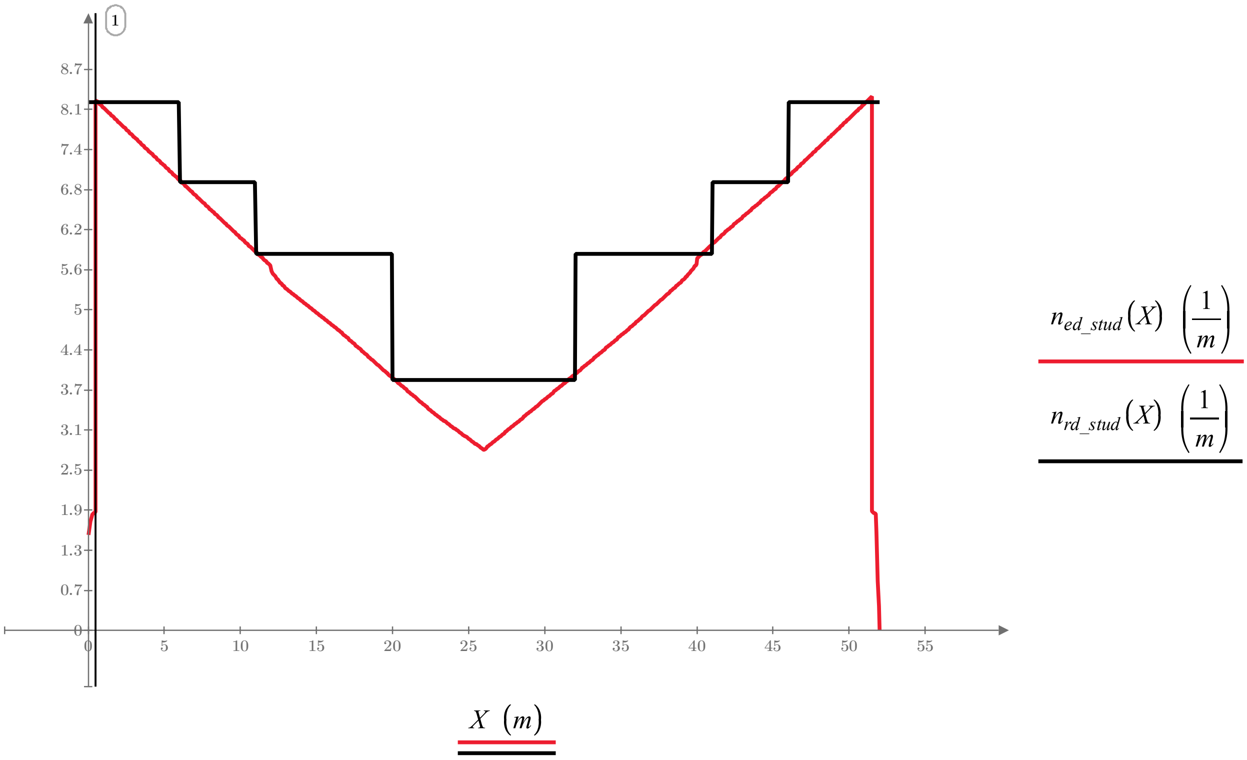
$\tau_{pe}:=\dfrac{S_{uf\_perm}\left(X_{check\_v}\right)\cdot V_{ULS3}\left(X_{check\_v}\right)}{I_{y\_perm}\left(X_{check\_v}\right)}=101\ \dfrac{kN}{m}$ Shear force per meter between concrete and top flange for permanent loads

$\tau:=\left|\tau_{sh}+\tau_{pe}\right|=1000\ \dfrac{kN}{m}$ Total shear force per meter between concrete and top flange

$n_{ed} := \dfrac{\tau + 0.9\ max\left(q_{1k\_br}, q_{tr\_br}\right)}{P_{rd}} = 8.2\ \dfrac{1}{m}$ = $n_{ed\_stud}\left(X_{check\_v}\right) = 8.2\ \dfrac{1}{m}$ Number of studs needed per meter

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



6.7.1.1 Additional studs for full anchorage

Case 1: Temperature and shrinkage causes the slab to contract and therefore they are working in the opposite direction as the shear flow from ULS- loads during bending.

The anchorage length is calculated according to SS-EN 1994-2, 6.9 (3).

$B_{out} = 2.525\ m$ Distance from centre web to outer part of edge beam

$B_{in} = 2.8\ m$ Distance from web to centerline bridge

$b := max\left(B_{out}, B_{in}\right) = 2.8\ m$

$l_{anch} := 1.5 \cdot b = 4.2\ m$ Anchorage length - SS-EN 1994-2, 6.9 (3)

$N_{cs\_stud} = 771\ kN$ Shrinkage force imposed on studs - calculated in chapter 4

$\left|N_{temp\_stud_1}\right| = 1755\ kN$ Temperature force imposed on studs causing contraction - calculated in chapter 4

$$n_{ed\_cs} := \frac{1.0\ N_{cs\_stud}}{l_{anch} \cdot P_{rd}} = 1.5\ \frac{1}{m}$$

$$n_{ed\_temp} := \frac{1.5\ \left|N_{temp\_stud_1}\right|}{l_{anch} \cdot P_{rd}} = 5.1\ \frac{1}{m}$$

$$n_{ed\_anch.1} := n_{ed\_cs} + n_{ed\_temp} = 6.6\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 8.2\ \frac{1}{m}$$

$$check_1 := \mathbf{if}\left(n_{ed\_anch.1} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_1 = \text{“No extra studs are needed”}$$

Case 2: Temperature causes the slab to expand and therefore working in the same direction as the shear flow from ULS- loads during bending. Shrinkage causes the slab to contract, i.e. working in the opposite direction.

$$-n_{ed\_cs} = -1.5\ \frac{1}{m}$$

$$N_{temp\_stud_2} = 1420\ kN$$

Temperature force imposed on studs causing expansion - calculated in chapter 4

$$n_{ed\_temp} := \frac{1.5 \cdot 0.6\ N_{temp\_stud_2}}{l_{anch} \cdot P_{rd}} = 2.5\ \frac{1}{m}$$

$$n_{ed\_stud}(0\ m) = 1.5\ \frac{1}{m}$$

$$n_{ed\_anch.2} := n_{ed\_stud}(0\ m) + n_{ed\_temp} - n_{ed\_cs} = 2.5\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 8.2\ \frac{1}{m}$$

$$check_2 := \mathbf{if}\left(n_{ed\_anch.2} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_2 = \text{“No extra studs are needed”}$$

Extra studs needed due to shrinkage and temperature

$n_{rd\_stud.adj}(X)$ Needed amount of studs with regards to extra anchorage due to temperature and shrinkage

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section (ULS- loads)

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section

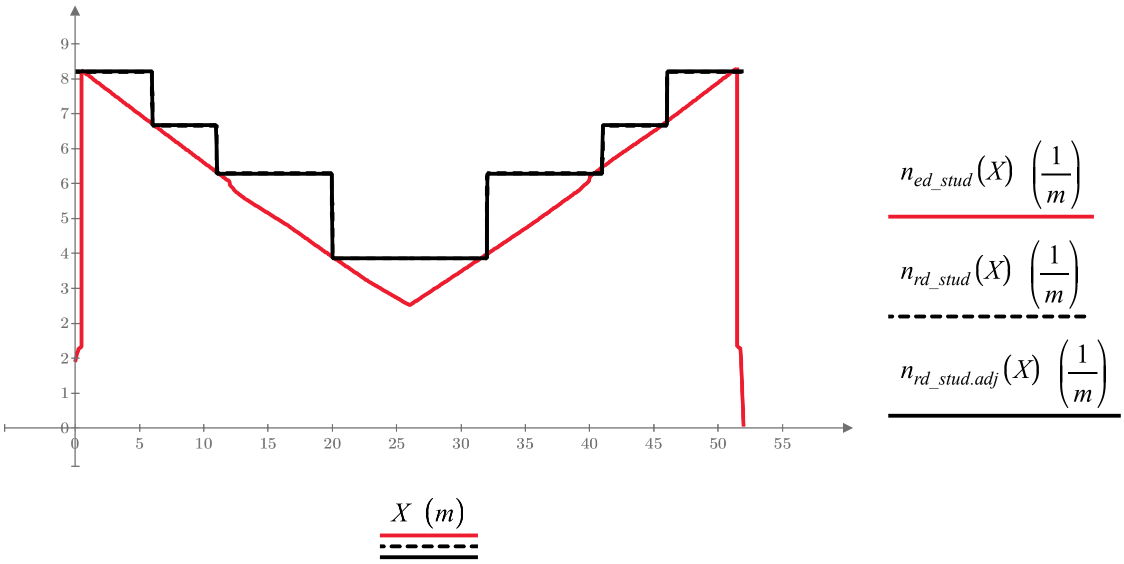


Table. Showing the adjusted need for studs near supports

X	n_{rd}	$n_{rd.adj}$
0	8.2	**8.2**
0.5	8.2	**8.2**
4.2	8.2	**8.2**
6	7.3	**7.3**
11	6.2	**6.2**
20	4.3	**4.3**

6.7.2 Fatigue limit state

6.7.2.1 Capacity

The fatigue capacity is calculated according to SS-EN 1993-1-9, Table 8.5 and SS-EN 1994-2, 6.8.3.

$\Delta\tau_c := 90\ MPa$ SS-EN 1993-1-9 - Table 8.5 (10)

$\Delta\tau_{E2} = \lambda_v \cdot \Delta\tau_c$

$\lambda_{v.1} := 1.55$ Bridge length less than 100m - SS-EN 1994-2 6.8.6.2 (4)

$Q_{mi} := 410\ kN$ Mean weight of large vehicles in the slow lane

$Q_0 := 480\ kN$

$N_{obs} := 0.05 \cdot 10^6$

$N_0 := 0.5 \cdot 10^6$

$\lambda_{v.2} := \dfrac{Q_{mi}}{Q_0} \cdot \left(\dfrac{N_{obs}}{N_0}\right)^{\frac{1}{8}} = 0.64$ SS-EN 1994-2 6.8.6.2 (4) and SS-EN 1993-2 Eq. 9.10

$t_{Ld} := 120$ Expected service life [years]

$\lambda_{v.3} := \left(\dfrac{120}{100}\right)^{\frac{1}{8}} = 1.02$

$\lambda_{v.4} := 1.0$ TSFS 2018:57 - 27 ch. 3 §

$\lambda_v := \lambda_{v.1} \cdot \lambda_{v.2} \cdot \lambda_{v.3} \cdot \lambda_{v.4} = 1.02$

$\gamma_{Ff} := 1.0$ SS-EN 1993-2, 9.3 (1)

$\gamma_{mF} := 1.0$ SS-EN 1994-2, 2.4.1.2 (6)

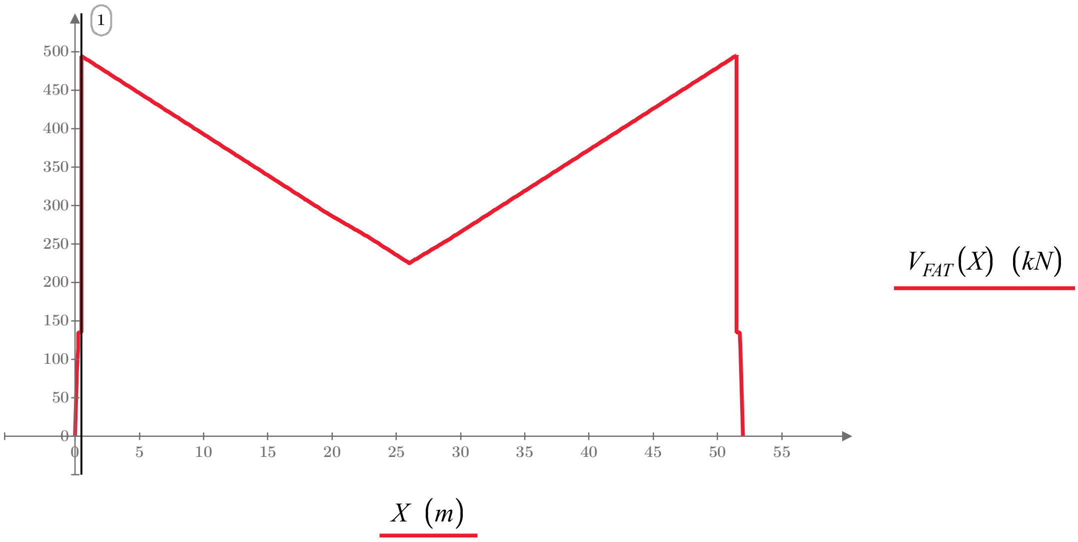
$\gamma_{Ff} \cdot \Delta\sigma_{E2} < \dfrac{\Delta\tau_c}{\gamma_{mF}}$

$F_{rd\_stud} := \Delta\tau_c \cdot \dfrac{\pi \cdot d_{stud}^{\ 2}}{4} = 34.2\ kN$ Shear fatigue capacity - one stud

$F_{rd\_stud} := \dfrac{F_{rd\_stud}}{\lambda_v} = 33.7\ kN$ Considering trafic load

6.7.2.2 Fatigue load

$V_{FAT}(X)$ Fatigue load from trafic - see Appendix X



$$\tau \cdot b = \frac{SV}{I} = \frac{stud\_capacity}{m}$$

$$S_{uf} := S_{uf\_short}(X_{check\_v}) = 0.081\ m^3$$

$$V := V_{FAT}(X_{check\_v}) = 494\ kN$$

$$I := I_{y\_short}(X_{check\_v}) = 0.228\ m^4$$

$$\Delta\tau := \frac{S_{uf} \cdot V}{I} = 175\ \frac{kN}{m}$$

$F_{rd\_stud} = 34\ kN$ Fatigue capacity - one stud

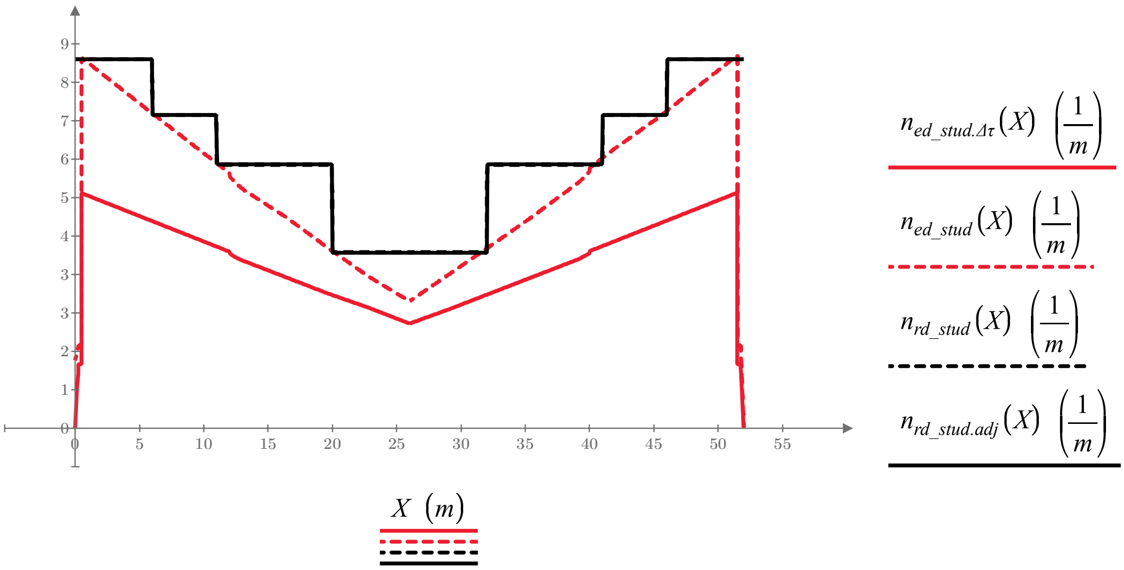
$$n_{ed\_\Delta\tau} := \frac{\Delta\tau}{F_{rd\_stud}} = 5.2\ \frac{1}{m} \quad = \quad n_{ed\_stud.\Delta\tau}(X_{check\_v}) = 5.2\ \frac{1}{m}$$

6.7.2.3 Design of studs with regards to fatigue

$n_{ed\_stud.\Delta\tau}(X)$ Needed amount of studs with regards to fatigue

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



Extra need of studs with regards to fatigue

$$Check := \textbf{if}\left(max\left(\frac{n_{ed\_stud.\Delta\tau}(X)}{n_{rd\_stud}(X)}\right), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$Check = \text{“No extra studs are needed”}$$

6.7.3 Summary - design of studs

Table. Showing the stud design for half of the span

X	$n_{stud.used}$	$n_{stud.need}$	$n_{stud.fat}$
0	**8.2**	1.5	0.0
0.5	**8.2**	8.2	5.2
4.2	**8.2**	7.4	4.8
6	**7.3**	7.0	4.6
11	**6.2**	5.9	4.0
20	**4.3**	3.9	2.9

6.8 Utilization rates

$\eta_{\sigma.u.max} = 100\%$	Stresses in top flange
$\eta_{\sigma.l.max} = 98\%$	Stresses in lower flange
$\eta_{\sigma.c.max} = 27\%$	Stresses in concrete
$\eta_{V.max} = 98\%$	Shear capacity
$\eta_{breathing} = 9\%$	Breathing

9 Material savings

The material savings that are achieved thanks to redesigning the bridge with corrugated webs and using stainless steel is compared to the old design. However it is important to note that dependent on the utilization rates the savings may always not be comparable. For this bridge the highest utilization ratios are larger than 95% (close to 99,8%) and therefore close to comparable.

9.1 Main girder

The main girder is redesigned with a corrugated web and a slightly slimmer design. Over the full bridge length this decreases the weight of the bridge. All savings are presented for the full bridge (full width). Consideration has been taken to the extra web length arising from the corrugation.

$V_{old} = 13.25\ m^3$ Steel volume - girder with flat web; original design

$V_{new} = 9.24\ m^3$ Steel volume - girder with corrugated web; new design

$m_{old\_girder} := 7850\ \frac{kg}{m^3} \cdot V_{old} = 104\ 10^3 \cdot kg$ Weight of girder - flat web; original design

$m_{new\_girder} := 7551\ \frac{kg}{m^3} \cdot V_{new} = 70\ 10^3 \cdot kg$ Weight of girder - corrugated web; new design

$\eta_{girder} := 1 - \frac{m_{new\_girder}}{m_{old\_girder}} = 33\%$ Material saving [%] - steel girder

$\Delta m_{saving\_girder} := m_{old\_girder} - m_{new\_girder} = 34\ 10^3\ kg$ Material saving [kg] - steel girder

9.2 Studs

$V_{stud\_old} = 72750\ cm^3$ Volume - studs; original design

$V_{stud\_new} = 48490\ cm^3$ Volume - studs; new design

$m_{old\_stud} := 7850\ \frac{kg}{m^3} \cdot V_{stud\_old} = 571\ kg$ Mass - studs; original design

$m_{new\_stud} := 7551\ \frac{kg}{m^3} \cdot V_{stud\_new} = 366\ kg$ Mass - studs; new design

$\eta_{stud} := 1 - \frac{m_{new\_stud}}{m_{old\_stud}} = 36\%$ Material saving [%] - studs

$\Delta m_{saving\_stud} := m_{old\_stud} - m_{new\_stud} = 205\ kg$ Material saving [kg] - studs

9.3 Cross-beams

$V_{cb}=0.59\ m^3$ Volume cross-beams old and new design

$m_{old\_cb}:=7850\ \frac{kg}{m^3}\cdot V_{cb}=4597\ kg$ Mass - cross-beams; original design

$m_{new\_cb}:=7551\ \frac{kg}{m^3}\cdot V_{cb}=4422\ kg$ Mass - cross-beams; new design

$\eta_{cb}:=1-\frac{m_{new\_cb}}{m_{old\_cb}}=4\%$ Material saving [%] - studs

$\Delta m_{saving\_cb}:=m_{old\_cb}-m_{new\_cb}=175\ kg$ Material saving [kg] - studs

9.4 Welds

$V_{weld\_old}=0.0051\ m^3$ Volume - welds; original design

$V_{weld\_new}=0.0038\ m^3$ Volume - welds; new design

$m_{old\_weld}:=7850\ \frac{kg}{m^3}\cdot V_{weld\_old}=40\ kg$ Mass - welds; original design

$m_{new\_weld}:=7551\ \frac{kg}{m^3}\cdot V_{weld\_new}=29\ kg$ Mass - welds; new design

$\eta_{weld}:=1-\frac{m_{new\_weld}}{m_{old\_weld}}=28\%$ Material saving [%] - welds

$\Delta m_{saving\_weld}:=m_{old\_weld}-m_{new\_weld}=11\ kg$ Material saving [kg] - welds

9.5 Total savings

$m_{old\_bridge}:=m_{old\_girder}+m_{old\_stud}+m_{old\_cb}=109\ 10^3\ kg$ Total mass of original design

$m_{new\_bridge}:=m_{new\_girder}+m_{new\_stud}+m_{new\_cb}=75\ 10^3\ kg$ Total mass of new design

$\eta_{bridge}:=1-\frac{m_{new\_bridge}}{m_{old\_bridge}}=31.8\%$ Material saving [%] - full bridge

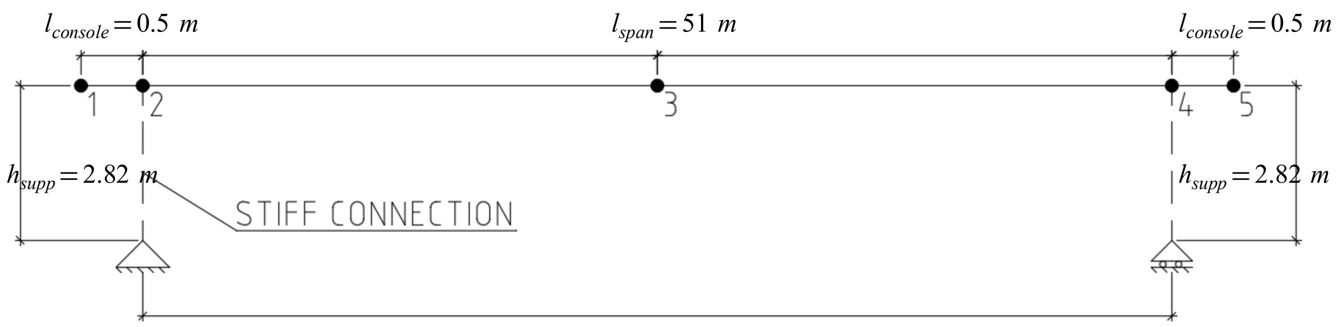
$\Delta m_{saving.tot}:=\Delta m_{saving\_girder}+\Delta m_{saving\_stud}+\Delta m_{saving\_cb}+\Delta m_{saving\_weld}=35\ 10^3\ kg$
Material saving [kg] - full bridge

Extracts - Calculation report $1.375h_w$

3 System

In the following chapter the input data for the bridge's geometries is presented. The cross-sectional parameters that are presented is preliminary and used for the system analysis. In chapter 5 the final design is presented.

3.1 Primary system - longitudinal



The bridge is modelled as a simply supported bridge in the software StripStep-2. Due to the beams depth the supports are set offset from the neutral axis. This is modelled with a stiff connection in the system analysis.

3.1.1 Cross-section dimensions

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

New height of girder is taken as: $Round(2050 \cdot 1.375, 10) = 2820$

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	600	8	2737	48	850
500	35	600	8	2737	48	850

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	600	8	2737	48	850
10500	35	600	8	2737	48	850
10500	42	600	6	2723	55	850
11300	42	800	6	2723	55	1180
25500	42	800	6	2723	55	1180

Important! Two exactly the same values will not work with the linterp- function. Therefore 0.1 millimeter must be added for a X-value where you want to cross-sectional properties at the same time.

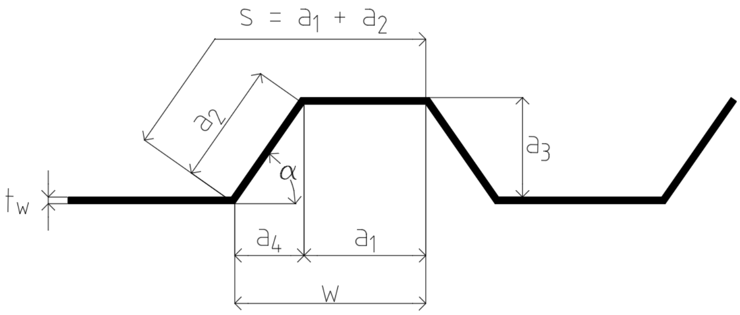
Element 3

S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	600	8	2737	48	850
500	35	600	8	2737	48	850

$$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=2820\ mm$$

$$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=2820\ mm$$

3.1.2 Corrugation shape



$a_{c1}:=150\ mm$ Flat-fold length

$\alpha_c:=36\ deg$ Corrugation angle

$a_{c3}:=80\ mm$ Corrugation depth

$a_{c2}:=\dfrac{a_{c3}}{sin\left(\alpha_c\right)}=136\ mm$ Length of angled part

$a_{c4}:=\dfrac{a_{c3}}{tan\left(\alpha_c\right)}=110\ mm$ Length of hypopythis

$s_c:=a_{c1}+a_{c2}=286\ mm$ Length of corrugation

$w_c:=a_{c1}+a_{c4}=260\ mm$ Straight length

$r_c:=\dfrac{s_c}{w_c}=1.10$ Ratio corrugation/flat-fold length

3.1.3.2 Effective width, steel flanges

Calculation of the effective width in the steel flanges with regards to shear lag is calculated according to SS-EN 1993-1-5 3.2.

$b_{eff\_f} = \beta \cdot b_0 + \frac{t_w}{2}$

Effective width with regards to shear lag under elastic conditions - Equation (3.1)

$X_{check\_uf} := \frac{L_{bridge}}{2} = 26.0\ m$

$X_{check\_lf} := 0.2\ L_{bridge} = 10.4\ m$

<u>Upper flange</u>

$\alpha_0 := 1.0$

Equation given in Table 3.1. For webs without any longitudinal stiffeners $\alpha_0 = 1.0$.

$bfu := b_{fu}\left(X_{check\_uf}\right) = 800\ mm$

Width of upper flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_uf}\right) = 6\ mm$

Thickness of web - see Appendix B - Preliminary sizing

$b0 := \frac{bfu - tw}{2} = 397\ mm$

Figure 3.2 (Notations for shear lag)

$\kappa fu := \frac{\alpha_0 \cdot b0}{L_{span}} = 0.008$ = $\kappa_{fu}\left(X_{check\_uf}\right) = 0.008$

Table 3.1

$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fu \le 0.02 \\ \quad \| \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fu \le 0.70 \\ \quad \left\| \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fu^2} \right. \\ \text{else if } 0.70 < \kappa fu \\ \quad \left\| \beta \leftarrow \dfrac{1}{8.6\ \kappa fu^2} \right. \end{array} \right. = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for upper flange

$2\ \beta_{check} \cdot b0 + tw = 800\ mm$

$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad \| b \leftarrow bfu \\ \text{else} \\ \quad \| b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfu\right) \\ b \end{array} \right. = 0.800\ m \quad = \quad b_{eff\_fu}\left(X_{check\_uf}\right) = 0.800\ m$$

<u>Lower flange</u>

$bfl := b_{fl}\left(X_{check\_lf}\right) = 850\ mm$ Width of lower flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_lf}\right) = 8\ mm$ Thickness of web - see Appendix B - Preliminary sizing

$b0 := \dfrac{bfl - tw}{2} = 421\ mm$ Figure 3.2 (Notations for shear lag)

$\kappa fl := \dfrac{\alpha_0 \cdot b0}{L_{span}} = 0.008$ = $\kappa_{fl}\left(X_{check\_lf}\right) = 0.008$ Table 3.1

$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fl \le 0.02 \\ \quad \| \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fl \le 0.70 \\ \quad \left\| \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fl^2} \right. \\ \text{else if } 0.70 < \kappa fl \\ \quad \left\| \beta \leftarrow \dfrac{1}{8.6\ \kappa fl^2} \right. \end{array} \right. = 1.00$$

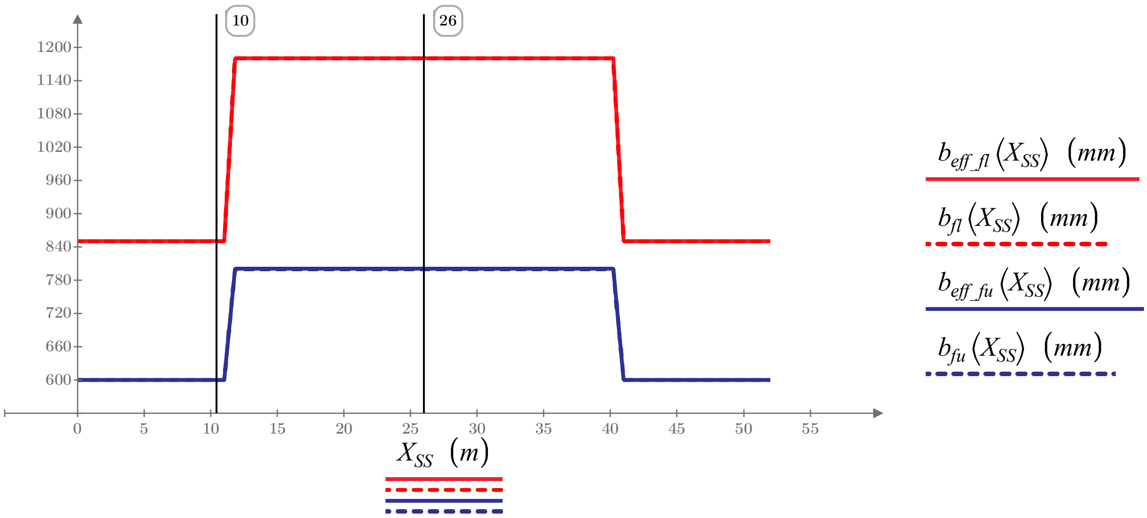
β for sagging bending, one-span bridge

Calculating the effective flange width for lower flange

$2\ \beta_{check} \cdot b0 + tw = 850\ mm$

$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad \| b \leftarrow bfl \\ \text{else} \\ \quad \| b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfl\right) \\ b \end{array} \right. = 0.850\ m$$

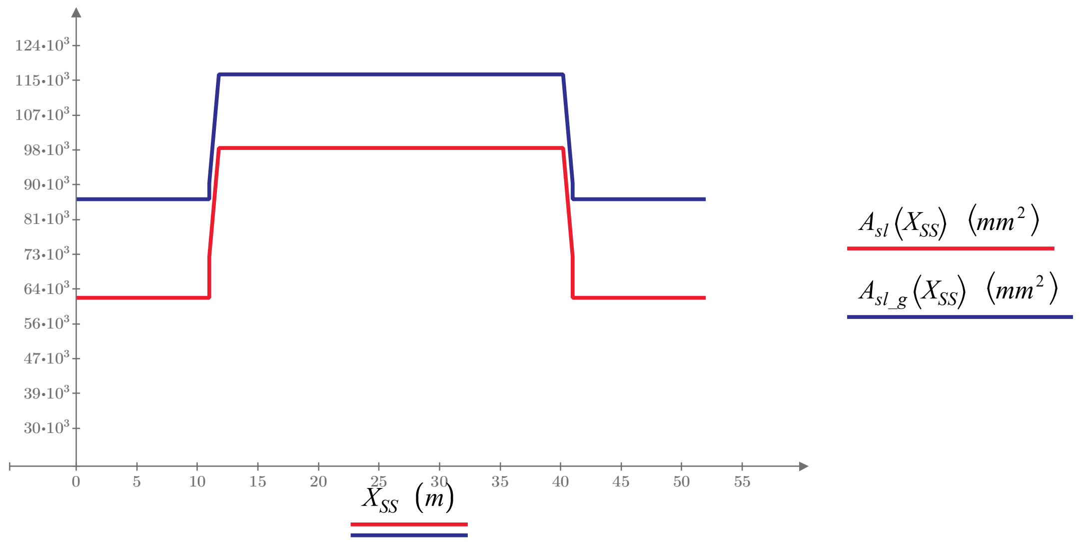
= $b_{eff\_fl}\left(X_{check\_lf}\right) = 0.850\ m$



3.1.4.2 Cross-sectional constants during construction

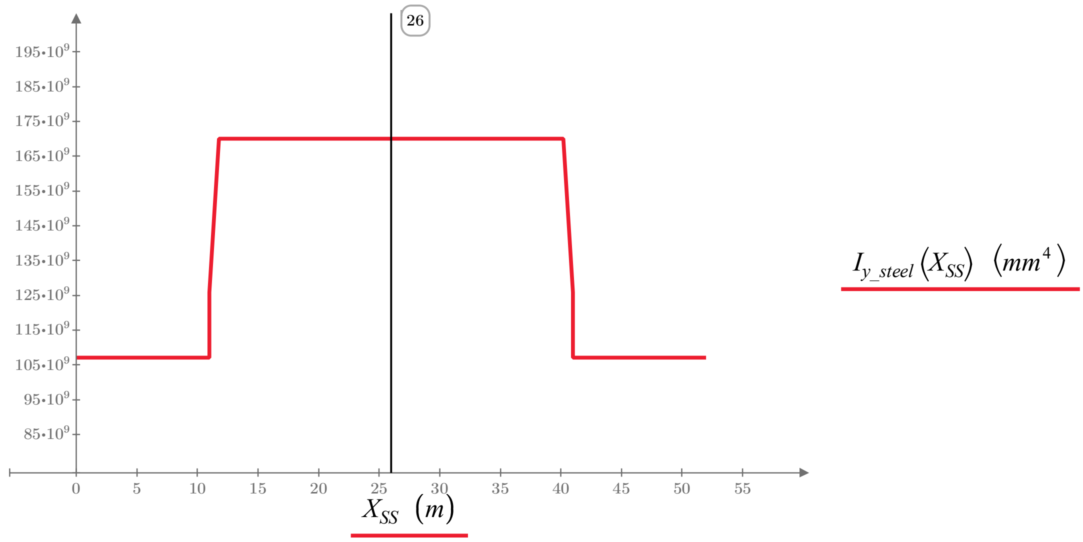
The bridge is checked so that the steel girder (alone) can withstand the loads that are imposed during construction such as the self-weight of curing concrete.

A_{sl_g} is the area of the steel including the web, for calculation of self-weight
A_{sl} is the area of the steel excluding the web, for stiffness calculations



$A_{sl}\left(\frac{L_{bridge}}{2}\right) = 0.099\ m^2$ Area at mid-span

I_{y_steel} is the area of the steel excluding the web, for stiffness calculations



$I_{y\_steel}\left(\frac{L_{bridge}}{2}\right) = 0.1701\ m^4$ Stiffness at mid-span

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]	[kN/m]
0.000	-1.852	2.820	1070.415	618	6.485
0.500	-1.852	2.820	1070.415	618	6.485

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]	[kN/m]
0.000	-1.852	2.820	1070.415	618	6.485
10.500	-1.852	2.820	1070.415	618	6.485
10.500	-1.822	2.820	1257.867	720	6.790
11.300	-1.847	2.820	1700.716	985	8.795
25.500	-1.847	2.820	1700.716	985	8.795
39.700	-1.847	2.820	1700.716	985	8.795
40.500	-1.822	2.820	1257.867	720	6.790
40.500	-1.852	2.820	1070.415	618	6.485
51.000	-1.852	2.820	1070.415	618	6.485

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]	[kN/m]
0.000	-1.852	2.820	1070.415	618	6.485
0.500	-1.852	2.820	1070.415	618	6.485

System model used in Strip-Step2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.82	ZR	
					2
3	51.500	0.00	-2.82	YZR	
					3
4	52.000	0.00			-3

3.1.4.3 Cross-sectional constants for variable loads (short term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.217	3.140	3125.63	3297
0.500	-0.217	3.140	3125.63	3297

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.217	3.140	3125.63	3297
10.500	-0.217	3.140	3125.63	3297
10.500	-0.260	3.140	3508.14	3398
11.300	-0.380	3.140	4624.60	3664
25.500	-0.380	3.140	4624.60	3664
39.700	-0.380	3.140	4624.60	3664
40.500	-0.260	3.140	3508.14	3398
40.500	-0.217	3.140	3125.63	3297
51.000	-0.217	3.140	3125.63	3297

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.217	3.140	3125.63	3297
0.500	-0.217	3.140	3125.63	3297

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.82	ZR	
					2
3	51.500	0.00	-2.82	YZR	
					3
4	52.000	0.00			-3

3.1.4.4 Cross-sectional constants for additional permanent loads (long term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	[$10^4 m^4$]	[$10^4 m^2$]
0	-0.685	3.140	2527.859	1471
0.5	-0.685	3.140	2527.859	1471

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	[$10^4 m^4$]	[$10^4 m^2$]
0	-0.685	3.140	2527.859	1471
10.5	-0.685	3.140	2527.859	1471
10.5	-0.747	3.140	2797.657	1572
11.3	-0.916	3.140	3549.003	1838
25.5	-0.916	3.140	3549.003	1838
39.7	-0.916	3.140	3549.003	1838
40.5	-0.747	3.140	2797.657	1572
40.5	-0.685	3.140	2527.859	1471
51	-0.685	3.140	2527.859	1471

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	[$10^4 m^4$]	[$10^4 m^2$]
0	-0.685	3.140	2527.859	1471
0.5	-0.685	3.140	2527.859	1471

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.82	ZR	
					2
3	51.500	0.00	-2.82	YZR	
					3
4	52.000	0.00			-3

3.1.4.5 Cross-sectional constants for shrinkage analysis

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.582	3.140	2657.903	1675
0.5	-0.582	3.140	2657.903	1675

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.582	3.140	2657.903	1675
10.5	-0.582	3.140	2657.903	1675
10.5	-0.643	3.140	2948.236	1777
11.3	-0.808	3.140	3763.696	2042
25.5	-0.808	3.140	3763.696	2042
39.7	-0.808	3.140	3763.696	2042
40.5	-0.643	3.140	2948.236	1777
40.5	-0.582	3.140	2657.903	1675
51	-0.582	3.140	2657.903	1675

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.582	3.140	2657.903	1675
0.5	-0.582	3.140	2657.903	1675

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-2.82	ZR	
					2
3	51.500	0.00	-2.82	YZR	
					3
4	52.000	0.00			-3

4 Loads and load combinations

4.1 Permanent loads

4.1.1 Self-weight

4.1.1.1 Steel

The self-weight is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: STEEL

$\rho_{st} = 75.51 \ \frac{kN}{m^3}$ — Self-weight of stainless steel - SS-EN 10088-1:2014 Table E.1 or E.2

The self-weight for each element is calculated in chapter 3.

4.1.1.2 Concrete

The self-weight of the slab is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: SLAB

$Wet\_Concrete := \text{"NO"}$ — "YES" or "NO" dependent if the previous designer has used the weight of wet concrete

$\rho_c := \mathbf{if}\left(Wet\_Concrete = \text{"NO"}, 25 \ \frac{kN}{m^3}, 26 \ \frac{kN}{m^3}\right) = 25 \ \frac{kN}{m^3}$ — Self-weight of concrete (reinforced) - SS-EN 1992-1-1 Table A.1

$A_{slab} = 3.47 \ m^2$ — Area of slab, see chapter 3

$g_{slab} := \frac{A_{slab} \cdot \rho_c}{2} = 43.4 \ \frac{kN}{m}$ — Self-weight of concrete slab, (half of the load goes to each girder) - applied in the casting stage

If the previous designer has considered that the hardened concrete has a smaller self-weight a reduction in self-weight is applied in the system analysis for permanent loads.

Appendix X. Name in Strip-Step2: AVSLAB

$$g_{slab.perm} := \mathbf{if}\left(Wet\_Concrete = \text{"NO"}, 0 \ \frac{kN}{m}, \frac{A_{slab} \cdot (-1) \ \frac{kN}{m^3}}{2}\right) = 0.000 \ \frac{kN}{m}$$

4.1.2 Shrinkage

The shrinkage force is calculated and used in Strip-Step2, Appendix X. Name in Strip-Step2: E:SHRINK

$h_0 = 341\ mm$ — Equivalent thickness, calculated in chapter 2

$t = 120\ yr$

$t = 43829.1\ day$

$f_{cm} = 43\ MPa$ — Mean compressive strength - see chapter 2

$t_s := 1\ day$

$RH := 80\%$ — Krav Brobyggande B.3.1.5

$$\beta_{ds} := \frac{\frac{t-t_s}{day}}{\left(\frac{t-t_s}{day}\right) + 0.04 \cdot \sqrt{\left(\frac{h_0}{1\ mm}\right)^3}} = 0.994$$

SS-EN 1992-1-1 3.1.4 Equation 3.10

$$k_h := \begin{cases} 0.85 - \left(\frac{h_0}{mm} - 200\right) \cdot 0.001 & \text{if } 200\ mm \le h_0 < 300\ mm \\ 0.75 - \left(\frac{h_0}{mm} - 300\right) \cdot 0.0005 & \text{else if } 300\ mm \le h_0 < 500\ mm \\ 0.7 & \text{else if } h_0 \ge 500\ mm \\ 1.0 & \text{else} \end{cases} = 0.73$$

SS-EN 1992-1-1 3.1.4 table 3.3

$RH_0 := 100\%$

$\alpha_{ds1} := 4$ — SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$\alpha_{ds2} := 0.12$ — SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0}\right)^3\right) = 0.76$$

SS-EN 1992-1-1 Appendix B.2 equation B12

$f_{cmo} := 10\ MPa$ — SS-EN 1992-1-1 Appendix B.2 (1)

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}\right)}\right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$

SS-EN 1992-1-1 Appendix B.2 equation B11

$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 1.84 \cdot 10^{-4}$ Drying shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.9

$\varepsilon_{ca0} := 2.5 \cdot \left(\frac{f_{ck} - f_{cmo}}{MPa}\right) \cdot 10^{-6} = 6.3 \cdot 10^{-5}$ SS-EN 1992-1-1 3.1.4 Equation 3.12

$\beta_{as} := 1 - e^{\left(-0.2 \cdot \sqrt{\frac{t - t_s}{day}}\right)} = 1.0$ SS-EN 1992-1-1 3.1.4 Equation 3.13

$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca0} = 6.25 \cdot 10^{-5}$ Autogenous shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.11

$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.8

4.1.2.1 Shrinkage force

The shrinkage force and corresponding moment is calculated accordingly:

$n_{L\_cs} = 14.91$ Modular ratio accounting for creep and shrinkage

$n_{L\_short} = 5.88$ Modular ratio

$E_{c.eff} := \frac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab (half of the cross-section used in system analysis)

$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5206\ kN$ Total shrinkage force

$e_{cs} := z_{tp\_cs}(0\ m) = 0.582\ m$ The shrinkage force is applied in the center of gravity for the composite section

$M_{cs} := F_{cs} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right) = 3864\ kN \cdot m$ Total bending moment

Shrinkage - anchorage of studs

Forces in the concrete

$$F_{c\_cs} := F_{cs} \cdot \left(1 - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{sl\_cs}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{y\_cs}(0\ m)} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2} \right)^2 \right) = 779\ kN$$ Force in concrete

$$b_{eff}(0\ m) = 4.925\ m$$ Effective width of flange

$$M_{c\_cs} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_cs}} \cdot {h_{m.slab}}^3}{I_{y\_cs}(0\ m)} \cdot M_{cs} = 13\ kN \cdot m$$ Moment in concrete

$$\sigma_1 := \frac{F_{c\_cs}}{A_{slab.fic}} - \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot {h_{m.slab}}^3} = 0.34\ MPa$$ Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{F_{c\_cs}}{A_{slab.fic}} + \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot {h_{m.slab}}^3} = 0.65\ MPa$$ Stress in concrete, lower edge of slab

$$N_{cs} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = 779\ kN \quad = \quad F_{c\_cs} = 779\ kN$$ Force imposed on studs caused by shrinkage (used in chapter 6)

4.2.2.1 Horizontal side trafic load

$P_{h\_side\_car} := max\left(\begin{bmatrix} 25\% \cdot Q_{1k\_br} \\ 25\% \cdot Q_{tr\_br} \end{bmatrix}\right) = 109.1\ kN$ Horisontal side force from the acceleration load (used in chapter 7)

$P_{v\_side\_car} := P_{h\_side\_car} \cdot \dfrac{h_{m.slab}}{2 \cdot B_{in}} = 6.2\ kN$ Vertical side force from the acceleration load (used in chapter 7)

4.2.3 Temperature load

Temperatures are determined according to SS-EN 1991-1-5, 6.1.3 unless otherwise stated.

Load case 1 - local temperature differences for Hudiksvall

$T_0 = 10\ °$ A.1(3)

$T_{min} = -38\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{max} = 34\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4\ ° = -34\ °$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min} = 44\ °$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4\ ° = 38\ °$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0 = 28\ °$ Expansion - Equation 6.1

$\Delta T := T_{e.max} - T_{e.min} = 72\ °$ Total temperature difference

Load case 2 - either of the components are larger than the other

$\Delta T_{c2st} := 15\ °$ Temperature difference between concrete and steel - SS-EN 1991-1-5, 6.1.6

4.2.3.1 Coefficients of thermal linear expansion

For composite bridges normally it is suggested to use the same thermal linear expansion coefficient, according to SS-EN 1991-1-5 Table C.1. However for stainless steels the thermal linear expansion coefficient is much larger than for concrete and hence a more through calculation is needed for the strain.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

4.2.3.2 Strains for the different load cases

$\Delta\varepsilon_{LC.1\_con} := -\left(\alpha_{ss} - \alpha_c\right) \cdot \frac{\Delta T_{N.con}}{\circ} = -26.4\ 10^{-5}$ Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp} := \left(\alpha_{ss} - \alpha_c\right) \cdot \frac{\Delta T_{N.exp}}{\circ} = 16.8\ 10^{-5}$ Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss} \cdot \frac{\Delta T_{c2st}}{\circ} = 24.0\ 10^{-5}$ Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c \cdot \frac{\Delta T_{c2st}}{\circ} = 15.0\ 10^{-5}$ Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st}, \Delta\varepsilon_{LC.2\_c}\right) = 24.0\ 10^{-5}$ Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp_1} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con} = -50.4\ 10^{-5}$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp_2} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp} = 40.8\ 10^{-5}$ Maximum strain difference; temperature raises and the steel heatens up even higher

4.2.3.3 Temperature load - global analysis

The concrete is transformed to steel. Already defined parameters are calculated in chapter 3.

$\frac{b_{eff}(0\ m)}{n_{L\_short}} = 0.837\ m$ Width of transformed concrete

$\frac{A_{slab.fic}}{n_{L\_short}} = 0.268\ m^2$ Area of transformed concrete

$A_{sl}(0\ m) = 0.062\ m^2$ Area of composite section

$z_{tp\_short}(0\ m) = 217.2\ mm$ Distance from top of concrete to center of gravity for composite section

$I_{y\_short}(0\ m) = 0.313\ m^4$ Moment of inertia for composite section

$F_{temp} := \varepsilon_{temp} \cdot E_s \cdot A_{sl}(0\ m) = \begin{bmatrix} -6229 \\ 5043 \end{bmatrix} kN$ Force on composite section

$e_{temp\_F} := z_{tp\_short}(0\ m) = 217.2\ mm$ Level at which the force is imposed on the system

$e_{temp\_M} := z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) = 1.635\ m$ Eccentricity for the bending moment

$$M_{temp} := F_{temp} \cdot e_{temp\_M} = \begin{bmatrix} -10183 \\ 8243 \end{bmatrix} kN \cdot m$$

Bending moment - composite section

Anchorage of temperature load imposed on studs

Concrete

$$N_{c\_temp} := F_{temp} \cdot \left(\frac{\dfrac{A_{slab.fic}}{n_{L\_short}}}{A_{sl\_short}(0\ m)} - \frac{\dfrac{A_{slab.fic}}{n_{L\_short}}}{I_{y\_short}(0\ m)} \cdot \left(\left(z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) \right) \cdot \left(z_{tp\_short}(0\ m) + \frac{h_{m.slab}}{2} \right) \right) \right) = \begin{bmatrix} -1770 \\ 1433 \end{bmatrix} kN$$

$$M_{c\_temp} := \frac{\dfrac{1}{12} \cdot \dfrac{b_{eff}(0\ m)}{n_{L\_short}} \cdot h_{m.slab}^{3}}{I_{y\_short}(0\ m)} \cdot M_{temp} = \begin{bmatrix} -74 \\ 60 \end{bmatrix} kN \cdot m$$

$$\sigma_1 := \frac{N_{c\_temp}}{A_{slab.fic}} - \frac{M_{c\_temp}}{\dfrac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{2}} = \begin{bmatrix} -0.24 \\ 0.19 \end{bmatrix} MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{N_{c\_temp}}{A_{slab.fic}} + \frac{M_{c\_temp}}{\dfrac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{2}} = \begin{bmatrix} -2.01 \\ 1.63 \end{bmatrix} MPa$$

Stress in concrete, lower edge of slab

$$N_{temp} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = \begin{bmatrix} -1770 \\ 1433 \end{bmatrix} kN$$

Force imposed on studs caused by shrinkage (used in chapter 6 when calculating the anchorage of the slab by the studs)

5 Capacity checks during construction

The capacity check that is carried out in this chapter is bending moment capacity with respect to lateral torsional buckling in the casting phase

In the casting phase the normalforces in the cross-section are neglectible.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

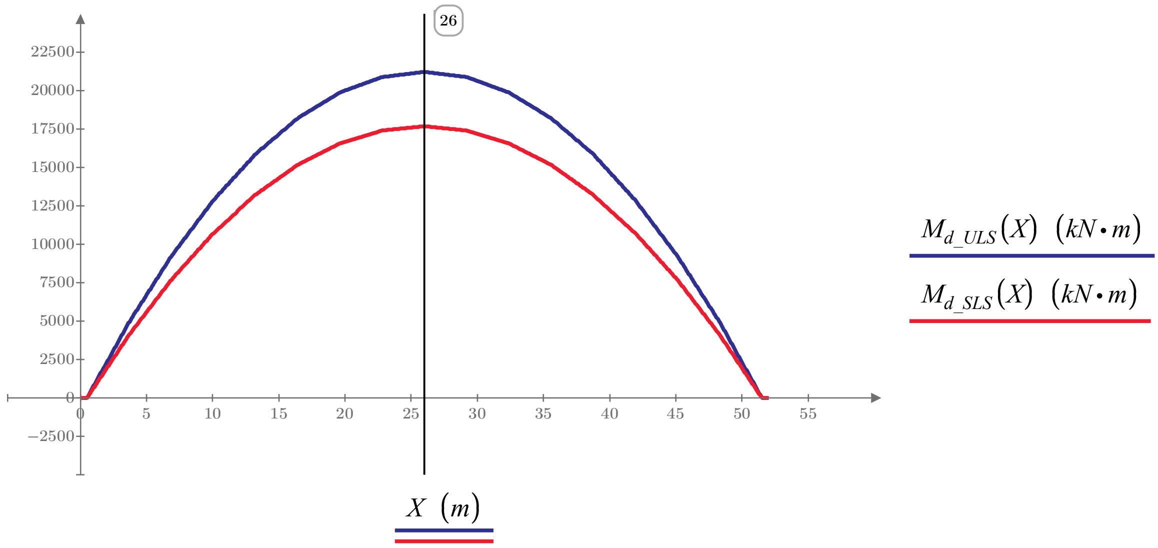
$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

5.1 Load effects

Load effects retrieved from Strip-Step2, Appendix X.

Bending moment

M_{d_ULS} Bending moment descending from permanent loads during construction in ultimate limit state
M_{d_SLS} Bending moment descending from permanent loads during construction in service limit state

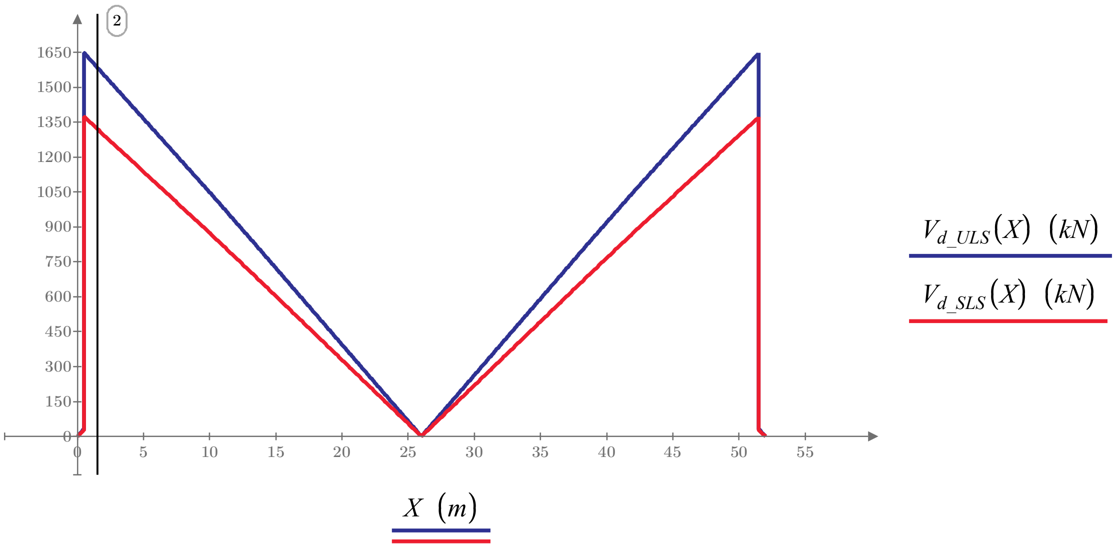


$M_{d\_ULS}(X_{check\_m}) = 21218\ kN \cdot m$ Bending moment at control point - ULS

$M_{d\_SLS}(X_{check\_m}) = 17681\ kN \cdot m$ Bending moment at control point - SLS

Shear force

V_{d_ULS} Shear force descending from permanent loads during construction in ultimate limit state
V_{d_SLS} Shear force descending from permanent loads during construction in service limit state



$V_{d\_ULS}(X_{check\_v}) = 1586\ kN$ Shear force at control point - ULS

$V_{d\_SLS}(X_{check\_v}) = 1321\ kN$ Shear force at control point - SLS

5.2 Redesign of cross-section

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange
f_{yw}	Chosen yield strength - web

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)	f_{yw_1}
0	35	675	7	2735	50	600	f_{y_10mm}
500	35	675	7	2735	50	600	f_{y_10mm}

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)	f_{yw_2}
0	35	675	7	2735	50	600	f_{y_10mm}
10500	35	675	7	2735	50	600	f_{y_10mm}
10500	45	675	5	2720	55	600	$f_{y\_6.4mm}$
11300	45	675	5	2720	55	825	$f_{y\_6.4mm}$
25500	45	675	5	2720	55	825	$f_{y\_6.4mm}$

Element 3

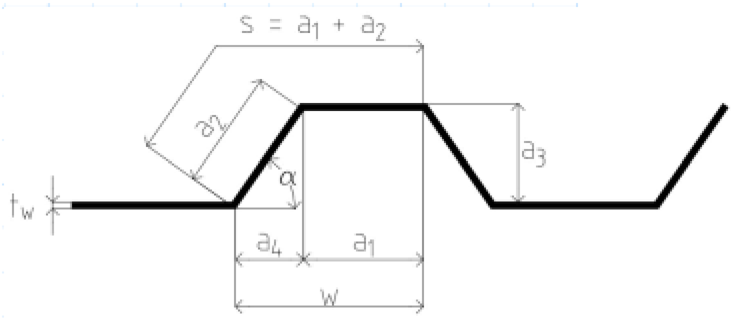
S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)	f_{yw_3}
0	35	675	7	2735	50	600	f_{y_10mm}
500	35	675	7	2735	50	600	f_{y_10mm}

$$h_{tot1} := mean\left(t_{fu\_1} + h_{w\_1} + t_{fl\_1}\right) = 2820\ mm$$

Check to see that the girder height is kept constant

$$h_{tot2} := mean\left(t_{fu\_2} + h_{w\_2} + t_{fl\_2}\right) = 2820\ mm$$

5.2.1 Shape of corrugation



$a_{c1} := 135\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 80\ mm$ Corrugation depth

$a_{weld} := 5\ mm$ Weld throat thickness

$a_{c2} := \frac{a_{c3}}{sin(\alpha_c)} = 136\ mm$ Legnth of hypotenuse

$a_{c4} := \frac{a_{c3}}{tan(\alpha_c)} = 110\ mm$ Length of corrugation depth, see figure

$s_c := a_{c1} + a_{c2} = 271\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 245\ mm$ Straight length

$r_c := \frac{s_c}{w_c} = 1.106$ Ratio corrugation/straight length

5.2.2 Cross-section classification

The cross-section classes is determined according to SS-EN 1993 1-4 5.2.2 with updated limits from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated and does not contribute to the axial stiffness the web is not classified.

$$c_w(x) := h_w(x) - 2 \cdot \sqrt{2} \cdot a_{weld}$$

$$c_{uf}(x) := \frac{b_{fu}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on upper flange

$$c_{lf}(x) := \frac{b_{fl}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on lower flange

Cross-section class, upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength of top flange

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

SS-EN 1993-1-4 5.2.2 Table 5.2

$$csc_{uf}(x) := \begin{array}{l} \text{if } \frac{c_{uf}(x)}{t_{fu}(x)} \le 9\ \varepsilon_{uf} \\ \quad \text{"csc1"} \\ \text{else if } 9\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 10\ \varepsilon_{uf} \\ \quad \text{"csc2"} \\ \text{else if } 10\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 14\ \varepsilon_{uf} \\ \quad \text{"csc3"} \\ \text{else} \\ \quad \text{"csc4"} \end{array}$$

Cross-section class upper flange

$csc_{uf}(X_{check\_m}) = \text{"csc3"}$ — Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{uf}(X_{check\_v}) = \text{"csc4"}$ — Cross-section class at $X_{check\_v} = 1.500\ m$

Cross-section class, lower flange

$E_s = 200\ GPa$ Modulus of elasticity

$f_{ylf} = 450\ MPa$ Proof strength of top flange

$\varepsilon_{lf} := \sqrt{\frac{235}{f_{ylf}} \cdot \frac{E_s}{210000}} = 0.71$ SS-EN 1993-1-4 5.2.2 Table 5.2

Cross-section class lower flange

$$csc_{lf}(x) := \begin{cases} \text{"csc1"} & \text{if } \frac{c_{lf}(x)}{t_{fl}(x)} \le 9\ \varepsilon_{lf} \\ \text{"csc2"} & \text{else if } 9\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 10\ \varepsilon_{lf} \\ \text{"csc3"} & \text{else if } 10\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 14\ \varepsilon_{lf} \\ \text{"csc4"} & \text{else} \end{cases}$$

$csc_{lf}(X_{check\_m}) = \text{"csc3"}$ Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{lf}(X_{check\_v}) = \text{"csc2"}$ Cross-section class at $X_{check\_v} = 1.500\ m$

5.2.3 Plate buckling of compressive flange

If the compressed flange is in cross-section class four an effective width of the compressed flange is calculated according to SS-EN 1993-1-4 5.2.3. Updated values are taken from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated the buckling factor k_σ is calculated according to SS-EN 1993-1-5 D.2.1 (2)

$a_{bend} := a_{c1} + 2\ a_{c4} = 355\ mm$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$c_u := c_{uf}\left(X_{check\_m}\right) = 368\ mm$ — Width of outstand flange from weld toe to free edge

$t_u := t_{fu}\left(X_{check\_m}\right) = 45\ mm$ — Thickness of upper flange

$b_u := b_{fu}\left(X_{check\_m}\right) = 675\ mm$ — Width of upper flange

$\varepsilon_{uf} = 0.71$

$k_{\sigma 1} := 0.43 + \left(\frac{c_u}{a_{bend}}\right)^2 = 1.5$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_{\sigma 2} := 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_\sigma := min\left(k_{\sigma 1}, k_{\sigma 2}\right) = 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

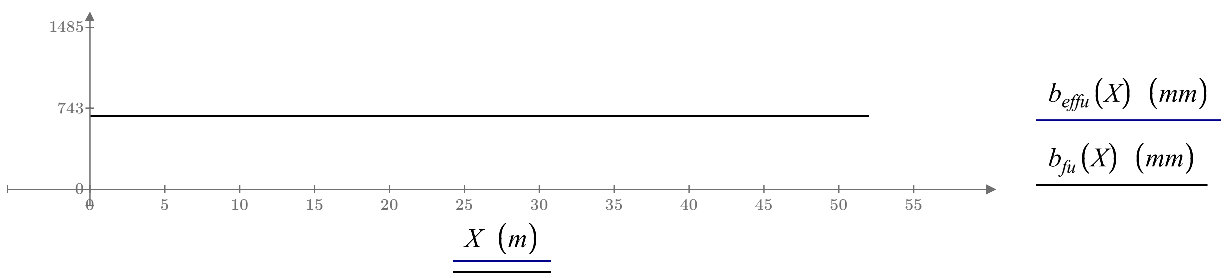
$$\lambda_p := \frac{\frac{c_u}{t_u}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_\sigma}} = 0.53$$

Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho := \mathbf{if}\left(\lambda_p \le 0.748, 1.0, \frac{\lambda_p - 0.188}{{\lambda_p}^2}\right) = 1.00$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

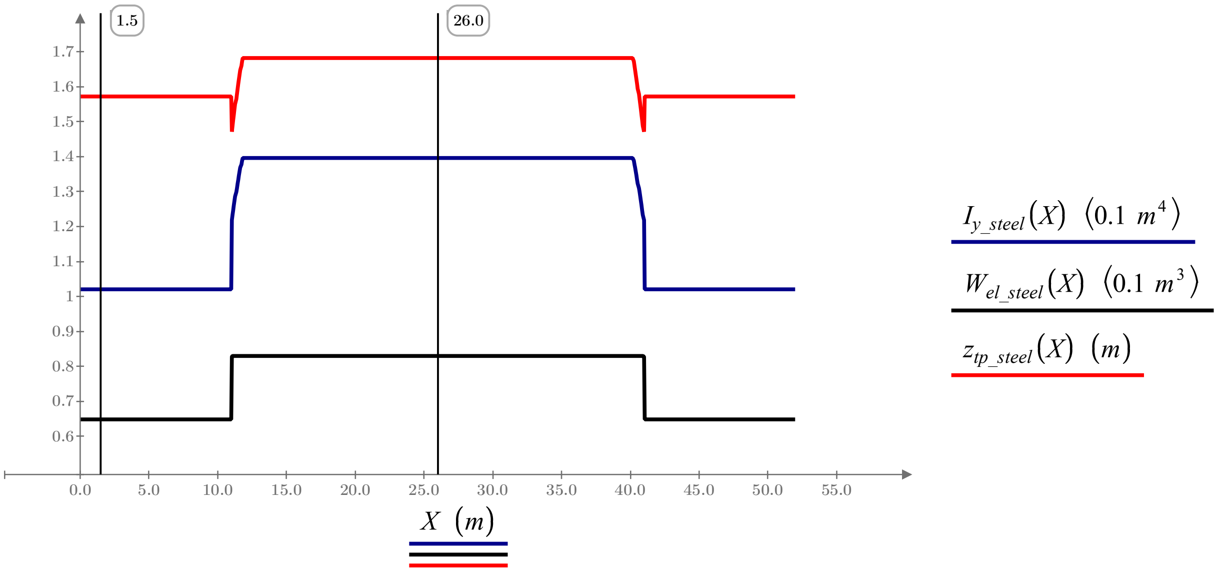
$b_{eff} := b_u \cdot \rho = 675\ mm$ = $b_{effu}\left(X_{check\_m}\right) = 675\ mm$



$b_{fu}(x) := b_{effu}(x)$ — Renaming the width of flange in order to minimize errors

5.2.6 New cross-sectional constants during casting

$I_{y\_steel}(x)$	Stiffness of steel girder alone (excluding the web)
$z_{tp\_steel}(x)$	Distance from the top of the top flange to the center of gravity for the steel section
$W_{el\_steel}(x)$	Elastic bending stiffness of steel girder alone (excluding the web)



Coordinates and cross-sectional constants for control calculations

$X_{check\_m} = 26\ m$	X- coordinate for control calculations, bending moment
$A_{sl}(X_{check\_m}) = 0.076\ m^2$	Area
$I_{y\_steel}(X_{check\_m}) = (140 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_m}) = 1.682\ m$	Center of gravity
$W_{el\_steel}(X_{check\_m}) = (83 \cdot 10^{-3})\ m^3$	Elastic bending resistance
$X_{check\_v} = 1.5\ m$	X- coordinate for control calculations, shear force
$A_{sl}(X_{check\_v}) = 0.054\ m^2$	Area
$I_{y\_steel}(X_{check\_v}) = (102 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_v}) = 1.571\ m$	Center of gravity
$W_{el\_steel}(X_{check\_v}) = (65 \cdot 10^{-3})\ m^3$	Elastic bending resistance

5.2.4 Lateral torsional buckling of compressive flange - SS-EN 1993-1-4 5.4.2.1

Simplified method only consisdering buckling of top flange according to SS-EN 1993-1-4 5.4.2.1

$\alpha_{LT} := 0.76$ — Buckling curve d, welded open cross-section

$l_{cr} := 7.29\ m$ — Distance between the cross-beams

$b_{ef} := b_{effu}\left(X_{check\_m}\right) = 675\ mm$ — Width of upper flange

$t_f := t_{fu}\left(X_{check\_m}\right) = 45\ mm$ — Thickness of upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength

$I_{zf} := \dfrac{b_{ef}^{\ 3} \cdot t_f}{12} = 0.001\ m^4$ — Moment of inertia, upper flange

$N_{crLT} := \dfrac{\pi^2 \cdot E_s \cdot I_{zf}}{l_{cr}^{\ 2}} = 42837\ kN$ — Critical buckling load

$\lambda_{LT\_u} := \sqrt{\dfrac{b_{ef} \cdot t_f \cdot f_{yuf}}{N_{crLT}}} = 0.565$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.9

$\Phi_{LT\_u} := 0.5 \cdot \left(1 + \alpha_{LT} \cdot \left(\lambda_{LT\_u} - 0.2\right) + \lambda_{LT\_u}^{\ 2}\right) = 0.8$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.7

$\chi_{LT\_u} := min\left(\dfrac{1}{\Phi_{LT\_u} + \sqrt{\Phi_{LT\_u}^{\ 2} - \lambda_{LT\_u}^{\ 2}}}, 1\right) = 0.734$ = $\chi_{LT}\left(X_{check\_m}\right) = 0.734$

SS-EN 1993-1-4 5.4.2.1 Equation 5.6

$h_{w\_u} := h_w\left(X_{check\_m}\right) = 2720\ mm$ — Height of web

$t_l := t_{fl}\left(X_{check\_m}\right) = 55\ mm$ — Thickness of lower flange

$k_{fl} := 1.1$ — Increase in capacity due to simlified method used, SS-EN 1993-1-1 6.3.2.4 (2)B

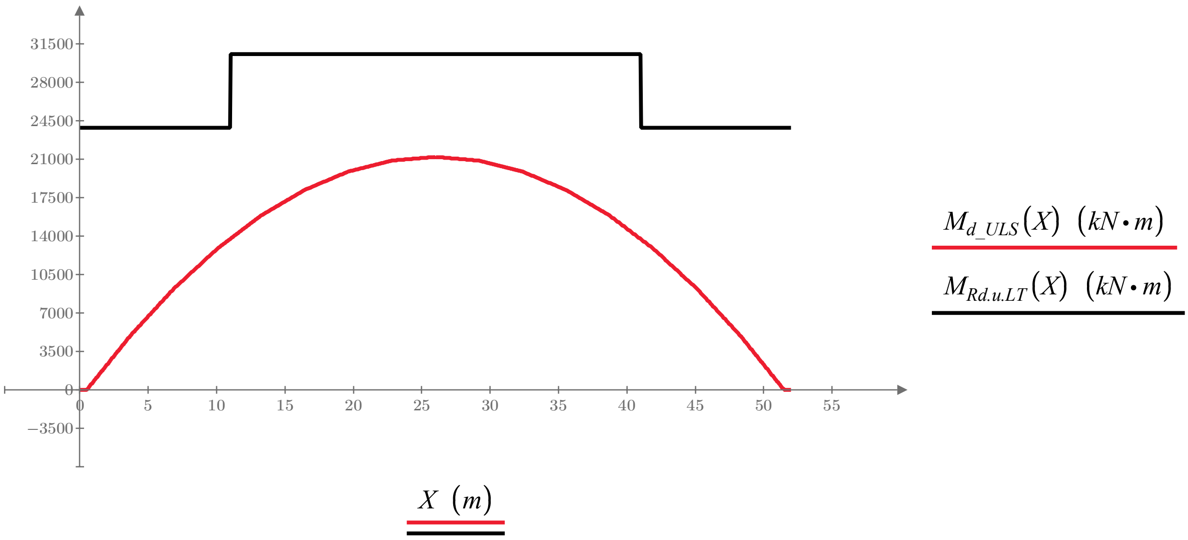
$M_{Rd.u.LT\_u} := \dfrac{b_{ef} \cdot t_f \cdot k_{fl} \cdot \chi_{LT\_u} \cdot f_{yuf}}{\gamma_{M1}} \left(h_{w\_u} + \dfrac{t_f + t_l}{2}\right) = 30576\ kN \cdot m$ = $M_{Rd.u.LT}\left(X_{check\_m}\right) = 30576\ kN \cdot m$

SS-EN 1993-1-5 D.2.1 Equation D.1

5.2.5 Check of lateral torsional buckling

Check of the buckling capacity of the girders is performed.

$M_{Rd.u.LT}(X)$	Capacity of the girder against LT- buckling during casting
$M_{d\_ULS}(X)$	Load effects during casting, Appendix X



$$\eta_{max} := max\left(\frac{M_{d\_ULS}(X)}{M_{Rd.u.LT}(X)}\right) = 69\%$$

Maximum utilization rate of the steel cross-section during casting

5.3 Stresses in steel cross-section

The stresses in the top and bottom flange is calculated for the loading senario to be able to superposition them with the other loadcases to determinte the ultimate capacity of the composite section.

Upper flange

Control calculation at midspan

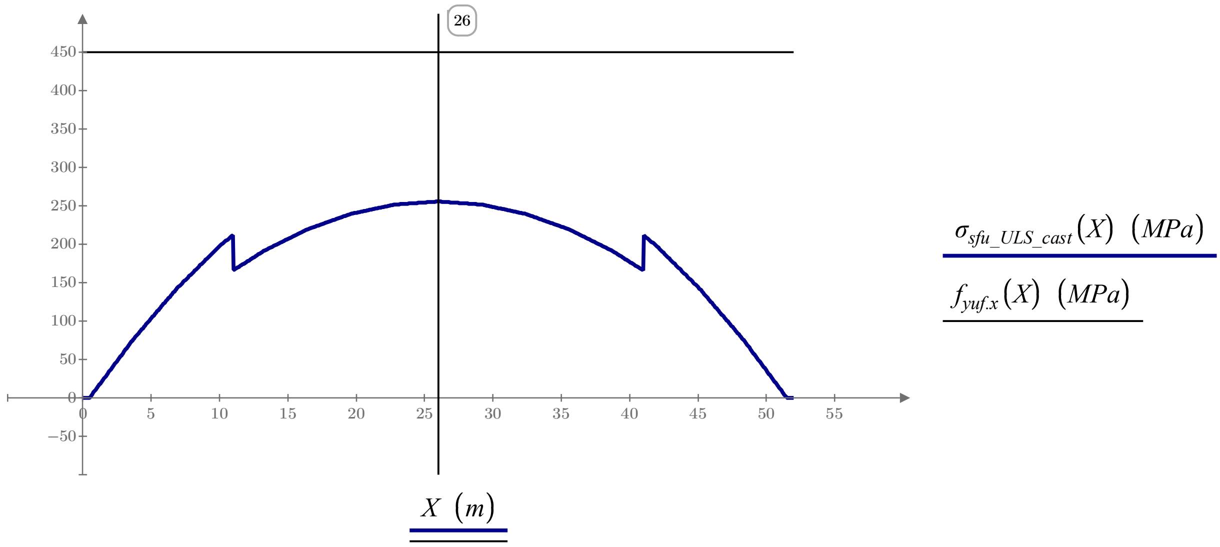
$M := M_{d\_ULS}(X_{check\_m}) = 21218\ kN \cdot m$ Load effext at midspan

$I := I_{y\_steel}(X_{check\_m}) = 0.14\ m^4$ Moment of inertia at midspan

$z := z_{tp\_steel}(X_{check\_m}) = 1.682\ m$ Centre of gravity at midspan maesured from the top of the beam

$\sigma := \frac{M}{I} \cdot z = 256\ MPa$ = $\sigma_{sfu\_ULS\_cast}(X_{check\_m}) = 256\ MPa$

$\sigma_{sfu\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\frac{\sigma_{sfu\_ULS\_cast}(X)}{f_{yuf}}\right) = 57\%$

Lower flange

Check calculation at midspan

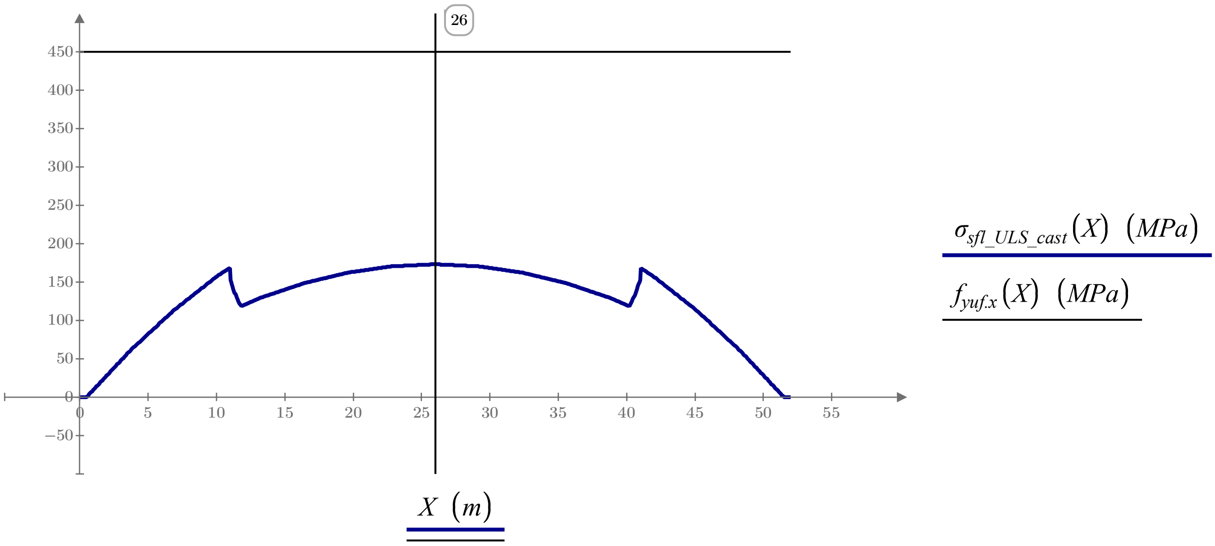
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 21218\ kN \cdot m$ — Load effect at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.14\ m^4$ — Moment of inertia at midspan

$z := z_{tplf\_steel}\left(X_{check\_m}\right) = 1.138\ m$ — Centre of gravity at midspan measured from the bottom of the beam

$$\sigma := \frac{M}{I} \cdot z = 173\ MPa \quad = \quad \sigma_{sfl\_ULS\_cast}\left(X_{check\_m}\right) = 173\ MPa$$

$\sigma_{sfl\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$$\eta_{max} := max\left(\frac{\sigma_{sfl\_ULS\_cast}(X)}{f_{ylf}}\right) = 38\%$$

6 Capacity checks - Ultimate limit state, global

The capacity checks that are to be carried out are bending moment capacity, shear capacity, web breathing and design of studs, both in ULS and due to fatigue

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 0.5\ m$ Coordinate for control calculations - shear force

6.1 Load effects in ULS

6.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

Bending moment in the ultimate limit state

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{tr}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

M_{d_ULS} Design bending moment effect from all loads

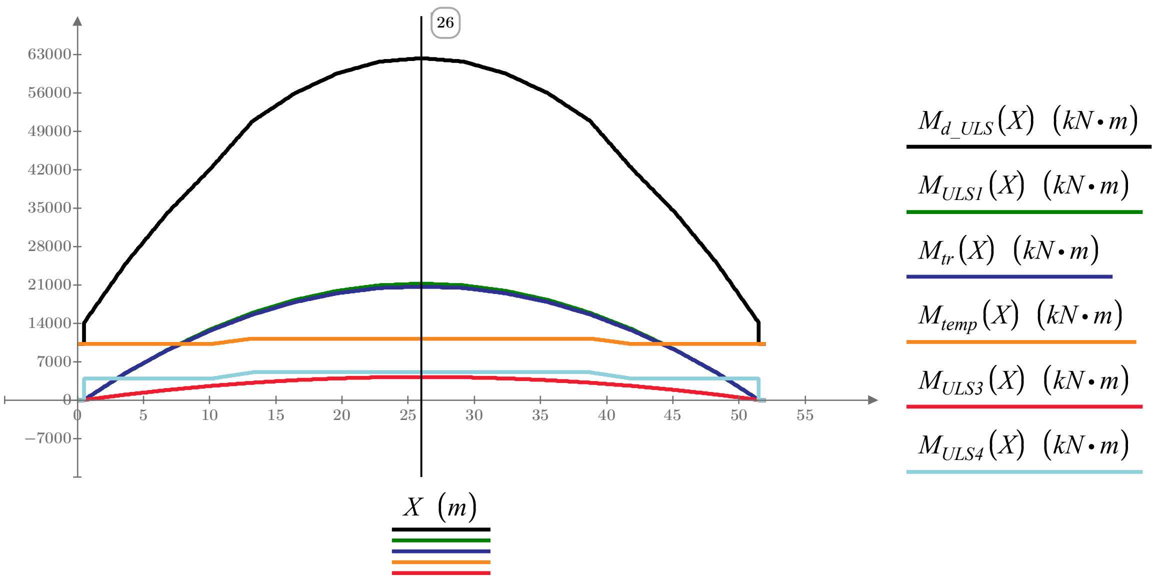
$M_{ULS1}(x)$ Bending moment during casting (Appendix X)

$M_{tr}(x)$ Bending moment from multi component loads (Appendix X)

$M_{tr}(x)$ Bending moment from temperature loads (Appendix X)

$M_{ULS3}(x)$ Bending moment from additional permanent loads after construction (Appendix X)

$M_{ULS4}(x)$ Bending moment from shrinkage (Appendix X)

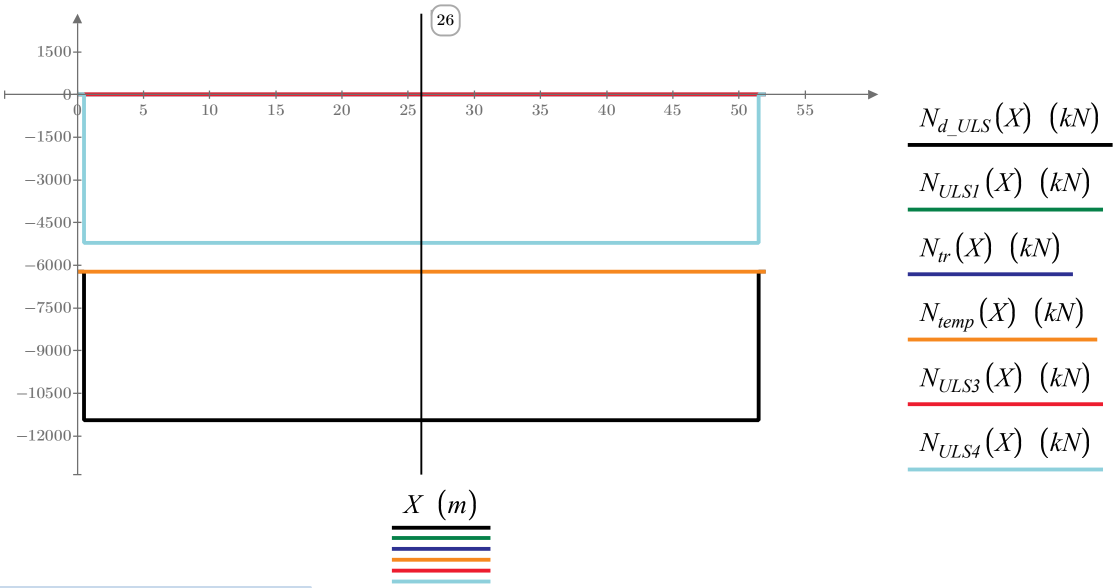


$M_{d\_ULS}(X_{check\_m}) = 62296\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{tr}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design normal force from all loads
$N_{ULS1}(x)$	Normal force during casting (Appendix X)
$N_{tr}(x)$	Normal force from multi components loads (Appendix X)
$N_{temp}(x)$	Normal force from temperature loads (Appendix X)
$N_{ULS3}(x)$	Normal force from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Normal force from shrinkage (Appendix X)

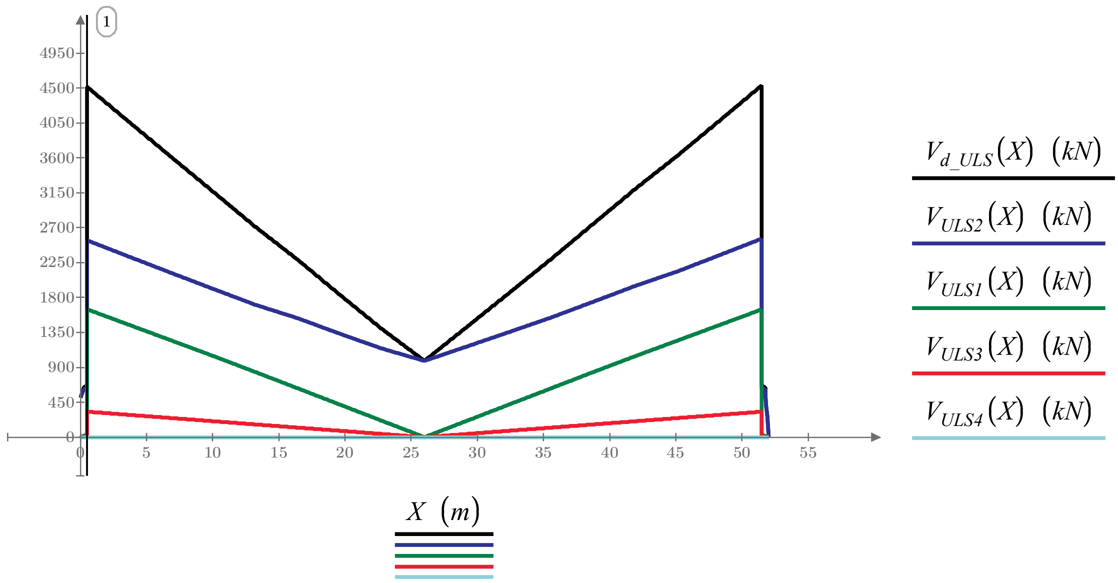


$N_{d\_ULS}(X_{check\_m}) = -11435\ kN$ Normal force at control point - ULS

6.1.2 Shear force

$$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{ULS3}(x) + V_{ULS4}(x)$$

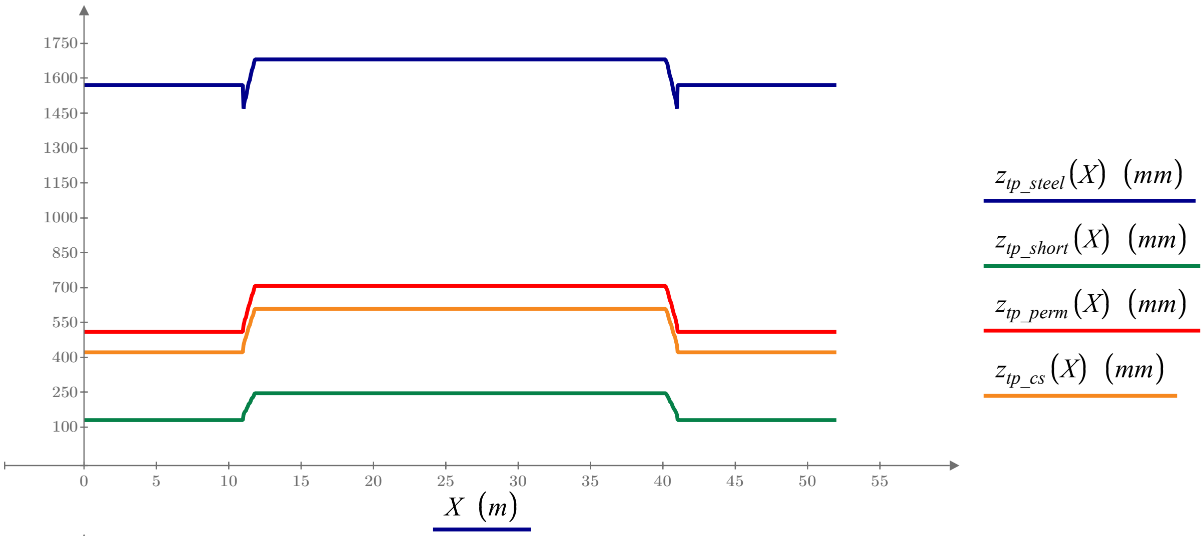
V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (both temperature and traffic) (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)

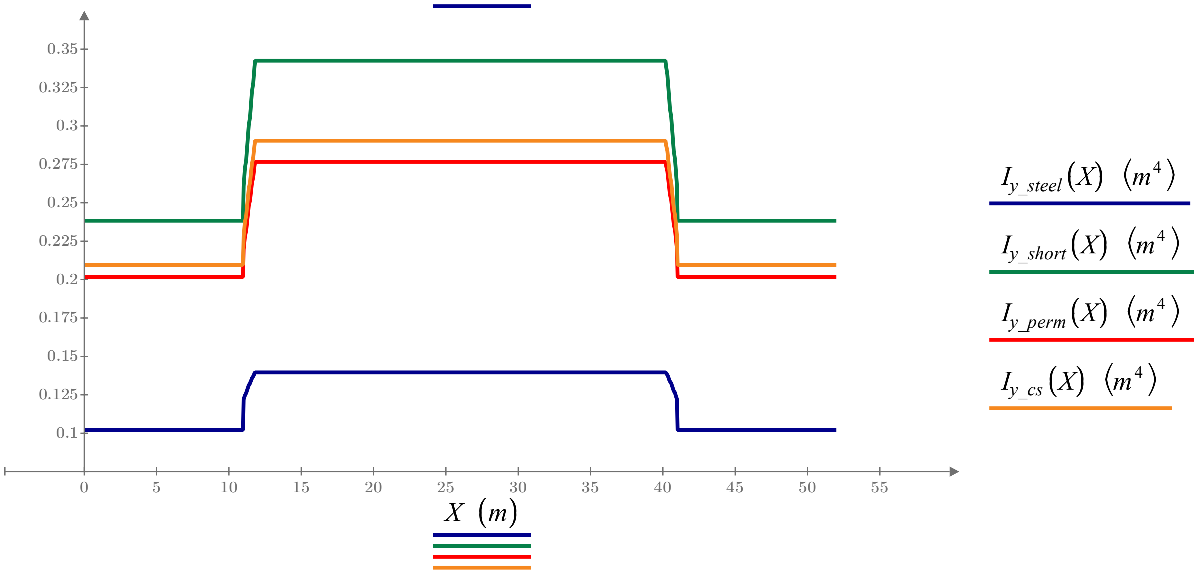


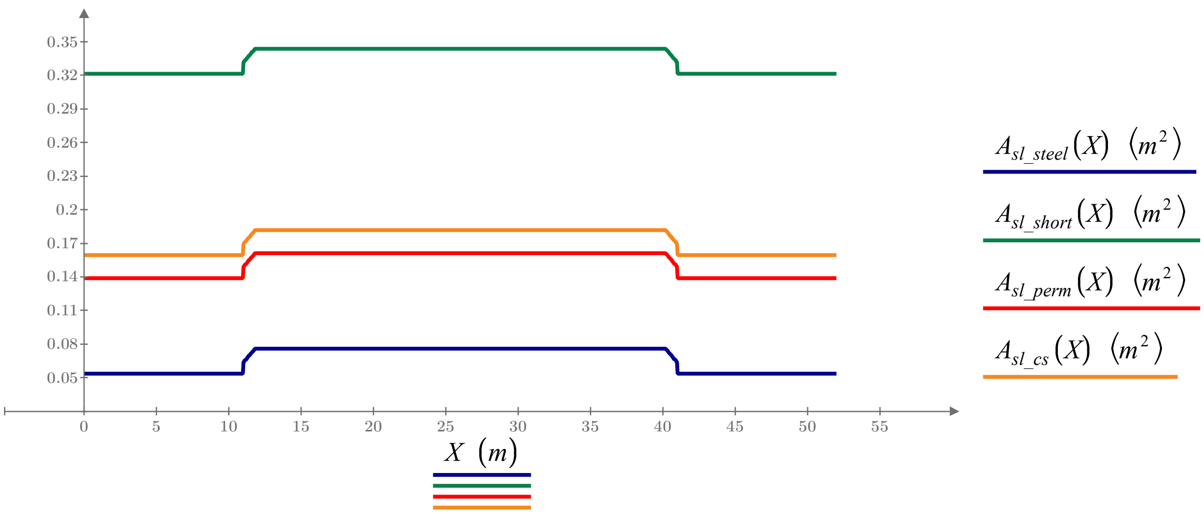
$V_{d\_ULS}(X_{check\_v}) = 4516\ kN$ Shear force at control point - ULS

6.2 Cross-sectional constants

For calculations, see chapter 5.







6.3 Stresses in steel cross-section

The stresses is calculated for each load case taking into acount load duration and creep. The stresses are then superpositioned.

6.3.1 Stresses during casting

$M := M_{ULS1}\left(X_{check\_m}\right) = 21218\ kN \cdot m$ Bending moment

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.14\ m^4$ Moment of inertia

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 1682\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2820\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot -z = -256\ MPa$ = $\sigma_{s.u.cast}\left(X_{check\_m}\right) = -256\ MPa$ Stresses in upper flange from loads durng casting

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 173\ MPa$ = $\sigma_{s.l.cast}\left(X_{check\_m}\right) = 173\ MPa$ Stresses in lower flange from loads durng casting

6.3.2 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses must be calculated for that specific section. The worst load case is determined dependent on the largest stress in each part whereas the multi-component load is the main load for the lower flange, and the temperature load is the worst load case for the upper flange.

6.3.2.1 Multi component loads (traffic)

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = -0.02\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.344\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.342\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) = 246\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2820\ mm$ Height of girder

$\sigma_{s.u} := \frac{0.9\ N}{A} + \frac{0.9\ M}{I} \cdot (-z) = -13\ MPa$ = $\sigma_{s.u.tr}\left(X_{check\_m}\right) = -13\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{1.5\ N}{A} + \frac{1.5\ M}{I} \cdot (h - z) = 233\ MPa$ = $\sigma_{s.l.tr}\left(X_{check\_m}\right) = 233\ MPa$ Stresses in lower flange from short term loads

6.3.2.2 Temperature loads

$M := M_{temp}\left(X_{check\_m}\right) = 11199\ kN \cdot m$ Bending moment

$N := N_{temp}\left(X_{check\_m}\right) = -6229\ kN$ Normal force

$A_s := A_{sl\_steel}\left(X_{check\_m}\right) = 0.076\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.344\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}\left(X_{check\_m}\right) = 0.14\ m^4$ Moment of inertia - steel section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.342\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}\left(X_{check\_m}\right) = 1682\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 246\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}\left(X_{check\_m}\right) = 2820\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 4569\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(1 - \left(\frac{A_s}{A} + \frac{A_s}{I}\left(z_s - z\right)^2\right)\right) = -2014\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{1.5\ N_s}{A_s} + \frac{1.5\ M_s}{I_s} \cdot \left(-z_s\right) = -122\ MPa$ = $\sigma_{s.u.temp}\left(X_{check\_m}\right) = -122\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{0.9\ N_s}{A_s} + \frac{0.9\ M_s}{I_s} \cdot \left(h - z_s\right) = 10\ MPa$ = $\sigma_{s.l.temp}\left(X_{check\_m}\right) = 10\ MPa$ Stresses in lower flange from short term loads

6.3.3 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.276\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) = 706\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2820\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot (-z) = -11\ MPa$ = $\sigma_{s.u.perm}(X_{check\_m}) = -11\ MPa$ Stresses in upper flange from additional permanent loads

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 32\ MPa$ = $\sigma_{s.l.perm}(X_{check\_m}) = 32\ MPa$ Stresses in lower flange from additional permanent loads

6.3.4 Stresses due to shrinkage

In the same manner as the temperature load must be calculated for the steel section, the shrinkage which acts on the concrete section must be converted for the steel section.

$M := M_{ULS4}(X_{check\_m}) = 5040\ kN \cdot m$ Bending moment

$N := N_{ULS4}(X_{check\_m}) = -5206\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.076\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_cs}(X_{check\_m}) = 0.181\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.14\ m^4$ Moment of inertia - steel section

$I := I_{y\_cs}(X_{check\_m}) = 0.29\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 1682\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_cs}(X_{check\_m}) = 609\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 2820\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 2425\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(\frac{A_s}{A} - \frac{A_s}{I} \cdot \left(z - \frac{h_{m.slab}}{2}\right) \cdot \left(z_s + \frac{h_{m.slab}}{2} - \left(z - \frac{h_{m.slab}}{2}\right)\right)\right) = -1323\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (-z_s) = -47\ MPa$ = $\sigma_{s.u.shrink}(X_{check\_m}) = -47\ MPa$ Stresses in upper flange due to shrinkage

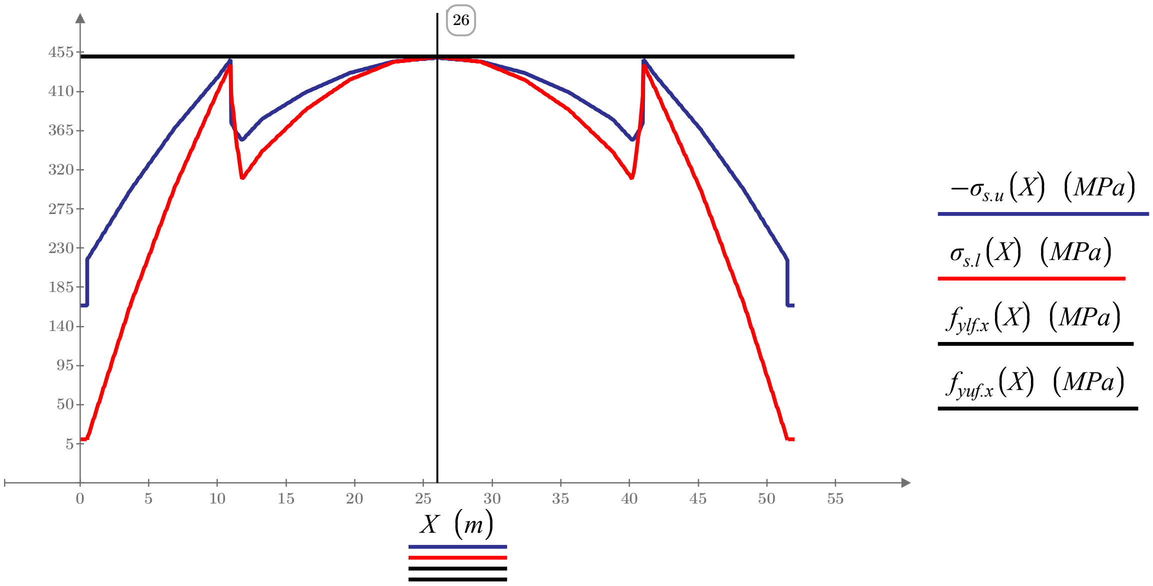
$\sigma_{s.l} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (h - z_s) = 2\ MPa$ = $\sigma_{s.l.shrink}(X_{check\_m}) = 2\ MPa$ Stresses in lower flange due to shrinkage

6.3.5 Summary of stresses

The stresses from the different phases are summarised accordingly.

$$\sigma_{s.u}(x) := \sigma_{s.u.cast}(x) + \sigma_{s.u.shrink}(x) + \sigma_{s.u.perm}(x) + \sigma_{s.u.tr}(x) + \sigma_{s.u.temp}(x)$$ Stresses in upper flange

$$\sigma_{s.l}(x) := \sigma_{s.l.cast}(x) + \sigma_{s.l.shrink}(x) + \sigma_{s.l.perm}(x) + \sigma_{s.l.tr}(x) + \sigma_{s.l.temp}(x)$$ Stresses in lower flange



$$\eta_{\sigma.u.max} := max\left(\frac{-\sigma_{s.u}(X)}{f_{yuf.x}(X)}\right) = 100\%$$ Maximum utilization rate - upper flange

$$\eta_{\sigma.l.max} := max\left(\frac{\sigma_{s.l}(X)}{f_{ylf.x}(X)}\right) = 100\%$$ Maximum utilization rate - lower flange

6.4 Calculation of stresses in concrete

The stresses in the concrete is calculated with the same principle as the steel taking creep into account.

6.4.1 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses for the concrete must be calculated separately.

6.4.1.1 Traffic and wind loads

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.344\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.342\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) + h_{m.slab} = 566\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 2820\ mm$ Height of girder

$n_{\Gamma} := n_{L_2} = 5.88$ Modular ratio

$\sigma_c := \left(\dfrac{N}{A} + \dfrac{-M \cdot z}{I}\right) \cdot \dfrac{1}{n_{\Gamma}} = -6\ MPa$ = $\sigma_{c.short}\left(X_{check\_m}\right) = -6\ MPa$ Stresses in concrete from short-term loads

6.4.1.2 Temperature load

$M := M_{temp}\left(X_{check\_m}\right) = 11199\ kN \cdot m$

$N := N_{temp}\left(X_{check\_m}\right) = -6229\ kN$

$n_{\Gamma} := n_{L_2} = 5.88$ Modular ratio

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.344\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_{\Gamma}} = 0.268\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.342\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}(X_{check\_m}) \cdot \frac{h_{m.slab}^{3}}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}(X_{check\_m})}{n_\Gamma} \cdot \frac{h_{m.slab}^{3}}{12} = 0.002\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_short}(X_{check\_m}) = 246\ mm$ Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \frac{I_{c.eff}}{I} \cdot M = 75\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(\frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot (z_s - z) \cdot \left(z + \frac{h_{m.slab}}{2}\right)\right) = -2014\ kN$ Normal force imposed on concrete slab

$\sigma_c := \left(\frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c}\right) = 0.39\ MPa$ = $\sigma_{c.temp}(X_{check\_m}) = 0.39\ MPa$ Stresses in concrete from temperature loads

6.4.2 Stresses due to additional permanent loads

$M := M_{ULS3}(X_{check\_m}) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}(X_{check\_m}) = 0.276\ m^4$ Moment of inertia

$z := z_{tp\_perm}(X_{check\_m}) + h_{m.slab} = 1026\ mm$ Center of gravity from top flange

$h := h_{beam}(X_{check\_m}) = 2820\ mm$ Height of girder

$n_\Gamma := n_{L_1} = 18.48$ Creep

$\sigma_c := \frac{-M \cdot z}{I} \cdot \frac{1}{n_\Gamma} = -0.8\ MPa$ = $\sigma_{c.perm}(X_{check\_m}) = -0.8\ MPa$ Stresses in concrete from permanent loads

6.4.3 Stresses due to shrinkage

$M := M_{ULS4}\left(X_{check\_m}\right) = 5040\ kN \cdot m$ Bending moment

$N := N_{ULS4}\left(X_{check\_m}\right) = -5206\ kN$ Normal force

$n_{\Gamma} := n_{L_3} = 14.91$ Modular ratio considering creep

$A := A_{sl\_cs}\left(X_{check\_m}\right) = 0.181\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_{\Gamma}} = 0.106\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_cs}\left(X_{check\_m}\right) = 0.29\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \dfrac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \dfrac{b_{eff}\left(X_{check\_m}\right)}{n_{\Gamma}} \cdot \dfrac{h_{m.slab}^{\ 3}}{12} = 0.001\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_cs}\left(X_{check\_m}\right) = 609\ mm$ Center of gravity - composite section

$z_c := \dfrac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \dfrac{I_{c.eff}}{I} \cdot M = 16\ kN \cdot m$ Bending moment imposed on concrete slab

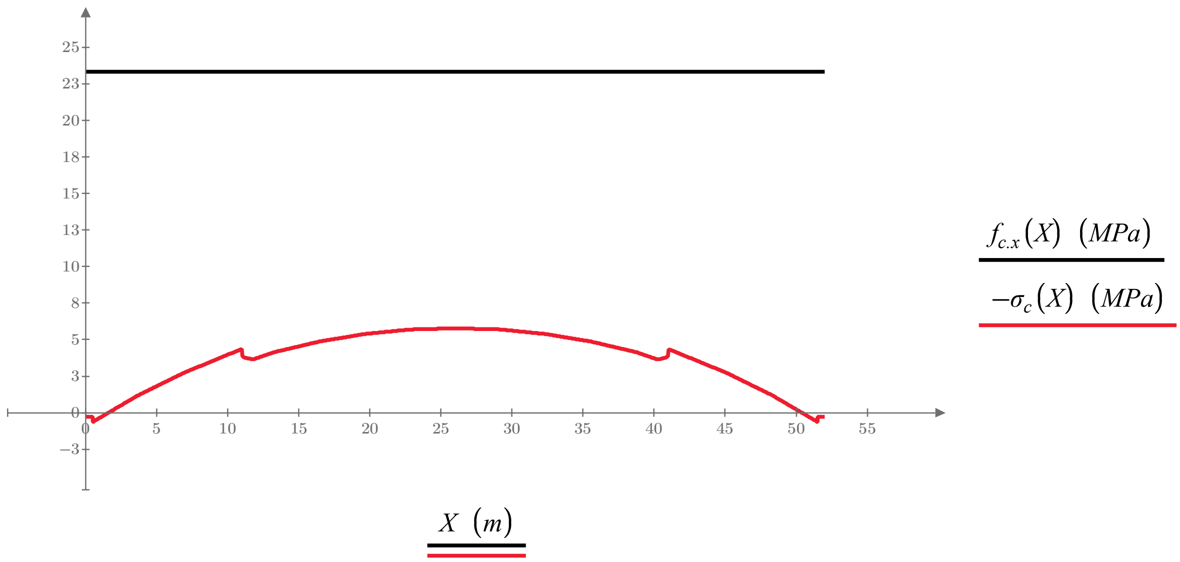
$N_c := N \cdot \left(1 - \dfrac{A_{c.eff}}{A} - \dfrac{A_{c.eff}}{I} \cdot \left(z + \dfrac{h_{m.slab}}{2}\right)^2\right) = -1052\ kN$ Normal force imposed on concrete slab

$\sigma_c := \dfrac{-N_c}{A_c} + \dfrac{M_c \cdot -z_c}{I_c} = 0.5\ MPa$ = $\sigma_{c.shrink}\left(X_{check\_m}\right) = 0.5\ MPa$ Stresses in concrete from shrinkage

6.4.4 Summary of stresses

$$\sigma_c(x) := \sigma_{c.short}(x) + \sigma_{c.temp}(x) + \sigma_{c.perm}(x) + \sigma_{c.shrink}(x)$$

Stresses in concrete



$$\eta_{\sigma.c.max} := max\left(\frac{-\sigma_c(X)}{f_{c.x}(X)}\right) = 25\%$$

Maximum utilization rate - upper flange

6.5 Shear capacity

The shear capacity of the corrugated steel girder is calculated accoridng to SS-EN 1993-1-5, Appendix D.
The local- and global buckling factor are calculated according to SS-EN 1993-1-5, Appendix D.

Local buckling factor

$a_{c1} = 135\ mm$ $\quad a_{c2} = 136\ mm$ — Corrugation geometries

$a_{cmax} := max\left(a_{c1}, a_{c2}\right) = 136\ mm$ — SS-EN 1993-1-5 D.2.2.(2)

$tw := t_w\left(X_{check\_v}\right) = 7\ mm$ — Web thickness

$\tau_{cr} := 4.83 \cdot E_s \cdot \left(\frac{tw}{a_{cmax}}\right)^2 = 2555\ MPa$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.7

$\lambda_c := \sqrt{\frac{f_{yw}\left(X_{check\_v}\right)}{\tau_{cr} \cdot \sqrt{3}}} = 0.329$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.6

$\chi_l := min\left(\frac{1.15}{0.9 + \lambda_c}, 1\right) = 0.94$ $=$ $\chi_{c.l}\left(X_{check\_v}\right) = 0.94$ — Reduction factor local buckling - SS-EN 1993-1-5 D.2.2.(2) Equation D.5

Global buckling factor

$D_X := \frac{E_s \cdot tw^3}{12 \cdot \left(1 - \nu^2\right)} \cdot \frac{w_c}{s_c} = 6\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$D_Z := \frac{E_s \cdot tw \cdot {a_{c3}}^2}{12} \cdot \frac{\left(3\ a_{c1} + a_{c2}\right)}{w_c} = 1648\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$hw := h_w\left(X_{check\_v}\right) = 2735\ mm$ — Web height

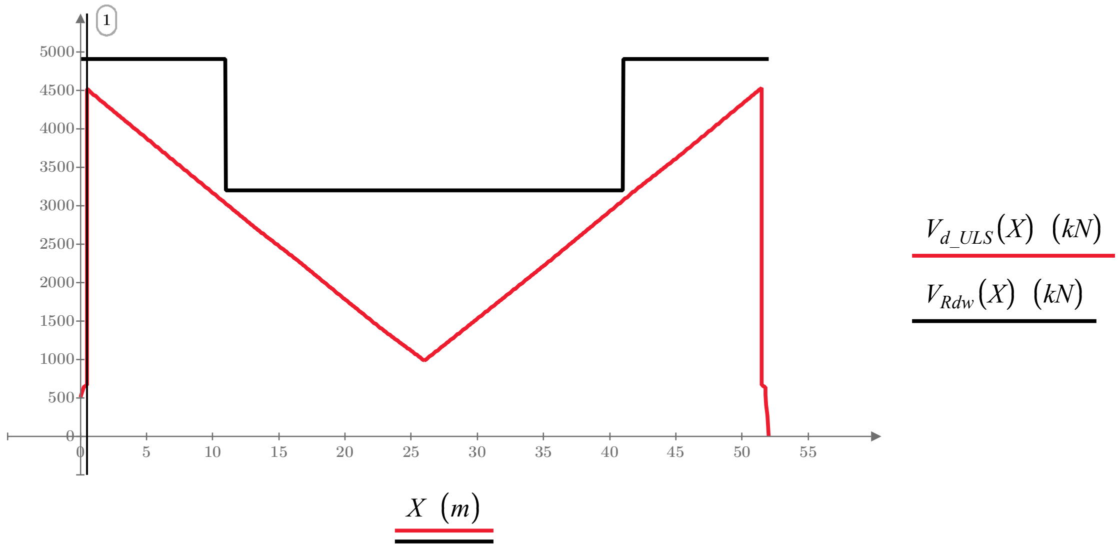
$\tau := \frac{32.4}{tw \cdot hw^2} \cdot \sqrt[4]{\frac{D_X}{N \cdot m} \cdot \frac{{D_Z}^3}{\left(N \cdot m\right)^3}}\ N \cdot m = 247\ MPa$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.10

$\lambda := \sqrt{\frac{f_{yw}\left(X_{check\_v}\right)}{\tau \cdot \sqrt{3}}} = 1.06$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.9

$\chi_g := min\left(\frac{1.5}{0.5 + \lambda^2}, 1\right) = 0.93$ $=$ $\chi_{c.g}\left(X_{check\_v}\right) = 0.93$ — Global buckling factor SS-EN 1993-1-5 D.2.2.(3) Equation D.8

$\chi_C := min\left(\chi_g, \chi_l\right) = 0.93$ $=$ $\chi_c\left(X_{check\_v}\right) = 0.93$ — SS-EN 1993-1-5 D.2.2

$V_{Rd} := \chi_C \cdot \frac{f_{yw}\left(X_{check\_v}\right)}{\gamma_{M1} \cdot \sqrt{3}} \cdot hw \cdot tw = 4908\ kN$ $=$ $V_{Rdw}\left(X_{check\_v}\right) = 4908\ kN$ — SS-EN 1993-1-5 D.2.2.(1) Eq.D.4



$\eta_{V.max} = 96\%$ Maximum utilization rate

6.7 Studs

6.7.1 Ultimate limit state

The capacity of the studs in the ultimate limit state are calculated according to SS-EN 1994-2 6.6.3

$d_{stud} \equiv 22\ mm$ — Diameter of stud

$h_{stud} \equiv 200\ mm$ — Length of stud

$f_{ub} = 800\ MPa$ — Characteristic strength of studs

$f_{u\_stud} := min\left(f_{ub}, 500\ MPa\right) = 500\ MPa$ — Ultimate strength shear stud - SS-EN 1994-2 6.6.3.1 (1)

$\frac{h_{stud}}{d_{stud}} = 9$ — Ratio height- diameter

$$\alpha_{stud} := \left\| \begin{array}{l} \text{if } 3 \le \frac{h_{stud}}{d_{stud}} \le 4 \\ \quad \left\| a \leftarrow 0.2 \cdot \left(1 + \frac{h_{stud}}{d_{stud}}\right) \right. \\ \text{else if } 4 < \frac{h_{stud}}{d_{stud}} \\ \quad \left\| a \leftarrow 1 \right. \\ a \end{array} \right. = 1.0$$

Correction factor for length to diameter ratio shear stud SS-EN 1994-2 6.6.3.1 (1)

$$P_{rd} := min\left(\left\| \begin{array}{c} \frac{0.8 \cdot f_{u\_stud} \cdot \pi \cdot d_{stud}^{2}}{4 \cdot \gamma_V} \\ \frac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^{2} \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \end{array} \right\| \right) = 122\ kN$$

Capacity of one shear stud SS-EN 1994-2 6.6.3.1 Eq: 6.18,6.19

$S_{uf\_short}\left(X_{check\_v}\right) = \left(77 \cdot 10^{-3}\right)\ m^{3}$ — Second moment of area

$V_{ULS2}\left(X_{check\_v}\right) = 2538\ kN$ — Shear force

$I_{y\_short}\left(X_{check\_v}\right) = \left(238 \cdot 10^{-3}\right)\ m^{4}$ — Moment of inertia

$$\tau_{sh} := \frac{S_{uf\_short}\left(X_{check\_v}\right) \cdot V_{ULS2}\left(X_{check\_v}\right)}{I_{y\_short}\left(X_{check\_v}\right)} = 824\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for short term loads

$S_{uf\_perm}\left(X_{check\_v}\right) = \left(57 \cdot 10^{-3}\right)\ m^{3}$ — Second moment of area

$V_{ULS3}\left(X_{check\_v}\right) = 329\ kN$ Shear force

$I_{y\_perm}\left(X_{check\_v}\right) = \left(201 \cdot 10^{-3}\right)\ m^4$ Moment of inertia

$$\tau_{pe} := \frac{S_{uf\_perm}\left(X_{check\_v}\right) \cdot V_{ULS3}\left(X_{check\_v}\right)}{I_{y\_perm}\left(X_{check\_v}\right)} = 93\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for permanent loads

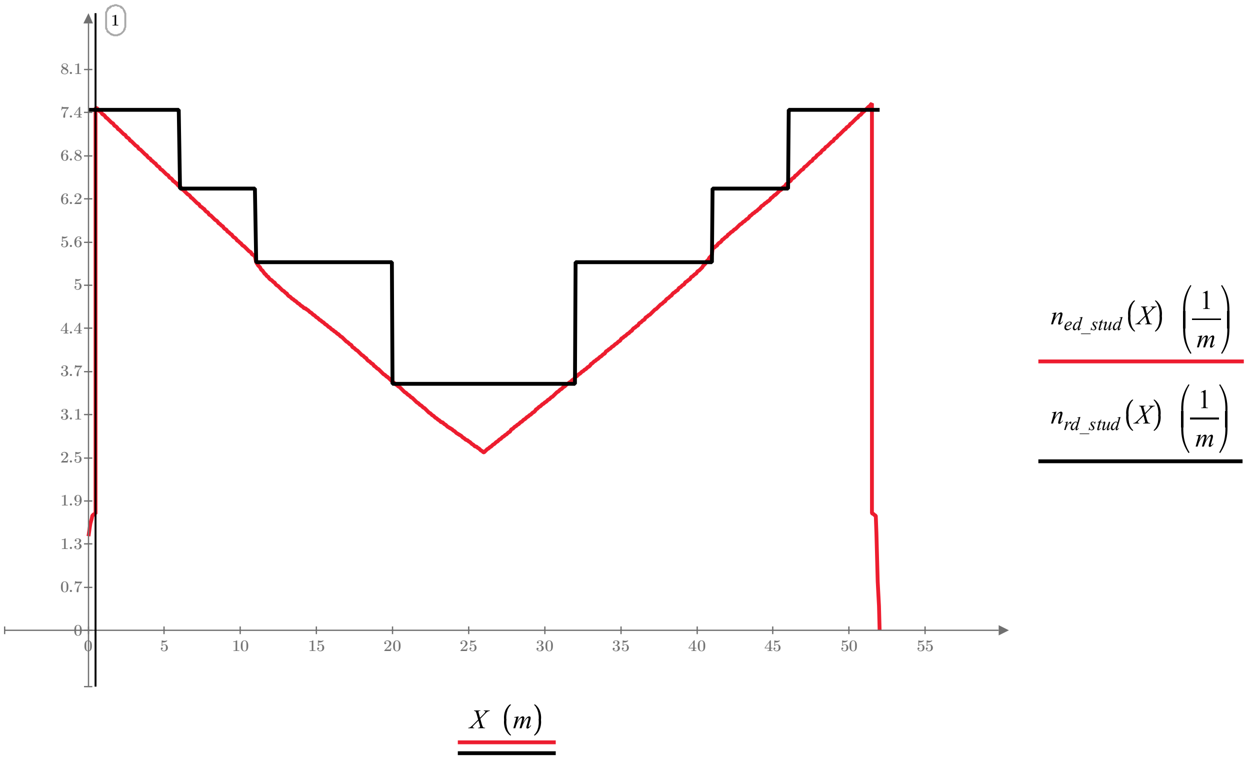
$$\tau := \left|\tau_{sh} + \tau_{pe}\right| = 917\ \frac{kN}{m}$$

Total shear force per meter between concrete and top flange

$n_{ed} := \dfrac{\tau + 0.9\ max\left(q_{1k\_br}, q_{tr\_br}\right)}{P_{rd}} = 7.5\ \dfrac{1}{m}$ = $n_{ed\_stud}\left(X_{check\_v}\right) = 7.5\ \dfrac{1}{m}$ Number of studs needed per meter

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



6.7.1.1 Additional studs for full anchorage

Case 1: Temperature and shrinkage causes the slab to contract and therefore they are working in the opposite direction as the shear flow from ULS- loads during bending.

The anchorage length is calculated according to SS-EN 1994-2, 6.9 (3).

$B_{out} = 2.525\ m$ Distance from centre web to outer part of edge beam

$B_{in} = 2.8\ m$ Distance from web to centerline bridge

$b := max\left(B_{out}, B_{in}\right) = 2.8\ m$

$l_{anch} := 1.5 \cdot b = 4.2\ m$ Anchorage length - SS-EN 1994-2, 6.9 (3)

$N_{cs\_stud} = 779\ kN$ Shrinkage force imposed on studs - calculated in chapter 4

$\left|N_{temp\_stud_1}\right| = 1770\ kN$ Temperature force imposed on studs causing contraction - calculated in chapter 4

$$n_{ed\_cs} := \frac{1.0\ N_{cs\_stud}}{l_{anch} \cdot P_{rd}} = 1.5\ \frac{1}{m}$$

$$n_{ed\_temp} := \frac{1.5\ \left|N_{temp\_stud_1}\right|}{l_{anch} \cdot P_{rd}} = 5.2\ \frac{1}{m}$$

$$n_{ed\_anch.1} := n_{ed\_cs} + n_{ed\_temp} = 6.7\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 7.5\ \frac{1}{m}$$

$$check_1 := \mathbf{if}\left(n_{ed\_anch.1} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_1 = \text{“No extra studs are needed”}$$

Case 2: Temperature causes the slab to expand and therefore working in the same direction as the shear flow from ULS- loads during bending. Shrinkage causes the slab to contract, i.e. working in the opposite direction.

$$-n_{ed\_cs} = -1.5\ \frac{1}{m}$$

$$N_{temp\_stud_2} = 1433\ kN$$

Temperature force imposed on studs causing expansion - calculated in chapter 4

$$n_{ed\_temp} := \frac{1.5 \cdot 0.6\ N_{temp\_stud_2}}{l_{anch} \cdot P_{rd}} = 2.5\ \frac{1}{m}$$

$$n_{ed\_stud}(0\ m) = 1.4\ \frac{1}{m}$$

$$n_{ed\_anch.2} := n_{ed\_stud}(0\ m) + n_{ed\_temp} - n_{ed\_cs} = 2.4\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 7.5\ \frac{1}{m}$$

$$check_2 := \mathbf{if}\left(n_{ed\_anch.2} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_2 = \text{“No extra studs are needed”}$$

Extra studs needed due to shrinkage and temperature

$n_{rd\_stud.adj}(X)$ Needed amount of studs with regards to extra anchorage due to temperature and shrinkage

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section (ULS- loads)

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section

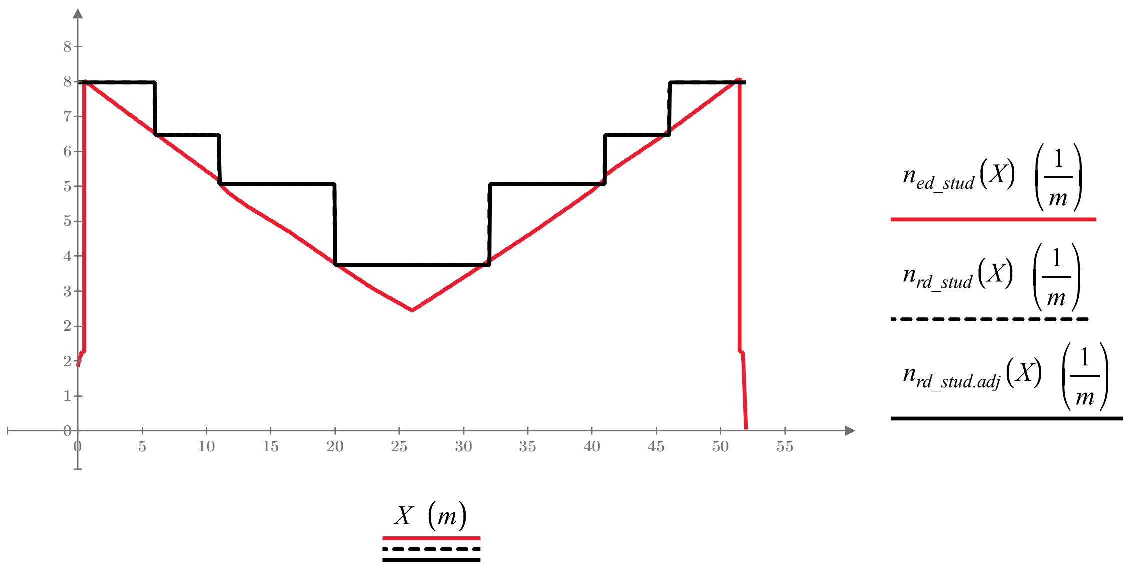


Table. Showing the adjusted need for studs near supports

X	n_{rd}	$n_{rd.adj}$
0	7.5	**7.5**
0.5	7.5	**7.5**
4.2	7.5	**7.5**
6	6.7	**6.7**
11	5.6	**5.6**
20	4.0	**4.0**

6.7.2 Fatigue limit state

6.7.2.1 Capacity

The fatigue capacity is calculated according to SS-EN 1993-1-9, Table 8.5 and SS-EN 1994-2, 6.8.3.

$\Delta\tau_c := 90\ MPa$ SS-EN 1993-1-9 - Table 8.5 (10)

$\Delta\tau_{E2} = \lambda_v \cdot \Delta\tau_c$

$\lambda_{v.1} := 1.55$ Bridge length less than 100m - SS-EN 1994-2 6.8.6.2 (4)

$Q_{mi} := 410\ kN$ Mean weight of large vehicles in the slow lane

$Q_0 := 480\ kN$

$N_{obs} := 0.05 \cdot 10^6$

$N_0 := 0.5 \cdot 10^6$

$\lambda_{v.2} := \frac{Q_{mi}}{Q_0} \cdot \left(\frac{N_{obs}}{N_0}\right)^{\frac{1}{8}} = 0.64$ SS-EN 1994-2 6.8.6.2 (4) and SS-EN 1993-2 Eq. 9.10

$t_{Ld} := 120$ Expected service life [years]

$\lambda_{v.3} := \left(\frac{120}{100}\right)^{\frac{1}{8}} = 1.02$

$\lambda_{v.4} := 1.0$ TSFS 2018:57 - 27 ch. 3 §

$\lambda_v := \lambda_{v.1} \cdot \lambda_{v.2} \cdot \lambda_{v.3} \cdot \lambda_{v.4} = 1.02$

$\gamma_{Ff} := 1.0$ SS-EN 1993-2, 9.3 (1)

$\gamma_{mF} := 1.0$ SS-EN 1994-2, 2.4.1.2 (6)

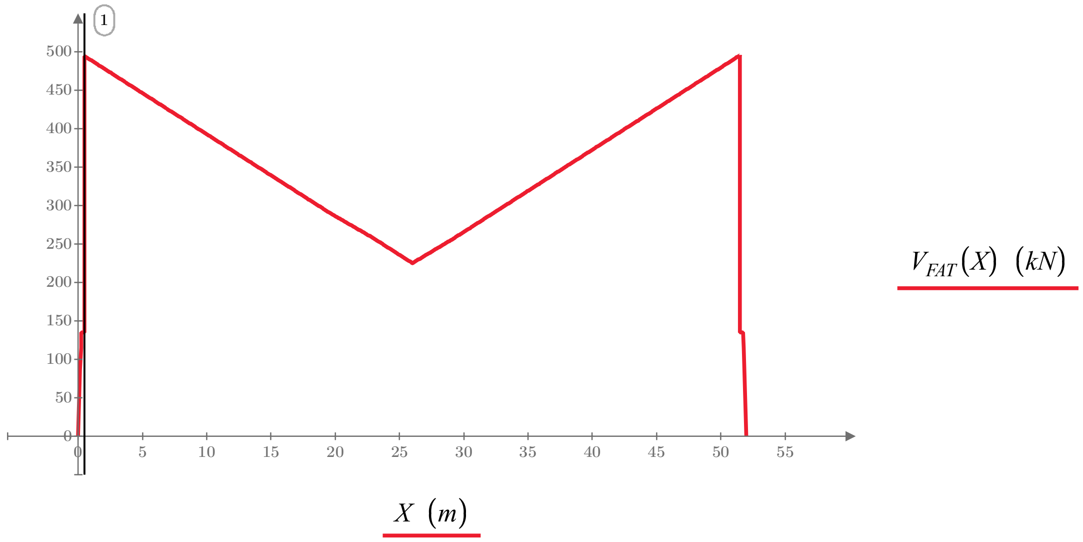
$\gamma_{Ff} \cdot \Delta\sigma_{E2} < \frac{\Delta\tau_c}{\gamma_{mF}}$

$F_{rd\_stud} := \Delta\tau_c \cdot \frac{\pi \cdot d_{stud}^{\ 2}}{4} = 34.2\ kN$ Shear fatigue capacity - one stud

$F_{rd\_stud} := \frac{F_{rd\_stud}}{\lambda_v} = 33.7\ kN$ Considering trafic load

6.7.2.2 Fatigue load

$V_{FAT}(X)$ Fatigue load from trafic - see Appendix X



$$\tau \cdot b = \frac{SV}{I} = \frac{stud\_capacity}{m}$$

$$S_{uf} := S_{uf\_short}\left(X_{check\_v}\right) = 0.077\ m^3$$

$$V := V_{FAT}\left(X_{check\_v}\right) = 494\ kN$$

$$I := I_{y\_short}\left(X_{check\_v}\right) = 0.238\ m^4$$

$$\Delta\tau := \frac{S_{uf} \cdot V}{I} = 161\ \frac{kN}{m}$$

$F_{rd\_stud} = 34\ kN$ Fatigue capacity - one stud

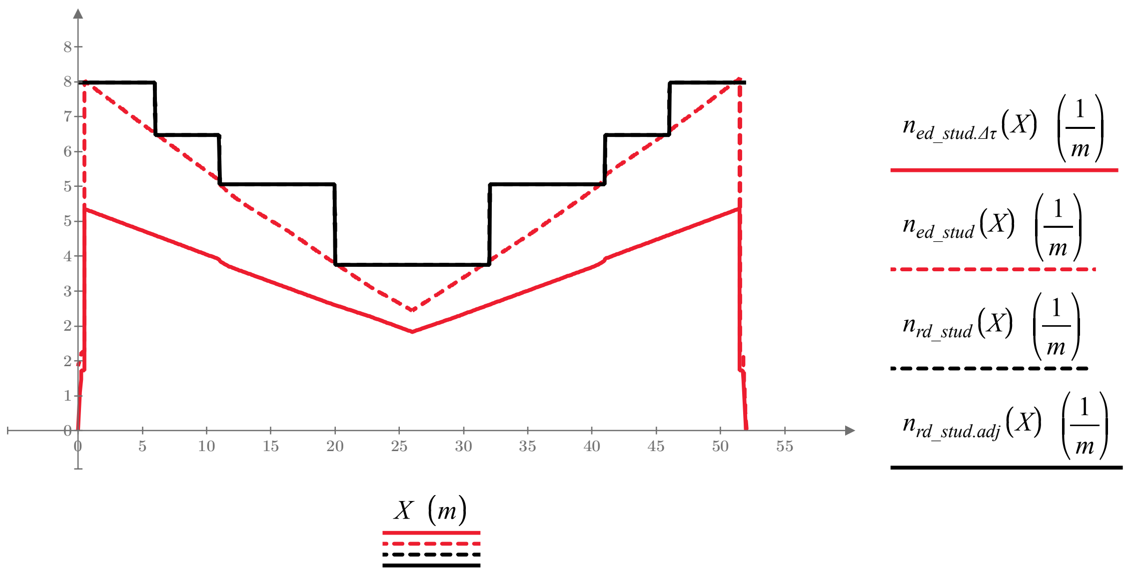
$$n_{ed\_\Delta\tau} := \frac{\Delta\tau}{F_{rd\_stud}} = 4.8\ \frac{1}{m} \quad = \quad n_{ed\_stud.\Delta\tau}\left(X_{check\_v}\right) = 4.8\ \frac{1}{m}$$

6.7.2.3 Design of studs with regards to fatigue

$n_{ed\_stud.\Delta\tau}(X)$ Needed amount of studs with regards to fatigue

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



Extra need of studs with regards to fatigue

$$Check := \textbf{if}\left(max\left(\frac{n_{ed\_stud.\Delta\tau}(X)}{n_{rd\_stud}(X)}\right), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$Check =$ “No extra studs are needed”

6.7.3 Summary - design of studs

Table. Showing the stud design for half of the span

X	$n_{stud.used}$	$n_{stud.need}$	$n_{stud.fat}$
0	**7.5**	1.4	0.0
0.5	**7.5**	7.5	4.8
4.2	**7.5**	6.8	4.4
6	**6.7**	6.4	4.2
11	**5.6**	5.3	3.7
20	**4.0**	3.6	2.7

6.8 Utilization rates

$\eta_{\sigma.u.max} = 100\%$	Stresses in top flange
$\eta_{\sigma.l.max} = 100\%$	Stresses in lower flange
$\eta_{\sigma.c.max} = 25\%$	Stresses in concrete
$\eta_{V.max} = 96\%$	Shear capacity
$\eta_{breathing} = 10\%$	Breathing

9 Material savings

The material savings that are achieved thanks to redesigning the bridge with corrugated webs and using stainless steel is compared to the old design. However it is important to note that dependent on the utilization rates the savings may always not be comparable. For this bridge the highest utilization ratios are larger than 95% (close to 99,8%) and therefore close to comparable.

9.1 Main girder

The main girder is redesigned with a corrugated web and a slightly slimmer design. Over the full bridge length this decreases the weight of the bridge. All savings are presented for the full bridge (full width). Consideration has been taken to the extra web length arising from the corrugation.

$V_{old} = 13.25\ m^3$ Steel volume - girder with flat web; original design

$V_{new} = 8.72\ m^3$ Steel volume - girder with corrugated web; new design

$m_{old\_girder} := 7850\ \frac{kg}{m^3} \cdot V_{old} = 104\ 10^3 \cdot kg$ Weight of girder - flat web; original design

$m_{new\_girder} := 7551\ \frac{kg}{m^3} \cdot V_{new} = 66\ 10^3 \cdot kg$ Weight of girder - corrugated web; new design

$\eta_{girder} := 1 - \frac{m_{new\_girder}}{m_{old\_girder}} = 37\%$ Material saving [%] - steel girder

$\Delta m_{saving\_girder} := m_{old\_girder} - m_{new\_girder} = 38\ 10^3\ kg$ Material saving [kg] - steel girder

9.2 Studs

$V_{stud\_old} = 72750\ cm^3$ Volume - studs; original design

$V_{stud\_new} = 44330\ cm^3$ Volume - studs; new design

$m_{old\_stud} := 7850\ \frac{kg}{m^3} \cdot V_{stud\_old} = 571\ kg$ Mass - studs; original design

$m_{new\_stud} := 7551\ \frac{kg}{m^3} \cdot V_{stud\_new} = 335\ kg$ Mass - studs; new design

$\eta_{stud} := 1 - \frac{m_{new\_stud}}{m_{old\_stud}} = 41\%$ Material saving [%] - studs

$\Delta m_{saving\_stud} := m_{old\_stud} - m_{new\_stud} = 236\ kg$ Material saving [kg] - studs

9.3 Cross-beams

$V_{cb}=0.59\ m^3$ Volume cross-beams old and new design

$m_{old\_cb}:=7850\ \frac{kg}{m^3}\cdot V_{cb}=4597\ kg$ Mass - cross-beams; original design

$m_{new\_cb}:=7551\ \frac{kg}{m^3}\cdot V_{cb}=4422\ kg$ Mass - cross-beams; new design

$\eta_{cb}:=1-\frac{m_{new\_cb}}{m_{old\_cb}}=4\%$ Material saving [%] - studs

$\Delta m_{saving\_cb}:=m_{old\_cb}-m_{new\_cb}=175\ kg$ Material saving [kg] - studs

9.4 Welds

$V_{weld\_old}=0.0051\ m^3$ Volume - welds; original design

$V_{weld\_new}=0.0038\ m^3$ Volume - welds; new design

$m_{old\_weld}:=7850\ \frac{kg}{m^3}\cdot V_{weld\_old}=40\ kg$ Mass - welds; original design

$m_{new\_weld}:=7551\ \frac{kg}{m^3}\cdot V_{weld\_new}=29\ kg$ Mass - welds; new design

$\eta_{weld}:=1-\frac{m_{new\_weld}}{m_{old\_weld}}=28\%$ Material saving [%] - welds

$\Delta m_{saving\_weld}:=m_{old\_weld}-m_{new\_weld}=11\ kg$ Material saving [kg] - welds

9.5 Total savings

$m_{old\_bridge}:=m_{old\_girder}+m_{old\_stud}+m_{old\_cb}=109\ 10^3\ kg$ Total mass of original design

$m_{new\_bridge}:=m_{new\_girder}+m_{new\_stud}+m_{new\_cb}=71\ 10^3\ kg$ Total mass of new design

$\eta_{bridge}:=1-\frac{m_{new\_bridge}}{m_{old\_bridge}}=35.4\%$ Material saving [%] - full bridge

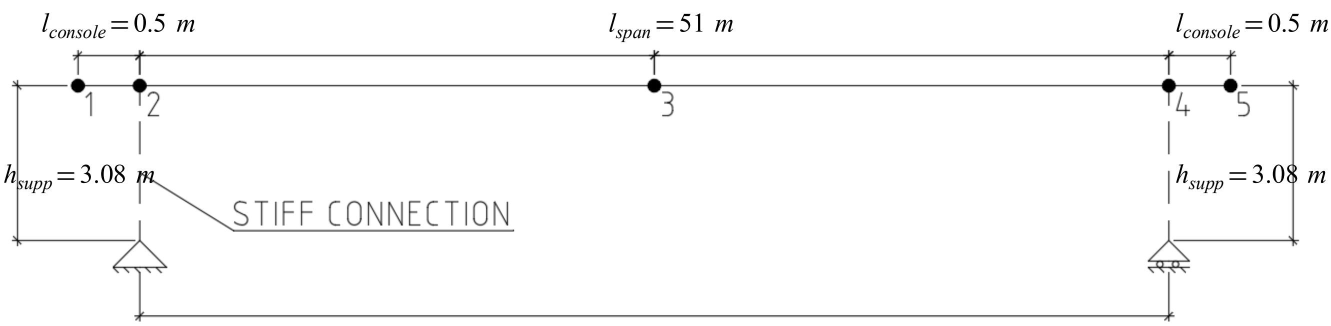
$\Delta m_{saving.tot}:=\Delta m_{saving\_girder}+\Delta m_{saving\_stud}+\Delta m_{saving\_cb}+\Delta m_{saving\_weld}=39\ 10^3\ kg$
Material saving [kg] - full bridge

Extracts - Calculation report 1.5h_w

3 System

In the following chapter the input data for the bridge's geometries is presented. The cross-sectional parameters that are presented is preliminary and used for the system analysis. In chapter 5 the final design is presented.

3.1 Primary system - longitudinal



The bridge is modelled as a simply supported bridge in the software StripStep-2. Due to the beams depth the supports are set offset from the neutral axis. This is modelled with a stiff connection in the system analysis.

3.1.1 Cross-section dimensions

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

The new height of the girder is calculated as: $2050\ mm \cdot 1.5 = 3075\ mm$

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	600	8	2990	50	700
500	35	600	8	2990	50	700

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	600	8	2990	50	700
10500	35	600	8	2990	50	700
10500	40	600	6	2980	55	700
11300	40	700	6	2980	55	900
25500	40	700	6	2980	55	900

Important! Two exactly the same values will not work with the linterp- function. Therefore 0.1 millimeter must be added for a X-value where you want to cross-sectional properties at the same time.

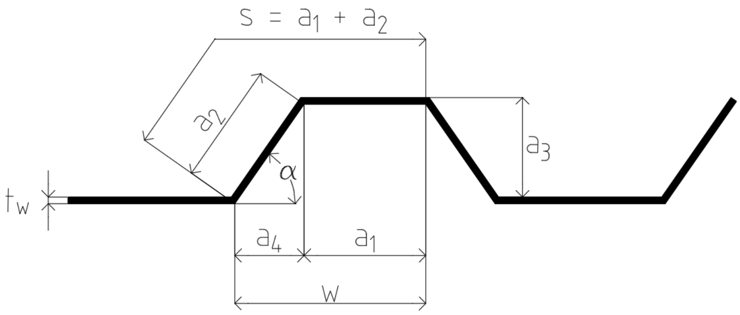
Element 3

S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	600	8	2990	50	700
500	35	600	8	2990	50	700

$$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=3075\ mm$$

$$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=3075\ mm$$

3.1.2 Corrugation shape



$a_{c1}:=120\ mm$ Flat-fold length

$\alpha_c:=36\ deg$ Corrugation angle

$a_{c3}:=70\ mm$ Corrugation depth

$a_{c2}:=\dfrac{a_{c3}}{sin\left(\alpha_c\right)}=119\ mm$ Length of angled part

$a_{c4}:=\dfrac{a_{c3}}{tan\left(\alpha_c\right)}=96\ mm$ Length of hypopythis

$s_c:=a_{c1}+a_{c2}=239\ mm$ Length of corrugation

$w_c:=a_{c1}+a_{c4}=216\ mm$ Straight length

$r_c:=\dfrac{s_c}{w_c}=1.11$ Ratio corrugation/flat-fold length

3.1.3.2 Effective width, steel flanges

Calculation of the effective width in the steel flanges with regards to shear lag is calculated according to SS-EN 1993-1-5 3.2.

$$b_{eff\_f} = \beta \cdot b_0 + \frac{t_w}{2}$$

Effective width with regards to shear lag under elastic conditions - Equation (3.1)

$$X_{check\_uf} := \frac{L_{bridge}}{2} = 26.0\ m$$

$$X_{check\_lf} := 0.2\ L_{bridge} = 10.4\ m$$

Upper flange

$$\alpha_0 := 1.0$$

Equation given in Table 3.1. For webs without any longitudinal stiffeners $\alpha_0 = 1.0$.

$$bfu := b_{fu}\left(X_{check\_uf}\right) = 700\ mm$$

Width of upper flange - see Appendix B - Preliminary sizing

$$tw := t_w\left(X_{check\_uf}\right) = 6\ mm$$

Thickness of web - see Appendix B - Preliminary sizing

$$b0 := \frac{bfu - tw}{2} = 347\ mm$$

Figure 3.2 (Notations for shear lag)

$$\kappa fu := \frac{\alpha_0 \cdot b0}{L_{span}} = 0.007 \quad = \quad \kappa_{fu}\left(X_{check\_uf}\right) = 0.007$$

Table 3.1

$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fu \le 0.02 \\ \quad \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fu \le 0.70 \\ \quad \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fu^2} \\ \text{else if } 0.70 < \kappa fu \\ \quad \beta \leftarrow \dfrac{1}{8.6\ \kappa fu^2} \end{array} \right\| = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for upper flange

$$2\ \beta_{check} \cdot b0 + tw = 700\ mm$$

$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad b \leftarrow bfu \\ \text{else} \\ \quad b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfu\right) \\ b \end{array} \right\| = 0.700\ m \quad = \quad b_{eff\_fu}\left(X_{check\_uf}\right) = 0.700\ m$$

Lower flange

$bfl := b_{fl}\left(X_{check\_lf}\right) = 700\ mm$ — Width of lower flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_lf}\right) = 8\ mm$ — Thickness of web - see Appendix B - Preliminary sizing

$b0 := \dfrac{bfl - tw}{2} = 346\ mm$ — Figure 3.2 (Notations for shear lag)

$\kappa fl := \dfrac{\alpha_0 \cdot b0}{L_{span}} = 0.007$ = $\kappa_{fl}\left(X_{check\_lf}\right) = 0.007$ — Table 3.1

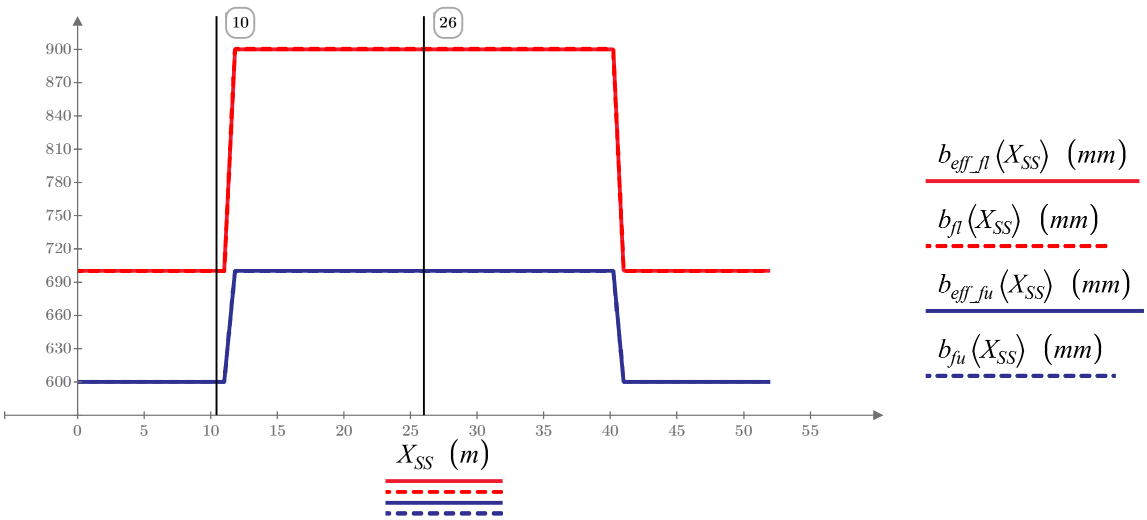
$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fl \le 0.02 \\ \quad \| \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fl \le 0.70 \\ \quad \| \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fl^2} \\ \text{else if } 0.70 < \kappa fl \\ \quad \| \beta \leftarrow \dfrac{1}{8.6\ \kappa fl^2} \end{array} \right. = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for lower flange

$2\ \beta_{check} \cdot b0 + tw = 700\ mm$

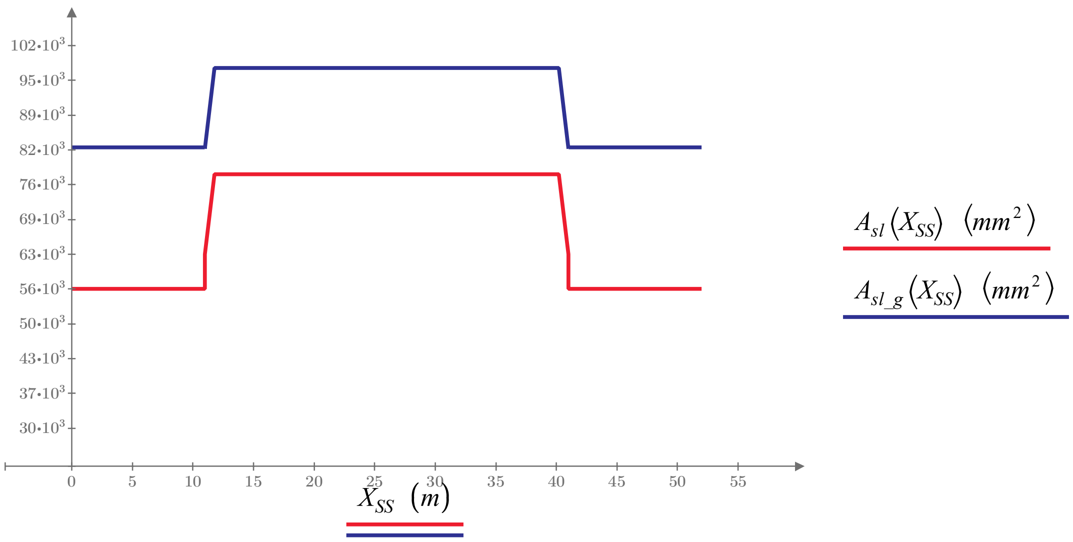
$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad \| b \leftarrow bfl \\ \text{else} \\ \quad \| b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfl\right) \\ b \end{array} \right. = 0.700\ m \quad = \quad b_{eff\_fl}\left(X_{check\_lf}\right) = 0.700\ m$$



3.1.4.2 Cross-sectional constants during construction

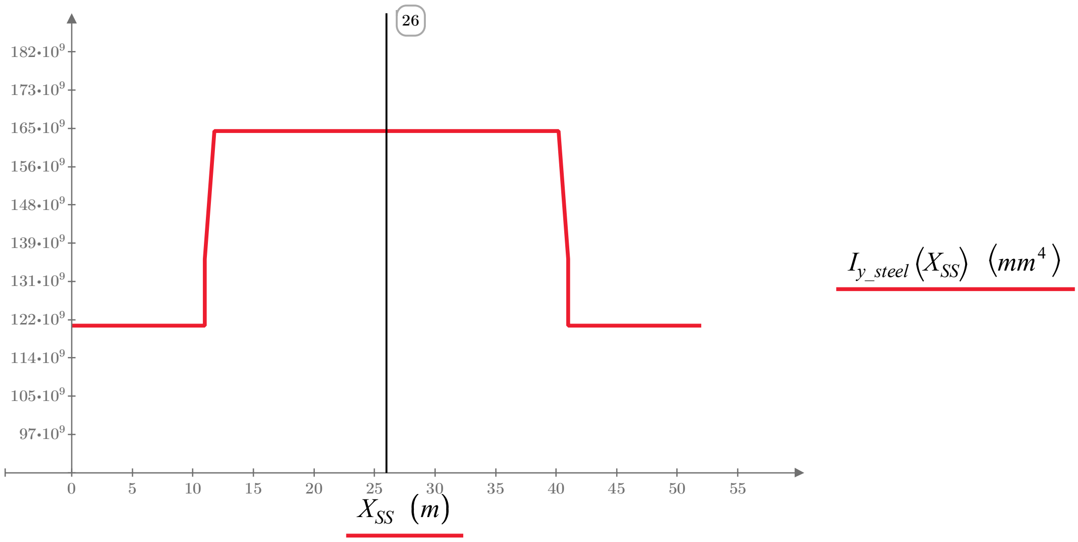
The bridge is checked so that the steel girder (alone) can withstand the loads that are imposed during construction such as the self-weight of curing concrete.

A_{sl_g} is the area of the steel including the web, for calculation of self-weight
A_{sl} is the area of the steel excluding the web, for stiffness calculations



$$A_{sl}\left(\frac{L_{bridge}}{2}\right) = 0.078\ m^2$$ Area at mid-span

I_{y_steel} is the area of the steel excluding the web, for stiffness calculations



$$I_{y\_steel}\left(\frac{L_{bridge}}{2}\right) = 0.1639\ m^4$$ Stiffness at mid-span

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]	[kN/m]
0.000	-1.913	3.075	1207.077	560	6.225
0.500	-1.913	3.075	1207.077	560	6.225

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]	[kN/m]
0.000	-1.913	3.075	1207.077	560	6.225
10.500	-1.913	3.075	1207.077	560	6.225
10.500	-1.885	3.075	1355.194	625	6.212
11.300	-1.954	3.075	1639.354	775	7.344
25.500	-1.954	3.075	1639.354	775	7.344
39.700	-1.954	3.075	1639.354	775	7.344
40.500	-1.885	3.075	1355.194	625	6.212
40.500	-1.913	3.075	1207.077	560	6.225
51.000	-1.913	3.075	1207.077	560	6.225

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]	[kN/m]
0.000	-1.913	3.075	1207.077	560	6.225
0.500	-1.913	3.075	1207.077	560	6.225

System model used in Strip-Step2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.075	ZR	
					2
3	51.500	0.00	-3.075	YZR	
					3
4	52.000	0.00			-3

3.1.4.3 Cross-sectional constants for variable loads (short term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.198	3.395	3219.87	3239
0.500	-0.198	3.395	3219.87	3239

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.198	3.395	3219.87	3239
10.500	-0.198	3.395	3219.87	3239
10.500	-0.227	3.395	3497.10	3304
11.300	-0.314	3.395	4347.57	3454
25.500	-0.314	3.395	4347.57	3454
39.700	-0.314	3.395	4347.57	3454
40.500	-0.227	3.395	3497.10	3304
40.500	-0.198	3.395	3219.87	3239
51.000	-0.198	3.395	3219.87	3239

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.198	3.395	3219.87	3239
0.500	-0.198	3.395	3219.87	3239

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.075	ZR	
					2
3	51.500	0.00	-3.075	YZR	
					3
4	52.000	0.00			-3

3.1.4.4 Cross-sectional constants for additional permanent loads (long term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]
0	-0.662	3.395	2666.540	1413
0.5	-0.662	3.395	2666.540	1413

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]
0	-0.662	3.395	2666.540	1413
10.5	-0.662	3.395	2666.540	1413
10.5	-0.705	3.395	2870.529	1478
11.3	-0.846	3.395	3460.363	1628
25.5	-0.846	3.395	3460.363	1628
39.7	-0.846	3.395	3460.363	1628
40.5	-0.705	3.395	2870.529	1478
40.5	-0.662	3.395	2666.540	1413
51	-0.662	3.395	2666.540	1413

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	[10^4m^4]	[10^4m^2]
0	-0.662	3.395	2666.540	1413
0.5	-0.662	3.395	2666.540	1413

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.075	ZR	
					2
3	51.500	0.00	-3.075	YZR	
					3
4	52.000	0.00			-3

3.1.4.5 Cross-sectional constants for shrinkage analysis

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.558	3.395	2788.880	1617
0.5	-0.558	3.395	2788.880	1617

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.558	3.395	2788.880	1617
10.5	-0.558	3.395	2788.880	1617
10.5	-0.600	3.395	3006.644	1682
11.3	-0.734	3.395	3646.093	1832
25.5	-0.734	3.395	3646.093	1832
39.7	-0.734	3.395	3646.093	1832
40.5	-0.600	3.395	3006.644	1682
40.5	-0.558	3.395	2788.880	1617
51	-0.558	3.395	2788.880	1617

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.558	3.395	2788.880	1617
0.5	-0.558	3.395	2788.880	1617

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.075	ZR	
					2
3	51.500	0.00	-3.075	YZR	
					3
4	52.000	0.00			-3

4 Loads and load combinations

4.1 Permanent loads

4.1.1 Self-weight

4.1.1.1 Steel

The self-weight is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: STEEL

$\rho_{st} = 75.51 \ \frac{kN}{m^3}$ Self-weight of stainless steel - SS-EN 10088-1:2014 Table E.1 or E.2

The self-weight for each element is calculated in chapter 3.

4.1.1.2 Concrete

The self-weight of the slab is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: SLAB

$Wet\_Concrete := \text{“NO”}$ "YES" or "NO" dependent if the previous designer has used the weight of wet concrete

$\rho_c := \mathbf{if}\left(Wet\_Concrete = \text{“NO”}, 25 \ \frac{kN}{m^3}, 26 \ \frac{kN}{m^3}\right) = 25 \ \frac{kN}{m^3}$ Self-weight of concrete (reinforced) - SS-EN 1992-1-1 Table A.1

$A_{slab} = 3.47 \ m^2$ Area of slab, see chapter 3

$g_{slab} := \frac{A_{slab} \cdot \rho_c}{2} = 43.4 \ \frac{kN}{m}$ Self-weight of concrete slab, (half of the load goes to each girder) - applied in the casting stage

If the previous designer has considered that the hardened concrete has a smaller self-weight a reduction in self-weight is applied in the system analysis for permanent loads.

Appendix X. Name in Strip-Step2: AVSLAB

$$g_{slab.perm} := \mathbf{if}\left(Wet\_Concrete = \text{“NO”}, 0 \ \frac{kN}{m}, \frac{A_{slab} \cdot (-1) \ \frac{kN}{m^3}}{2}\right) = 0.000 \ \frac{kN}{m}$$

4.1.2 Shrinkage

The shrinkage force is calculated and used in Strip-Step2, Appendix X. Name in Strip-Step2: E:SHRINK

$h_0 = 341\ mm$ Equivalent thickness, calculated in chapter 2

$t = 120\ yr$

$t = 43829.1\ day$

$f_{cm} = 43\ MPa$ Mean compressive strength - see chapter 2

$t_s := 1\ day$

$RH := 80\%$ Krav Brobyggande B.3.1.5

$$\beta_{ds} := \frac{\frac{t - t_s}{day}}{\left(\frac{t - t_s}{day}\right) + 0.04 \cdot \sqrt{\left(\frac{h_0}{1\ mm}\right)^3}} = 0.994$$ SS-EN 1992-1-1 3.1.4 Equation 3.10

$$k_h := \begin{array}{l} \text{if } 200\ mm \le h_0 < 300\ mm \\ \quad \left\| 0.85 - \left(\frac{h_0}{mm} - 200\right) \cdot 0.001 \right. \\ \text{else if } 300\ mm \le h_0 < 500\ mm \\ \quad \left\| 0.75 - \left(\frac{h_0}{mm} - 300\right) \cdot 0.0005 \right. \\ \text{else if } h_0 \ge 500\ mm \\ \quad \| 0.7 \\ \text{else} \\ \quad \| 1.0 \end{array} = 0.73$$ SS-EN 1992-1-1 3.1.4 table 3.3

$RH_0 := 100\%$

$\alpha_{ds1} := 4$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$\alpha_{ds2} := 0.12$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0}\right)^3\right) = 0.76$$ SS-EN 1992-1-1 Appendix B.2 equation B12

$f_{cmo} := 10\ MPa$ SS-EN 1992-1-1 Appendix B.2 (1)

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}\right)}\right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$ SS-EN 1992-1-1 Appendix B.2 equation B11

$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 1.84 \cdot 10^{-4}$ Drying shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.9

$\varepsilon_{ca0} := 2.5 \cdot \left(\frac{f_{ck} - f_{cmo}}{MPa}\right) \cdot 10^{-6} = 6.3 \cdot 10^{-5}$ SS-EN 1992-1-1 3.1.4 Equation 3.12

$\beta_{as} := 1 - e^{\left(-0.2 \cdot \sqrt{\frac{t - t_s}{day}}\right)} = 1.0$ SS-EN 1992-1-1 3.1.4 Equation 3.13

$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca0} = 6.25 \cdot 10^{-5}$ Autogenous shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.11

$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.8

4.1.2.1 Shrinkage force

The shrinkage force and corresponding moment is calculated accordingly:

$n_{L\_cs} = 14.91$ Modular ratio accounting for creep and shrinkage

$n_{L\_short} = 5.88$ Modular ratio

$E_{c.eff} := \frac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab (half of the cross-section used in system analysis)

$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5206\ kN$ Total shrinkage force

$e_{cs} := z_{tp\_cs}(0\ m) = 0.558\ m$ The shrinkage force is applied in the center of gravity for the composite section

$M_{cs} := F_{cs} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right) = 3737\ kN \cdot m$ Total bending moment

Shrinkage - anchorage of studs

Forces in the concrete

$$F_{c\_cs} := F_{cs} \cdot \left(1 - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{sl\_cs}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{y\_cs}(0\ m)} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right)^2\right) = 785\ kN$$

Force in concrete

$$b_{eff}(0\ m) = 4.925\ m$$

Effective width of flange

$$M_{c\_cs} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_cs}} \cdot h_{m.slab}^{\ 3}}{I_{y\_cs}(0\ m)} \cdot M_{cs} = 12\ kN \cdot m$$

Moment in concrete

$$\sigma_1 := \frac{F_{c\_cs}}{A_{slab.fic}} - \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 3}} = 0.35\ MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{F_{c\_cs}}{A_{slab.fic}} + \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 3}} = 0.64\ MPa$$

Stress in concrete, lower edge of slab

$$N_{cs} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = 785\ kN \quad = \quad F_{c\_cs} = 785\ kN$$

Force imposed on studs caused by shrinkage (used in chapter 6)

4.2.2.1 Horizontal side trafic load

$P_{h\_side\_car} := max\left(\begin{bmatrix} 25\% \cdot Q_{1k\_br} \\ 25\% \cdot Q_{tr\_br} \end{bmatrix}\right) = 109.1\ kN$ Horisontal side force from the acceleration load (used in chapter 7)

$P_{v\_side\_car} := P_{h\_side\_car} \cdot \dfrac{h_{m.slab}}{2 \cdot B_{in}} = 6.2\ kN$ Vertical side force from the acceleration load (used in chapter 7)

4.2.3 Temperature load

Temperatures are determined according to SS-EN 1991-1-5, 6.1.3 unless otherwise stated.

Load case 1 - local temperature differences for Hudiksvall

$T_0 = 10\ °$ A.1(3)

$T_{min} = -38\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{max} = 34\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4\ ° = -34\ °$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min} = 44\ °$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4\ ° = 38\ °$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0 = 28\ °$ Expansion - Equation 6.1

$\Delta T := T_{e.max} - T_{e.min} = 72\ °$ Total temperature difference

Load case 2 - either of the components are larger than the other

$\Delta T_{c2st} := 15\ °$ Temperature difference between concrete and steel - SS-EN 1991-1-5, 6.1.6

4.2.3.1 Coefficients of thermal linear expansion

For composite bridges normally it is suggested to use the same thermal linear expansion coefficient, according to SS-EN 1991-1-5 Table C.1. However for stainless steels the thermal linear expansion coefficient is much larger than for concrete and hence a more through calculation is needed for the strain.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

4.2.3.2 Strains for the different load cases

$\Delta\varepsilon_{LC.1\_con} := -(\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.con}}{\circ} = -26.4\ 10^{-5}$ Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp} := (\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.exp}}{\circ} = 16.8\ 10^{-5}$ Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss} \cdot \frac{\Delta T_{c2st}}{\circ} = 24.0\ 10^{-5}$ Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c \cdot \frac{\Delta T_{c2st}}{\circ} = 15.0\ 10^{-5}$ Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st}, \Delta\varepsilon_{LC.2\_c}\right) = 24.0\ 10^{-5}$ Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp_1} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con} = -50.4\ 10^{-5}$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp_2} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp} = 40.8\ 10^{-5}$ Maximum strain difference; temperature raises and the steel heatens up even higher

4.2.3.3 Temperature load - global analysis

The concrete is transformed to steel. Already defined parameters are calculated in chapter 3.

$\frac{b_{eff}(0\ m)}{n_{L\_short}} = 0.837\ m$ Width of transformed concrete

$\frac{A_{slab.fic}}{n_{L\_short}} = 0.268\ m^2$ Area of transformed concrete

$A_{sl}(0\ m) = 0.056\ m^2$ Area of composite section

$z_{tp\_short}(0\ m) = 198.5\ mm$ Distance from top of concrete to center of gravity for composite section

$I_{y\_short}(0\ m) = 0.322\ m^4$ Moment of inertia for composite section

$F_{temp} := \varepsilon_{temp} \cdot E_s \cdot A_{sl}(0\ m) = \begin{bmatrix} -5645 \\ 4570 \end{bmatrix} kN$ Force on composite section

$e_{temp\_F} := z_{tp\_short}(0\ m) = 198.5\ mm$ Level at which the force is imposed on the system

$e_{temp\_M} := z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) = 1.714\ m$ Eccentricity for the bending moment

$$M_{temp} := F_{temp} \cdot e_{temp\_M} = \begin{bmatrix} -9677 \\ 7834 \end{bmatrix} kN \cdot m$$

Bending moment - composite section

Anchorage of temperature load imposed on studs

Concrete

$$N_{c\_temp} := F_{temp} \cdot \left(\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{A_{sl\_short}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_short}}}{I_{y\_short}(0\ m)} \cdot \left(\left(z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) \right) \cdot \left(z_{tp\_short}(0\ m) + \frac{h_{m.slab}}{2} \right) \right) \right) = \begin{bmatrix} -1783 \\ 1444 \end{bmatrix} kN$$

$$M_{c\_temp} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_short}} \cdot h_{m.slab}^{\ 3}}{I_{y\_short}(0\ m)} \cdot M_{temp} = \begin{bmatrix} -69 \\ 56 \end{bmatrix} kN \cdot m$$

$$\sigma_1 := \frac{N_{c\_temp}}{A_{slab.fic}} - \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 2}} = \begin{bmatrix} -0.31 \\ 0.25 \end{bmatrix} MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{N_{c\_temp}}{A_{slab.fic}} + \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 2}} = \begin{bmatrix} -1.95 \\ 1.58 \end{bmatrix} MPa$$

Stress in concrete, lower edge of slab

$$N_{temp} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = \begin{bmatrix} -1783 \\ 1444 \end{bmatrix} kN$$

Force imposed on studs caused by shrinkage (used in chapter 6 when calculating the anchorage of the slab by the studs)

5 Capacity checks during construction

The capacity check that is carried out in this chapter is bending moment capacity with respect to lateral torsional buckling in the casting phase

In the casting phase the normalforces in the cross-section are neglectible.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

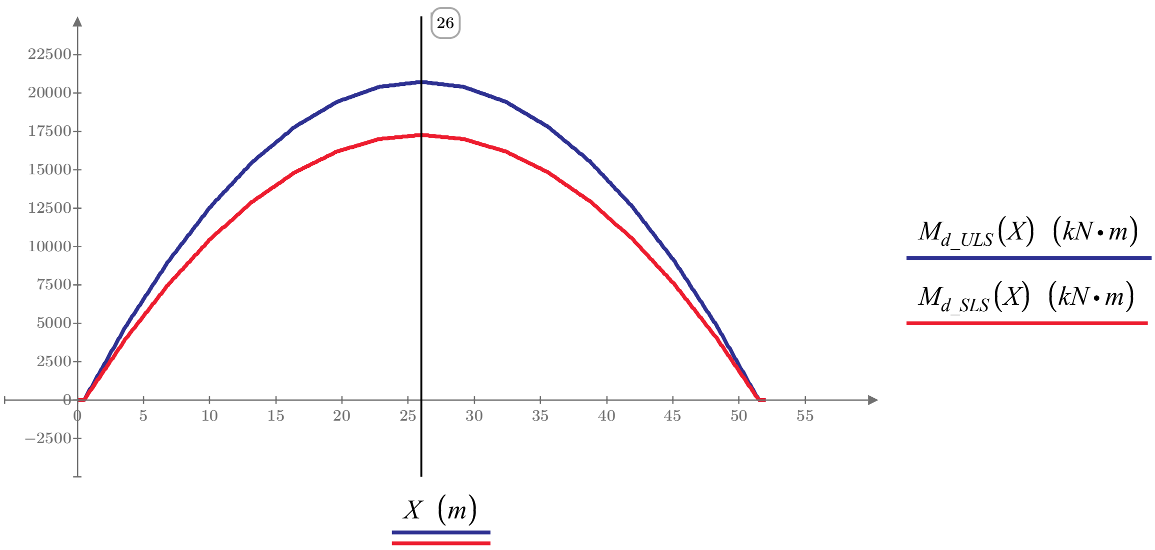
$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

5.1 Load effects

Load effects retrieved from Strip-Step2, Appendix X.

Bending moment

M_{d_ULS} Bending moment descending from permanent loads during construction in ultimate limit state
M_{d_SLS} Bending moment descending from permanent loads during construction in service limit state

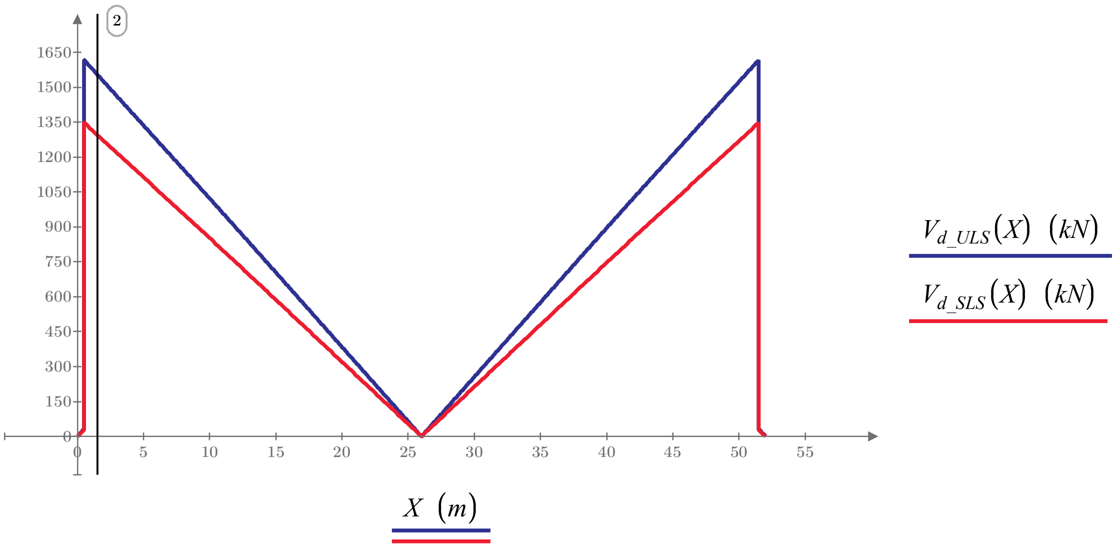


$M_{d\_ULS}(X_{check\_m}) = 20728\ kN \cdot m$ Bending moment at control point - ULS

$M_{d\_SLS}(X_{check\_m}) = 17274\ kN \cdot m$ Bending moment at control point - SLS

Shear force

V_{d_ULS} Shear force descending from permanent loads during construction in ultimate limit state
V_{d_SLS} Shear force descending from permanent loads during construction in service limit state



$V_{d\_ULS}(X_{check\_v}) = 1555\ kN$ Shear force at control point - ULS

$V_{d\_SLS}(X_{check\_v}) = 1296\ kN$ Shear force at control point - SLS

5.2 Redesign of cross-section

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange
f_{yw}	Chosen yield strength - web

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)	f_{yw_1}
0	35	580	7	2990	50	550	f_{y_10mm}
500	35	580	7	2990	50	550	f_{y_10mm}

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)	f_{yw_2}
0	35	580	7	2990	50	550	f_{y_10mm}
10500	35	580	7	2990	50	550	f_{y_10mm}
10500	40	580	5	2980	55	550	$f_{y\_6.4mm}$
11300	40	675	5	2980	55	750	$f_{y\_6.4mm}$
25500	40	675	5	2980	55	750	$f_{y\_6.4mm}$

Element 3

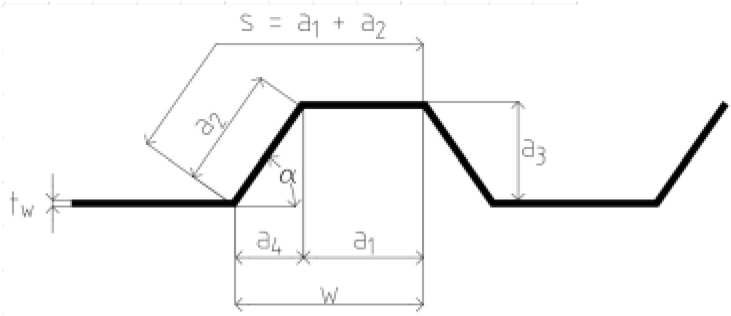
S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)	f_{yw_3}
0	35	580	7	2990	50	550	f_{y_10mm}
500	35	580	7	2990	50	550	f_{y_10mm}

$$h_{tot1} := mean\left(t_{fu\_1} + h_{w\_1} + t_{fl\_1}\right) = 3075\ mm$$

$$h_{tot2} := mean\left(t_{fu\_2} + h_{w\_2} + t_{fl\_2}\right) = 3075\ mm$$

Check to see that the girder height is kept constant

5.2.1 Shape of corrugation



$a_{c1} := 153\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 90\ mm$ Corrugation depth

$a_{weld} := 5\ mm$ Weld throat thickness

$a_{c2} := \frac{a_{c3}}{\sin(\alpha_c)} = 153\ mm$ Legnth of hypotenuse

$a_{c4} := \frac{a_{c3}}{\tan(\alpha_c)} = 124\ mm$ Length of corrugation depth, see figure

$s_c := a_{c1} + a_{c2} = 306\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 277\ mm$ Straight length

$r_c := \frac{s_c}{w_c} = 1.106$ Ratio corrugation/straight length

5.2.2 Cross-section classification

The cross-section classes is determined according to SS-EN 1993 1-4 5.2.2 with updated limits from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated and does not contribute to the axial stiffness the web is not classified.

$$c_w(x) := h_w(x) - 2 \cdot \sqrt{2} \cdot a_{weld}$$

$$c_{uf}(x) := \frac{b_{fu}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on upper flange

$$c_{lf}(x) := \frac{b_{fl}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on lower flange

<u>Cross-section class, upper flange</u>

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength of top flange

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

SS-EN 1993-1-4 5.2.2 Table 5.2

$$csc_{uf}(x) := \begin{array}{l} \text{if } \frac{c_{uf}(x)}{t_{fu}(x)} \le 9\ \varepsilon_{uf} \\ \quad \text{“csc1”} \\ \text{else if } 9\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 10\ \varepsilon_{uf} \\ \quad \text{“csc2”} \\ \text{else if } 10\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 14\ \varepsilon_{uf} \\ \quad \text{“csc3”} \\ \text{else} \\ \quad \text{“csc4”} \end{array}$$

Cross-section class upper flange

$csc_{uf}(X_{check\_m}) = \text{“csc3”}$ — Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{uf}(X_{check\_v}) = \text{“csc3”}$ — Cross-section class at $X_{check\_v} = 1.500\ m$

Cross-section class, lower flange

$E_s = 200\ GPa$ Modulus of elasticity

$f_{ylf} = 450\ MPa$ Proof strength of top flange

$\varepsilon_{lf} := \sqrt{\frac{235}{f_{ylf}} \cdot \frac{E_s}{210000}} = 0.71$ SS-EN 1993-1-4 5.2.2 Table 5.2

Cross-section class lower flange

$$csc_{lf}(x) := \begin{cases} \text{if } \frac{c_{lf}(x)}{t_{fl}(x)} \le 9\ \varepsilon_{lf} \\ \quad \text{"csc1"} \\ \text{else if } 9\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 10\ \varepsilon_{lf} \\ \quad \text{"csc2"} \\ \text{else if } 10\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 14\ \varepsilon_{lf} \\ \quad \text{"csc3"} \\ \text{else} \\ \quad \text{"csc4"} \end{cases}$$

$csc_{lf}(X_{check\_m}) = \text{"csc3"}$ Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{lf}(X_{check\_v}) = \text{"csc1"}$ Cross-section class at $X_{check\_v} = 1.500\ m$

5.2.3 Plate buckling of compressive flange

If the compressed flange is in cross-section class four an effective width of the compressed flange is calculated according to SS-EN 1993-1-4 5.2.3. Updated values are taken from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated the buckling factor k_σ is calculated according to SS-EN 1993-1-5 D.2.1 (2)

$a_{bend} := a_{c1} + 2\ a_{c4} = 401\ mm$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$c_u := c_{uf}\left(X_{check\_m}\right) = 373\ mm$ Width of outstand flange from weld toe to free edge

$t_u := t_{fu}\left(X_{check\_m}\right) = 40\ mm$ Thickness of upper flange

$b_u := b_{fu}\left(X_{check\_m}\right) = 675\ mm$ Width of upper flange

$\varepsilon_{uf} = 0.71$

$k_{\sigma 1} := 0.43 + \left(\frac{c_u}{a_{bend}}\right)^2 = 1.3$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_{\sigma 2} := 0.6$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_\sigma := min\left(k_{\sigma 1}, k_{\sigma 2}\right) = 0.6$ SS-EN 1993-1-5 D.2.2.(1) Equation D.4

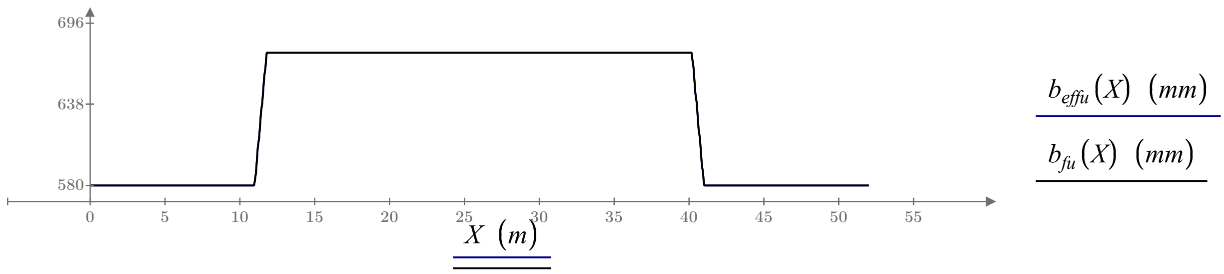
$$\lambda_p := \frac{\frac{c_u}{t_u}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_\sigma}} = 0.6$$

Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho := \mathbf{if}\left(\lambda_p \le 0.748, 1.0, \frac{\lambda_p - 0.188}{\lambda_p^{\ 2}}\right) = 1.00$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

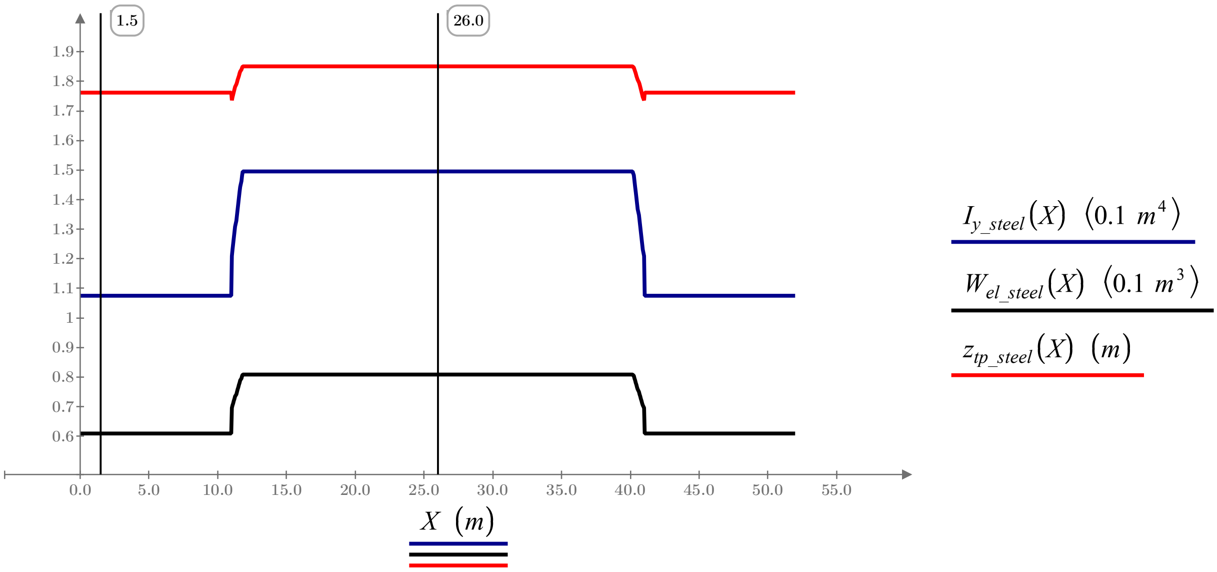
$b_{eff} := b_u \cdot \rho = 675\ mm$ = $b_{effu}\left(X_{check\_m}\right) = 675\ mm$



$b_{fu}(x) := b_{effu}(x)$ Renaming the width of flange in order to minimize errors

5.2.6 New cross-sectional constants during casting

$I_{y\_steel}(x)$	Stiffness of steel girder alone (excluding the web)
$z_{tp\_steel}(x)$	Distance from the top of the top flange to the center of gravity for the steel section
$W_{el\_steel}(x)$	Elastic bending stiffness of steel girder alone (excluding the web)



Coordinates and cross-sectional constants for control calculations

$X_{check\_m} = 26\ m$	X- coordinate for control calculations, bending moment
$A_{sl}(X_{check\_m}) = 0.068\ m^2$	Area
$I_{y\_steel}(X_{check\_m}) = (150 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_m}) = 1.85\ m$	Center of gravity
$W_{el\_steel}(X_{check\_m}) = (81 \cdot 10^{-3})\ m^3$	Elastic bending resistance
$X_{check\_v} = 1.5\ m$	X- coordinate for control calculations, shear force
$A_{sl}(X_{check\_v}) = 0.048\ m^2$	Area
$I_{y\_steel}(X_{check\_v}) = (107 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_v}) = 1.762\ m$	Center of gravity
$W_{el\_steel}(X_{check\_v}) = (61 \cdot 10^{-3})\ m^3$	Elastic bending resistance

5.2.4 Lateral torsional buckling of compressive flange - SS-EN 1993-1-4 5.4.2.1

Simplified method only consisdering buckling of top flange according to SS-EN 1993-1-4 5.4.2.1

$\alpha_{LT} := 0.76$ — Buckling curve d, welded open cross-section

$l_{cr} := 7.29\ m$ — Distance between the cross-beams

$b_{ef} := b_{effu}\left(X_{check\_m}\right) = 675\ mm$ — Width of upper flange

$t_f := t_{fu}\left(X_{check\_m}\right) = 40\ mm$ — Thickness of upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength

$I_{zf} := \dfrac{b_{ef}^{\ 3} \cdot t_f}{12} = 0.001\ m^4$ — Moment of inertia, upper flange

$N_{crLT} := \dfrac{\pi^2 \cdot E_s \cdot I_{zf}}{l_{cr}^{\ 2}} = 38077\ kN$ — Critical buckling load

$\lambda_{LT\_u} := \sqrt{\dfrac{b_{ef} \cdot t_f \cdot f_{yuf}}{N_{crLT}}} = 0.565$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.9

$\Phi_{LT\_u} := 0.5 \cdot \left(1 + \alpha_{LT} \cdot \left(\lambda_{LT\_u} - 0.2\right) + \lambda_{LT\_u}^{\ 2}\right) = 0.8$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.7

$\chi_{LT\_u} := min\left(\dfrac{1}{\Phi_{LT\_u} + \sqrt{\Phi_{LT\_u}^{\ 2} - \lambda_{LT\_u}^{\ 2}}}, 1\right) = 0.734$ = $\chi_{LT}\left(X_{check\_m}\right) = 0.734$

SS-EN 1993-1-4 5.4.2.1 Equation 5.6

$h_{w\_u} := h_w\left(X_{check\_m}\right) = 2980\ mm$ — Height of web

$t_l := t_{fl}\left(X_{check\_m}\right) = 55\ mm$ — Thickness of lower flange

$k_{fl} := 1.1$ — Increase in capacity due to simlified method used, SS-EN 1993-1-1 6.3.2.4 (2)B

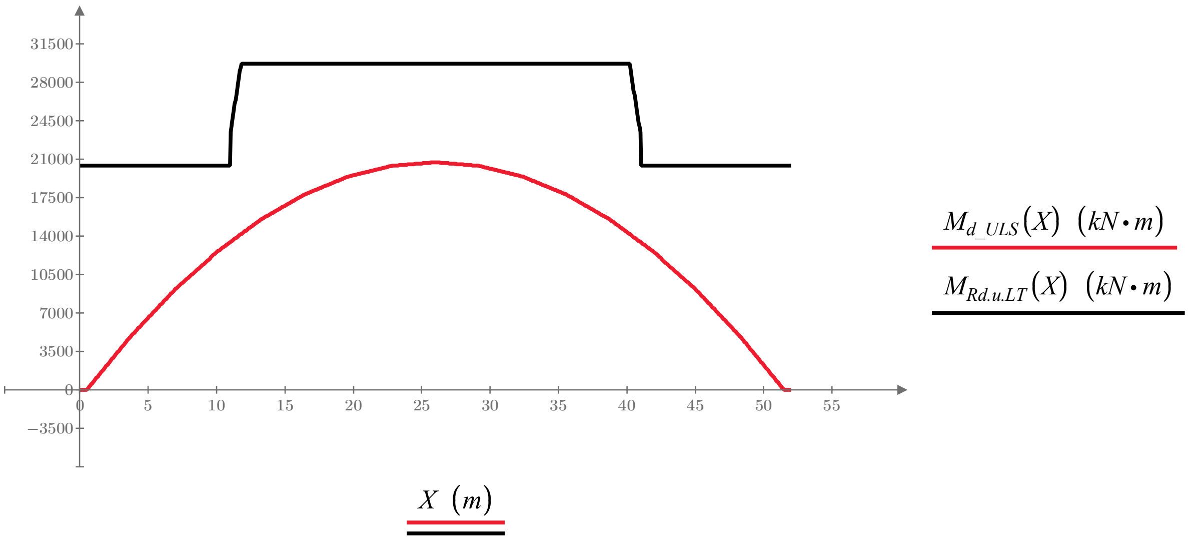
$M_{Rd.u.LT\_u} := \dfrac{b_{ef} \cdot t_f \cdot k_{fl} \cdot \chi_{LT\_u} \cdot f_{yuf}}{\gamma_{M1}} \left(h_{w\_u} + \dfrac{t_f + t_l}{2}\right) = 29705\ kN \cdot m$ = $M_{Rd.u.LT}\left(X_{check\_m}\right) = 29705\ kN \cdot m$

SS-EN 1993-1-5 D.2.1 Equation D.1

5.2.5 Check of lateral torsional buckling

Check of the buckling capacity of the girders is performed.

$M_{Rd.u.LT}(X)$	Capacity of the girder against LT- buckling during casting
$M_{d\_ULS}(X)$	Load effects during casting, Appendix X



$$\eta_{max} := max\left(\frac{M_{d\_ULS}(X)}{M_{Rd.u.LT}(X)}\right) = 70\%$$

Maximum utilization rate of the steel cross-section during casting

5.3 Stresses in steel cross-section

The stresses in the top and bottom flange is calculated for the loading senario to be able to superposition them with the other loadcases to determinte the ultimate capacity of the composite section.

Upper flange

Control calculation at midspan

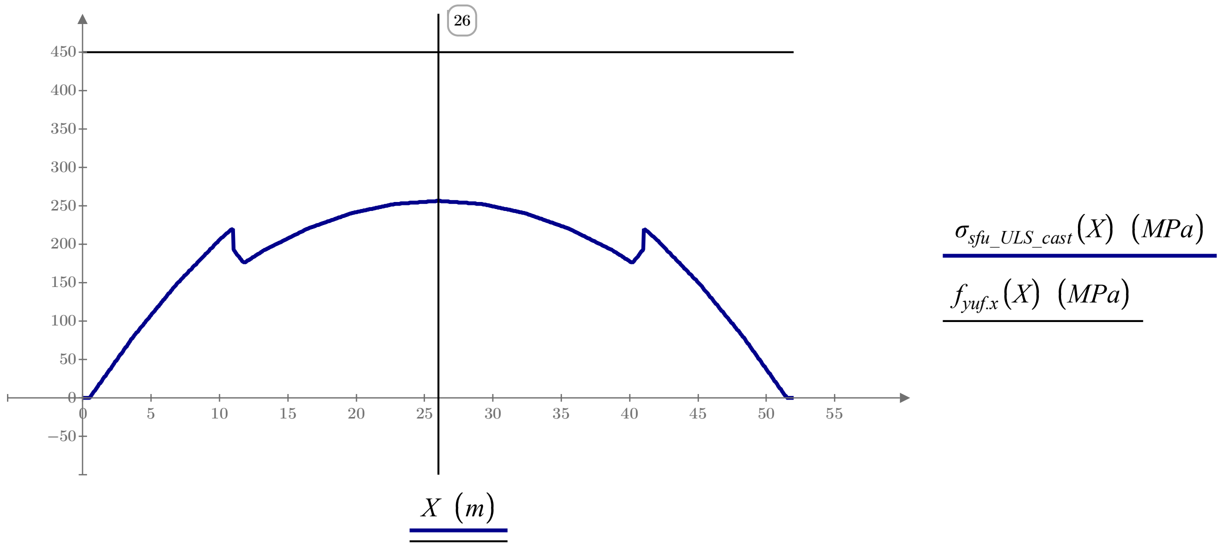
$M := M_{d\_ULS}(X_{check\_m}) = 20728\ kN \cdot m$ Load effext at midspan

$I := I_{y\_steel}(X_{check\_m}) = 0.15\ m^4$ Moment of inertia at midspan

$z := z_{tp\_steel}(X_{check\_m}) = 1.85\ m$ Centre of gravity at midspan maesured from the top of the beam

$\sigma := \frac{M}{I} \cdot z = 256\ MPa$ = $\sigma_{sfu\_ULS\_cast}(X_{check\_m}) = 256\ MPa$

$\sigma_{sfu\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$$\eta_{max} := max\left(\frac{\sigma_{sfu\_ULS\_cast}(X)}{f_{yuf}}\right) = 57\%$$

Lower flange

Check calculation at midspan

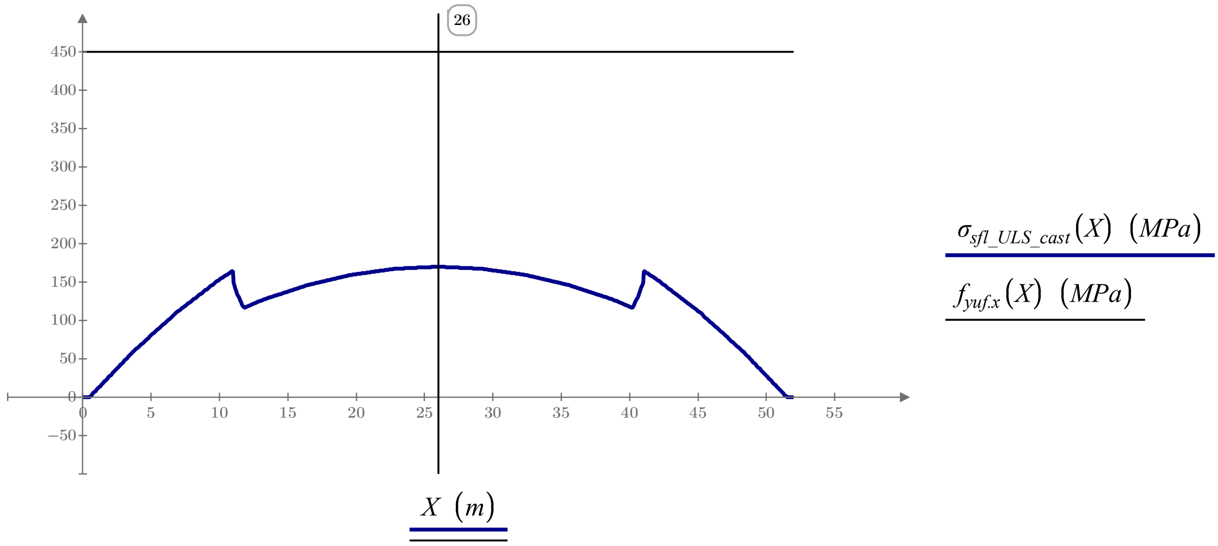
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 20728\ kN \cdot m$ Load effect at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.15\ m^4$ Moment of inertia at midspan

$z := z_{tplf\_steel}\left(X_{check\_m}\right) = 1.225\ m$ Centre of gravity at midspan measured from the bottom of the beam

$\sigma := \frac{M}{I} \cdot z = 170\ MPa$ = $\sigma_{sfl\_ULS\_cast}\left(X_{check\_m}\right) = 170\ MPa$

$\sigma_{sfl\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\frac{\sigma_{sfl\_ULS\_cast}(X)}{f_{ylf}}\right) = 38\%$

6 Capacity checks - Ultimate limit state, global

The capacity checks that are to be carried out are bending moment capacity, shear capacity, web breathing and design of studs, both in ULS and due to fatigue

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 0.5\ m$ Coordinate for control calculations - shear force

6.1 Load effects in ULS

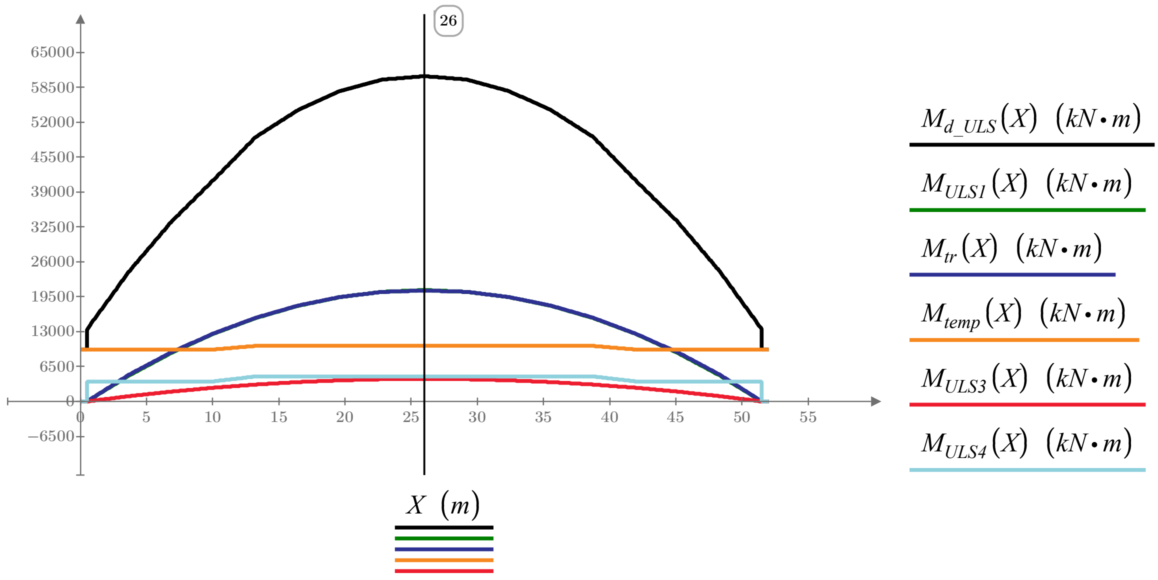
6.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

Bending moment in the ultimate limit state

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{tr}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

M_{d_ULS}	Design bending moment effect from all loads
$M_{ULS1}(x)$	Bending moment during casting (Appendix X)
$M_{tr}(x)$	Bending moment from multi component loads (Appendix X)
$M_{tr}(x)$	Bending moment from temperature loads (Appendix X)
$M_{ULS3}(x)$	Bending moment from additional permanent loads after construction (Appendix X)
$M_{ULS4}(x)$	Bending moment from shrinkage (Appendix X)

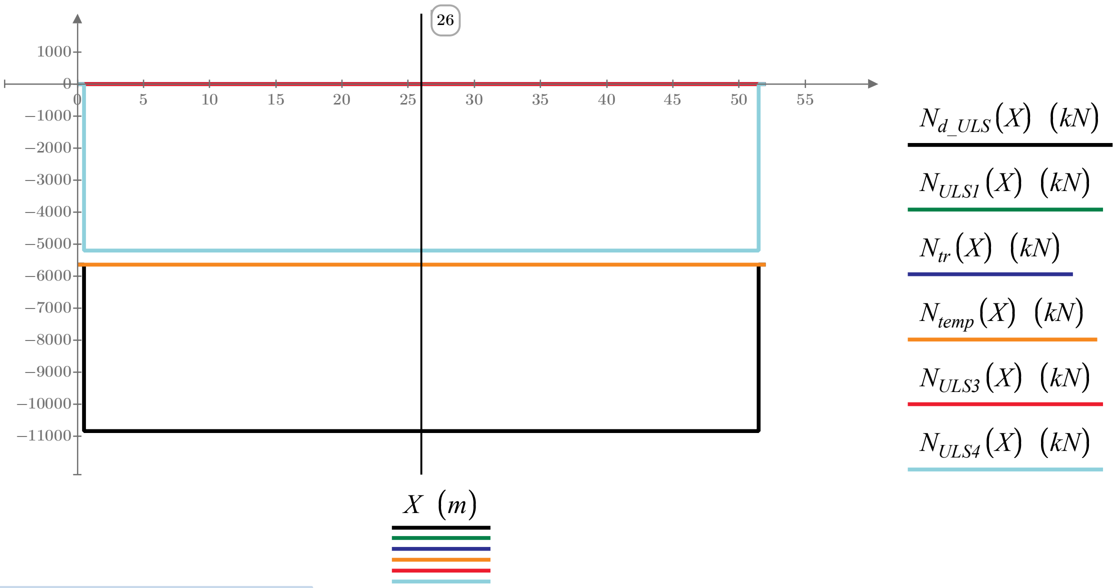


$M_{d\_ULS}(X_{check\_m}) = 60552\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{tr}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design normal force from all loads
$N_{ULS1}(x)$	Normal force during casting (Appendix X)
$N_{tr}(x)$	Normal force from multi components loads (Appendix X)
$N_{temp}(x)$	Normal force from temperature loads (Appendix X)
$N_{ULS3}(x)$	Normal force from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Normal force from shrinkage (Appendix X)

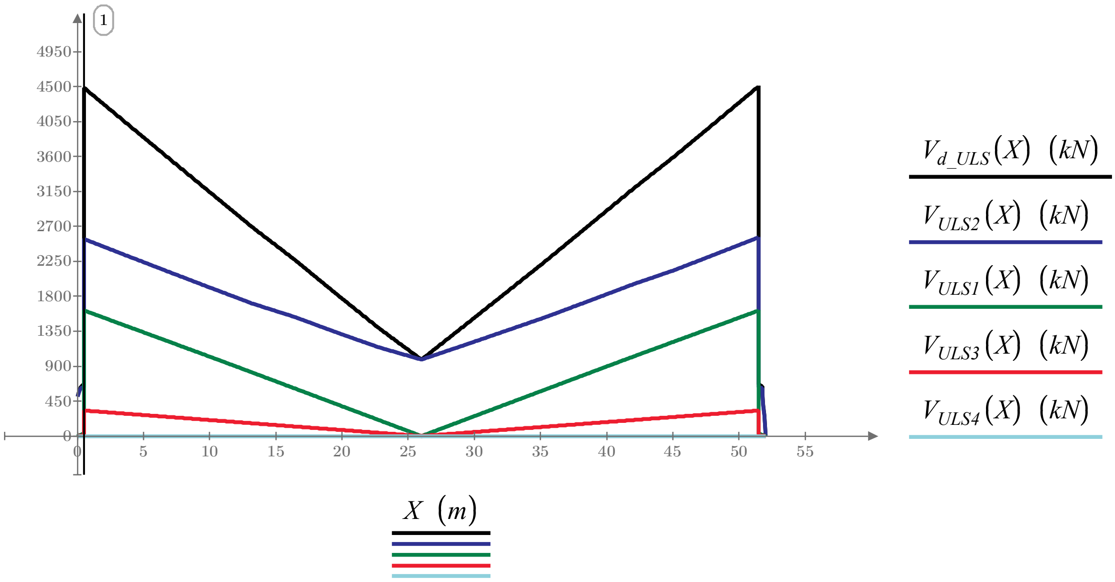


$N_{d\_ULS}(X_{check\_m}) = -10851\ kN$ Normal force at control point - ULS

6.1.2 Shear force

$$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{ULS3}(x) + V_{ULS4}(x)$$

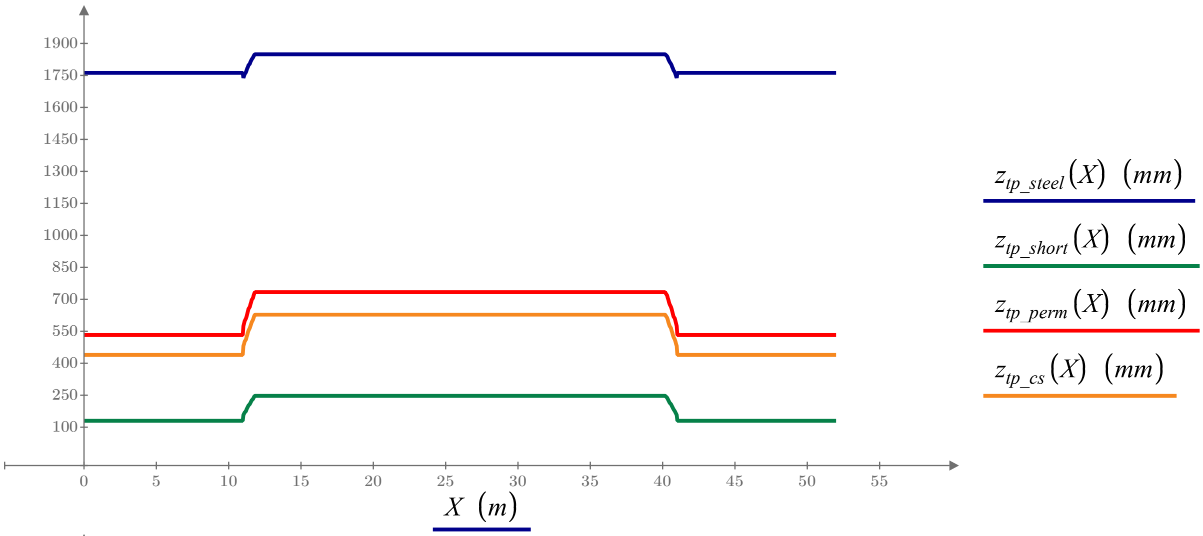
V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (both temperature and traffic) (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)

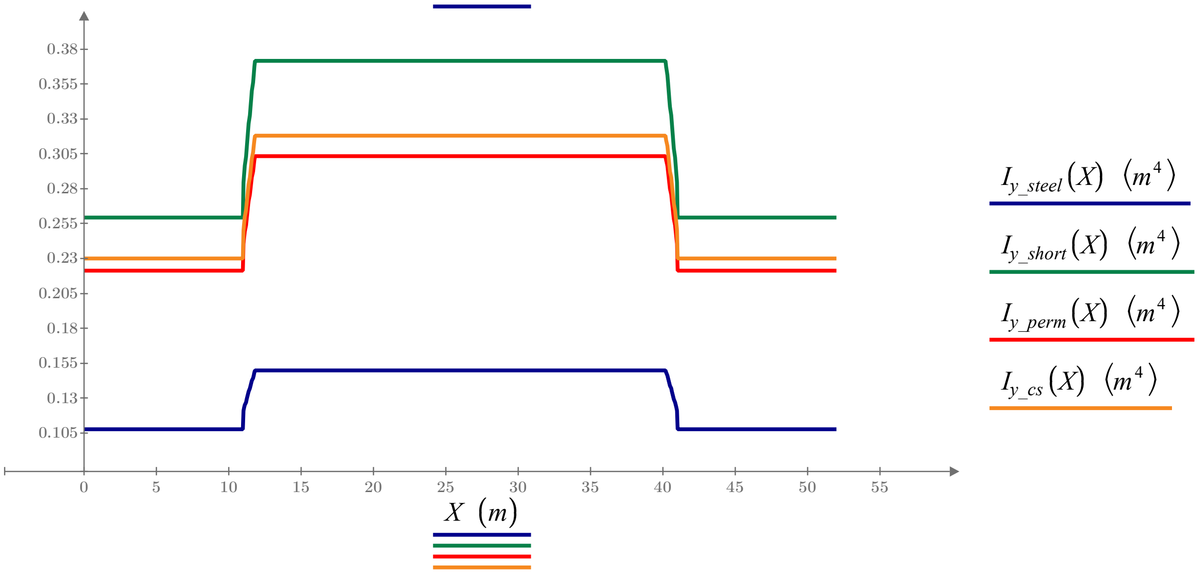


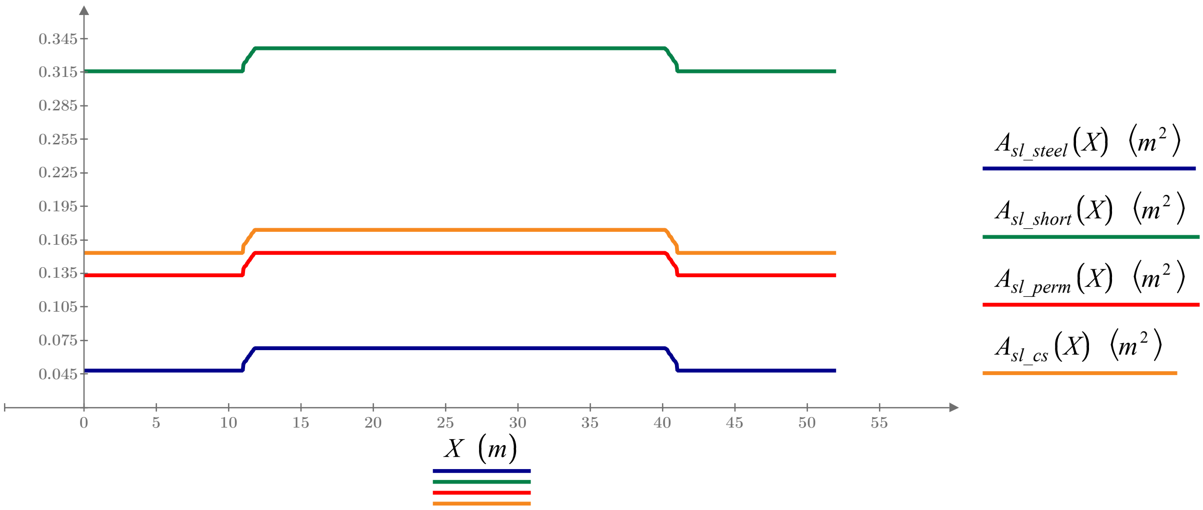
$V_{d\_ULS}(X_{check\_v}) = 4485\ kN$ Shear force at control point - ULS

6.2 Cross-sectional constants

For calculations, see chapter 5.







6.3 Stresses in steel cross-section

The stresses is calculated for each load case taking into acount load duration and creep. The stresses are then superpositioned.

6.3.1 Stresses during casting

$M := M_{ULS1}\left(X_{check\_m}\right) = 20728\ kN \cdot m$ Bending moment

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.15\ m^4$ Moment of inertia

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 1850\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3075\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot -z = -256\ MPa$ = $\sigma_{s.u.cast}\left(X_{check\_m}\right) = -256\ MPa$ Stresses in upper flange from loads durng casting

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 170\ MPa$ = $\sigma_{s.l.cast}\left(X_{check\_m}\right) = 170\ MPa$ Stresses in lower flange from loads durng casting

6.3.2 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses must be calculated for that specific section. The worst load case is determined dependent on the largest stress in each part whereas the multi-component load is the main load for the lower flange, and the temperature load is the worst load case for the upper flange.

6.3.2.1 Multi component loads (traffic)

$M := M_{tr}\left(X_{check\_m}\right) = 20646\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0.01\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.336\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.372\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) = 248\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3075\ mm$ Height of girder

$\sigma_{s.u} := \frac{0.9\ N}{A} + \frac{0.9\ M}{I} \cdot (-z) = -12\ MPa$ = $\sigma_{s.u.tr}\left(X_{check\_m}\right) = -12\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{1.5\ N}{A} + \frac{1.5\ M}{I} \cdot (h - z) = 236\ MPa$ = $\sigma_{s.l.tr}\left(X_{check\_m}\right) = 236\ MPa$ Stresses in lower flange from short term loads

6.3.2.2 Temperature loads

$M := M_{temp}\left(X_{check\_m}\right) = 10332\ kN \cdot m$ Bending moment

$N := N_{temp}\left(X_{check\_m}\right) = -5645\ kN$ Normal force

$A_s := A_{sl\_steel}\left(X_{check\_m}\right) = 0.068\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.336\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}\left(X_{check\_m}\right) = 0.15\ m^4$ Moment of inertia - steel section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.372\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}\left(X_{check\_m}\right) = 1850\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 248\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}\left(X_{check\_m}\right) = 3075\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 4159\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(1 - \left(\frac{A_s}{A} + \frac{A_s}{I}\left(z_s - z\right)^2\right)\right) = -1839\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{1.5\ N_s}{A_s} + \frac{1.5\ M_s}{I_s} \cdot \left(-z_s\right) = -118\ MPa$ = $\sigma_{s.u.temp}\left(X_{check\_m}\right) = -118\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{0.9\ N_s}{A_s} + \frac{0.9\ M_s}{I_s} \cdot \left(h - z_s\right) = 6\ MPa$ = $\sigma_{s.l.temp}\left(X_{check\_m}\right) = 6\ MPa$ Stresses in lower flange from short term loads

6.3.3 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.303\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) = 733\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3075\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot (-z) = -10\ MPa$ = $\sigma_{s.u.perm}(X_{check\_m}) = -10\ MPa$ Stresses in upper flange from additional permanent loads

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 32\ MPa$ = $\sigma_{s.l.perm}(X_{check\_m}) = 32\ MPa$ Stresses in lower flange from additional permanent loads

6.3.4 Stresses due to shrinkage

In the same manner as the temperature load must be calculated for the steel section, the shrinkage which acts on the concrete section must be converted for the steel section.

$M := M_{ULS4}(X_{check\_m}) = 4653\ kN \cdot m$ Bending moment

$N := N_{ULS4}(X_{check\_m}) = -5206\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.068\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_cs}(X_{check\_m}) = 0.174\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.15\ m^4$ Moment of inertia - steel section

$I := I_{y\_cs}(X_{check\_m}) = 0.318\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 1850\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_cs}(X_{check\_m}) = 628\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 3075\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 2189\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(\frac{A_s}{A} - \frac{A_s}{I} \cdot \left(z - \frac{h_{m.slab}}{2} \right) \cdot \left(z_s + \frac{h_{m.slab}}{2} - \left(z - \frac{h_{m.slab}}{2} \right) \right) \right) = -1235\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (-z_s) = -45\ MPa$ = $\sigma_{s.u.shrink}(X_{check\_m}) = -45\ MPa$ Stresses in upper flange due to shrinkage

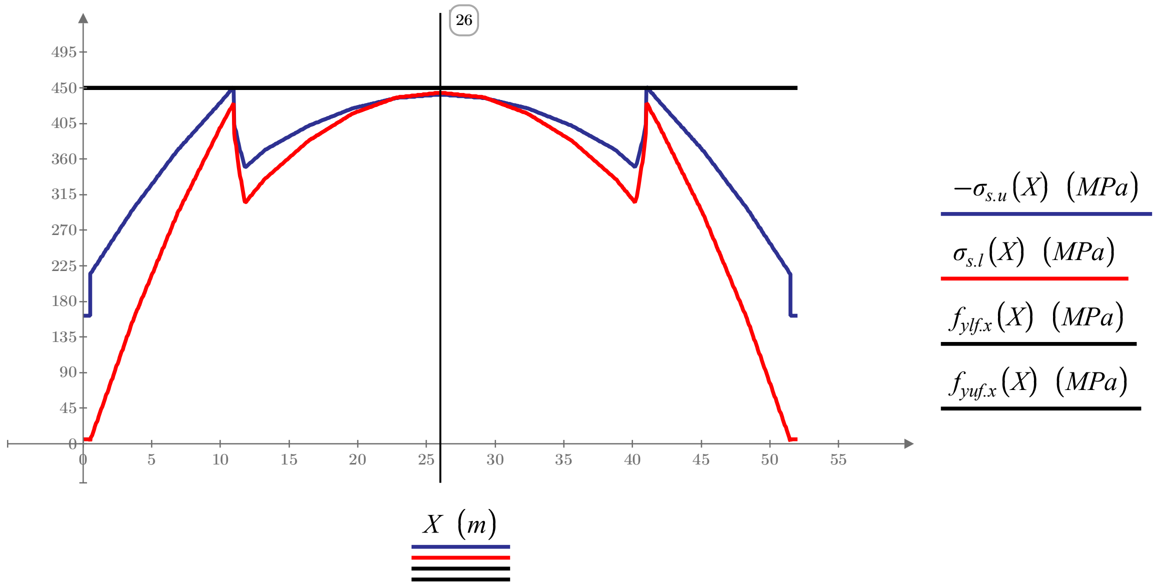
$\sigma_{s.l} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (h - z_s) = 0\ MPa$ = $\sigma_{s.l.shrink}(X_{check\_m}) = 0\ MPa$ Stresses in lower flange due to shrinkage

6.3.5 Summary of stresses

The stresses from the different phases are summarised accordingly.

$\sigma_{s.u}(x) := \sigma_{s.u.cast}(x) + \sigma_{s.u.shrink}(x) + \sigma_{s.u.perm}(x) + \sigma_{s.u.tr}(x) + \sigma_{s.u.temp}(x)$ Stresses in upper flange

$\sigma_{s.l}(x) := \sigma_{s.l.cast}(x) + \sigma_{s.l.shrink}(x) + \sigma_{s.l.perm}(x) + \sigma_{s.l.tr}(x) + \sigma_{s.l.temp}(x)$ Stresses in lower flange



$\eta_{\sigma.u.max} := max\left(\frac{-\sigma_{s.u}(X)}{f_{yuf.x}(X)}\right) = 100\%$ Maximum utilization rate - upper flange

$\eta_{\sigma.l.max} := max\left(\frac{\sigma_{s.l}(X)}{f_{ylf.x}(X)}\right) = 99\%$ Maximum utilization rate - lower flange

6.4 Calculation of stresses in concrete

The stresses in the concrete is calculated with the same principle as the steel taking creep into account.

6.4.1 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses for the concrete must be calculated separately.

6.4.1.1 Traffic and wind loads

$M := M_{tr}\left(X_{check\_m}\right) = 20646\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.336\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.372\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) + h_{m.slab} = 568\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3075\ mm$ Height of girder

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$\sigma_c := \left(\dfrac{N}{A} + \dfrac{-M \cdot z}{I}\right) \cdot \dfrac{1}{n_\Gamma} = -5\ MPa$ = $\sigma_{c.short}\left(X_{check\_m}\right) = -5\ MPa$ Stresses in concrete from short-term loads

6.4.1.2 Temperature load

$M := M_{temp}\left(X_{check\_m}\right) = 10332\ kN \cdot m$

$N := N_{temp}\left(X_{check\_m}\right) = -5645\ kN$

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.336\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_\Gamma} = 0.268\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.372\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}\left(X_{check\_m}\right)}{n_\Gamma} \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.002\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 248\ mm$ Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \frac{I_{c.eff}}{I} \cdot M = 64\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(\frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot \left(z_s - z\right) \cdot \left(z + \frac{h_{m.slab}}{2}\right)\right) = -1839\ kN$ Normal force imposed on concrete slab

$\sigma_c := \left(\frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c}\right) = 0.41\ MPa$ = $\sigma_{c.temp}\left(X_{check\_m}\right) = 0.41\ MPa$ Stresses in concrete from temperature loads

6.4.2 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.303\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) + h_{m.slab} = 1053\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3075\ mm$ Height of girder

$n_\Gamma := n_{L_1} = 18.48$ Creep

$\sigma_c := \frac{-M \cdot z}{I} \cdot \frac{1}{n_\Gamma} = -0.8\ MPa$ = $\sigma_{c.perm}\left(X_{check\_m}\right) = -0.8\ MPa$ Stresses in concrete from permanent loads

6.4.3 Stresses due to shrinkage

$M := M_{ULS4}\left(X_{check\_m}\right) = 4653\ kN \cdot m$ — Bending moment

$N := N_{ULS4}\left(X_{check\_m}\right) = -5206\ kN$ — Normal force

$n_{\Gamma} := n_{L_3} = 14.91$ — Modular ratio considering creep

$A := A_{sl\_cs}\left(X_{check\_m}\right) = 0.174\ m^2$ — Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \frac{1}{n_{\Gamma}} = 0.106\ m^2$ — Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ — Cross-section area - concrete section

$I := I_{y\_cs}\left(X_{check\_m}\right) = 0.318\ m^4$ — Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.013\ m^4$ — Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}\left(X_{check\_m}\right)}{n_{\Gamma}} \cdot \frac{h_{m.slab}^{\ 3}}{12} = 0.001\ m^4$ — Moment of inertia - effective concrete section

$z := z_{tp\_cs}\left(X_{check\_m}\right) = 628\ mm$ — Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ — Center of gravity - concrete section

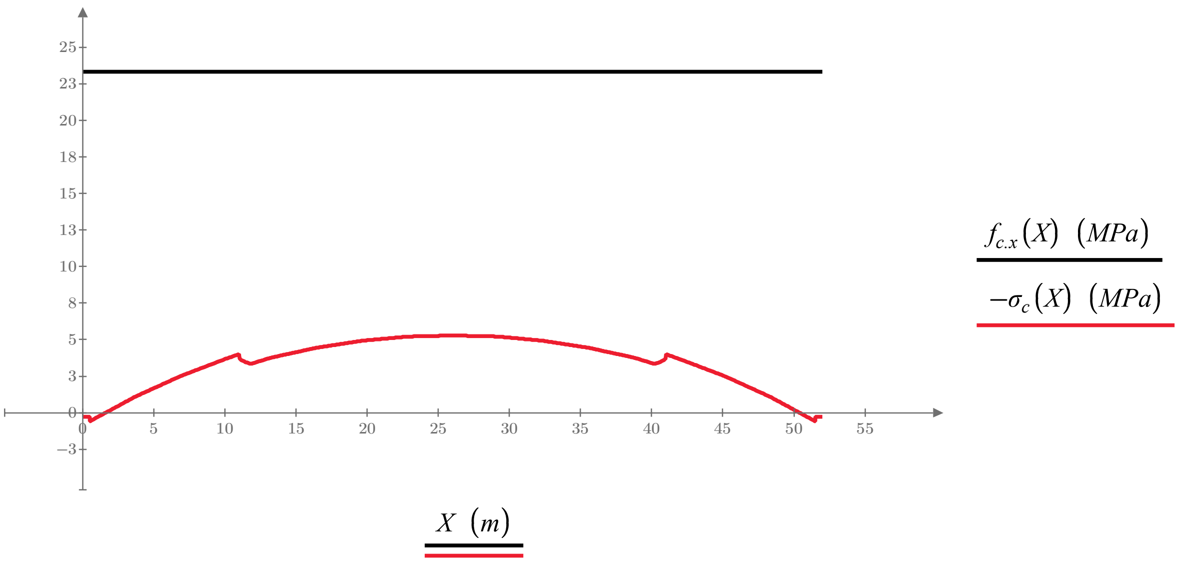
$M_c := \frac{I_{c.eff}}{I} \cdot M = 13\ kN \cdot m$ — Bending moment imposed on concrete slab

$N_c := N \cdot \left(1 - \frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot \left(z + \frac{h_{m.slab}}{2}\right)^2\right) = -966\ kN$ — Normal force imposed on concrete slab

$\sigma_c := \frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c} = 0.5\ MPa$ $=$ $\sigma_{c.shrink}\left(X_{check\_m}\right) = 0.5\ MPa$ — Stresses in concrete from shrinkage

6.4.4 Summary of stresses

$\sigma_c(x) := \sigma_{c.short}(x) + \sigma_{c.temp}(x) + \sigma_{c.perm}(x) + \sigma_{c.shrink}(x)$ Stresses in concrete



$\eta_{\sigma.c.max} := max\left(\frac{-\sigma_c(X)}{f_{c.x}(X)}\right) = 23\%$ Maximum utilization rate - upper flange

6.5 Shear capacity

The shear capacity of the corrugated steel girder is calculated accoridng to SS-EN 1993-1-5, Appendix D.
The local- and global buckling factor are calculated according to SS-EN 1993-1-5, Appendix D.

<u>Local buckling factor</u>

$a_{c1} = 153\ mm$ $\quad a_{c2} = 153\ mm$ — Corrugation geometries

$a_{cmax} := max\left(a_{c1}, a_{c2}\right) = 153\ mm$ — SS-EN 1993-1-5 D.2.2.(2)

$tw := t_w\left(X_{check\_v}\right) = 7\ mm$ — Web thickness

$\tau_{cr} := 4.83 \cdot E_s \cdot \left(\dfrac{tw}{a_{cmax}}\right)^2 = 2019\ MPa$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.7

$\lambda_c := \sqrt{\dfrac{f_{yw}\left(X_{check\_v}\right)}{\tau_{cr} \cdot \sqrt{3}}} = 0.37$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.6

$\chi_l := min\left(\dfrac{1.15}{0.9 + \lambda_c}, 1\right) = 0.91 \quad = \quad \chi_{c.l}\left(X_{check\_v}\right) = 0.91$ — Reduction factor local buckling - SS-EN 1993-1-5 D.2.2.(2) Equation D.5

<u>Global buckling factor</u>

$D_X := \dfrac{E_s \cdot tw^3}{12 \cdot \left(1 - \nu^2\right)} \cdot \dfrac{w_c}{s_c} = 6\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$D_Z := \dfrac{E_s \cdot tw \cdot a_{c3}^{\ 2}}{12} \cdot \dfrac{\left(3\ a_{c1} + a_{c2}\right)}{w_c} = 2089\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$hw := h_w\left(X_{check\_v}\right) = 2990\ mm$ — Web height

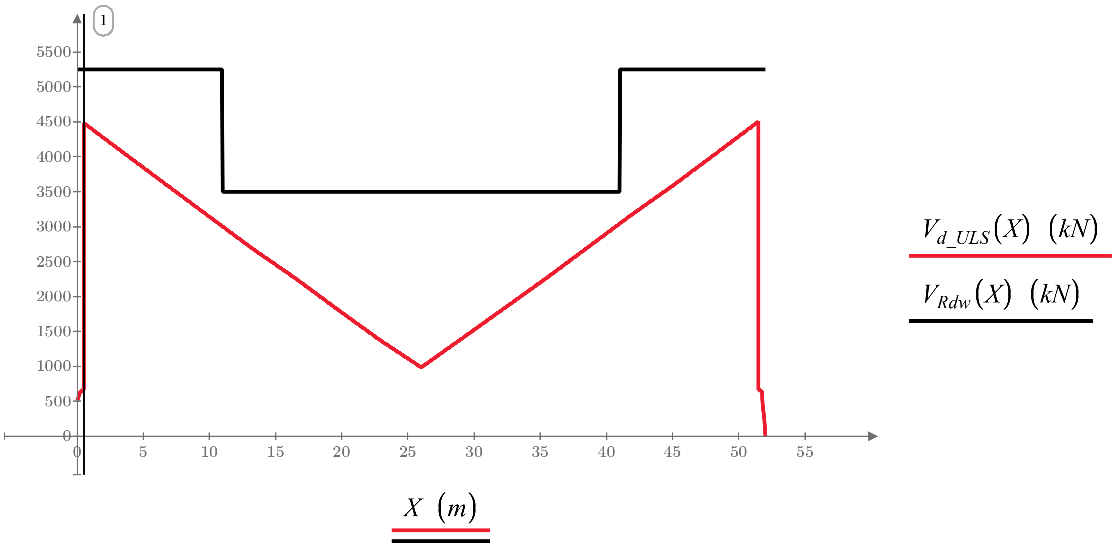
$\tau := \dfrac{32.4}{tw \cdot hw^2} \cdot \sqrt[4]{\dfrac{D_X}{N \cdot m} \cdot \dfrac{D_Z^{\ 3}}{\left(N \cdot m\right)^3}}\ N \cdot m = 247\ MPa$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.10

$\lambda := \sqrt{\dfrac{f_{yw}\left(X_{check\_v}\right)}{\tau \cdot \sqrt{3}}} = 1.06$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.9

$\chi_g := min\left(\dfrac{1.5}{0.5 + \lambda^2}, 1\right) = 0.92 \quad = \quad \chi_{c.g}\left(X_{check\_v}\right) = 0.92$ — Global buckling factor SS-EN 1993-1-5 D.2.2.(3) Equation D.8

$\chi_C := min\left(\chi_g, \chi_l\right) = 0.91 \quad = \quad \chi_c\left(X_{check\_v}\right) = 0.91$ — SS-EN 1993-1-5 D.2.2

$V_{Rd} := \chi_C \cdot \dfrac{f_{yw}\left(X_{check\_v}\right)}{\gamma_{M1} \cdot \sqrt{3}} \cdot hw \cdot tw = 5250\ kN \quad = \quad V_{Rdw}\left(X_{check\_v}\right) = 5250\ kN$ — SS-EN 1993-1-5 D.2.2.(1) Eq.D.4



$\eta_{V.max} = 87\%$ Maximum utilization rate

6.7 Studs

6.7.1 Ultimate limit state

The capacity of the studs in the ultimate limit state are calculated according to SS-EN 1994-2 6.6.3

$d_{stud} \equiv 22\ mm$ Diameter of stud

$h_{stud} \equiv 200\ mm$ Length of stud

$f_{ub} = 800\ MPa$ Characteristic strength of studs

$f_{u\_stud} := min\left(f_{ub}, 500\ MPa\right) = 500\ MPa$ Ultimate strength shear stud - SS-EN 1994-2 6.6.3.1 (1)

$\frac{h_{stud}}{d_{stud}} = 9$ Ratio height- diameter

$$\alpha_{stud} := \begin{vmatrix} \text{if } 3 \le \frac{h_{stud}}{d_{stud}} \le 4 \\ \quad \| a \leftarrow 0.2 \cdot \left(1 + \frac{h_{stud}}{d_{stud}}\right) \\ \text{else if } 4 < \frac{h_{stud}}{d_{stud}} \\ \quad \| a \leftarrow 1 \\ a \end{vmatrix} = 1.0$$

Correction factor for length to diameter ratio shear stud SS-EN 1994-2 6.6.3.1 (1)

$$P_{rd} := min\left(\begin{Vmatrix} \frac{0.8 \cdot f_{u\_stud} \cdot \pi \cdot d_{stud}^{2}}{4 \cdot \gamma_V} \\ \frac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^{2} \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \end{Vmatrix}\right) = 122\ kN$$

Capacity of one shear stud SS-EN 1994-2 6.6.3.1 Eq: 6.18,6.19

$S_{uf\_short}\left(X_{check\_v}\right) = \left(78 \cdot 10^{-3}\right)\ m^3$ Second moment of area

$V_{ULS2}\left(X_{check\_v}\right) = 2538\ kN$ Shear force

$I_{y\_short}\left(X_{check\_v}\right) = \left(260 \cdot 10^{-3}\right)\ m^4$ Moment of inertia

$$\tau_{sh} := \frac{S_{uf\_short}\left(X_{check\_v}\right) \cdot V_{ULS2}\left(X_{check\_v}\right)}{I_{y\_short}\left(X_{check\_v}\right)} = 762\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for short term loads

$S_{uf\_perm}\left(X_{check\_v}\right) = \left(59 \cdot 10^{-3}\right)\ m^3$ Second moment of area

$V_{ULS3}\left(X_{check\_v}\right)=329\ kN$ Shear force

$I_{y\_perm}\left(X_{check\_v}\right)=\left(221 \cdot 10^{-3}\right)\ m^4$ Moment of inertia

$$\tau_{pe}:=\frac{S_{uf\_perm}\left(X_{check\_v}\right) \cdot V_{ULS3}\left(X_{check\_v}\right)}{I_{y\_perm}\left(X_{check\_v}\right)}=87\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for permanent loads

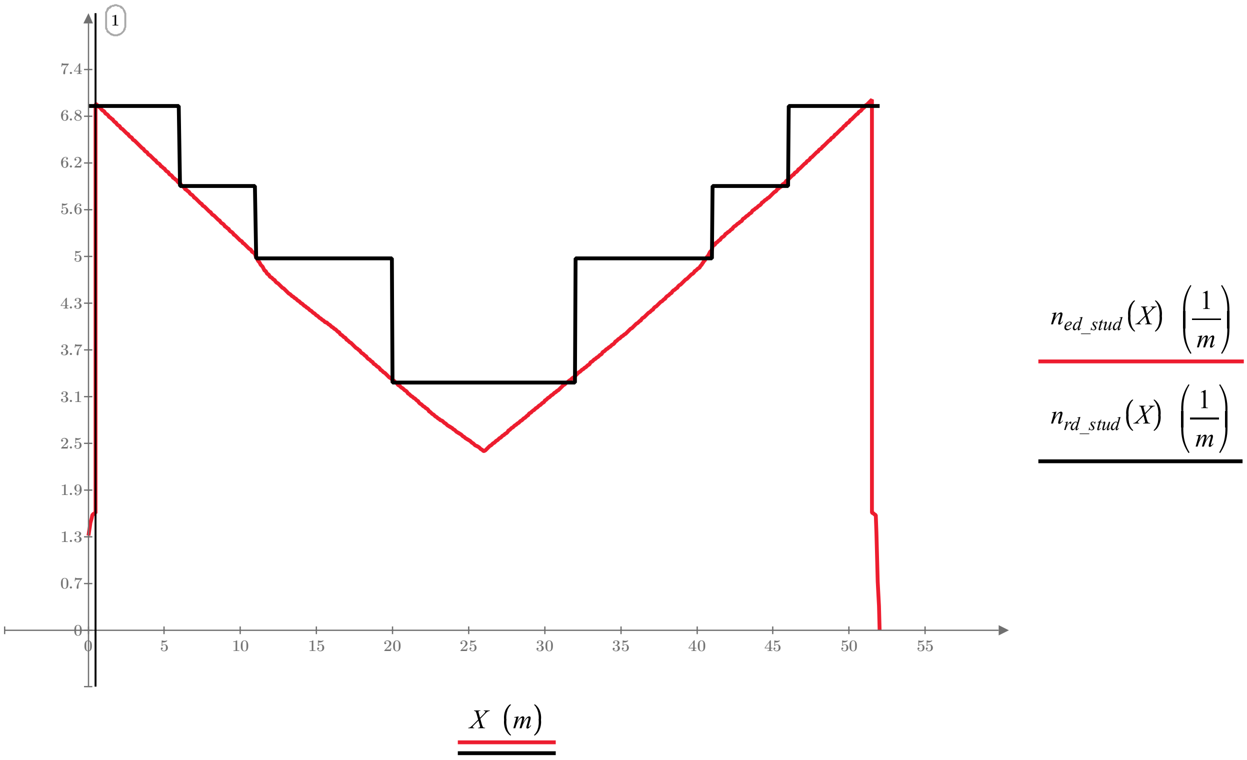
$$\tau:=\left|\tau_{sh}+\tau_{pe}\right|=850\ \frac{kN}{m}$$

Total shear force per meter between concrete and top flange

$n_{ed} := \dfrac{\tau + 0.9\ max\left(q_{1k\_br}, q_{tr\_br}\right)}{P_{rd}} = 7.0\ \dfrac{1}{m}$ = $\boxed{n_{ed\_stud}\left(X_{check\_v}\right) = 7.0\ \dfrac{1}{m}}$ Number of studs needed per meter

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



6.7.1.1 Additional studs for full anchorage

Case 1: Temperature and shrinkage causes the slab to contract and therefore they are working in the opposite direction as the shear flow from ULS- loads during bending.

The anchorage length is calculated according to SS-EN 1994-2, 6.9 (3).

$B_{out} = 2.525\ m$ Distance from centre web to outer part of edge beam

$B_{in} = 2.8\ m$ Distance from web to centerline bridge

$b := max\left(B_{out}, B_{in}\right) = 2.8\ m$

$l_{anch} := 1.5 \cdot b = 4.2\ m$ Anchorage length - SS-EN 1994-2, 6.9 (3)

$N_{cs\_stud} = 785\ kN$ Shrinkage force imposed on studs - calculated in chapter 4

$\left|N_{temp\_stud_1}\right| = 1783\ kN$ Temperature force imposed on studs causing contraction - calculated in chapter 4

$$n_{ed\_cs} := \frac{1.0\ N_{cs\_stud}}{l_{anch} \cdot P_{rd}} = 1.5\ \frac{1}{m}$$

$$n_{ed\_temp} := \frac{1.5\ \left|N_{temp\_stud_1}\right|}{l_{anch} \cdot P_{rd}} = 5.2\ \frac{1}{m}$$

$$n_{ed\_anch.1} := n_{ed\_cs} + n_{ed\_temp} = 6.7\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 6.9\ \frac{1}{m}$$

$$check_1 := \textbf{if}\left(n_{ed\_anch.1} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_1 = \text{“No extra studs are needed”}$$

Case 2: Temperature causes the slab to expand and therefore working in the same direction as the shear flow from ULS- loads during bending. Shrinkage causes the slab to contract, i.e. working in the opposite direction.

$$-n_{ed\_cs} = -1.5\ \frac{1}{m}$$

$$N_{temp\_stud_2} = 1444\ kN$$

Temperature force imposed on studs causing expansion
- calculated in chapter 4

$$n_{ed\_temp} := \frac{1.5 \cdot 0.6\ N_{temp\_stud_2}}{l_{anch} \cdot P_{rd}} = 2.5\ \frac{1}{m}$$

$$n_{ed\_stud}(0\ m) = 1.3\ \frac{1}{m}$$

$$n_{ed\_anch.2} := n_{ed\_stud}(0\ m) + n_{ed\_temp} - n_{ed\_cs} = 2.3\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 6.9\ \frac{1}{m}$$

$$check_2 := \textbf{if}\left(n_{ed\_anch.2} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_2 = \text{“No extra studs are needed”}$$

Extra studs needed due to shrinkage and temperature

$n_{rd\_stud.adj}(X)$ Needed amount of studs with regards to extra anchorage due to temperature and shrinkage

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section (ULS- loads)

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section

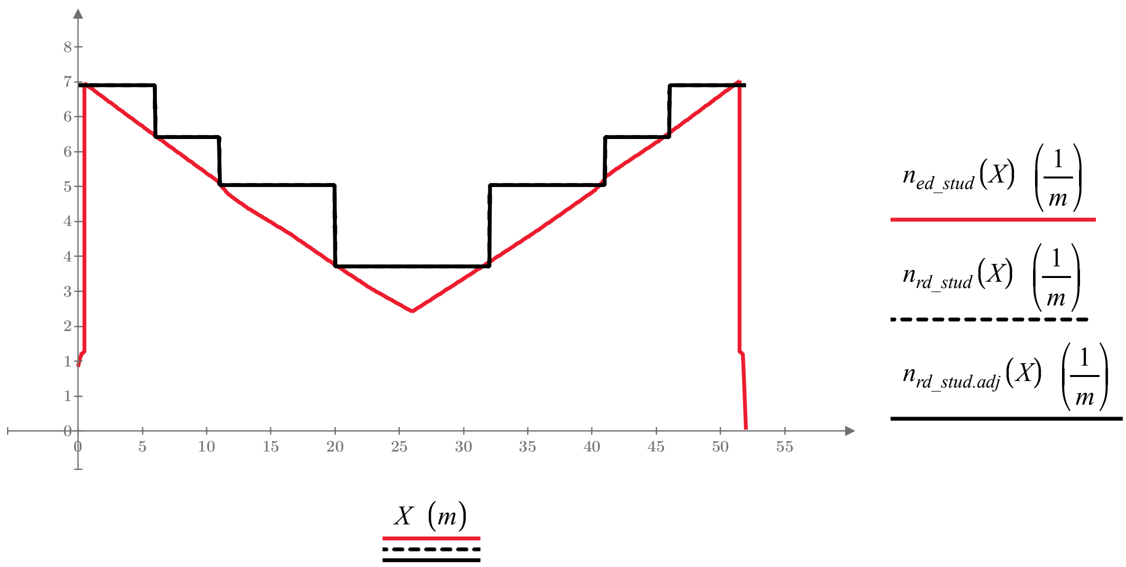


Table. Showing the adjusted need for studs near supports

X	n_{rd}	$n_{rd.adj}$
0	6.9	**6.9**
0.5	6.9	**6.9**
4.2	6.9	**6.9**
6	6.2	**6.2**
11	5.2	**5.2**
20	3.7	**3.7**

6.7.2 Fatigue limit state

6.7.2.1 Capacity

The fatigue capacity is calculated according to SS-EN 1993-1-9, Table 8.5 and SS-EN 1994-2, 6.8.3.

$\Delta\tau_c := 90\ MPa$ SS-EN 1993-1-9 - Table 8.5 (10)

$\Delta\tau_{E2} = \lambda_v \cdot \Delta\tau_c$

$\lambda_{v.1} := 1.55$ Bridge length less than 100m - SS-EN 1994-2 6.8.6.2 (4)

$Q_{mi} := 410\ kN$ Mean weight of large vehicles in the slow lane

$Q_0 := 480\ kN$

$N_{obs} := 0.05 \cdot 10^6$

$N_0 := 0.5 \cdot 10^6$

$\lambda_{v.2} := \frac{Q_{mi}}{Q_0} \cdot \left(\frac{N_{obs}}{N_0}\right)^{\frac{1}{8}} = 0.64$ SS-EN 1994-2 6.8.6.2 (4) and SS-EN 1993-2 Eq. 9.10

$t_{Ld} := 120$ Expected service life [years]

$\lambda_{v.3} := \left(\frac{120}{100}\right)^{\frac{1}{8}} = 1.02$

$\lambda_{v.4} := 1.0$ TSFS 2018:57 - 27 ch. 3 §

$\lambda_v := \lambda_{v.1} \cdot \lambda_{v.2} \cdot \lambda_{v.3} \cdot \lambda_{v.4} = 1.02$

$\gamma_{Ff} := 1.0$ SS-EN 1993-2, 9.3 (1)

$\gamma_{mF} := 1.0$ SS-EN 1994-2, 2.4.1.2 (6)

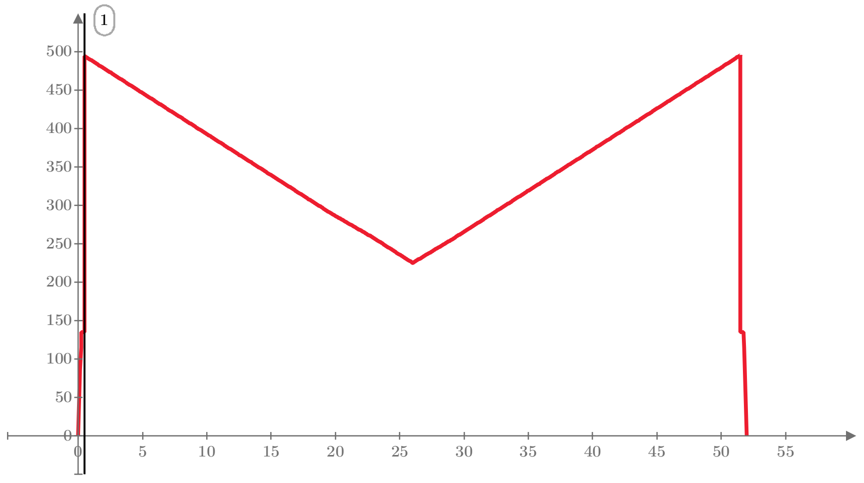
$\gamma_{Ff} \cdot \Delta\sigma_{E2} < \frac{\Delta\tau_c}{\gamma_{mF}}$

$F_{rd\_stud} := \Delta\tau_c \cdot \frac{\pi \cdot d_{stud}^2}{4} = 34.2\ kN$ Shear fatigue capacity - one stud

$F_{rd\_stud} := \frac{F_{rd\_stud}}{\lambda_v} = 33.7\ kN$ Considering trafic load

6.7.2.2 Fatigue load

$V_{FAT}(X)$ Fatigue load from trafic - see Appendix X



$$\tau \cdot b = \frac{SV}{I} = \frac{stud\_capacity}{m}$$

$$S_{uf} := S_{uf\_short}\left(X_{check\_v}\right) = 0.078 \ m^3$$

$$V := V_{FAT}\left(X_{check\_v}\right) = 494 \ kN$$

$$I := I_{y\_short}\left(X_{check\_v}\right) = 0.26 \ m^4$$

$$\Delta\tau := \frac{S_{uf} \cdot V}{I} = 148 \ \frac{kN}{m}$$

$F_{rd\_stud} = 34 \ kN$ Fatigue capacity - one stud

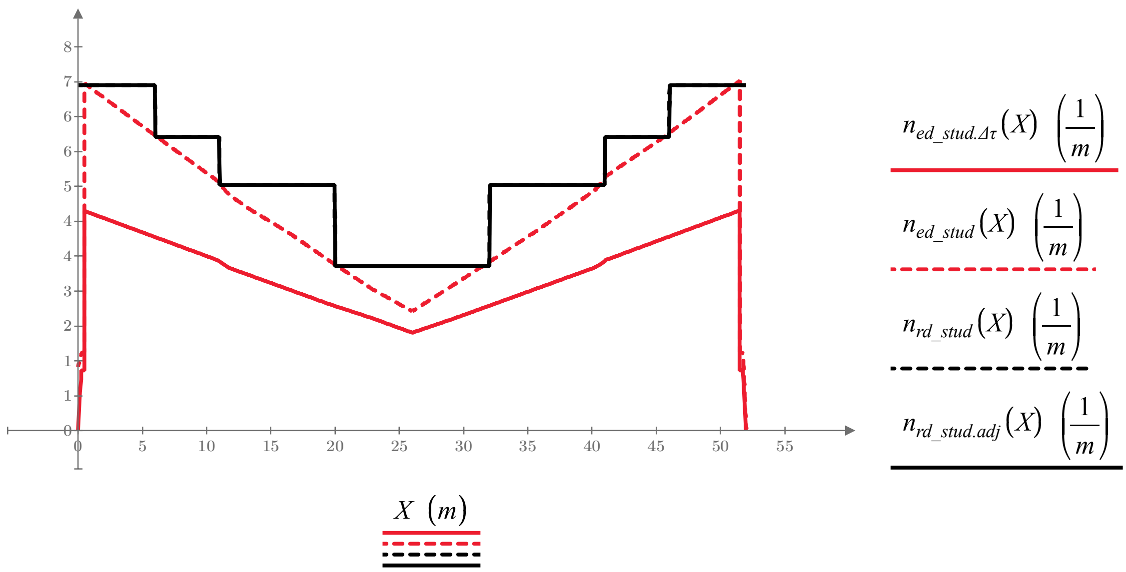
$$n_{ed\_\Delta\tau} := \frac{\Delta\tau}{F_{rd\_stud}} = 4.4 \ \frac{1}{m} \quad = \quad n_{ed\_stud.\Delta\tau}\left(X_{check\_v}\right) = 4.4 \ \frac{1}{m}$$

6.7.2.3 Design of studs with regards to fatigue

$n_{ed\_stud.\Delta\tau}(X)$ Needed amount of studs with regards to fatigue

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



Extra need of studs with regards to fatigue

$$Check := \mathbf{if}\left(max\left(\frac{n_{ed\_stud.\Delta\tau}(X)}{n_{rd\_stud}(X)}\right), \text{"No extra studs are needed"}, \text{"Extra studs are needed"}\right)$$

$$Check = \text{"No extra studs are needed"}$$

6.7.3 Summary - design of studs

Table. Showing the stud design for half of the span

X	$n_{stud.used}$	$n_{stud.need}$	$n_{stud.fat}$
0	**6.9**	1.3	0.0
0.5	**6.9**	7.0	4.4
4.2	**6.9**	6.3	4.1
6	**6.2**	5.9	3.9
11	**5.2**	5.0	3.4
20	**3.7**	3.3	2.5

6.8 Utilization rates

$\eta_{\sigma.u.max} = 100\%$	Stresses in top flange
$\eta_{\sigma.l.max} = 99\%$	Stresses in lower flange
$\eta_{\sigma.c.max} = 23\%$	Stresses in concrete
$\eta_{V.max} = 87\%$	Shear capacity
$\eta_{breathing} = 12\%$	Breathing

9 Material savings

The material savings that are achieved thanks to redesigning the bridge with corrugated webs and using stainless steel is compared to the old design. However it is important to note that dependent on the utilization rates the savings may always not be comparable. For this bridge the highest utilization ratios are larger than 95% (close to 99,8%) and therefore close to comparable.

9.1 Main girder

The main girder is redesigned with a corrugated web and a slightly slimmer design. Over the full bridge length this decreases the weight of the bridge. All savings are presented for the full bridge (full width). Consideration has been taken to the extra web length arising from the corrugation.

$V_{old} = 13.25\ m^3$ Steel volume - girder with flat web; original design

$V_{new} = 8.18\ m^3$ Steel volume - girder with corrugated web; new design

$m_{old\_girder} := 7850\ \frac{kg}{m^3} \cdot V_{old} = 104\ 10^3 \cdot kg$ Weight of girder - flat web; original design

$m_{new\_girder} := 7551\ \frac{kg}{m^3} \cdot V_{new} = 62\ 10^3 \cdot kg$ Weight of girder - corrugated web; new design

$\eta_{girder} := 1 - \frac{m_{new\_girder}}{m_{old\_girder}} = 41\%$ Material saving [%] - steel girder

$\Delta m_{saving\_girder} := m_{old\_girder} - m_{new\_girder} = 42\ 10^3\ kg$ Material saving [kg] - steel girder

9.2 Studs

$V_{stud\_old} = 72750\ cm^3$ Volume - studs; original design

$V_{stud\_new} = 41120\ cm^3$ Volume - studs; new design

$m_{old\_stud} := 7850\ \frac{kg}{m^3} \cdot V_{stud\_old} = 571\ kg$ Mass - studs; original design

$m_{new\_stud} := 7551\ \frac{kg}{m^3} \cdot V_{stud\_new} = 310\ kg$ Mass - studs; new design

$\eta_{stud} := 1 - \frac{m_{new\_stud}}{m_{old\_stud}} = 46\%$ Material saving [%] - studs

$\Delta m_{saving\_stud} := m_{old\_stud} - m_{new\_stud} = 261\ kg$ Material saving [kg] - studs

9.3 Cross-beams

$V_{cb}=0.59\ m^3$ Volume cross-beams old and new design

$m_{old\_cb}:=7850\ \frac{kg}{m^3}\cdot V_{cb}=4597\ kg$ Mass - cross-beams; original design

$m_{new\_cb}:=7551\ \frac{kg}{m^3}\cdot V_{cb}=4422\ kg$ Mass - cross-beams; new design

$\eta_{cb}:=1-\frac{m_{new\_cb}}{m_{old\_cb}}=4\%$ Material saving [%] - studs

$\Delta m_{saving\_cb}:=m_{old\_cb}-m_{new\_cb}=175\ kg$ Material saving [kg] - studs

9.4 Welds

$V_{weld\_old}=0.0051\ m^3$ Volume - welds; original design

$V_{weld\_new}=0.0038\ m^3$ Volume - welds; new design

$m_{old\_weld}:=7850\ \frac{kg}{m^3}\cdot V_{weld\_old}=40\ kg$ Mass - welds; original design

$m_{new\_weld}:=7551\ \frac{kg}{m^3}\cdot V_{weld\_new}=29\ kg$ Mass - welds; new design

$\eta_{weld}:=1-\frac{m_{new\_weld}}{m_{old\_weld}}=28\%$ Material saving [%] - welds

$\Delta m_{saving\_weld}:=m_{old\_weld}-m_{new\_weld}=11\ kg$ Material saving [kg] - welds

9.5 Total savings

$m_{old\_bridge}:=m_{old\_girder}+m_{old\_stud}+m_{old\_cb}=109\ 10^3\ kg$ Total mass of original design

$m_{new\_bridge}:=m_{new\_girder}+m_{new\_stud}+m_{new\_cb}=67\ 10^3\ kg$ Total mass of new design

$\eta_{bridge}:=1-\frac{m_{new\_bridge}}{m_{old\_bridge}}=39.1\%$ Material saving [%] - full bridge

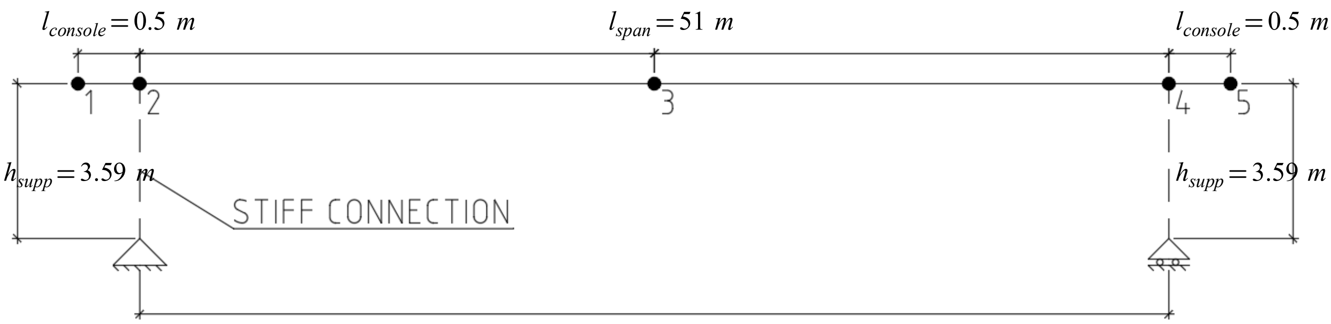
$\Delta m_{saving.tot}:=\Delta m_{saving\_girder}+\Delta m_{saving\_stud}+\Delta m_{saving\_cb}+\Delta m_{saving\_weld}=43\ 10^3\ kg$
Material saving [kg] - full bridge

Extracts - Calculation report 1.75h_w

3 System

In the following chapter the input data for the bridge's geometries is presented. The cross-sectional parameters that are presented is preliminary and used for the system analysis. In chapter 5 the final design is presented.

3.1 Primary system - longitudinal



The bridge is modelled as a simply supported bridge in the software StripStep-2. Due to the beams depth the supports are set offset from the neutral axis. This is modelled with a stiff connection in the system analysis.

3.1.1 Cross-section dimensions

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

The new height of the girder is calculated as: $Round(2050\ 1.75, 10) = 3590$

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	600	8	3505	50	700
500	35	600	8	3505	50	700

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	600	8	3505	50	700
10500	35	600	8	3505	50	700
10500	40	600	6	3495	55	700
11300	40	700	6	3495	55	900
25500	40	700	6	3495	55	900

Important! Two exactly the same values will not work with the linterp- function. Therefore 0.1 millimeter must be added for a X-value where you want to cross-sectional properties at the same time.

Element 3

S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	600	8	3505	50	700
500	35	600	8	3505	50	700

$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=3590\ mm$

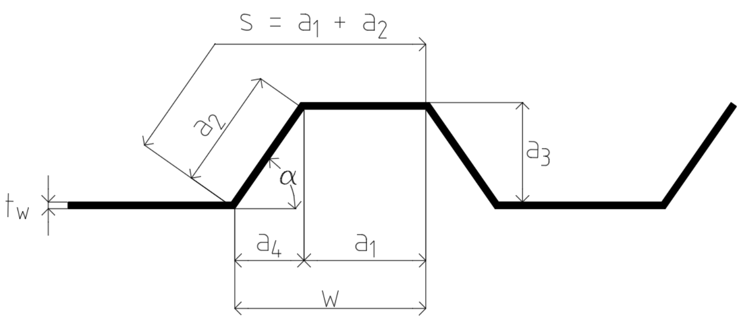
$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=3590\ mm$

3.1.2 Corrugation shape

$a_{c1}:=120\ mm$ Flat-fold length

$\alpha_c:=36\ deg$ Corrugation angle

$a_{c3}:=70\ mm$ Corrugation depth



$a_{c2}:=\dfrac{a_{c3}}{\sin\left(\alpha_c\right)}=119\ mm$ Length of angled part

$a_{c4}:=\dfrac{a_{c3}}{\tan\left(\alpha_c\right)}=96\ mm$ Length of hypopythis

$s_c:=a_{c1}+a_{c2}=239\ mm$ Length of corrugation

$w_c:=a_{c1}+a_{c4}=216\ mm$ Straight length

$r_c:=\dfrac{s_c}{w_c}=1.11$ Ratio corrugation/flat-fold length

3.1.3.2 Effective width, steel flanges

Calculation of the effective width in the steel flanges with regards to shear lag is calculated according to SS-EN 1993-1-5 3.2.

$b_{eff\_f} = \beta \cdot b_0 + \frac{t_w}{2}$

Effective width with regards to shear lag under elastic conditions - Equation (3.1)

$X_{check\_uf} := \frac{L_{bridge}}{2} = 26.0\ m$

$X_{check\_lf} := 0.2\ L_{bridge} = 10.4\ m$

Upper flange

$\alpha_0 := 1.0$

Equation given in Table 3.1. For webs without any longitudinal stiffeners $\alpha_0 = 1.0$.

$bfu := b_{fu}\left(X_{check\_uf}\right) = 700\ mm$

Width of upper flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_uf}\right) = 6\ mm$

Thickness of web - see Appendix B - Preliminary sizing

$b0 := \frac{bfu - tw}{2} = 347\ mm$

Figure 3.2 (Notations for shear lag)

$\kappa fu := \frac{\alpha_0 \cdot b0}{L_{span}} = 0.007$ = $\kappa_{fu}\left(X_{check\_uf}\right) = 0.007$

Table 3.1

$$\beta_{check} := \begin{array}{l} \text{if } \kappa fu \le 0.02 \\ \quad \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fu \le 0.70 \\ \quad \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fu^2} \\ \text{else if } 0.70 < \kappa fu \\ \quad \beta \leftarrow \dfrac{1}{8.6\ \kappa fu^2} \end{array} = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for upper flange

$2\ \beta_{check} \cdot b0 + tw = 700\ mm$

$$b_{ef} := \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad b \leftarrow bfu \\ \text{else} \\ \quad b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfu\right) \\ b \end{array} = 0.700\ m$$ = $b_{eff\_fu}\left(X_{check\_uf}\right) = 0.700\ m$

Lower flange

$bfl := b_{fl}\left(X_{check\_lf}\right) = 700\ mm$ Width of lower flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_lf}\right) = 8\ mm$ Thickness of web - see Appendix B - Preliminary sizing

$b0 := \dfrac{bfl - tw}{2} = 346\ mm$ Figure 3.2 (Notations for shear lag)

$\kappa fl := \dfrac{\alpha_0 \cdot b0}{L_{span}} = 0.007$ = $\kappa_{fl}\left(X_{check\_lf}\right) = 0.007$ Table 3.1

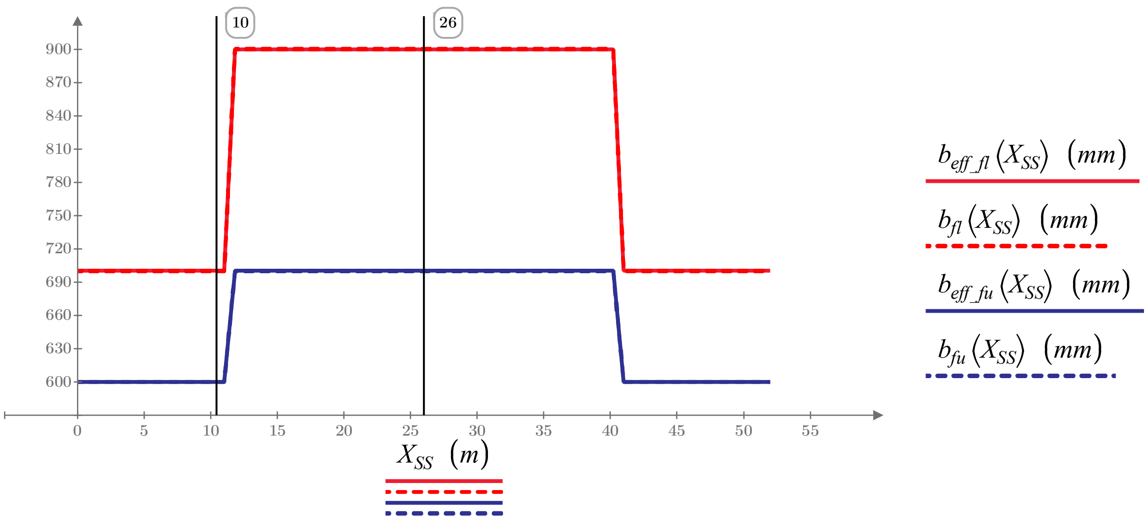
$$\beta_{check} := \left\| \begin{array}{l} \text{if } \kappa fl \le 0.02 \\ \quad \| \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fl \le 0.70 \\ \quad \left\| \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fl^2} \right. \\ \text{else if } 0.70 < \kappa fl \\ \quad \left\| \beta \leftarrow \dfrac{1}{8.6\ \kappa fl^2} \right. \end{array} \right. = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for lower flange

$2\ \beta_{check} \cdot b0 + tw = 700\ mm$

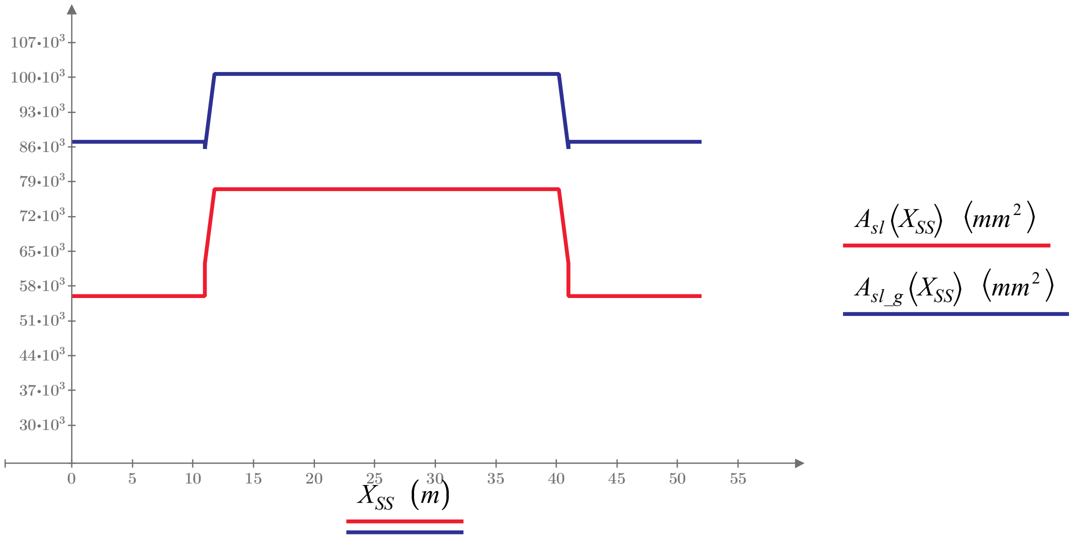
$$b_{ef} := \left\| \begin{array}{l} \text{if } \beta_{check} = 1.0 \\ \quad \| b \leftarrow bfl \\ \text{else} \\ \quad \| b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfl\right) \\ b \end{array} \right. = 0.700\ m$$ = $b_{eff\_fl}\left(X_{check\_lf}\right) = 0.700\ m$



3.1.4.2 Cross-sectional constants during construction

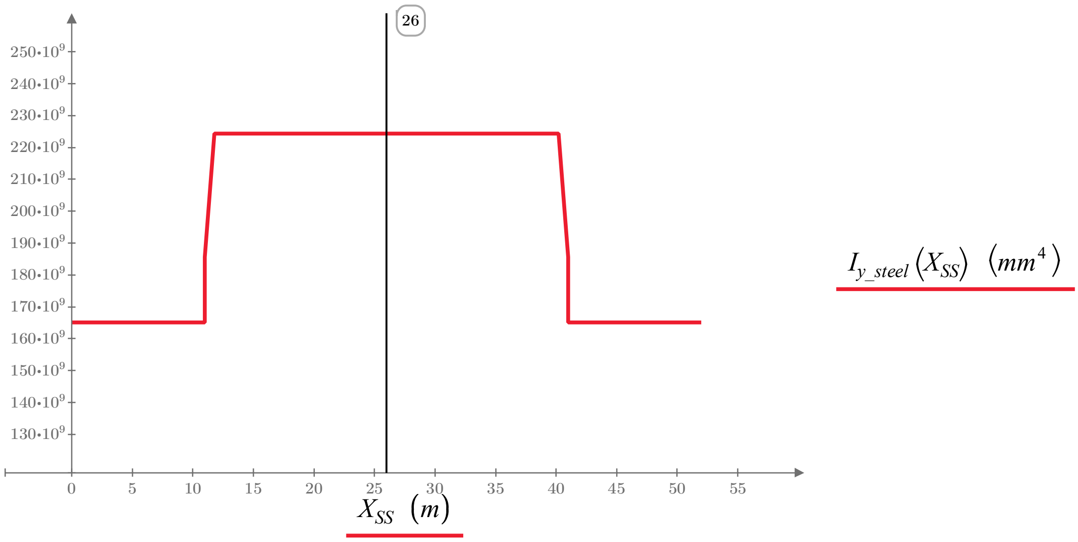
The bridge is checked so that the steel girder (alone) can withstand the loads that are imposed during construction such as the self-weight of curing concrete.

A_{sl_g} is the area of the steel including the web, for calculation of self-weight
A_{sl} is the area of the steel excluding the web, for stiffness calculations



$$A_{sl}\left(\frac{L_{bridge}}{2}\right)=0.078\ m^2$$ Area at mid-span

I_{y_steel} is the area of the steel excluding the web, for stiffness calculations



$$I_{y\_steel}\left(\frac{L_{bridge}}{2}\right)=0.2244\ m^4$$ Stiffness at mid-span

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	$[10^4m^4]$	$[10^4m^2]$	[kN/m]
0.000	-2.235	3.590	1651.844	560	6.569
0.500	-2.235	3.590	1651.844	560	6.569

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	$[10^4m^4]$	$[10^4m^2]$	[kN/m]
0.000	-2.235	3.590	1651.844	560	6.569
10.500	-2.235	3.590	1651.844	560	6.569
10.500	-2.202	3.590	1855.418	625	6.469
11.300	-2.283	3.590	2244.464	775	7.602
25.500	-2.283	3.590	2244.464	775	7.602
39.700	-2.283	3.590	2244.464	775	7.602
40.500	-2.202	3.590	1855.418	625	6.469
40.500	-2.235	3.590	1651.844	560	6.569
51.000	-2.235	3.590	1651.844	560	6.569

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A	g
[m]	[m]	[m]	$[10^4m^4]$	$[10^4m^2]$	[kN/m]
0.000	-2.235	3.590	1651.844	560	6.569
0.500	-2.235	3.590	1651.844	560	6.569

System model used in Strip-Step2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.59	ZR	
					2
3	51.500	0.00	-3.59	YZR	
					3
4	52.000	0.00			-3

3.1.4.3 Cross-sectional constants for variable loads (short term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0.000	-0.254	3.910	4330.65	3239
0.500	-0.254	3.910	4330.65	3239

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0.000	-0.254	3.910	4330.65	3239
10.500	-0.254	3.910	4330.65	3239
10.500	-0.287	3.910	4705.82	3304
11.300	-0.388	3.910	5853.53	3454
25.500	-0.388	3.910	5853.53	3454
39.700	-0.388	3.910	5853.53	3454
40.500	-0.287	3.910	4705.82	3304
40.500	-0.254	3.910	4330.65	3239
51.000	-0.254	3.910	4330.65	3239

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0.000	-0.254	3.910	4330.65	3239
0.500	-0.254	3.910	4330.65	3239

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.59	ZR	
					2
3	51.500	0.00	-3.59	YZR	
					3
4	52.000	0.00			-3

3.1.4.4 Cross-sectional constants for additional permanent loads (long term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^{4}m^{4}]$	A $[10^{4}m^{2}]$
0	-0.789	3.910	3597.338	1413
0.5	-0.789	3.910	3597.338	1413

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^{4}m^{4}]$	A $[10^{4}m^{2}]$
0	-0.789	3.910	3597.338	1413
10.5	-0.789	3.910	3597.338	1413
10.5	-0.839	3.910	3874.961	1478
11.3	-1.003	3.910	4673.921	1628
25.5	-1.003	3.910	4673.921	1628
39.7	-1.003	3.910	4673.921	1628
40.5	-0.839	3.910	3874.961	1478
40.5	-0.789	3.910	3597.338	1413
51	-0.789	3.910	3597.338	1413

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^{4}m^{4}]$	A $[10^{4}m^{2}]$
0	-0.789	3.910	3597.338	1413
0.5	-0.789	3.910	3597.338	1413

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.59	ZR	
					2
3	51.500	0.00	-3.59	YZR	
					3
4	52.000	0.00			-3

3.1.4.5 Cross-sectional constants for shrinkage analysis

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.669	3.910	3760.040	1617
0.5	-0.669	3.910	3760.040	1617

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.669	3.910	3760.040	1617
10.5	-0.669	3.910	3760.040	1617
10.5	-0.718	3.910	4056.001	1682
11.3	-0.873	3.910	4921.372	1832
25.5	-0.873	3.910	4921.372	1832
39.7	-0.873	3.910	4921.372	1832
40.5	-0.718	3.910	4056.001	1682
40.5	-0.669	3.910	3760.040	1617
51	-0.669	3.910	3760.040	1617

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0	-0.669	3.910	3760.040	1617
0.5	-0.669	3.910	3760.040	1617

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-3.59	ZR	
					2
3	51.500	0.00	-3.59	YZR	
					3
4	52.000	0.00			-3

4 Loads and load combinations

4.1 Permanent loads

4.1.1 Self-weight

4.1.1.1 Steel

The self-weight is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: STEEL

$\rho_{st} = 75.51 \ \frac{kN}{m^3}$ Self-weight of stainless steel - SS-EN 10088-1:2014 Table E.1 or E.2

The self-weight for each element is calculated in chapter 3.

4.1.1.2 Concrete

The self-weight of the slab is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: SLAB

$Wet\_Concrete := \text{"NO"}$ "YES" or "NO" dependent if the previous designer has used the weight of wet concrete

$\rho_c := \textbf{if}\left(Wet\_Concrete = \text{"NO"}, 25 \ \frac{kN}{m^3}, 26 \ \frac{kN}{m^3}\right) = 25 \ \frac{kN}{m^3}$ Self-weight of concrete (reinforced) - SS-EN 1992-1-1 Table A.1

$A_{slab} = 3.47 \ m^2$ Area of slab, see chapter 3

$g_{slab} := \frac{A_{slab} \cdot \rho_c}{2} = 43.4 \ \frac{kN}{m}$ Self-weight of concrete slab, (half of the load goes to each girder) - applied in the casting stage

If the previous designer has considered that the hardened concrete has a smaller self-weight a reduction in self-weight is applied in the system analysis for permanent loads.

Appendix X. Name in Strip-Step2: AVSLAB

$$g_{slab.perm} := \textbf{if}\left(Wet\_Concrete = \text{"NO"}, 0 \ \frac{kN}{m}, \frac{A_{slab} \cdot (-1) \ \frac{kN}{m^3}}{2}\right) = 0.000 \ \frac{kN}{m}$$

4.1.2 Shrinkage

The shrinkage force is calculated and used in Strip-Step2, Appendix X. Name in Strip-Step2: E:SHRINK

$h_0 = 341\ mm$ Equivalent thickness, calculated in chapter 2

$t = 120\ yr$

$t = 43829.1\ day$

$f_{cm} = 43\ MPa$ Mean compressive strength - see chapter 2

$t_s := 1\ day$

$RH := 80\%$ Krav Brobyggande B.3.1.5

$$\beta_{ds} := \frac{\dfrac{t-t_s}{day}}{\left(\dfrac{t-t_s}{day}\right) + 0.04 \cdot \sqrt{\left(\dfrac{h_0}{1\ mm}\right)^3}} = 0.994$$

SS-EN 1992-1-1 3.1.4 Equation 3.10

$$k_h := \begin{array}{l} \text{if } 200\ mm \le h_0 < 300\ mm \\ \quad \left\| 0.85 - \left(\dfrac{h_0}{mm} - 200\right) \cdot 0.001 \right. \\ \text{else if } 300\ mm \le h_0 < 500\ mm \\ \quad \left\| 0.75 - \left(\dfrac{h_0}{mm} - 300\right) \cdot 0.0005 \right. \\ \text{else if } h_0 \ge 500\ mm \\ \quad \| 0.7 \\ \text{else} \\ \quad \| 1.0 \end{array} = 0.73$$

SS-EN 1992-1-1 3.1.4 table 3.3

$RH_0 := 100\%$

$\alpha_{ds1} := 4$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$\alpha_{ds2} := 0.12$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0}\right)^3\right) = 0.76$$

SS-EN 1992-1-1 Appendix B.2 equation B12

$f_{cmo} := 10\ MPa$ SS-EN 1992-1-1 Appendix B.2 (1)

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}\right)}\right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$

SS-EN 1992-1-1 Appendix B.2 equation B11

$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 1.84 \cdot 10^{-4}$ Drying shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.9

$\varepsilon_{ca0} := 2.5 \cdot \left(\frac{f_{ck} - f_{cmo}}{MPa}\right) \cdot 10^{-6} = 6.3 \cdot 10^{-5}$ SS-EN 1992-1-1 3.1.4 Equation 3.12

$\beta_{as} := 1 - e^{\left(-0.2 \cdot \sqrt{\frac{t - t_s}{day}}\right)} = 1.0$ SS-EN 1992-1-1 3.1.4 Equation 3.13

$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca0} = 6.25 \cdot 10^{-5}$ Autogenous shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.11

$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.8

4.1.2.1 Shrinkage force

The shrinkage force and corresponding moment is calculated accordingly:

$n_{L\_cs} = 14.91$ Modular ratio accounting for creep and shrinkage

$n_{L\_short} = 5.88$ Modular ratio

$E_{c.eff} := \frac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab (half of the cross-section used in system analysis)

$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5206\ kN$ Total shrinkage force

$e_{cs} := z_{tp\_cs}(0\ m) = 0.669\ m$ The shrinkage force is applied in the center of gravity for the composite section

$M_{cs} := F_{cs} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right) = 4317\ kN \cdot m$ Total bending moment

Shrinkage - anchorage of studs

<u>Forces in the concrete</u>

$$F_{c\_cs} := F_{cs} \cdot \left(1 - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{sl\_cs}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{y\_cs}(0\ m)} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right)^2\right) = 795\ kN$$ Force in concrete

$$b_{eff}(0\ m) = 4.925\ m$$ Effective width of flange

$$M_{c\_cs} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_cs}} \cdot h_{m.slab}^{\ 3}}{I_{y\_cs}(0\ m)} \cdot M_{cs} = 10\ kN \cdot m$$ Moment in concrete

$$\sigma_1 := \frac{F_{c\_cs}}{A_{slab.fic}} - \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 3}} = 0.38\ MPa$$ Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{F_{c\_cs}}{A_{slab.fic}} + \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 3}} = 0.63\ MPa$$ Stress in concrete, lower edge of slab

$$N_{cs} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = 795\ kN \quad = \quad F_{c\_cs} = 795\ kN$$ Force imposed on studs caused by shrinkage (used in chapter 6)

4.2.2.1 Horizontal side trafic load

$P_{h\_side\_car} := max\left(\begin{bmatrix} 25\% \cdot Q_{1k\_br} \\ 25\% \cdot Q_{tr\_br} \end{bmatrix}\right) = 109.1\ kN$ Horisontal side force from the acceleration load (used in chapter 7)

$P_{v\_side\_car} := P_{h\_side\_car} \cdot \dfrac{h_{m.slab}}{2 \cdot B_{in}} = 6.2\ kN$ Vertical side force from the acceleration load (used in chapter 7)

4.2.3 Temperature load

Temperatures are determined according to SS-EN 1991-1-5, 6.1.3 unless otherwise stated.

Load case 1 - local temperature differences for Hudiksvall

$T_0 = 10\ °$ A.1(3)

$T_{min} = -38\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{max} = 34\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4\ ° = -34\ °$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min} = 44\ °$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4\ ° = 38\ °$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0 = 28\ °$ Expansion - Equation 6.1

$\Delta T := T_{e.max} - T_{e.min} = 72\ °$ Total temperature difference

Load case 2 - either of the components are larger than the other

$\Delta T_{c2st} := 15\ °$ Temperature difference between concrete and steel - SS-EN 1991-1-5, 6.1.6

4.2.3.1 Coefficients of thermal linear expansion

For composite bridges normally it is suggested to use the same thermal linear expansion coefficient, according to SS-EN 1991-1-5 Table C.1. However for stainless steels the thermal linear expansion coefficient is much larger than for concrete and hence a more through calculation is needed for the strain.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

4.2.3.2 Strains for the different load cases

$\Delta\varepsilon_{LC.1\_con} := -(\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.con}}{\circ} = -26.4\ 10^{-5}$ Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp} := (\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.exp}}{\circ} = 16.8\ 10^{-5}$ Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss} \cdot \frac{\Delta T_{c2st}}{\circ} = 24.0\ 10^{-5}$ Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c \cdot \frac{\Delta T_{c2st}}{\circ} = 15.0\ 10^{-5}$ Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st}, \Delta\varepsilon_{LC.2\_c}\right) = 24.0\ 10^{-5}$ Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp_1} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con} = -50.4\ 10^{-5}$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp_2} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp} = 40.8\ 10^{-5}$ Maximum strain difference; temperature raises and the steel heatens up even higher

4.2.3.3 Temperature load - global analysis

The concrete is transformed to steel. Already defined parameters are calculated in chapter 3.

$\frac{b_{eff}(0\ m)}{n_{L\_short}} = 0.837\ m$ Width of transformed concrete

$\frac{A_{slab.fic}}{n_{L\_short}} = 0.268\ m^2$ Area of transformed concrete

$A_{sl}(0\ m) = 0.056\ m^2$ Area of composite section

$z_{tp\_short}(0\ m) = 254.1\ mm$ Distance from top of concrete to center of gravity for composite section

$I_{y\_short}(0\ m) = 0.433\ m^4$ Moment of inertia for composite section

$F_{temp} := \varepsilon_{temp} \cdot E_s \cdot A_{sl}(0\ m) = \begin{bmatrix} -5645 \\ 4570 \end{bmatrix} kN$ Force on composite section

$e_{temp\_F} := z_{tp\_short}(0\ m) = 254.1\ mm$ Level at which the force is imposed on the system

$e_{temp\_M} := z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) = 1.981\ m$ Eccentricity for the bending moment

$$M_{temp} := F_{temp} \cdot e_{temp\_M} = \begin{bmatrix} -11180 \\ 9050 \end{bmatrix} kN \cdot m$$

Bending moment - composite section

Anchorage of temperature load imposed on studs

<u>Concrete</u>

$$N_{c\_temp} := F_{temp} \cdot \left(\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{A_{sl\_short}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_short}}}{I_{y\_short}(0\ m)} \cdot \left(\left(z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) \right) \cdot \left(z_{tp\_short}(0\ m) + \frac{h_{m.slab}}{2} \right) \right) \right) = \begin{bmatrix} -1805 \\ 1461 \end{bmatrix} kN$$

$$M_{c\_temp} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_short}} \cdot h_{m.slab}^{\ 3}}{I_{y\_short}(0\ m)} \cdot M_{temp} = \begin{bmatrix} -59 \\ 48 \end{bmatrix} kN \cdot m$$

$$\sigma_1 := \frac{N_{c\_temp}}{A_{slab.fic}} - \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 2}} = \begin{bmatrix} -0.44 \\ 0.36 \end{bmatrix} MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{N_{c\_temp}}{A_{slab.fic}} + \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{\ 2}} = \begin{bmatrix} -1.85 \\ 1.50 \end{bmatrix} MPa$$

Stress in concrete, lower edge of slab

$$N_{temp} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = \begin{bmatrix} -1805 \\ 1461 \end{bmatrix} kN$$

Force imposed on studs caused by shrinkage (used in chapter 6 when calculating the anchorage of the slab by the studs)

5 Capacity checks during construction

The capacity check that is carried out in this chapter is bending moment capacity with respect to lateral torsional buckling in the casting phase

In the casting phase the normalforces in the cross-section are neglectible.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

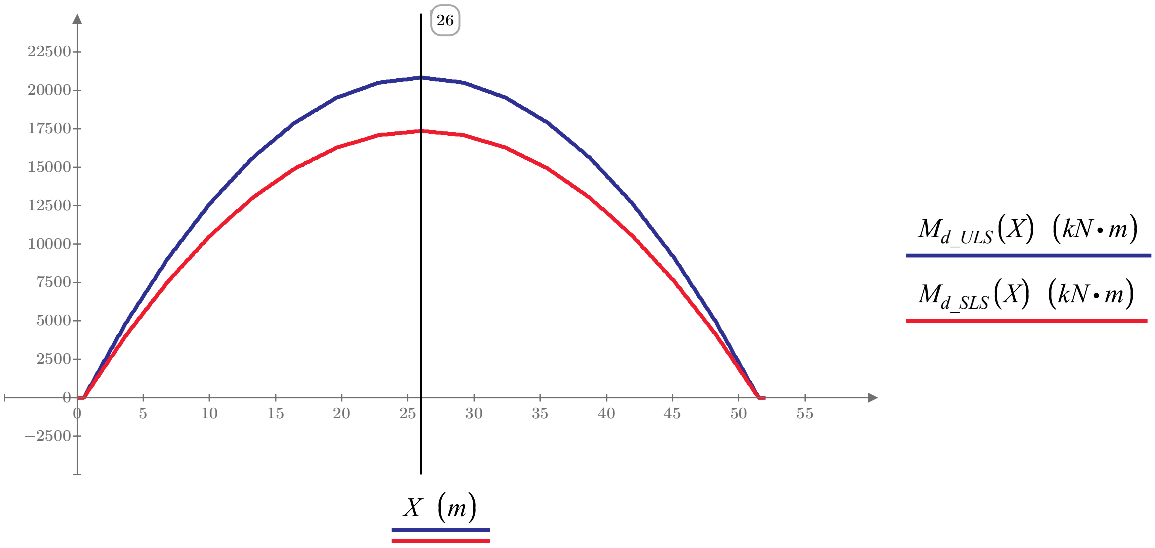
$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

5.1 Load effects

Load effects retrieved from Strip-Step2, Appendix X.

Bending moment

M_{d_ULS} Bending moment descending from permanent loads during construction in ultimate limit state
M_{d_SLS} Bending moment descending from permanent loads during construction in service limit state

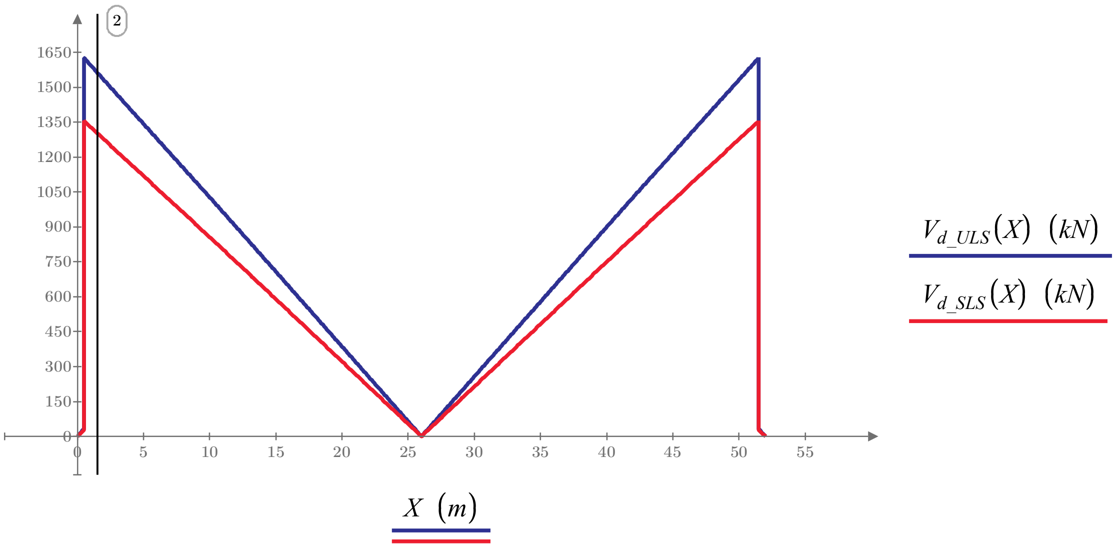


$M_{d\_ULS}(X_{check\_m}) = 20834\ kN \cdot m$ Bending moment at control point - ULS

$M_{d\_SLS}(X_{check\_m}) = 17361\ kN \cdot m$ Bending moment at control point - SLS

Shear force

V_{d_ULS} Shear force descending from permanent loads during construction in ultimate limit state
V_{d_SLS} Shear force descending from permanent loads during construction in service limit state



$V_{d\_ULS}(X_{check\_v}) = 1563\ kN$ Shear force at control point - ULS

$V_{d\_SLS}(X_{check\_v}) = 1303\ kN$ Shear force at control point - SLS

5.2 Redesign of cross-section

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	600	6	3510	45	600
500	35	600	6	3510	45	600

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	600	6	3510	45	600
12500	35	600	6	3510	45	600
12500	35	600	4	3505	50	600
13300	35	700	4	3505	50	700
25500	35	700	4	3505	50	700

10500
10500
11300

Element 3

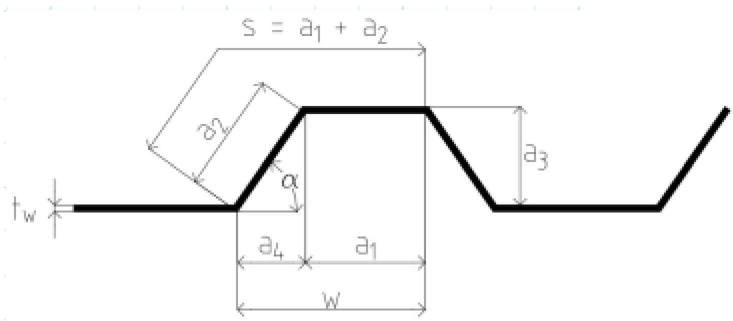
S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	600	6	3510	45	600
500	35	600	6	3510	45	600

$$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=3590\ mm$$

$$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=3590\ mm$$

Check to see that the girder height is kept constant

5.2.1 Shape of corrugation



$a_{c1} := 185\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 110\ mm$ Corrugation depth

$a_{weld} := 5\ mm$ Weld throat thickness

$a_{c2} := \dfrac{a_{c3}}{sin(\alpha_c)} = 187\ mm$ Legnth of hypotenuse

$a_{c4} := \dfrac{a_{c3}}{tan(\alpha_c)} = 151\ mm$ Length of corrugation depth, see figure

$s_c := a_{c1} + a_{c2} = 372\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 336\ mm$ Straight length

$r_c := \dfrac{s_c}{w_c} = 1.106$ Ratio corrugation/straight length

5.2.2 Cross-section classification

The cross-section classes is determined according to SS-EN 1993 1-4 5.2.2 with updated limits from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated and does not contribute to the axial stiffness the web is not classified.

$$c_w(x) := h_w(x) - 2 \cdot \sqrt{2} \cdot a_{weld}$$

$$c_{uf}(x) := \frac{b_{fu}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on upper flange

$$c_{lf}(x) := \frac{b_{fl}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on lower flange

<u>Cross-section class, upper flange</u>

$E_s = 200\ GPa$ Modulus of elasticity

$f_{yuf} = 450\ MPa$ Proof strength of top flange

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

SS-EN 1993-1-4 5.2.2 Table 5.2

$$csc_{uf}(x) := \begin{cases} \text{"csc1"} & \text{if } \frac{c_{uf}(x)}{t_{fu}(x)} \le 9\ \varepsilon_{uf} \\ \text{"csc2"} & \text{else if } 9\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 10\ \varepsilon_{uf} \\ \text{"csc3"} & \text{else if } 10\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 14\ \varepsilon_{uf} \\ \text{"csc4"} & \text{else} \end{cases}$$

Cross-section class upper flange

$csc_{uf}(X_{check\_m}) = \text{"csc4"}$ Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{uf}(X_{check\_v}) = \text{"csc3"}$ Cross-section class at $X_{check\_v} = 1.500\ m$

Cross-section class, lower flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{ylf} = 450\ MPa$ — Proof strength of top flange

$\varepsilon_{lf} := \sqrt{\frac{235}{f_{ylf}} \cdot \frac{E_s}{210000}} = 0.71$ — SS-EN 1993-1-4 5.2.2 Table 5.2

Cross-section class lower flange

$$csc_{lf}(x) := \begin{cases} \text{if } \frac{c_{lf}(x)}{t_{fl}(x)} \le 9\ \varepsilon_{lf} & \text{“csc1”} \\ \text{else if } 9\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 10\ \varepsilon_{lf} & \text{“csc2”} \\ \text{else if } 10\ \varepsilon_{lf} < \frac{c_{lf}(x)}{t_{fl}(x)} \le 14\ \varepsilon_{lf} & \text{“csc3”} \\ \text{else} & \text{“csc4”} \end{cases}$$

$csc_{lf}\left(X_{check\_m}\right) = \text{“csc3”}$ — Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{lf}\left(X_{check\_v}\right) = \text{“csc3”}$ — Cross-section class at $X_{check\_v} = 1.500\ m$

5.2.3 Plate buckling of compressive flange

If the compressed flange is in cross-section class four an effective width of the compressed flange is calculated according to SS-EN 1993-1-4 5.2.3. Updated values are taken from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated the buckling factor k_σ is calculated according to SS-EN 1993-1-5 D.2.1 (2)

$a_{bend} := a_{c1} + 2\ a_{c4} = 488\ mm$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$c_u := c_{uf}\left(X_{check\_m}\right) = 396\ mm$ — Width of outstand flange from weld toe to free edge

$t_u := t_{fu}\left(X_{check\_m}\right) = 35\ mm$ — Thickness of upper flange

$b_u := b_{fu}\left(X_{check\_m}\right) = 700\ mm$ — Width of upper flange

$\varepsilon_{uf} = 0.71$

$k_{\sigma 1} := 0.43 + \left(\frac{c_u}{a_{bend}}\right)^2 = 1.1$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_{\sigma 2} := 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_\sigma := min\left(k_{\sigma 1}, k_{\sigma 2}\right) = 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

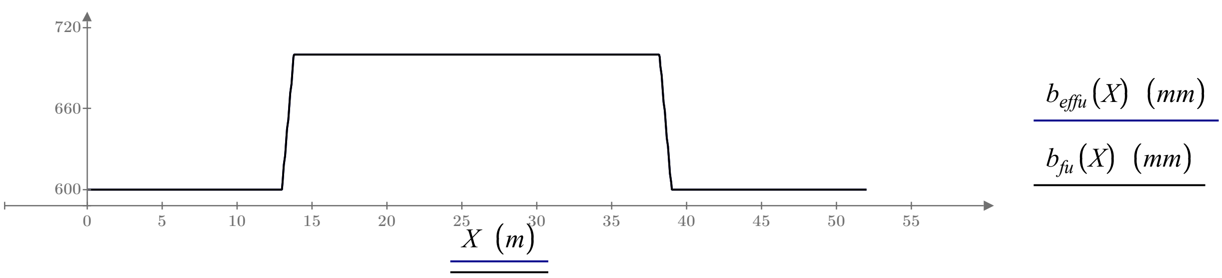
$$\lambda_p := \frac{\frac{c_u}{t_u}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_\sigma}} = 0.73$$

Slenderness of flange plate SS-EN 1993-1-1 (2)

$$\rho := \mathbf{if}\left(\lambda_p \le 0.748, 1.0, \frac{\lambda_p - 0.188}{\lambda_p^{\ 2}}\right) = 1.00$$

Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

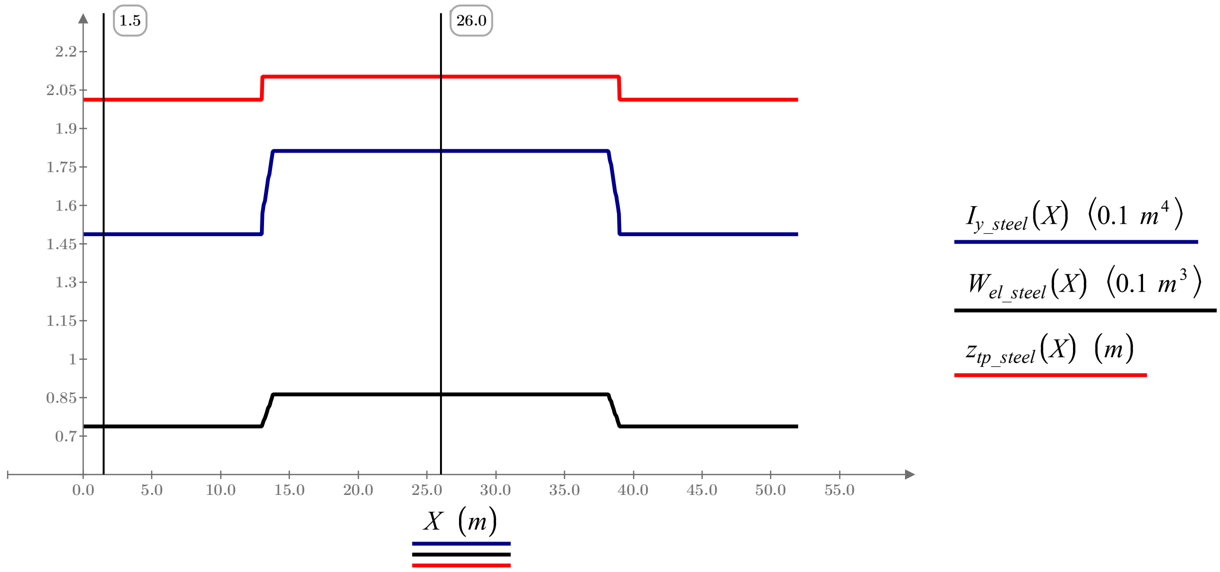
$b_{eff} := b_u \cdot \rho = 700\ mm$ = $b_{effu}\left(X_{check\_m}\right) = 700\ mm$



$b_{fu}(x) := b_{effu}(x)$ — Renaming the width of flange in order to minimize errors

5.2.6 New cross-sectional constants during casting

$I_{y\_steel}(x)$	Stiffness of steel girder alone (excluding the web)
$z_{tp\_steel}(x)$	Distance from the top of the top flange to the center of gravity for the steel section
$W_{el\_steel}(x)$	Elastic bending stiffness of steel girder alone (excluding the web)



Coordinates and cross-sectional constants for control calculations

$X_{check\_m} = 26\ m$	X- coordinate for control calculations, bending moment
$A_{sl}(X_{check\_m}) = 0.06\ m^2$	Area
$I_{y\_steel}(X_{check\_m}) = (181 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_m}) = 2.104\ m$	Center of gravity
$W_{el\_steel}(X_{check\_m}) = (86 \cdot 10^{-3})\ m^3$	Elastic bending resistance
$X_{check\_v} = 1.5\ m$	X- coordinate for control calculations, shear force
$A_{sl}(X_{check\_v}) = 0.048\ m^2$	Area
$I_{y\_steel}(X_{check\_v}) = (149 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_v}) = 2.014\ m$	Center of gravity
$W_{el\_steel}(X_{check\_v}) = (74 \cdot 10^{-3})\ m^3$	Elastic bending resistance

5.2.4 Lateral torsional buckling of compressive flange - SS-EN 1993-1-4 5.4.2.1

Simplified method only consisdering buckling of top flange according to SS-EN 1993-1-4 5.4.2.1

$\alpha_{LT} := 0.76$ — Buckling curve d, welded open cross-section

$l_{cr} := 7.29\ m$ — Distance between the cross-beams

$b_{ef} := b_{effu}\left(X_{check\_m}\right) = 700\ mm$ — Width of upper flange

$t_f := t_{fu}\left(X_{check\_m}\right) = 35\ mm$ — Thickness of upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength

$I_{zf} := \dfrac{b_{ef}^{\ 3} \cdot t_f}{12} = 0.001\ m^4$ — Moment of inertia, upper flange

$N_{crLT} := \dfrac{\pi^2 \cdot E_s \cdot I_{zf}}{l_{cr}^{\ 2}} = 37158\ kN$ — Critical buckling load

$\lambda_{LT\_u} := \sqrt{\dfrac{b_{ef} \cdot t_f \cdot f_{yuf}}{N_{crLT}}} = 0.545$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.9

$\Phi_{LT\_u} := 0.5 \cdot \left(1 + \alpha_{LT} \cdot \left(\lambda_{LT\_u} - 0.2\right) + \lambda_{LT\_u}^{\ 2}\right) = 0.78$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.7

$$\chi_{LT\_u} := min\left(\frac{1}{\Phi_{LT\_u} + \sqrt{\Phi_{LT\_u}^{\ 2} - \lambda_{LT\_u}^{\ 2}}}, 1\right) = 0.748 \quad = \quad \chi_{LT}\left(X_{check\_m}\right) = 0.748$$

SS-EN 1993-1-4 5.4.2.1 Equation 5.6

$h_{w\_u} := h_w\left(X_{check\_m}\right) = 3505\ mm$ — Height of web

$t_l := t_{fl}\left(X_{check\_m}\right) = 50\ mm$ — Thickness of lower flange

$k_{fl} := 1.1$ — Increase in capacity due to simlified method used, SS-EN 1993-1-1 6.3.2.4 (2)B

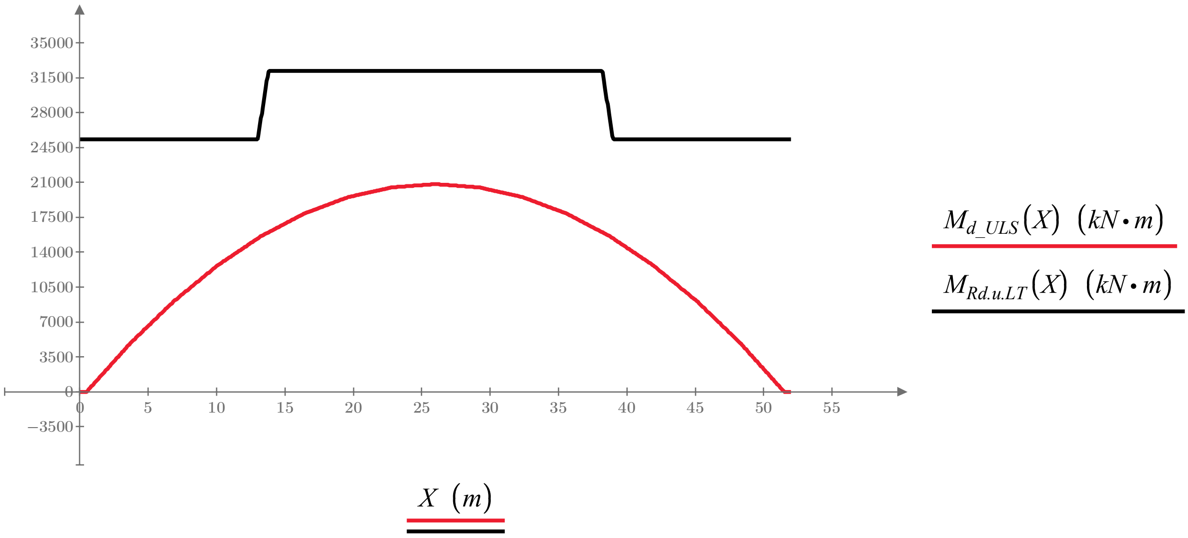
$$M_{Rd.u.LT\_u} := \frac{b_{ef} \cdot t_f \cdot k_{fl} \cdot \chi_{LT\_u} \cdot f_{yuf}}{\gamma_{M1}} \left(h_{w\_u} + \frac{t_f + t_l}{2}\right) = 32185\ kN \cdot m \quad = \quad M_{Rd.u.LT}\left(X_{check\_m}\right) = 32185\ kN \cdot m$$

SS-EN 1993-1-5 D.2.1 Equation D.1

5.2.5 Check of lateral torsional buckling

Check of the buckling capacity of the girders is performed.

$M_{Rd.u.LT}(X)$	Capacity of the girder against LT- buckling during casting
$M_{d\_ULS}(X)$	Load effects during casting, Appendix X



$$\eta_{max} := max\left(\frac{M_{d\_ULS}(X)}{M_{Rd.u.LT}(X)}\right) = 65\%$$

Maximum utilization rate of the steel cross-section during casting

5.3 Stresses in steel cross-section

The stresses in the top and bottom flange is calculated for the loading senario to be able to superposition them with the other loadcases to determinte the ultimate capacity of the composite section.

Upper flange

Control calculation at midspan

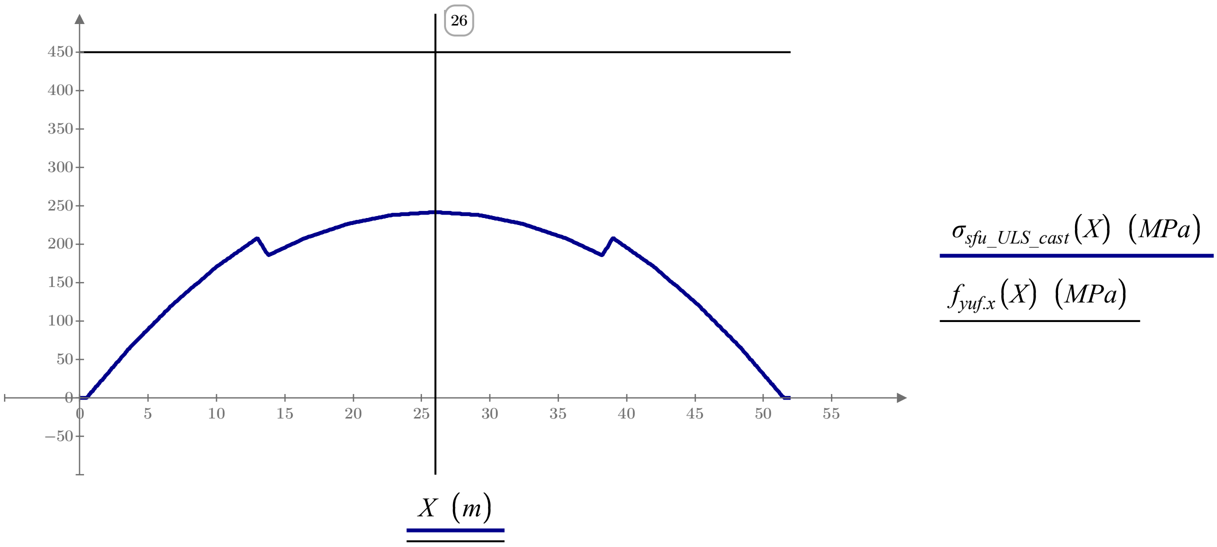
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 20834\ kN \cdot m$ Load effext at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.181\ m^4$ Moment of inertia at midspan

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 2.104\ m$ Centre of gravity at midspan maesured from the top of the beam

$\sigma := \frac{M}{I} \cdot z = 242\ MPa$ = $\sigma_{sfu\_ULS\_cast}\left(X_{check\_m}\right) = 242\ MPa$

$\sigma_{sfu\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\frac{\sigma_{sfu\_ULS\_cast}(X)}{f_{yuf}}\right) = 54\%$

Lower flange

Check calculation at midspan

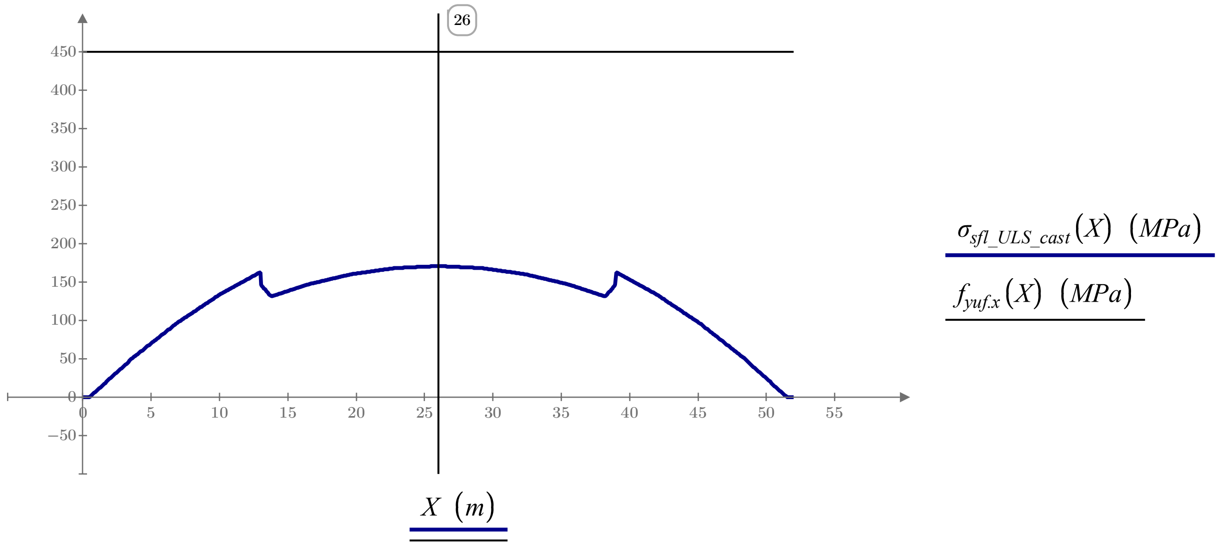
$M := M_{d\_ULS}(X_{check\_m}) = 20834\ kN \cdot m$ — Load effect at midspan

$I := I_{y\_steel}(X_{check\_m}) = 0.181\ m^4$ — Moment of inertia at midspan

$z := z_{tplf\_steel}(X_{check\_m}) = 1.486\ m$ — Centre of gravity at midspan measured from the bottom of the beam

$$\sigma := \frac{M}{I} \cdot z = 171\ MPa \quad = \quad \sigma_{sfl\_ULS\_cast}(X_{check\_m}) = 171\ MPa$$

$\sigma_{sfl\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$$\eta_{max} := max\left(\frac{\sigma_{sfl\_ULS\_cast}(X)}{f_{ylf}}\right) = 38\%$$

6 Capacity checks - Ultimate limit state, global

The capacity checks that are to be carried out are bending moment capacity, shear capacity, web breathing and design of studs, both in ULS and due to fatigue

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 0.5\ m$ Coordinate for control calculations - shear force

6.1 Load effects in ULS

6.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

Bending moment in the ultimate limit state

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{tr}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

M_{d_ULS} Design bending moment effect from all loads

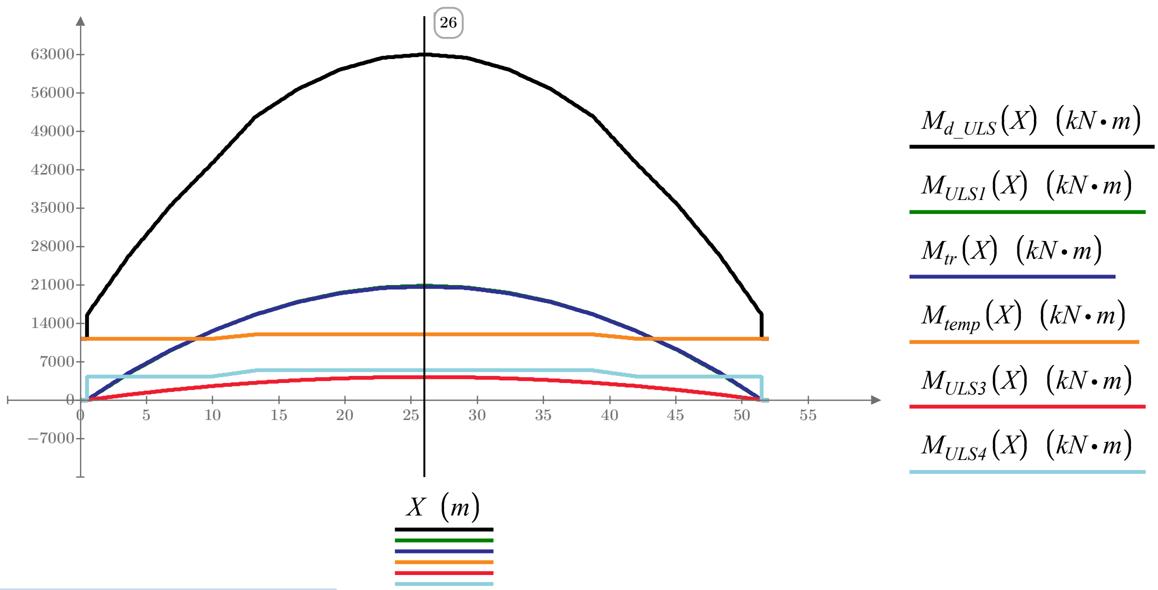
$M_{ULS1}(x)$ Bending moment during casting (Appendix X)

$M_{tr}(x)$ Bending moment from multi component loads (Appendix X)

$M_{tr}(x)$ Bending moment from temperature loads (Appendix X)

$M_{ULS3}(x)$ Bending moment from additional permanent loads after construction (Appendix X)

$M_{ULS4}(x)$ Bending moment from shrinkage (Appendix X)

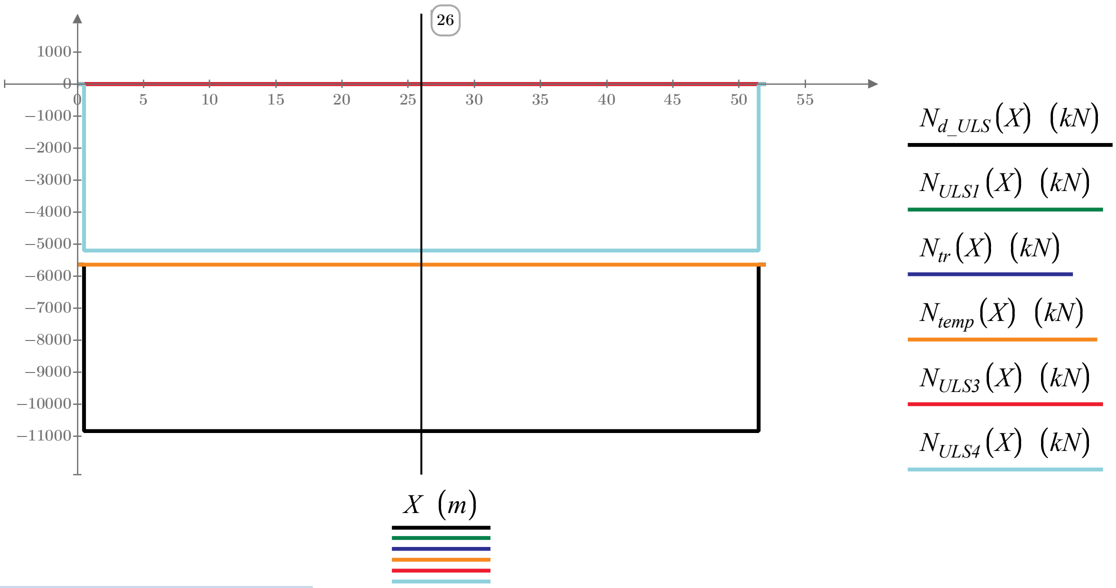


$M_{d\_ULS}(X_{check\_m}) = 62988\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{tr}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design normal force from all loads
$N_{ULS1}(x)$	Normal force during casting (Appendix X)
$N_{tr}(x)$	Normal force from multi components loads (Appendix X)
$N_{temp}(x)$	Normal force from temperature loads (Appendix X)
$N_{ULS3}(x)$	Normal force from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Normal force from shrinkage (Appendix X)

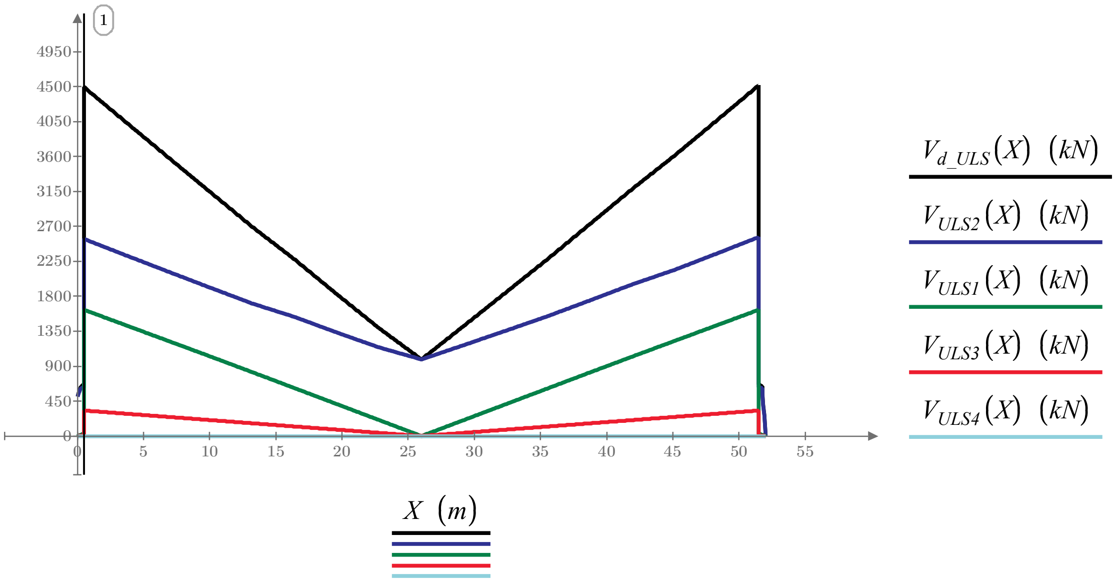


$N_{d\_ULS}(X_{check\_m}) = -10851\ kN$ Normal force at control point - ULS

6.1.2 Shear force

$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{ULS3}(x) + V_{ULS4}(x)$

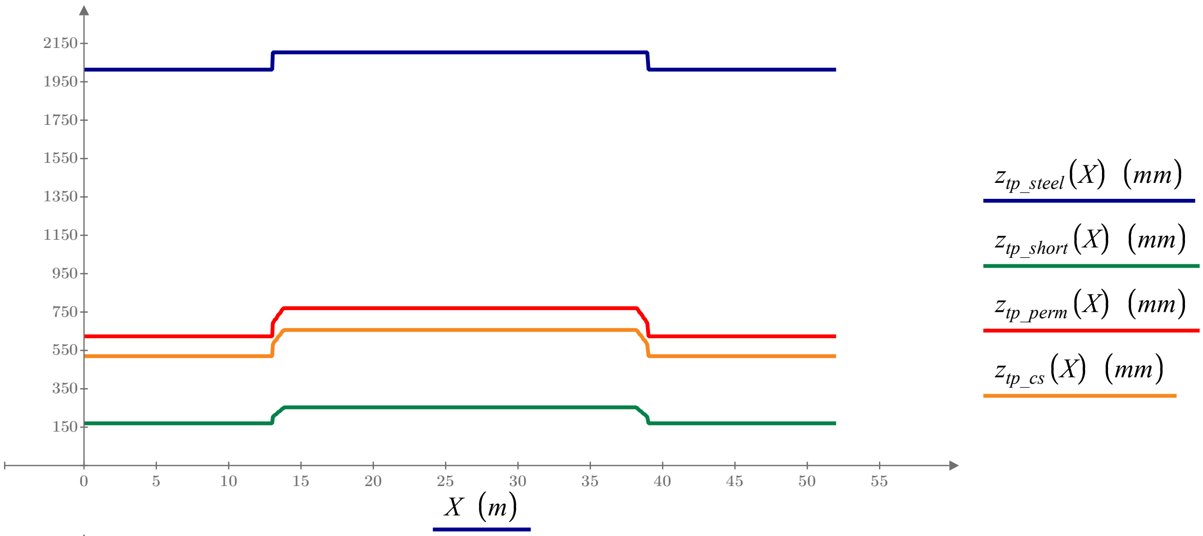
V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (both temperature and traffic) (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)

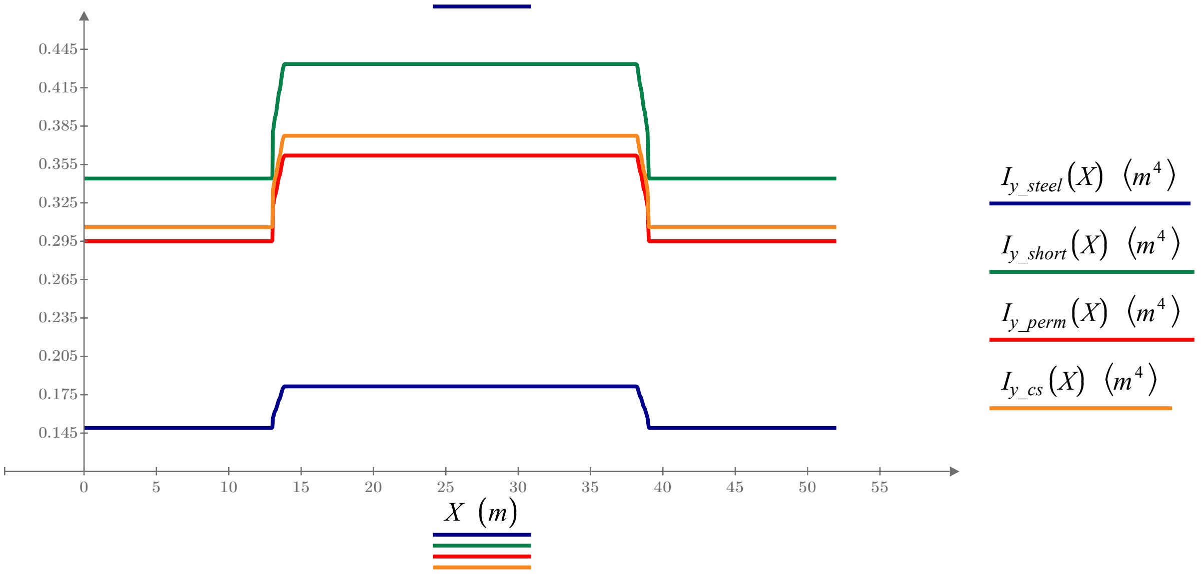


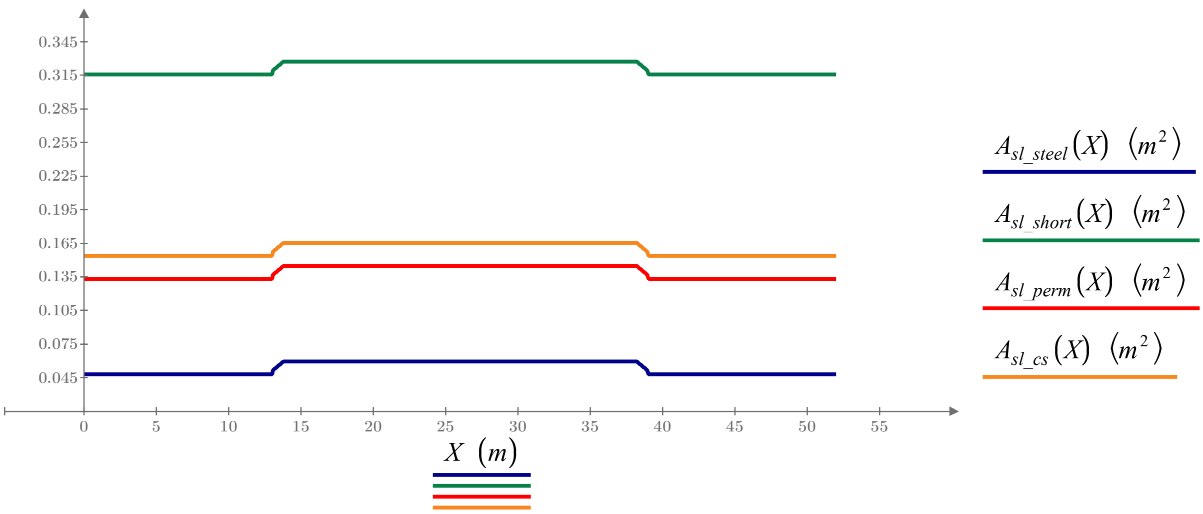
$V_{d\_ULS}(X_{check\_v}) = 4494\ kN$ Shear force at control point - ULS

6.2 Cross-sectional constants

For calculations, see chapter 5.







6.3 Stresses in steel cross-section

The stresses is calculated for each load case taking into acount load duration and creep. The stresses are then superpositioned.

6.3.1 Stresses during casting

$M := M_{ULS1}\left(X_{check\_m}\right) = 20834\ kN \cdot m$ Bending moment

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.181\ m^4$ Moment of inertia

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 2104\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3590\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot -z = -242\ MPa$ = $\sigma_{s.u.cast}\left(X_{check\_m}\right) = -242\ MPa$ Stresses in upper flange from loads durng casting

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 171\ MPa$ = $\sigma_{s.l.cast}\left(X_{check\_m}\right) = 171\ MPa$ Stresses in lower flange from loads durng casting

6.3.2 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses must be calculated for that specific section. The worst load case is determined dependent on the largest stress in each part whereas the multi-component load is the main load for the lower flange, and the temperature load is the worst load case for the upper flange.

6.3.2.1 Multi component loads (traffic)

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0.01\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.327\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.433\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) = 252\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3590\ mm$ Height of girder

$\sigma_{s.u} := \frac{0.9\ N}{A} + \frac{0.9\ M}{I} \cdot (-z) = -11\ MPa$ = $\sigma_{s.u.tr}\left(X_{check\_m}\right) = -11\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{1.5\ N}{A} + \frac{1.5\ M}{I} \cdot (h - z) = 239\ MPa$ = $\sigma_{s.l.tr}\left(X_{check\_m}\right) = 239\ MPa$ Stresses in lower flange from short term loads

6.3.2.2 Temperature loads

$M := M_{temp}\left(X_{check\_m}\right) = 11937\ kN \cdot m$ Bending moment

$N := N_{temp}\left(X_{check\_m}\right) = -5645\ kN$ Normal force

$A_s := A_{sl\_steel}\left(X_{check\_m}\right) = 0.06\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.327\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}\left(X_{check\_m}\right) = 0.181\ m^4$ Moment of inertia - steel section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.433\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}\left(X_{check\_m}\right) = 2104\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 252\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}\left(X_{check\_m}\right) = 3590\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 4997\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(1 - \left(\frac{A_s}{A} + \frac{A_s}{I}\left(z_s - z\right)^2\right)\right) = -1958\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{1.5\ N_s}{A_s} + \frac{1.5\ M_s}{I_s} \cdot \left(-z_s\right) = -136\ MPa$ = $\sigma_{s.u.temp}\left(X_{check\_m}\right) = -136\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{0.9\ N_s}{A_s} + \frac{0.9\ M_s}{I_s} \cdot \left(h - z_s\right) = 7\ MPa$ = $\sigma_{s.l.temp}\left(X_{check\_m}\right) = 7\ MPa$ Stresses in lower flange from short term loads

6.3.3 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4193\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.362\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) = 771\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3590\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot (-z) = -9\ MPa$ = $\sigma_{s.u.perm}(X_{check\_m}) = -9\ MPa$ Stresses in upper flange from additional permanent loads

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 33\ MPa$ = $\sigma_{s.l.perm}(X_{check\_m}) = 33\ MPa$ Stresses in lower flange from additional permanent loads

6.3.4 Stresses due to shrinkage

In the same manner as the temperature load must be calculated for the steel section, the shrinkage which acts on the concrete section must be converted for the steel section.

$M := M_{ULS4}(X_{check\_m}) = 5379\ kN \cdot m$ Bending moment

$N := N_{ULS4}(X_{check\_m}) = -5206\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.06\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_cs}(X_{check\_m}) = 0.165\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.181\ m^4$ Moment of inertia - steel section

$I := I_{y\_cs}(X_{check\_m}) = 0.377\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 2104\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_cs}(X_{check\_m}) = 655\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 3590\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 2585\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(\frac{A_s}{A} - \frac{A_s}{I} \cdot \left(z - \frac{h_{m.slab}}{2}\right) \cdot \left(z_s + \frac{h_{m.slab}}{2} - \left(z - \frac{h_{m.slab}}{2}\right)\right)\right) = -1156\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (-z_s) = -49\ MPa$ = $\sigma_{s.u.shrink}(X_{check\_m}) = -49\ MPa$ Stresses in upper flange due to shrinkage

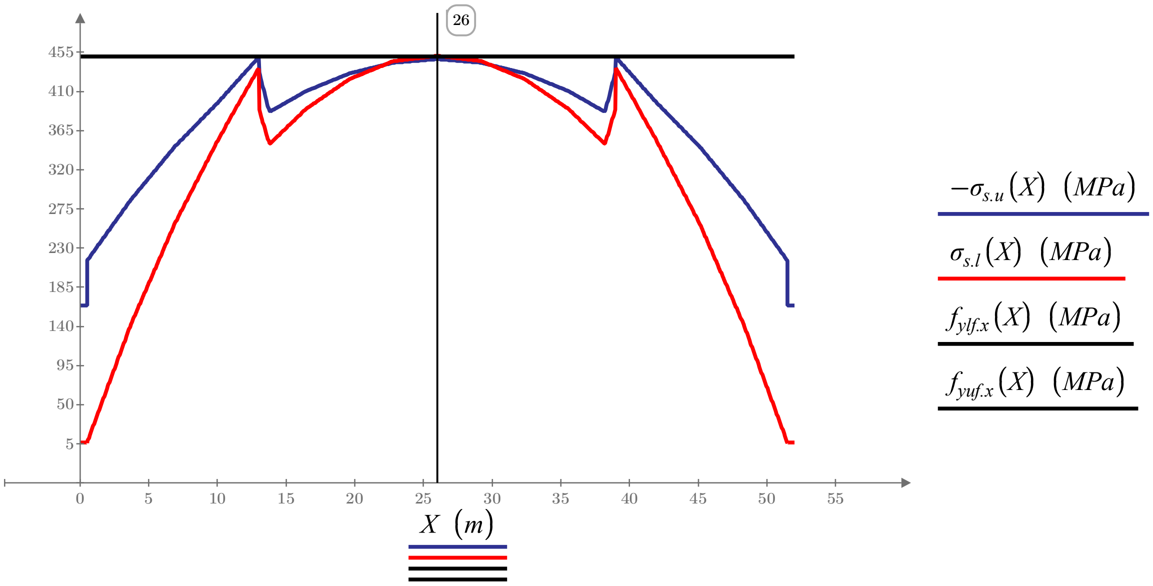
$\sigma_{s.l} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (h - z_s) = 2\ MPa$ = $\sigma_{s.l.shrink}(X_{check\_m}) = 2\ MPa$ Stresses in lower flange due to shrinkage

6.3.5 Summary of stresses

The stresses from the different phases are summarised accordingly.

$\sigma_{s.u}(x) := \sigma_{s.u.cast}(x) + \sigma_{s.u.shrink}(x) + \sigma_{s.u.perm}(x) + \sigma_{s.u.tr}(x) + \sigma_{s.u.temp}(x)$ Stresses in upper flange

$\sigma_{s.l}(x) := \sigma_{s.l.cast}(x) + \sigma_{s.l.shrink}(x) + \sigma_{s.l.perm}(x) + \sigma_{s.l.tr}(x) + \sigma_{s.l.temp}(x)$ Stresses in lower flange



$\eta_{\sigma.u.max} := max\left(\frac{-\sigma_{s.u}(X)}{f_{yuf.x}(X)}\right) = 100\%$ Maximum utilization rate - upper flange

$\eta_{\sigma.l.max} := max\left(\frac{\sigma_{s.l}(X)}{f_{ylf.x}(X)}\right) = 100\%$ Maximum utilization rate - lower flange

6.4 Calculation of stresses in concrete

The stresses in the concrete is calculated with the same principle as the steel taking creep into account.

6.4.1 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses for the concrete must be calculated separately.

6.4.1.1 Traffic and wind loads

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.327\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.433\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) + h_{m.slab} = 571\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3590\ mm$ Height of girder

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$\sigma_c := \left(\frac{N}{A} + \frac{-M \cdot z}{I}\right) \cdot \frac{1}{n_\Gamma} = -5\ MPa$ = $\sigma_{c.short}\left(X_{check\_m}\right) = -5\ MPa$ Stresses in concrete from short-term loads

6.4.1.2 Temperature load

$M := M_{temp}\left(X_{check\_m}\right) = 11937\ kN \cdot m$

$N := N_{temp}\left(X_{check\_m}\right) = -5645\ kN$

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.327\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \frac{1}{n_\Gamma} = 0.268\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.433\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \frac{h_{m.slab}{}^3}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}\left(X_{check\_m}\right)}{n_\Gamma} \cdot \frac{h_{m.slab}{}^3}{12} = 0.002\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 252\ mm$ Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \frac{I_{c.eff}}{I} \cdot M = 63\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(\frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot \left(z_s - z\right) \cdot \left(z + \frac{h_{m.slab}}{2}\right)\right) = -1958\ kN$ Normal force imposed on concrete slab

$\sigma_c := \left(\frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c}\right) = 0.49\ MPa$ = $\sigma_{c.temp}\left(X_{check\_m}\right) = 0.49\ MPa$ Stresses in concrete from temperature loads

6.4.2 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4193\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.362\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) + h_{m.slab} = 1090\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 3590\ mm$ Height of girder

$n_\Gamma := n_{L_1} = 18.48$ Creep

$\sigma_c := \frac{-M \cdot z}{I} \cdot \frac{1}{n_\Gamma} = -0.7\ MPa$ = $\sigma_{c.perm}\left(X_{check\_m}\right) = -0.7\ MPa$ Stresses in concrete from permanent loads

6.4.3 Stresses due to shrinkage

$M := M_{ULS4}\left(X_{check\_m}\right) = 5379\ kN \cdot m$ Bending moment

$N := N_{ULS4}\left(X_{check\_m}\right) = -5206\ kN$ Normal force

$n_{\Gamma} := n_{L_3} = 14.91$ Modular ratio considering creep

$A := A_{sl\_cs}\left(X_{check\_m}\right) = 0.165\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_{\Gamma}} = 0.106\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_cs}\left(X_{check\_m}\right) = 0.377\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \dfrac{{h_{m.slab}}^3}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \dfrac{b_{eff}\left(X_{check\_m}\right)}{n_{\Gamma}} \cdot \dfrac{{h_{m.slab}}^3}{12} = 0.001\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_cs}\left(X_{check\_m}\right) = 655\ mm$ Center of gravity - composite section

$z_c := \dfrac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

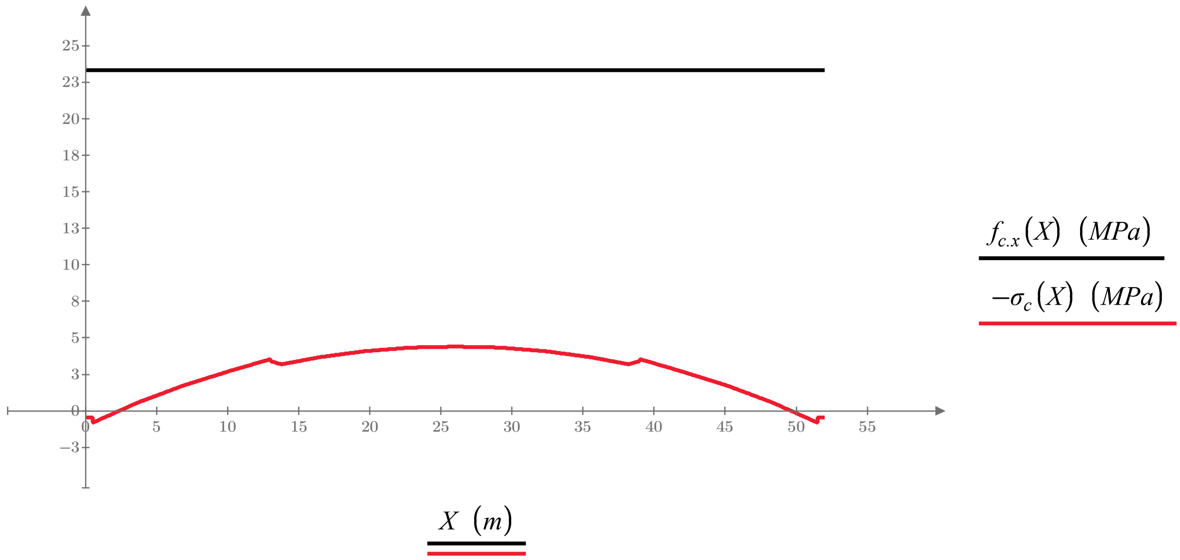
$M_c := \dfrac{I_{c.eff}}{I} \cdot M = 13\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(1 - \dfrac{A_{c.eff}}{A} - \dfrac{A_{c.eff}}{I} \cdot \left(z + \dfrac{h_{m.slab}}{2}\right)^2\right) = -905\ kN$ Normal force imposed on concrete slab

$\sigma_c := \dfrac{-N_c}{A_c} + \dfrac{M_c \cdot -z_c}{I_c} = 0.4\ MPa$ = $\sigma_{c.shrink}\left(X_{check\_m}\right) = 0.4\ MPa$ Stresses in concrete from shrinkage

6.4.4 Summary of stresses

$\sigma_c(x) := \sigma_{c.short}(x) + \sigma_{c.temp}(x) + \sigma_{c.perm}(x) + \sigma_{c.shrink}(x)$ Stresses in concrete



$\eta_{\sigma.c.max} := max\left(\dfrac{-\sigma_c(X)}{f_{c.x}(X)}\right) = 19\%$ Maximum utilization rate - upper flange

6.5 Shear capacity

The shear capacity of the corrugated steel girder is calculated accoridng to SS-EN 1993-1-5, Appendix D.
The local- and global buckling factor are calculated according to SS-EN 1993-1-5, Appendix D.

Local buckling factor

$a_{c1}=185\ mm$ $\qquad a_{c2}=187\ mm$ — Corrugation geometries

$a_{cmax}:=max\left(a_{c1},a_{c2}\right)=187\ mm$ — SS-EN 1993-1-5 D.2.2.(2)

$tw:=t_w\left(X_{check\_v}\right)=6\ mm$ — Web thickness

$\tau_{cr}:=4.83\cdot E_s\cdot\left(\dfrac{tw}{a_{cmax}}\right)^2=993\ MPa$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.7

$\lambda_c:=\sqrt{\dfrac{f_{yw}}{\tau_{cr}\cdot\sqrt{3}}}=0.555$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.6

$\chi_l:=min\left(\dfrac{1.15}{0.9+\lambda_c},1\right)=0.79$ = $\chi_{c.l}\left(X_{check\_v}\right)=0.79$ — Reduction factor local buckling - SS-EN 1993-1-5 D.2.2.(2) Equation D.5

Global buckling factor

$D_X:=\dfrac{E_s\cdot tw^3}{12\cdot\left(1-\nu^2\right)}\cdot\dfrac{w_c}{s_c}=4\ kN\cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$D_Z:=\dfrac{E_s\cdot tw\cdot a_{c3}^{\ 2}}{12}\cdot\dfrac{\left(3\ a_{c1}+a_{c2}\right)}{w_c}=2669\ kN\cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$hw:=h_w\left(X_{check\_v}\right)=3510\ mm$ — Web height

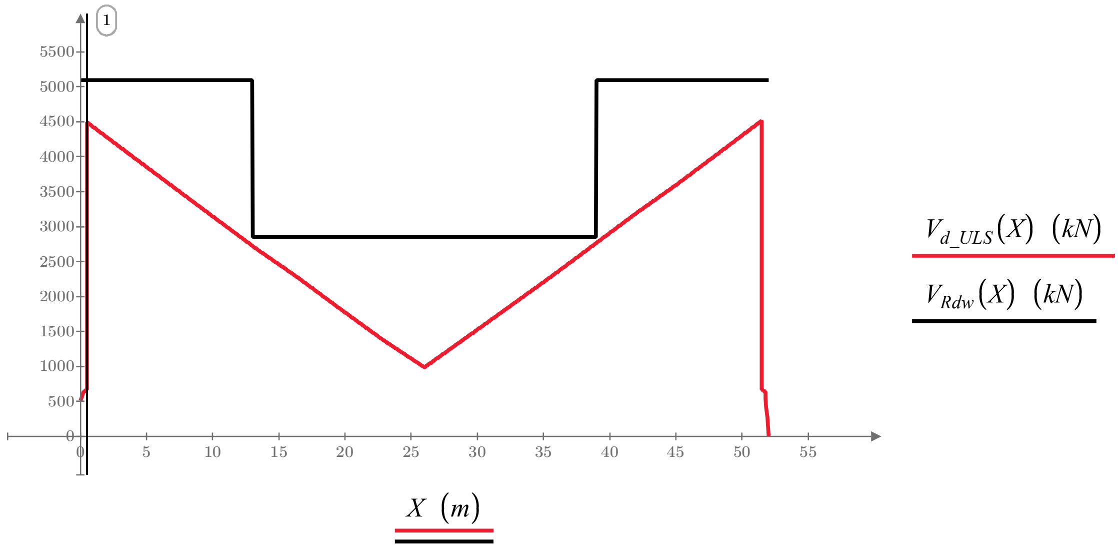
$\tau:=\dfrac{32.4}{tw\cdot hw^2}\cdot\sqrt[4]{\dfrac{D_X}{N\cdot m}\cdot\dfrac{D_Z^{\ 3}}{\left(N\cdot m\right)^3}}\ N\cdot m=224\ MPa$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.10

$\lambda:=\sqrt{\dfrac{f_{yw}}{\tau\cdot\sqrt{3}}}=1.17$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.9

$\chi_g:=min\left(\dfrac{1.5}{0.5+\lambda^2},1\right)=0.80$ = $\chi_{c.g}\left(X_{check\_v}\right)=0.80$ — Global buckling factor SS-EN 1993-1-5 D.2.2.(3) Equation D.8

$\chi_C:=min\left(\chi_g,\chi_l\right)=0.79$ = $\chi_c\left(X_{check\_v}\right)=0.79$ — SS-EN 1993-1-5 D.2.2

$V_{Rd}:=\chi_C\cdot\dfrac{f_{yw}}{\gamma_{M1}\cdot\sqrt{3}}\cdot hw\cdot tw=5093\ kN$ = $V_{Rdw}\left(X_{check\_v}\right)=5093\ kN$ — SS-EN 1993-1-5 D.2.2.(1) Eq.D.4



$\eta_{V.max} = 97\%$ Maximum utilization rate

6.7 Studs

6.7.1 Ultimate limit state

The capacity of the studs in the ultimate limit state are calculated according to SS-EN 1994-2 6.6.3

$d_{stud} \equiv 22\ mm$ — Diameter of stud

$h_{stud} \equiv 200\ mm$ — Length of stud

$f_{ub} = 800\ MPa$ — Characteristic strength of studs

$f_{u\_stud} := min\left(f_{ub}, 500\ MPa\right) = 500\ MPa$ — Ultimate strength shear stud - SS-EN 1994-2 6.6.3.1 (1)

$\frac{h_{stud}}{d_{stud}} = 9$ — Ratio height- diameter

$$\alpha_{stud} := \left\| \begin{array}{l} \text{if } 3 \le \frac{h_{stud}}{d_{stud}} \le 4 \\ \quad \left\| a \leftarrow 0.2 \cdot \left(1 + \frac{h_{stud}}{d_{stud}}\right) \right. \\ \text{else if } 4 < \frac{h_{stud}}{d_{stud}} \\ \quad \left\| a \leftarrow 1 \right. \\ a \end{array} \right| = 1.0$$

Correction factor for length to diameter ratio shear stud SS-EN 1994-2 6.6.3.1 (1)

$$P_{rd} := min\left(\left\| \begin{array}{c} \frac{0.8 \cdot f_{u\_stud} \cdot \pi \cdot d_{stud}^{2}}{4 \cdot \gamma_V} \\ \frac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^{2} \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \end{array} \right| \right) = 122\ kN$$

Capacity of one shear stud SS-EN 1994-2 6.6.3.1 Eq: 6.18,6.19

$S_{uf\_short}\left(X_{check\_v}\right) = \left(88 \cdot 10^{-3}\right)\ m^{3}$ — Second moment of area

$V_{ULS2}\left(X_{check\_v}\right) = 2538\ kN$ — Shear force

$I_{y\_short}\left(X_{check\_v}\right) = \left(344 \cdot 10^{-3}\right)\ m^{4}$ — Moment of inertia

$$\tau_{sh} := \frac{S_{uf\_short}\left(X_{check\_v}\right) \cdot V_{ULS2}\left(X_{check\_v}\right)}{I_{y\_short}\left(X_{check\_v}\right)} = 654\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for short term loads

$S_{uf\_perm}\left(X_{check\_v}\right) = \left(67 \cdot 10^{-3}\right)\ m^{3}$ — Second moment of area

$V_{ULS3}\left(X_{check\_v}\right)=329\ kN$ — Shear force

$I_{y\_perm}\left(X_{check\_v}\right)=\left(295\cdot 10^{-3}\right)\ m^4$ — Moment of inertia

$$\tau_{pe}:=\frac{S_{uf\_perm}\left(X_{check\_v}\right)\cdot V_{ULS3}\left(X_{check\_v}\right)}{I_{y\_perm}\left(X_{check\_v}\right)}=74\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for permanent loads

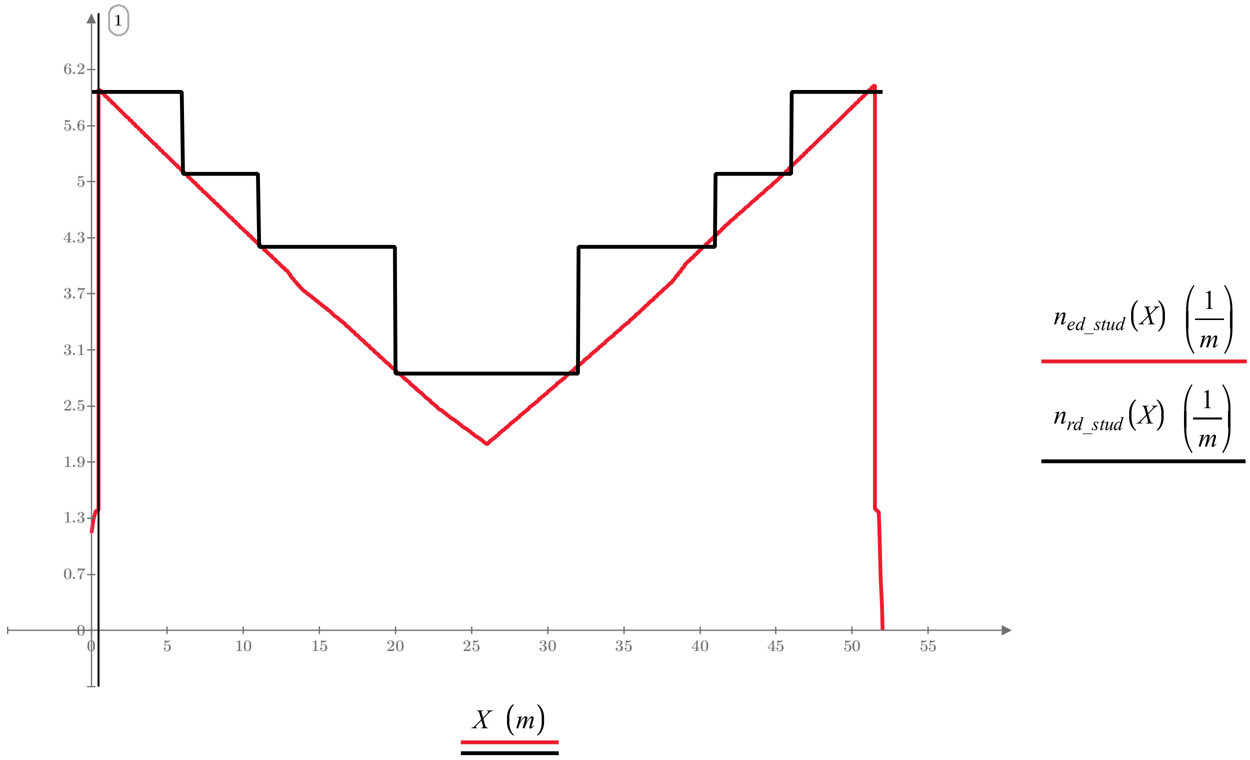
$$\tau:=\left|\tau_{sh}+\tau_{pe}\right|=728\ \frac{kN}{m}$$

Total shear force per meter between concrete and top flange

$n_{ed} := \dfrac{\tau + 0.9\ max\left(q_{1k\_br}, q_{tr\_br}\right)}{P_{rd}} = 6.0\ \dfrac{1}{m}$ = $n_{ed\_stud}\left(X_{check\_v}\right) = 6.0\ \dfrac{1}{m}$ Number of studs needed per meter

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



6.7.1.1 Additional studs for full anchorage

Case 1: Temperature and shrinkage causes the slab to contract and therefore they are working in the opposite direction as the shear flow from ULS- loads during bending.

The anchorage length is calculated according to SS-EN 1994-2, 6.9 (3).

$B_{out} = 2.525\ m$ Distance from centre web to outer part of edge beam

$B_{in} = 2.8\ m$ Distance from web to centerline bridge

$b := max\left(B_{out}, B_{in}\right) = 2.8\ m$

$l_{anch} := 1.5 \cdot b = 4.2\ m$ Anchorage length - SS-EN 1994-2, 6.9 (3)

$N_{cs\_stud} = 795\ kN$ Shrinkage force imposed on studs - calculated in chapter 4

$\left|N_{temp\_stud_1}\right| = 1805\ kN$ Temperature force imposed on studs causing contraction - calculated in chapter 4

$$n_{ed\_cs} := \frac{1.0\ N_{cs\_stud}}{l_{anch} \cdot P_{rd}} = 1.5\ \frac{1}{m}$$

$$n_{ed\_temp} := \frac{1.5\ \left|N_{temp\_stud_1}\right|}{l_{anch} \cdot P_{rd}} = 5.3\ \frac{1}{m}$$

$$n_{ed\_anch.1} := n_{ed\_cs} + n_{ed\_temp} = 6.8\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 5.9\ \frac{1}{m}$$

$$check_1 := \mathbf{if}\left(n_{ed\_anch.1} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_1 = \text{“Extra studs are needed”}$$

Case 2: Temperature causes the slab to expand and therefore working in the same direction as the shear flow from ULS- loads during bending. Shrinkage causes the slab to contract, i.e. working in the opposite direction.

$$-n_{ed\_cs} = -1.5\ \frac{1}{m}$$

$$N_{temp\_stud_2} = 1461\ kN$$

Temperature force imposed on studs causing expansion - calculated in chapter 4

$$n_{ed\_temp} := \frac{1.5 \cdot 0.6\ N_{temp\_stud_2}}{l_{anch} \cdot P_{rd}} = 2.6\ \frac{1}{m}$$

$$n_{ed\_stud}(0\ m) = 1.1\ \frac{1}{m}$$

$$n_{ed\_anch.2} := n_{ed\_stud}(0\ m) + n_{ed\_temp} - n_{ed\_cs} = 2.1\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 5.9\ \frac{1}{m}$$

$$check_2 := \mathbf{if}\left(n_{ed\_anch.2} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_2 = \text{“No extra studs are needed”}$$

Extra studs needed due to shrinkage and temperature

$n_{rd\_stud.adj}(X)$ Needed amount of studs with regards to extra anchorage due to temperature and shrinkage

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section (ULS- loads)

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section

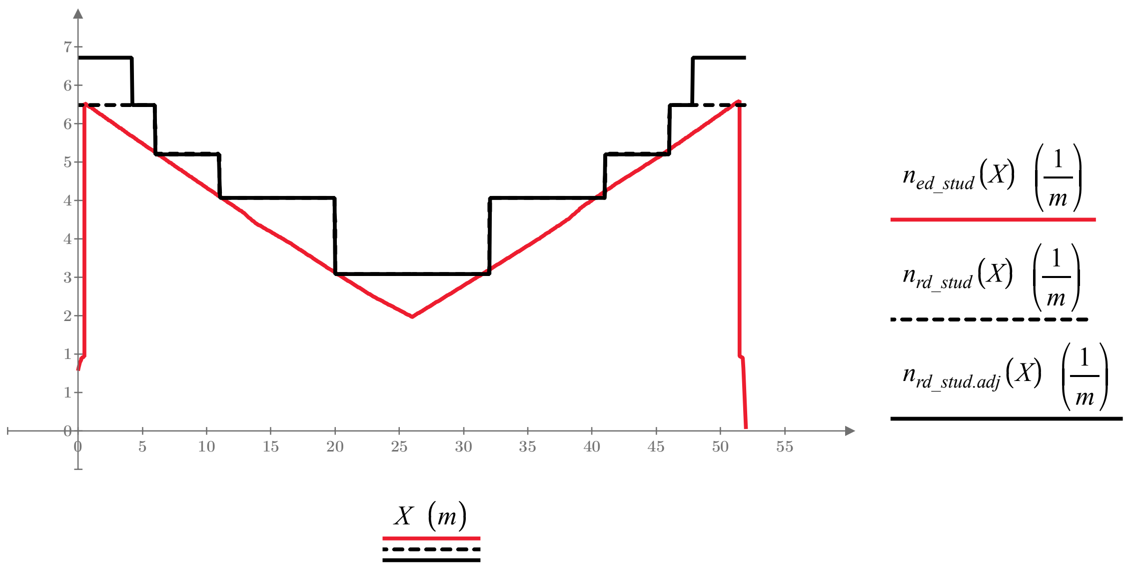


Table. Showing the adjusted need for studs near supports

X	n_{rd}	$n_{rd.adj}$
0	5.9	**6.8**
0.5	5.9	**6.8**
4.2	5.9	**6.0**
6	5.3	**5.3**
11	4.5	**4.5**
20	3.2	**3.2**

6.7.2 Fatigue limit state

6.7.2.1 Capacity

The fatigue capacity is calculated according to SS-EN 1993-1-9, Table 8.5 and SS-EN 1994-2, 6.8.3.

$\Delta\tau_c := 90\ MPa$ SS-EN 1993-1-9 - Table 8.5 (10)

$\Delta\tau_{E2} = \lambda_v \cdot \Delta\tau_c$

$\lambda_{v.1} := 1.55$ Bridge length less than 100m - SS-EN 1994-2 6.8.6.2 (4)

$Q_{mi} := 410\ kN$ Mean weight of large vehicles in the slow lane

$Q_0 := 480\ kN$

$N_{obs} := 0.05 \cdot 10^6$

$N_0 := 0.5 \cdot 10^6$

$\lambda_{v.2} := \frac{Q_{mi}}{Q_0} \cdot \left(\frac{N_{obs}}{N_0}\right)^{\frac{1}{8}} = 0.64$ SS-EN 1994-2 6.8.6.2 (4) and SS-EN 1993-2 Eq. 9.10

$t_{Ld} := 120$ Expected service life [years]

$\lambda_{v.3} := \left(\frac{120}{100}\right)^{\frac{1}{8}} = 1.02$

$\lambda_{v.4} := 1.0$ TSFS 2018:57 - 27 ch. 3 §

$\lambda_v := \lambda_{v.1} \cdot \lambda_{v.2} \cdot \lambda_{v.3} \cdot \lambda_{v.4} = 1.02$

$\gamma_{Ff} := 1.0$ SS-EN 1993-2, 9.3 (1)

$\gamma_{mF} := 1.0$ SS-EN 1994-2, 2.4.1.2 (6)

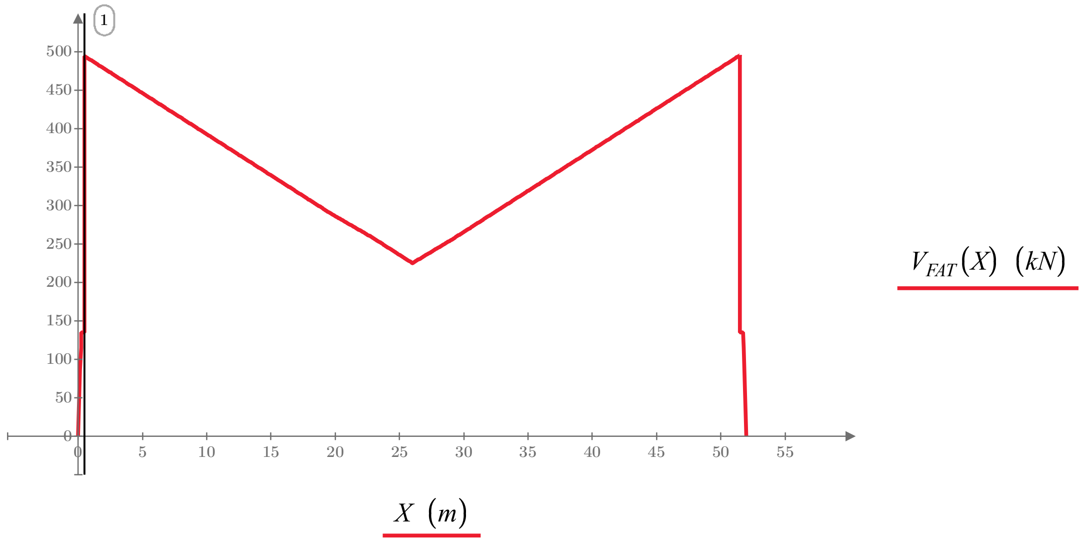
$\gamma_{Ff} \cdot \Delta\sigma_{E2} < \frac{\Delta\tau_c}{\gamma_{mF}}$

$F_{rd\_stud} := \Delta\tau_c \cdot \frac{\pi \cdot {d_{stud}}^2}{4} = 34.2\ kN$ Shear fatigue capacity - one stud

$F_{rd\_stud} := \frac{F_{rd\_stud}}{\lambda_v} = 33.7\ kN$ Considering trafic load

6.7.2.2 Fatigue load

$V_{FAT}(X)$ Fatigue load from trafic - see Appendix X



$$\tau \cdot b = \frac{SV}{I} = \frac{stud\_capacity}{m}$$

$$S_{uf} := S_{uf\_short}(X_{check\_v}) = 0.088\ m^3$$

$$V := V_{FAT}(X_{check\_v}) = 494\ kN$$

$$I := I_{y\_short}(X_{check\_v}) = 0.344\ m^4$$

$$\Delta\tau := \frac{S_{uf} \cdot V}{I} = 127\ \frac{kN}{m}$$

$F_{rd\_stud} = 34\ kN$ Fatigue capacity - one stud

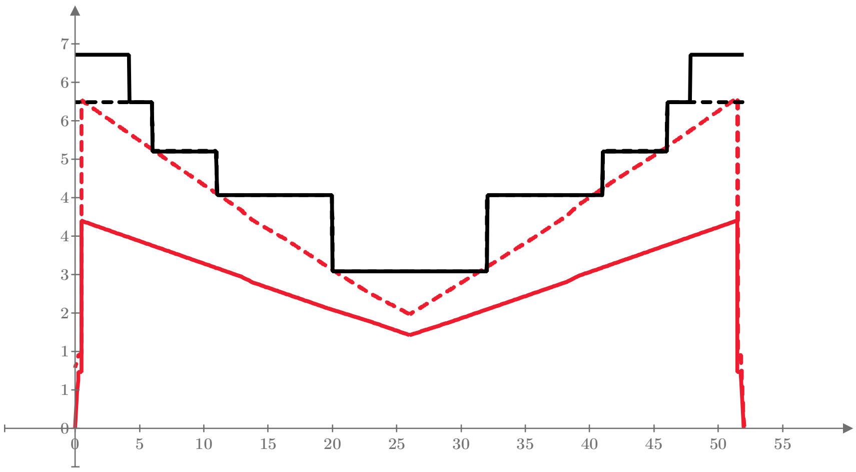
$$n_{ed\_\Delta\tau} := \frac{\Delta\tau}{F_{rd\_stud}} = 3.8\ \frac{1}{m} \quad = \quad n_{ed\_stud.\Delta\tau}(X_{check\_v}) = 3.8\ \frac{1}{m}$$

6.7.2.3 Design of studs with regards to fatigue

$n_{ed\_stud.\Delta\tau}(X)$ Needed amount of studs with regards to fatigue

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



Extra need of studs with regards to fatigue

$$Check := \mathbf{if}\left(max\left(\frac{n_{ed\_stud.\Delta\tau}(X)}{n_{rd\_stud}(X)}\right), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$Check = $ “No extra studs are needed”

6.7.3 Summary - design of studs

Table. Showing the stud design for half of the span

X	$n_{stud.used}$	$n_{stud.need}$	$n_{stud.fat}$
0	**6.8**	1.1	0.0
0.5	**6.8**	6.0	3.8
4.2	**6.0**	5.4	3.5
6	**5.3**	5.1	3.3
11	**4.5**	4.3	2.9
20	**3.2**	2.9	2.2

6.8 Utilization rates

$\eta_{\sigma.u.max} = 100\%$	Stresses in top flange
$\eta_{\sigma.l.max} = 100\%$	Stresses in lower flange
$\eta_{\sigma.c.max} = 19\%$	Stresses in concrete
$\eta_{V.max} = 97\%$	Shear capacity
$\eta_{breathing} = 27\%$	Breathing

9 Material savings

The material savings that are achieved thanks to redesigning the bridge with corrugated webs and using stainless steel is compared to the old design. However it is important to note that dependent on the utilization rates the savings may always not be comparable. For this bridge the highest utilization ratios are larger than 95% (close to 99,8%) and therefore close to comparable.

9.1 Main girder

The main girder is redesigned with a corrugated web and a slightly slimmer design. Over the full bridge length this decreases the weight of the bridge. All savings are presented for the full bridge (full width). Consideration has been taken to the extra web length arising from the corrugation.

$V_{old} = 13.25\ m^3$ Steel volume - girder with flat web; original design

$V_{new} = 7.59\ m^3$ Steel volume - girder with corrugated web; new design

$m_{old\_girder} := 7850\ \frac{kg}{m^3} \cdot V_{old} = 104\ 10^3 \cdot kg$ Weight of girder - flat web; original design

$m_{new\_girder} := 7551\ \frac{kg}{m^3} \cdot V_{new} = 57\ 10^3 \cdot kg$ Weight of girder - corrugated web; new design

$\eta_{girder} := 1 - \frac{m_{new\_girder}}{m_{old\_girder}} = 45\%$ Material saving [%] - steel girder

$\Delta m_{saving\_girder} := m_{old\_girder} - m_{new\_girder} = 47\ 10^3\ kg$ Material saving [kg] - steel girder

9.2 Studs

$V_{stud\_old} = 72750\ cm^3$ Volume - studs; original design

$V_{stud\_new} = 36440\ cm^3$ Volume - studs; new design

$m_{old\_stud} := 7850\ \frac{kg}{m^3} \cdot V_{stud\_old} = 571\ kg$ Mass - studs; original design

$m_{new\_stud} := 7551\ \frac{kg}{m^3} \cdot V_{stud\_new} = 275\ kg$ Mass - studs; new design

$\eta_{stud} := 1 - \frac{m_{new\_stud}}{m_{old\_stud}} = 52\%$ Material saving [%] - studs

$\Delta m_{saving\_stud} := m_{old\_stud} - m_{new\_stud} = 296\ kg$ Material saving [kg] - studs

9.3 Cross-beams

$V_{cb} = 0.59\ m^3$ Volume cross-beams old and new design

$m_{old\_cb} := 7850\ \frac{kg}{m^3} \cdot V_{cb} = 4597\ kg$ Mass - cross-beams; original design

$m_{new\_cb} := 7551\ \frac{kg}{m^3} \cdot V_{cb} = 4422\ kg$ Mass - cross-beams; new design

$\eta_{cb} := 1 - \frac{m_{new\_cb}}{m_{old\_cb}} = 4\%$ Material saving [%] - studs

$\Delta m_{saving\_cb} := m_{old\_cb} - m_{new\_cb} = 175\ kg$ Material saving [kg] - studs

9.4 Welds

$V_{weld\_old} = 0.005\ m^3$ Volume - welds; original design

$V_{weld\_new} = 0.0038\ m^3$ Volume - welds; new design

$m_{old\_weld} := 7850\ \frac{kg}{m^3} \cdot V_{weld\_old} = 40\ kg$ Mass - welds; original design

$m_{new\_weld} := 7551\ \frac{kg}{m^3} \cdot V_{weld\_new} = 28\ kg$ Mass - welds; new design

$\eta_{weld} := 1 - \frac{m_{new\_weld}}{m_{old\_weld}} = 28\%$ Material saving [%] - welds

$\Delta m_{saving\_weld} := m_{old\_weld} - m_{new\_weld} = 11\ kg$ Material saving [kg] - welds

9.5 Total savings

$m_{old\_bridge} := m_{old\_girder} + m_{old\_stud} + m_{old\_cb} = 109\ 10^3\ kg$ Total mass of original design

$m_{new\_bridge} := m_{new\_girder} + m_{new\_stud} + m_{new\_cb} = 62\ 10^3\ kg$ Total mass of new design

$\eta_{bridge} := 1 - \frac{m_{new\_bridge}}{m_{old\_bridge}} = 43.2\%$ Material saving [%] - full bridge

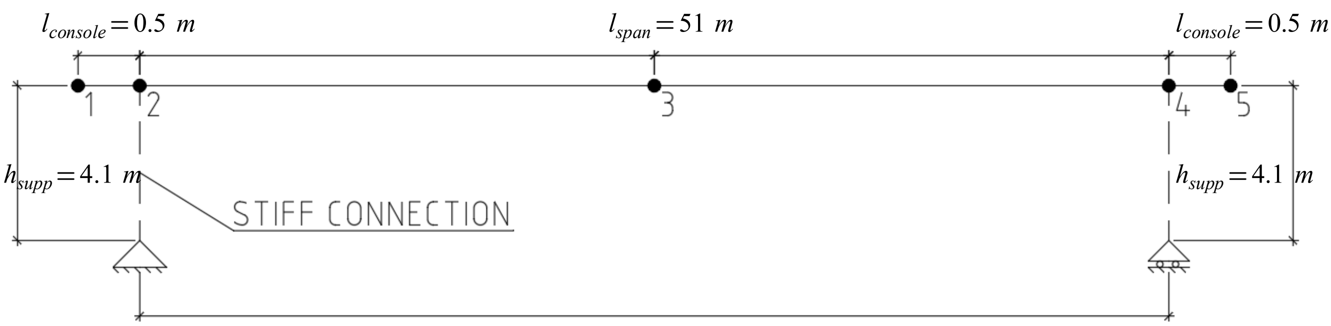
$\Delta m_{saving.tot} := \Delta m_{saving\_girder} + \Delta m_{saving\_stud} + \Delta m_{saving\_cb} + \Delta m_{saving\_weld} = 47\ 10^3\ kg$
Material saving [kg] - full bridge

Extracts - Calculation report 2.0h_w

3 System

In the following chapter the input data for the bridge's geometries is presented. The cross-sectional parameters that are presented is preliminary and used for the system analysis. In chapter 5 the final design is presented.

3.1 Primary system - longitudinal



The bridge is modelled as a simply supported bridge in the software StripStep-2. Due to the beams depth the supports are set offset from the neutral axis. This is modelled with a stiff connection in the system analysis.

3.1.1 Cross-section dimensions

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

The new height of the girder is calculated as: $Round(2050\ 2, 10) = 4100$

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	35	500	8	4020	45	600
500	35	500	8	4020	45	600

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	35	500	8	4020	45	600
10500	35	500	8	4020	45	600
10500	35	500	6	4015	50	600
11300	35	600	6	4015	50	725
25500	35	600	6	4015	50	725

Important! Two exactly the same values will not work with the linterp- function. Therefore 0.1 millimeter must be added for a X-value where you want to cross-sectional properties at the same time.

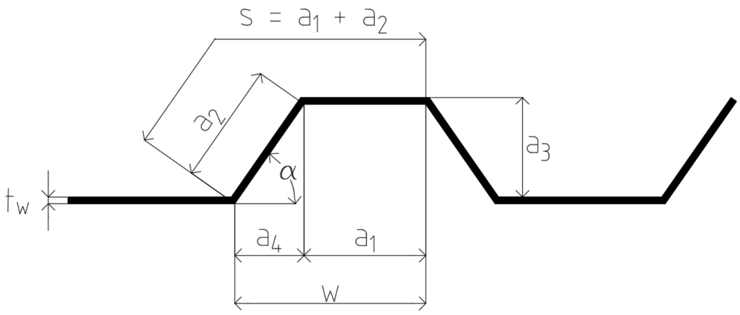
Element 3

S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	35	500	8	4020	45	600
500	35	500	8	4020	45	600

$$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=4100\ mm$$

$$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=4100\ mm$$

3.1.2 Corrugation shape



$a_{c1}:=185\ mm$ Flat-fold length

$\alpha_c:=36\ deg$ Corrugation angle

$a_{c3}:=110\ mm$ Corrugation depth

$a_{c2}:=\dfrac{a_{c3}}{\sin\left(\alpha_c\right)}=187\ mm$ Length of angled part

$a_{c4}:=\dfrac{a_{c3}}{\tan\left(\alpha_c\right)}=151\ mm$ Length of hypopythis

$s_c:=a_{c1}+a_{c2}=372\ mm$ Length of corrugation

$w_c:=a_{c1}+a_{c4}=336\ mm$ Straight length

$r_c:=\dfrac{s_c}{w_c}=1.11$ Ratio corrugation/flat-fold length

3.1.3.2 Effective width, steel flanges

Calculation of the effective width in the steel flanges with regards to shear lag is calculated according to SS-EN 1993-1-5 3.2.

$b_{eff\_f} = \beta \cdot b_0 + \frac{t_w}{2}$

Effective width with regards to shear lag under elastic conditions - Equation (3.1)

$X_{check\_uf} := \frac{L_{bridge}}{2} = 26.0\ m$

$X_{check\_lf} := 0.2\ L_{bridge} = 10.4\ m$

Upper flange

$\alpha_0 := 1.0$

Equation given in Table 3.1. For webs without any longitudinal stiffeners $\alpha_0 = 1.0$.

$bfu := b_{fu}\left(X_{check\_uf}\right) = 600\ mm$

Width of upper flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_uf}\right) = 6\ mm$

Thickness of web - see Appendix B - Preliminary sizing

$b0 := \frac{bfu - tw}{2} = 297\ mm$

Figure 3.2 (Notations for shear lag)

$\kappa fu := \frac{\alpha_0 \cdot b0}{L_{span}} = 0.006$ = $\kappa_{fu}\left(X_{check\_uf}\right) = 0.006$

Table 3.1

$$\beta_{check} := \begin{array}{|l} \text{if } \kappa fu \le 0.02 \\ \quad \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fu \le 0.70 \\ \quad \beta \leftarrow \frac{1}{1 + 6.4\ \kappa fu^2} \\ \text{else if } 0.70 < \kappa fu \\ \quad \beta \leftarrow \frac{1}{8.6\ \kappa fu^2} \end{array} = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for upper flange

$2\ \beta_{check} \cdot b0 + tw = 600\ mm$

$$b_{ef} := \begin{array}{|l} \text{if } \beta_{check} = 1.0 \\ \quad b \leftarrow bfu \\ \text{else} \\ \quad b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfu\right) \\ b \end{array} = 0.600\ m$$ = $b_{eff\_fu}\left(X_{check\_uf}\right) = 0.600\ m$

Lower flange

$bfl := b_{fl}\left(X_{check\_lf}\right) = 600\ mm$ Width of lower flange - see Appendix B - Preliminary sizing

$tw := t_w\left(X_{check\_lf}\right) = 8\ mm$ Thickness of web - see Appendix B - Preliminary sizing

$b0 := \dfrac{bfl - tw}{2} = 296\ mm$ Figure 3.2 (Notations for shear lag)

$\kappa fl := \dfrac{\alpha_0 \cdot b0}{L_{span}} = 0.006$ = $\kappa_{fl}\left(X_{check\_lf}\right) = 0.006$ Table 3.1

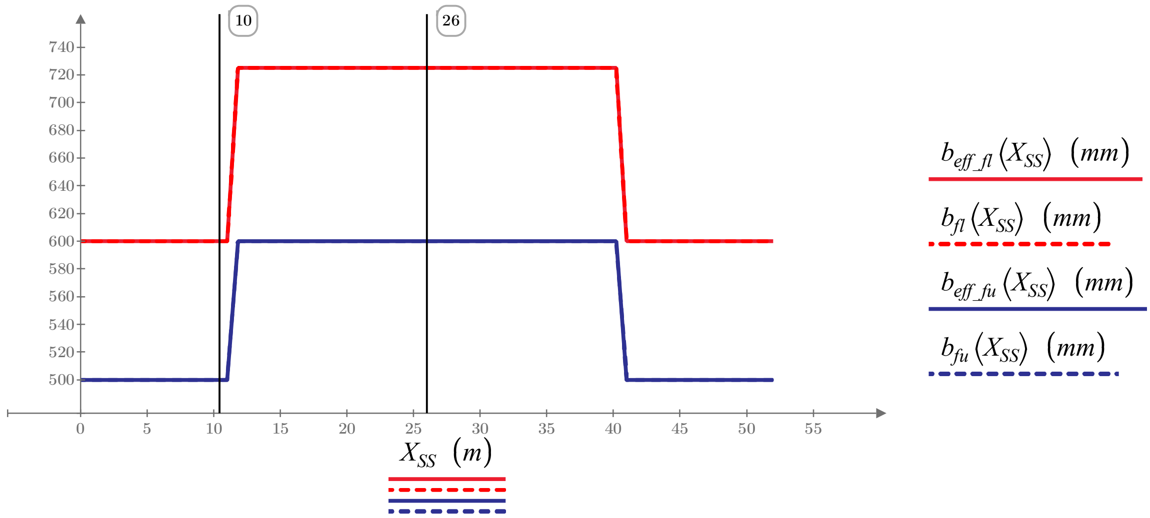
$$\beta_{check} := \begin{array}{|l} \text{if } \kappa fl \le 0.02 \\ \quad \beta \leftarrow 1.0 \\ \text{else if } 0.02 < \kappa fl \le 0.70 \\ \quad \beta \leftarrow \dfrac{1}{1 + 6.4\ \kappa fl^2} \\ \text{else if } 0.70 < \kappa fl \\ \quad \beta \leftarrow \dfrac{1}{8.6\ \kappa fl^2} \end{array} = 1.00$$

β for sagging bending, one-span bridge

Calculating the effective flange width for lower flange

$2\ \beta_{check} \cdot b0 + tw = 600\ mm$

$$b_{ef} := \begin{array}{|l} \text{if } \beta_{check} = 1.0 \\ \quad b \leftarrow bfl \\ \text{else} \\ \quad b \leftarrow min\left(2\ \beta_{check} \cdot b0 + tw, bfl\right) \\ b \end{array} = 0.600\ m$$ = $b_{eff\_fl}\left(X_{check\_lf}\right) = 0.600\ m$



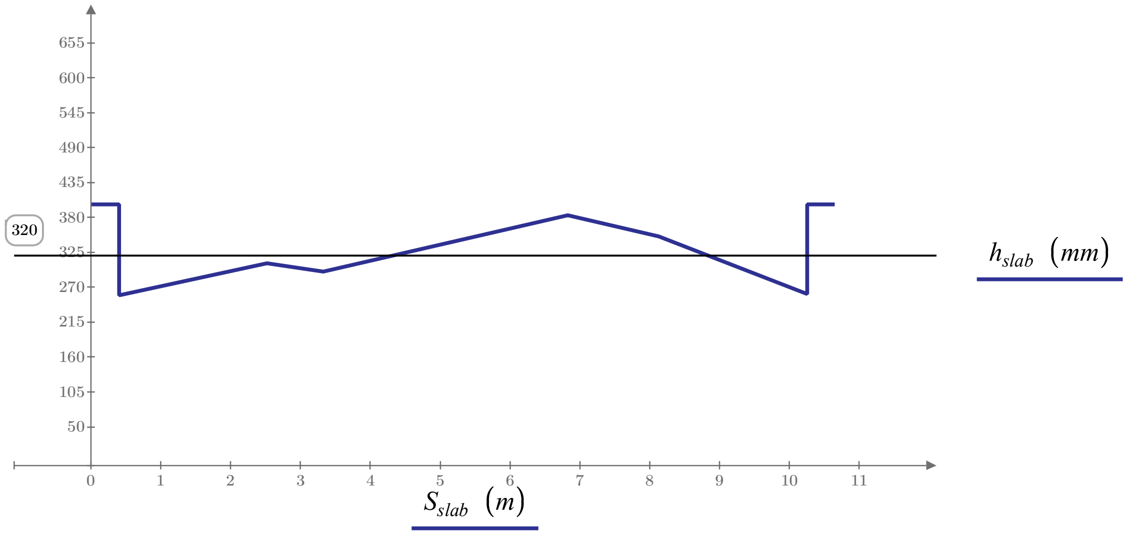
3.1.4 Cross-sectional constants for system analysis

For the system analysis an equivalent cross-section is calculated converting the concrete slab into steel.

3.1.4.1 Concrete slab

The thickness of the concrete slab varies over the width of the bridge. For calculation of the self-weight the different heights of the slab is integrated over the width, see the graph below.

h_{slab}	Height of slab over the transverse direction
S_{slab}	Transverse distance



$$A_{slab} = \sum_i \frac{\left(h_{slab_i} + h_{slab_{i+1}}\right)}{2} \cdot \left(S_{slab_{i+1}} - S_{slab_i}\right)$$ Principal for calculating the area of the slab

$A_{slab} = 3.472\ m^2$ Area of concrete slab

The stiffness of the concrete slab is calculated for the mean thickness, excluding the edge beam.

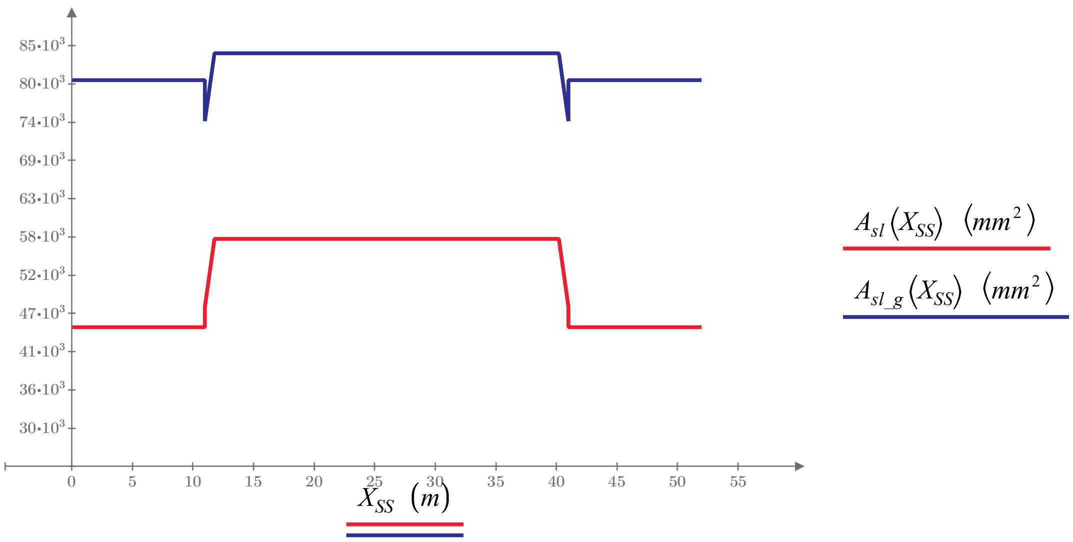
$h_{m.slab} = 320\ mm$ Mean slab height (includes height of heel)

$h_{klack} = 25\ mm$ Height of heel (for concrete slab)

3.1.4.2 Cross-sectional constants during construction

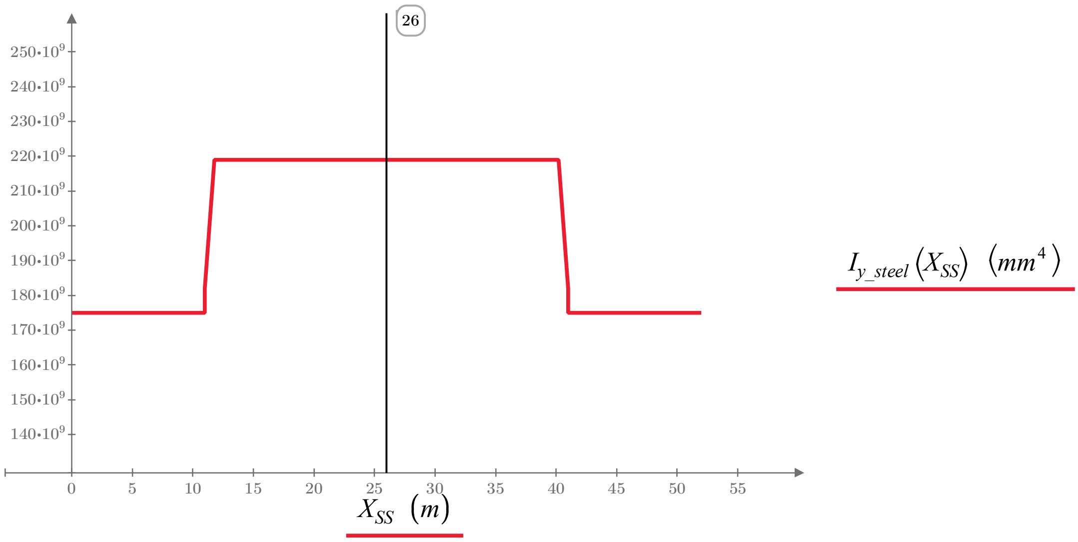
The bridge is checked so that the steel girder (alone) can withstand the loads that are imposed during construction such as the self-weight of curing concrete.

A_{sl_g} is the area of the steel including the web, for calculation of self-weight
A_{sl} is the area of the steel excluding the web, for stiffness calculations



$$A_{sl}\left(\frac{L_{bridge}}{2}\right) = 0.057\ m^2$$ Area at mid-span

I_{y_steel} is the area of the steel excluding the web, for stiffness calculations



$$I_{y\_steel}\left(\frac{L_{bridge}}{2}\right) = 0.2189\ m^4$$ Stiffness at mid-span

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]	g [kN/m]
0.000	-2.481	4.100	1750.288	445	6.047
0.500	-2.481	4.100	1750.288	445	6.047

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]	g [kN/m]
0.000	-2.481	4.100	1750.288	445	6.047
10.500	-2.481	4.100	1750.288	445	6.047
10.500	-2.580	4.100	1819.709	475	5.599
11.300	-2.587	4.100	2189.213	573	6.335
25.500	-2.587	4.100	2189.213	573	6.335
39.700	-2.587	4.100	2189.213	573	6.335
40.500	-2.580	4.100	1819.709	475	5.599
40.500	-2.481	4.100	1750.288	445	6.047
51.000	-2.481	4.100	1750.288	445	6.047

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]	g [kN/m]
0.000	-2.481	4.100	1750.288	445	6.047
0.500	-2.481	4.100	1750.288	445	6.047

System model used in Strip-Step2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-4.1	ZR	
					2
3	51.500	0.00	-4.1	YZR	
					3
4	52.000	0.00			-3

3.1.4.3 Cross-sectional constants for variable loads (short term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.216	4.420	4434.43	3124
0.500	-0.216	4.420	4434.43	3124

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.216	4.420	4434.43	3124
10.500	-0.216	4.420	4434.43	3124
10.500	-0.253	4.420	4871.73	3154
11.300	-0.324	4.420	5770.41	3251
25.500	-0.324	4.420	5770.41	3251
39.700	-0.324	4.420	5770.41	3251
40.500	-0.253	4.420	4871.73	3154
40.500	-0.216	4.420	4434.43	3124
51.000	-0.216	4.420	4434.43	3124

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y $[10^4 m^4]$	A $[10^4 m^2]$
0.000	-0.216	4.420	4434.43	3124
0.500	-0.216	4.420	4434.43	3124

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-4.1	ZR	
					2
3	51.500	0.00	-4.1	YZR	
					3
4	52.000	0.00			-3

3.1.4.4 Cross-sectional constants for additional permanent loads (long term)

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$
0	-0.746	4.420	3796.719	1298
0.5	-0.746	4.420	3796.719	1298

Element 2 - Node 2- 3

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$
0	-0.746	4.420	3796.719	1298
10.5	-0.746	4.420	3796.719	1298
10.5	-0.820	4.420	4117.360	1328
11.3	-0.943	4.420	4780.403	1425
25.5	-0.943	4.420	4780.403	1425
39.7	-0.943	4.420	4780.403	1425
40.5	-0.820	4.420	4117.360	1328
40.5	-0.746	4.420	3796.719	1298
51	-0.746	4.420	3796.719	1298

Element 3 - Node 3- 4

S	t_{offset}	h_{tot}	I_y	A
[m]	[m]	[m]	$[10^4 m^4]$	$[10^4 m^2]$
0	-0.746	4.420	3796.719	1298
0.5	-0.746	4.420	3796.719	1298

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-4.1	ZR	
					2
3	51.500	0.00	-4.1	YZR	
					3
4	52.000	0.00			-3

3.1.4.5 Cross-sectional constants for shrinkage analysis

Input for StripStep-2 (Appendix X)

Element 1 - Node 1- 2

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.622	4.420	3943.330	1502
0.5	-0.622	4.420	3943.330	1502

Element 2 - Node 2- 3

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.622	4.420	3943.330	1502
10.5	-0.622	4.420	3943.330	1502
10.5	-0.690	4.420	4289.387	1532
11.3	-0.805	4.420	4999.833	1630
25.5	-0.805	4.420	4999.833	1630
39.7	-0.805	4.420	4999.833	1630
40.5	-0.690	4.420	4289.387	1532
40.5	-0.622	4.420	3943.330	1502
51	-0.622	4.420	3943.330	1502

Element 3 - Node 3- 4

S [m]	t_{offset} [m]	h_{tot} [m]	I_y [$10^4 m^4$]	A [$10^4 m^2$]
0	-0.622	4.420	3943.330	1502
0.5	-0.622	4.420	3943.330	1502

System model used in StripStep-2

START NODE	Y [m]	Z [m]	DELTA Z [m]	REACTION	EL. NO
1	0.000	0.00			1
2	0.500	0.00	-4.1	ZR	
					2
3	51.500	0.00	-4.1	YZR	
					3
4	52.000	0.00			-3

4 Loads and load combinations

4.1 Permanent loads

4.1.1 Self-weight

4.1.1.1 Steel

The self-weight is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: STEEL

$\rho_{st} = 75.51 \ \frac{kN}{m^3}$ Self-weight of stainless steel - SS-EN 10088-1:2014 Table E.1 or E.2

The self-weight for each element is calculated in chapter 3.

4.1.1.2 Concrete

The self-weight of the slab is applied from node 1 to 4 in Strip-Step2, Appendix X. Name in Strip-Step2: SLAB

$Wet\_Concrete := \text{“NO”}$ "YES" or "NO" dependent if the previous designer has used the weight of wet concrete

$\rho_c := \mathbf{if}\left(Wet\_Concrete = \text{“NO”}, 25 \ \frac{kN}{m^3}, 26 \ \frac{kN}{m^3}\right) = 25 \ \frac{kN}{m^3}$ Self-weight of concrete (reinforced) - SS-EN 1992-1-1 Table A.1

$A_{slab} = 3.47 \ m^2$ Area of slab, see chapter 3

$g_{slab} := \frac{A_{slab} \cdot \rho_c}{2} = 43.4 \ \frac{kN}{m}$ Self-weight of concrete slab, (half of the load goes to each girder) - applied in the casting stage

If the previous designer has considered that the hardened concrete has a smaller self-weight a reduction in self-weight is applied in the system analysis for permanent loads.

Appendix X. Name in Strip-Step2: AVSLAB

$$g_{slab.perm} := \mathbf{if}\left(Wet\_Concrete = \text{“NO”}, 0 \ \frac{kN}{m}, \frac{A_{slab} \cdot (-1) \ \frac{kN}{m^3}}{2}\right) = 0.000 \ \frac{kN}{m}$$

4.1.2 Shrinkage

The shrinkage force is calculated and used in Strip-Step2, Appendix X. Name in Strip-Step2: E:SHRINK

$h_0 = 341\ mm$ Equivalent thickness, calculated in chapter 2

$t = 120\ yr$

$t = 43829.1\ day$

$f_{cm} = 43\ MPa$ Mean compressive strength - see chapter 2

$t_s := 1\ day$

$RH := 80\%$ Krav Brobyggande B.3.1.5

$$\beta_{ds} := \frac{\dfrac{t - t_s}{day}}{\left(\dfrac{t - t_s}{day}\right) + 0.04 \cdot \sqrt{\left(\dfrac{h_0}{1\ mm}\right)^3}} = 0.994$$ SS-EN 1992-1-1 3.1.4 Equation 3.10

$$k_h := \begin{array}{l} \text{if } 200\ mm \le h_0 < 300\ mm \\ \quad \left\| 0.85 - \left(\dfrac{h_0}{mm} - 200\right) \cdot 0.001 \right. \\ \text{else if } 300\ mm \le h_0 < 500\ mm \\ \quad \left\| 0.75 - \left(\dfrac{h_0}{mm} - 300\right) \cdot 0.0005 \right. \\ \text{else if } h_0 \ge 500\ mm \\ \quad \| 0.7 \\ \text{else} \\ \quad \| 1.0 \end{array} = 0.73$$ SS-EN 1992-1-1 3.1.4 table 3.3

$RH_0 := 100\%$

$\alpha_{ds1} := 4$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$\alpha_{ds2} := 0.12$ SS-EN 1992-1-1 Appendix B.2 (1), Cementclass N

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0}\right)^3\right) = 0.76$$ SS-EN 1992-1-1 Appendix B.2 equation B12

$f_{cmo} := 10\ MPa$ SS-EN 1992-1-1 Appendix B.2 (1)

$$\varepsilon_{cd.0} := 0.85 \cdot \left(\left(220 + 110 \cdot \alpha_{ds1}\right) \cdot e^{\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}}\right)}\right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$ SS-EN 1992-1-1 Appendix B.2 equation B11

$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd.0} = 1.84 \cdot 10^{-4}$ Drying shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.9

$\varepsilon_{ca0} := 2.5 \cdot \left(\dfrac{f_{ck} - f_{cmo}}{MPa}\right) \cdot 10^{-6} = 6.3 \cdot 10^{-5}$ SS-EN 1992-1-1 3.1.4 Equation 3.12

$\beta_{as} := 1 - e^{\left(-0.2 \cdot \sqrt{\frac{t - t_s}{day}}\right)} = 1.0$ SS-EN 1992-1-1 3.1.4 Equation 3.13

$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca0} = 6.25 \cdot 10^{-5}$ Autogenous shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.11

$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.46 \cdot 10^{-4}$ Total shrinkage - SS-EN 1992-1-1 3.1.4 Equation 3.8

4.1.2.1 Shrinkage force

The shrinkage force and corresponding moment is calculated accordingly:

$n_{L\_cs} = 14.91$ Modular ratio accounting for creep and shrinkage

$n_{L\_short} = 5.88$ Modular ratio

$E_{c.eff} := \dfrac{n_{L\_short}}{n_{L\_cs}} \cdot E_{cm} = 13.4\ GPa$ Effective modulus of elasticity for concrete

$A_{slab.fic} = 1.576\ m^2$ Area of concrete slab (half of the cross-section used in system analysis)

$F_{cs} := \varepsilon_{cs} \cdot E_{c.eff} \cdot A_{slab.fic} = 5206\ kN$ Total shrinkage force

$e_{cs} := z_{tp\_cs}(0\ m) = 0.622\ m$ The shrinkage force is applied in the center of gravity for the composite section

$M_{cs} := F_{cs} \cdot \left(z_{tp\_cs}(0\ m) + \dfrac{h_{m.slab}}{2}\right) = 4073\ kN \cdot m$ Total bending moment

Shrinkage - anchorage of studs

Forces in the concrete

$$F_{c\_cs} := F_{cs} \cdot \left(1 - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{A_{sl\_cs}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_cs}}}{I_{y\_cs}(0\ m)} \cdot \left(z_{tp\_cs}(0\ m) + \frac{h_{m.slab}}{2}\right)^2\right) = 687\ kN$$

Force in concrete

$$b_{eff}(0\ m) = 4.925\ m$$

Effective width of flange

$$M_{c\_cs} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_cs}} \cdot h_{m.slab}^{3}}{I_{y\_cs}(0\ m)} \cdot M_{cs} = 9\ kN \cdot m$$

Moment in concrete

$$\sigma_1 := \frac{F_{c\_cs}}{A_{slab.fic}} - \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{3}} = 0.32\ MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{F_{c\_cs}}{A_{slab.fic}} + \frac{M_{c\_cs} \cdot \frac{h_{m.slab}}{2}}{\frac{1}{12} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{3}} = 0.55\ MPa$$

Stress in concrete, lower edge of slab

$$N_{cs} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = 687\ kN \quad = \quad F_{c\_cs} = 687\ kN$$

Force imposed on studs caused by shrinkage (used in chapter 6)

4.2.2.1 Horizontal side trafic load

$P_{h\_side\_car} := max\left(\begin{bmatrix} 25\% \cdot Q_{1k\_br} \\ 25\% \cdot Q_{tr\_br} \end{bmatrix}\right) = 109.1\ kN$ Horisontal side force from the acceleration load (used in chapter 7)

$P_{v\_side\_car} := P_{h\_side\_car} \cdot \dfrac{h_{m.slab}}{2 \cdot B_{in}} = 6.2\ kN$ Vertical side force from the acceleration load (used in chapter 7)

4.2.3 Temperature load

Temperatures are determined according to SS-EN 1991-1-5, 6.1.3 unless otherwise stated.

Load case 1 - local temperature differences for Hudiksvall

$T_0 = 10\ °$ A.1(3)

$T_{min} = -38\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{max} = 34\ °$ TSFS 2018:57 - 8 ch 2 §

$T_{e.min} := T_{min} + 4\ ° = -34\ °$ Figure 6.1

$\Delta T_{N.con} := T_0 - T_{e.min} = 44\ °$ Contraction - Equation 6.1

$T_{e.max} := T_{max} + 4\ ° = 38\ °$ Figure 6.1

$\Delta T_{N.exp} := T_{e.max} - T_0 = 28\ °$ Expansion - Equation 6.1

$\Delta T := T_{e.max} - T_{e.min} = 72\ °$ Total temperature difference

Load case 2 - either of the components are larger than the other

$\Delta T_{c2st} := 15\ °$ Temperature difference between concrete and steel - SS-EN 1991-1-5, 6.1.6

4.2.3.1 Coefficients of thermal linear expansion

For composite bridges normally it is suggested to use the same thermal linear expansion coefficient, according to SS-EN 1991-1-5 Table C.1. However for stainless steels the thermal linear expansion coefficient is much larger than for concrete and hence a more through calculation is needed for the strain.

$\alpha_c := 10 \cdot 10^{-6}$ Thermal expansion coefficient - concrete - SS-EN 1991-1-5 Table C.1.

$\alpha_{ss} := 16 \cdot 10^{-6}$ Thermal expansion coefficient - stainless steel - SS-EN 1991-1-5 Table C.1.

$\alpha_{cs} := 12 \cdot 10^{-6}$ Thermal expansion coefficient - carbon steel - SS-EN 1991-1-5 Table C.1.

4.2.3.2 Strains for the different load cases

$\Delta\varepsilon_{LC.1\_con} := -(\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.con}}{\circ} = -26.4\ 10^{-5}$ Difference in strain between steel and concrete for contraction (temperature drop) - load case 1

$\Delta\varepsilon_{LC.1\_exp} := (\alpha_{ss} - \alpha_c) \cdot \frac{\Delta T_{N.exp}}{\circ} = 16.8\ 10^{-5}$ Difference in strain between steel and concrete for expansion (temperature raise) - load case 1

$\Delta\varepsilon_{LC.2\_st} := \alpha_{ss} \cdot \frac{\Delta T_{c2st}}{\circ} = 24.0\ 10^{-5}$ Difference in strain between steel and concrete for when the steel is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2\_c} := \alpha_c \cdot \frac{\Delta T_{c2st}}{\circ} = 15.0\ 10^{-5}$ Difference in strain between steel and concrete for when the concrete is 15 degrees warmer or colder than concrete - load case 2

$\Delta\varepsilon_{LC.2} := max\left(\Delta\varepsilon_{LC.2\_st}, \Delta\varepsilon_{LC.2\_c}\right) = 24.0\ 10^{-5}$ Only evaluating the worst case for load case 2, i.e. when there are a temperature drop or rise in the steel

$\varepsilon_{temp_1} := -\Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_con} = -50.4\ 10^{-5}$ Minimum strain difference; temperature drop and the steel drops even lower

$\varepsilon_{temp_2} := \Delta\varepsilon_{LC.2} + \Delta\varepsilon_{LC.1\_exp} = 40.8\ 10^{-5}$ Maximum strain difference; temperature raises and the steel heatens up even higher

4.2.3.3 Temperature load - global analysis

The concrete is transformed to steel. Already defined parameters are calculated in chapter 3.

$\frac{b_{eff}(0\ m)}{n_{L\_short}} = 0.837\ m$ Width of transformed concrete

$\frac{A_{slab.fic}}{n_{L\_short}} = 0.268\ m^2$ Area of transformed concrete

$A_{sl}(0\ m) = 0.045\ m^2$ Area of composite section

$z_{tp\_short}(0\ m) = 216.3\ mm$ Distance from top of concrete to center of gravity for composite section

$I_{y\_short}(0\ m) = 0.443\ m^4$ Moment of inertia for composite section

$F_{temp} := \varepsilon_{temp} \cdot E_s \cdot A_{sl}(0\ m) = \begin{bmatrix} -4486 \\ 3631 \end{bmatrix} kN$ Force on composite section

$e_{temp\_F} := z_{tp\_short}(0\ m) = 216.3\ mm$ Level at which the force is imposed on the system

$e_{temp\_M} := z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) = 2.265\ m$ Eccentricity for the bending moment

$$M_{temp} := F_{temp} \cdot e_{temp\_M} = \begin{bmatrix} -10158 \\ 8223 \end{bmatrix} kN \cdot m$$

Bending moment - composite section

Anchorage of temperature load imposed on studs

Concrete

$$N_{c\_temp} := F_{temp} \cdot \left(\frac{\frac{A_{slab.fic}}{n_{L\_short}}}{A_{sl\_short}(0\ m)} - \frac{\frac{A_{slab.fic}}{n_{L\_short}}}{I_{y\_short}(0\ m)} \cdot \left(\left(z_{tp\_steel}(0\ m) - z_{tp\_short}(0\ m) \right) \cdot \left(z_{tp\_short}(0\ m) + \frac{h_{m.slab}}{2} \right) \right) \right) = \begin{bmatrix} -1538 \\ 1245 \end{bmatrix} kN$$

$$M_{c\_temp} := \frac{\frac{1}{12} \cdot \frac{b_{eff}(0\ m)}{n_{L\_short}} \cdot h_{m.slab}^{3}}{I_{y\_short}(0\ m)} \cdot M_{temp} = \begin{bmatrix} -52 \\ 42 \end{bmatrix} kN \cdot m$$

$$\sigma_1 := \frac{N_{c\_temp}}{A_{slab.fic}} - \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{2}} = \begin{bmatrix} -0.35 \\ 0.29 \end{bmatrix} MPa$$

Stress in concrete, upper edge of slab

$$\sigma_2 := \frac{N_{c\_temp}}{A_{slab.fic}} + \frac{M_{c\_temp}}{\frac{1}{6} \cdot b_{eff}(0\ m) \cdot h_{m.slab}^{2}} = \begin{bmatrix} -1.60 \\ 1.29 \end{bmatrix} MPa$$

Stress in concrete, lower edge of slab

$$N_{temp} := \frac{\sigma_1 + \sigma_2}{2} \cdot A_{slab.fic} = \begin{bmatrix} -1538 \\ 1245 \end{bmatrix} kN$$

Force imposed on studs caused by shrinkage (used in chapter 6 when calculating the anchorage of the slab by the studs)

5 Capacity checks during construction

The capacity check that is carried out in this chapter is bending moment capacity with respect to lateral torsional buckling in the casting phase

In the casting phase the normalforces in the cross-section are neglectible.

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

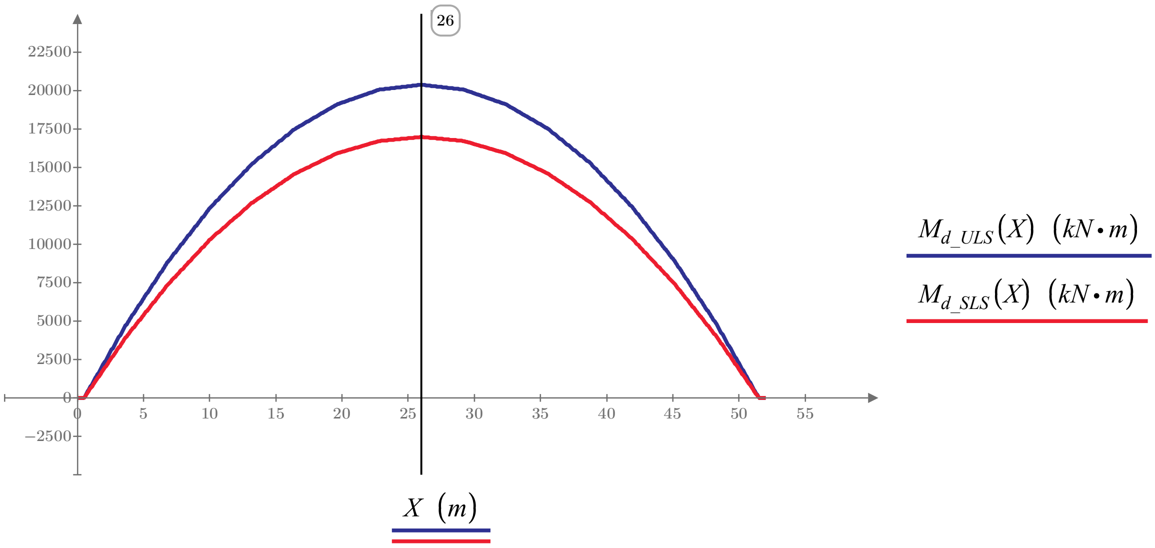
$X_{check\_v} = 1.5\ m$ Coordinate for control calculations - shear force

5.1 Load effects

Load effects retrieved from Strip-Step2, Appendix X.

Bending moment

M_{d_ULS} Bending moment descending from permanent loads during construction in ultimate limit state
M_{d_SLS} Bending moment descending from permanent loads during construction in service limit state

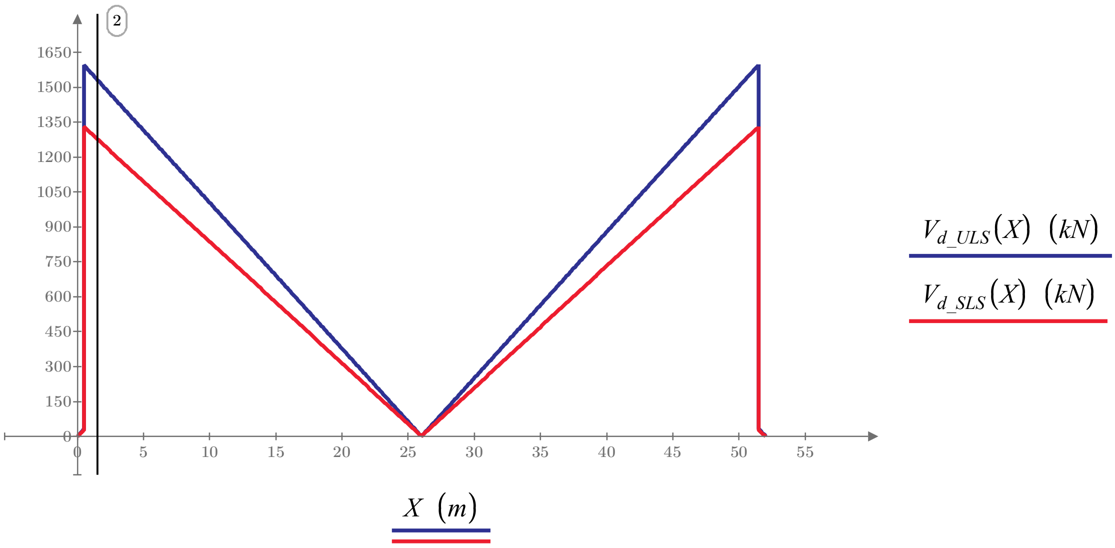


$M_{d\_ULS}(X_{check\_m}) = 20383\ kN \cdot m$ Bending moment at control point - ULS

$M_{d\_SLS}(X_{check\_m}) = 16986\ kN \cdot m$ Bending moment at control point - SLS

Shear force

V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
V_{d_SLS}	Shear force descending from permanent loads during construction in service limit state



$V_{d\_ULS}(X_{check\_v}) = 1533\ kN$ Shear force at control point - ULS

$V_{d\_SLS}(X_{check\_v}) = 1278\ kN$ Shear force at control point - SLS

5.2 Redesign of cross-section

S_{el}	Length coordinate
t_{fu}	Thickness of upper flange
b_{fu}	Width of upper flange
t_w	Thickness of web
h_w	Height of web
t_{fl}	Thickness of lower flange
b_{fl}	Width of lower flange

Element 1

S_{el1} (mm)	t_{fu_1} (mm)	b_{fu_1} (mm)	t_{w_1} (mm)	h_{w_1} (mm)	t_{fl_1} (mm)	b_{fl_1} (mm)
0	30	575	6	4025	45	525
500	30	575	6	4025	45	525

Element 2

S_{el2} (mm)	t_{fu_2} (mm)	b_{fu_2} (mm)	t_{w_2} (mm)	h_{w_2} (mm)	t_{fl_2} (mm)	b_{fl_2} (mm)
0	30	575	6	4025	45	525
12500	30	575	6	4025	45	525
12500	35	575	4	4015	50	525
13300	35	575	4	4015	50	625
25500	35	575	4	4015	50	625

10500
10500
11300

Element 3

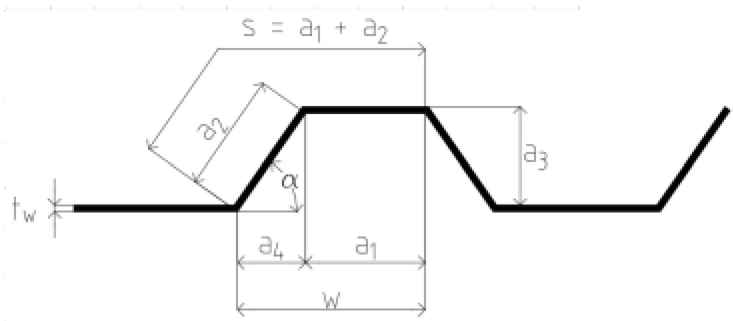
S_{el3} (mm)	t_{fu_3} (mm)	b_{fu_3} (mm)	t_{w_3} (mm)	h_{w_3} (mm)	t_{fl_3} (mm)	b_{fl_3} (mm)
0	30	575	6	4025	45	525
500	30	575	6	4025	45	525

$$mean\left(t_{fu\_1}+h_{w\_1}+t_{fl\_1}\right)=4100\ mm$$

$$mean\left(t_{fu\_2}+h_{w\_2}+t_{fl\_2}\right)=4100\ mm$$

Check to see that the girder height is kept constant

5.2.1 Shape of corrugation



$a_{c1} := 215\ mm$ Flatfold length

$\alpha_c := 36\ deg$ Corrugation degree

$a_{c3} := 125\ mm$ Corrugation depth

$a_{weld} := 5\ mm$ Weld throat thickness

$a_{c2} := \dfrac{a_{c3}}{sin\left(\alpha_c\right)} = 213\ mm$ Legnth of hypotenuse

$a_{c4} := \dfrac{a_{c3}}{tan\left(\alpha_c\right)} = 172\ mm$ Length of corrugation depth, see figure

$s_c := a_{c1} + a_{c2} = 428\ mm$ Length of corrugation

$w_c := a_{c1} + a_{c4} = 387\ mm$ Straight length

$r_c := \dfrac{s_c}{w_c} = 1.105$ Ratio corrugation/straight length

5.2.2 Cross-section classification

The cross-section classes is determined according to SS-EN 1993 1-4 5.2.2 with updated limits from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated and does not contribute to the axial stiffness the web is not classified.

$$c_w(x) := h_w(x) - 2 \cdot \sqrt{2} \cdot a_{weld}$$

$$c_{uf}(x) := \frac{b_{fu}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on upper flange

$$c_{lf}(x) := \frac{b_{fl}(x)}{2} + \frac{a_{c3}}{2} - \frac{t_w(x)}{2} - \sqrt{2} \cdot a_{weld}$$

Distance from web weld toe to free edge on lower flange

<u>Cross-section class, upper flange</u>

$$E_s = 200\ GPa$$

Modulus of elasticity

$$f_{yuf} = 450\ MPa$$

Proof strength of top flange

$$\varepsilon_{uf} := \sqrt{\frac{235}{f_{yuf}} \cdot \frac{E_s}{210000}} = 0.71$$

SS-EN 1993-1-4 5.2.2 Table 5.2

$$csc_{uf}(x) := \begin{cases} \text{“csc1”} & \text{if } \frac{c_{uf}(x)}{t_{fu}(x)} \le 9\ \varepsilon_{uf} \\ \text{“csc2”} & \text{else if } 9\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 10\ \varepsilon_{uf} \\ \text{“csc3”} & \text{else if } 10\ \varepsilon_{uf} < \frac{c_{uf}(x)}{t_{fu}(x)} \le 14\ \varepsilon_{uf} \\ \text{“csc4”} & \text{else} \end{cases}$$

Cross-section class upper flange

$$csc_{uf}(X_{check\_m}) = \text{“csc3”}$$

Cross-section class at $X_{check\_m} = 26.000\ m$

$$csc_{uf}(X_{check\_v}) = \text{“csc4”}$$

Cross-section class at $X_{check\_v} = 1.500\ m$

Cross-section class, lower flange

$E_s = 200\ GPa$ Modulus of elasticity

$f_{ylf} = 450\ MPa$ Proof strength of top flange

$\varepsilon_{lf} := \sqrt{\dfrac{235}{f_{ylf}} \cdot \dfrac{E_s}{210000}} = 0.71$ SS-EN 1993-1-4 5.2.2 Table 5.2

Cross-section class lower flange

$$csc_{lf}(x) := \begin{cases} \text{"csc1"} & \text{if } \dfrac{c_{lf}(x)}{t_{fl}(x)} \le 9\ \varepsilon_{lf} \\ \text{"csc2"} & \text{else if } 9\ \varepsilon_{lf} < \dfrac{c_{lf}(x)}{t_{fl}(x)} \le 10\ \varepsilon_{lf} \\ \text{"csc3"} & \text{else if } 10\ \varepsilon_{lf} < \dfrac{c_{lf}(x)}{t_{fl}(x)} \le 14\ \varepsilon_{lf} \\ \text{"csc4"} & \text{else} \end{cases}$$

$csc_{lf}(X_{check\_m}) = \text{"csc3"}$ Cross-section class at $X_{check\_m} = 26.000\ m$

$csc_{lf}(X_{check\_v}) = \text{"csc2"}$ Cross-section class at $X_{check\_v} = 1.500\ m$

5.2.3 Plate buckling of compressive flange

If the compressed flange is in cross-section class four an effective width of the compressed flange is calculated according to SS-EN 1993-1-4 5.2.3. Updated values are taken from SS-EN 1993 1-4:2006/A1:2015. Since the web is corrugated the buckling factor k_σ is calculated according to SS-EN 1993-1-5 D.2.1 (2)

$a_{bend} := a_{c1} + 2\ a_{c4} = 559\ mm$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$c_u := c_{uf}\left(X_{check\_m}\right) = 341\ mm$ — Width of outstand flange from weld toe to free edge

$t_u := t_{fu}\left(X_{check\_m}\right) = 35\ mm$ — Thickness of upper flange

$b_u := b_{fu}\left(X_{check\_m}\right) = 575\ mm$ — Width of upper flange

$\varepsilon_{uf} = 0.71$

$k_{\sigma 1} := 0.43 + \left(\dfrac{c_u}{a_{bend}}\right)^2 = 0.8$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

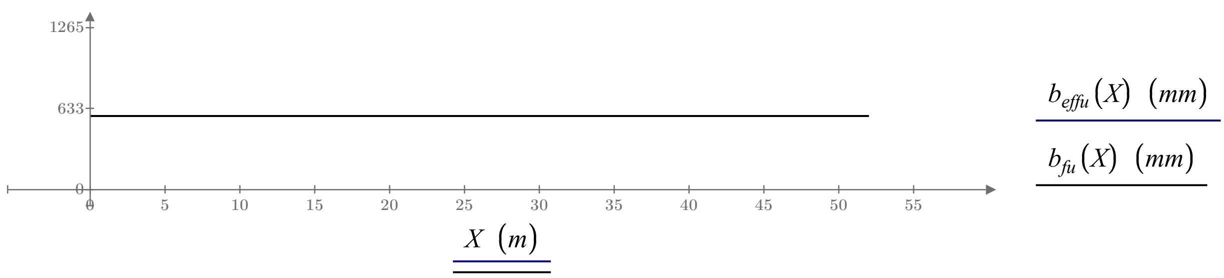
$k_{\sigma 2} := 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$k_\sigma := min\left(k_{\sigma 1}, k_{\sigma 2}\right) = 0.6$ — SS-EN 1993-1-5 D.2.2.(1) Equation D.4

$\lambda_p := \dfrac{\dfrac{c_u}{t_u}}{28.4 \cdot \varepsilon_{uf} \cdot \sqrt{k_\sigma}} = 0.63$ — Slenderness of flange plate SS-EN 1993-1-1 (2)

$\rho := \mathbf{if}\left(\lambda_p \le 0.748, 1.0, \dfrac{\lambda_p - 0.188}{{\lambda_p}^2}\right) = 1.00$ — Reduction of flange area SS-EN 1993-1-5 (2) 4.4 Equation 4.3. Same for carbon steel as for Stainless steel

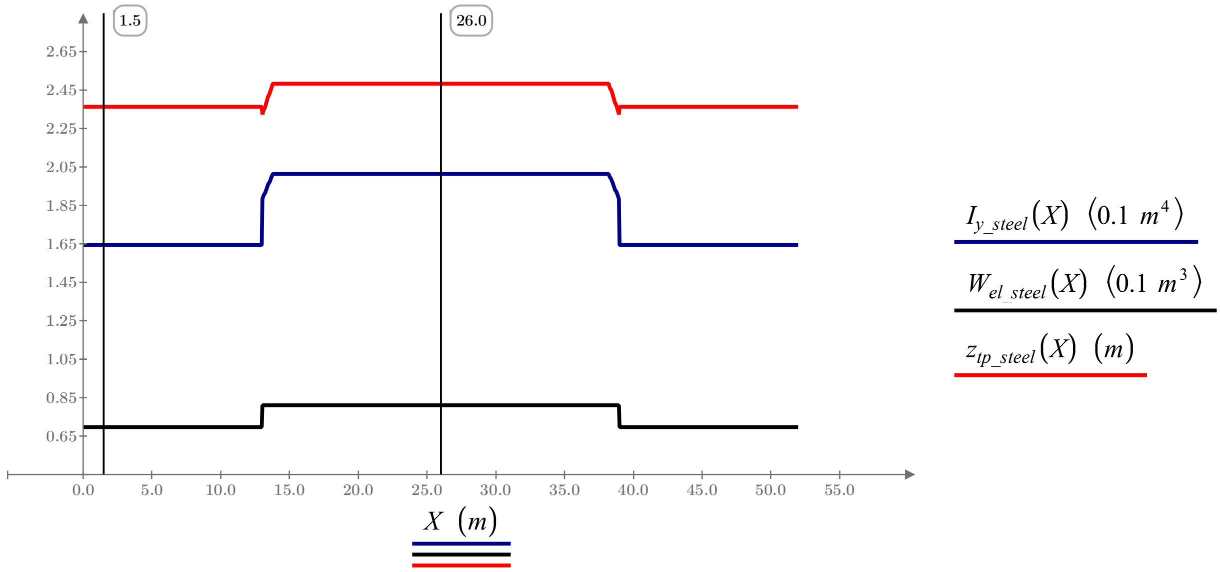
$b_{eff} := b_u \cdot \rho = 575\ mm$ = $b_{effu}\left(X_{check\_m}\right) = 575\ mm$



$b_{fu}(x) := b_{effu}(x)$ — Renaming the width of flange in order to minimize errors

5.2.6 New cross-sectional constants during casting

$I_{y\_steel}(x)$	Stiffness of steel girder alone (excluding the web)
$z_{tp\_steel}(x)$	Distance from the top of the top flange to the center of gravity for the steel section
$W_{el\_steel}(x)$	Elastic bending stiffness of steel girder alone (excluding the web)



Coordinates and cross-sectional constants for control calculations

$X_{check\_m} = 26\ m$	X- coordinate for control calculations, bending moment
$A_{sl}(X_{check\_m}) = 0.051\ m^2$	Area
$I_{y\_steel}(X_{check\_m}) = (202 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_m}) = 2.486\ m$	Center of gravity
$W_{el\_steel}(X_{check\_m}) = (81 \cdot 10^{-3})\ m^3$	Elastic bending resistance
$X_{check\_v} = 1.5\ m$	X- coordinate for control calculations, shear force
$A_{sl}(X_{check\_v}) = 0.041\ m^2$	Area
$I_{y\_steel}(X_{check\_v}) = (165 \cdot 10^{-3})\ m^4$	Stiffness
$z_{tp\_steel}(X_{check\_v}) = 2.363\ m$	Center of gravity
$W_{el\_steel}(X_{check\_v}) = (70 \cdot 10^{-3})\ m^3$	Elastic bending resistance

5.2.4 Lateral torsional buckling of compressive flange - SS-EN 1993-1-4 5.4.2.1

Simplified method only consisdering buckling of top flange according to SS-EN 1993-1-4 5.4.2.1

$\alpha_{LT} := 0.76$ — Buckling curve d, welded open cross-section

$l_{cr} := 7.29\ m$ — Distance between the cross-beams

$b_{ef} := b_{effu}\left(X_{check\_m}\right) = 575\ mm$ — Width of upper flange

$t_f := t_{fu}\left(X_{check\_m}\right) = 35\ mm$ — Thickness of upper flange

$E_s = 200\ GPa$ — Modulus of elasticity

$f_{yuf} = 450\ MPa$ — Proof strength

$I_{zf} := \dfrac{b_{ef}^{\ 3} \cdot t_f}{12} = 0.001\ m^4$ — Moment of inertia, upper flange

$N_{crLT} := \dfrac{\pi^2 \cdot E_s \cdot I_{zf}}{l_{cr}^{\ 2}} = 20595\ kN$ — Critical buckling load

$\lambda_{LT\_u} := \sqrt{\dfrac{b_{ef} \cdot t_f \cdot f_{yuf}}{N_{crLT}}} = 0.663$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.9

$\Phi_{LT\_u} := 0.5 \cdot \left(1 + \alpha_{LT} \cdot \left(\lambda_{LT\_u} - 0.2\right) + \lambda_{LT\_u}^{\ 2}\right) = 0.9$ — SS-EN 1993-1-4 5.4.2.1 Equation 5.7

$\chi_{LT\_u} := min\left(\dfrac{1}{\Phi_{LT\_u} + \sqrt{\Phi_{LT\_u}^2 - \lambda_{LT\_u}^{\ 2}}}, 1\right) = 0.667$ = $\chi_{LT}\left(X_{check\_m}\right) = 0.667$

SS-EN 1993-1-4 5.4.2.1 Equation 5.6

$h_{w\_u} := h_w\left(X_{check\_m}\right) = 4015\ mm$ — Height of web

$t_l := t_{fl}\left(X_{check\_m}\right) = 50\ mm$ — Thickness of lower flange

$k_{fl} := 1.1$ — Increase in capacity due to simlified method used, SS-EN 1993-1-1 6.3.2.4 (2)B

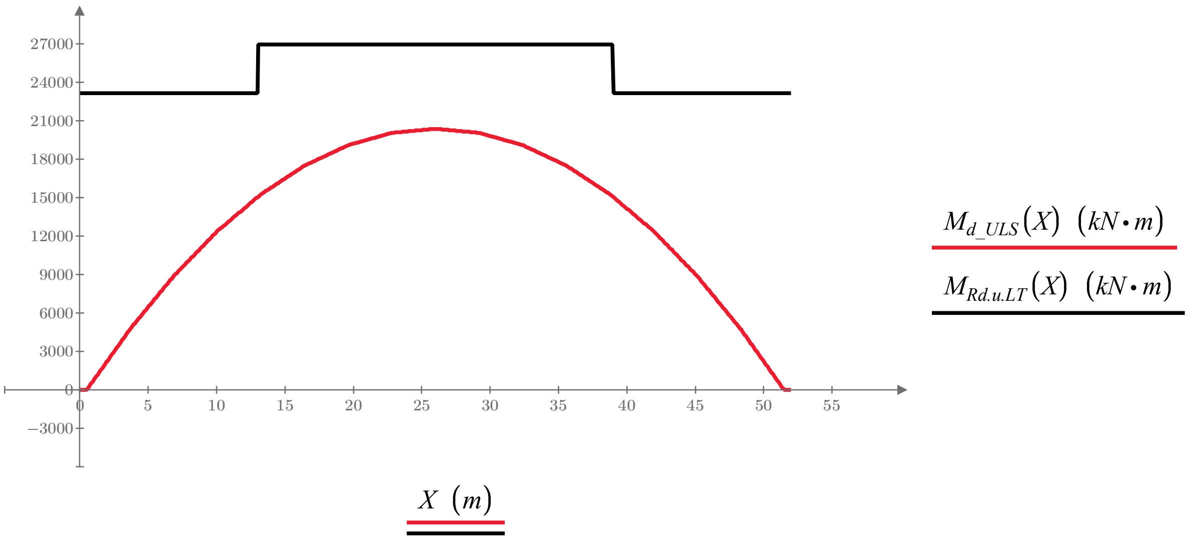
$M_{Rd.u.LT\_u} := \dfrac{b_{ef} \cdot t_f \cdot k_{fl} \cdot \chi_{LT\_u} \cdot f_{yuf}}{\gamma_{M1}} \left(h_{w\_u} + \dfrac{t_f + t_l}{2}\right) = 26979\ kN \cdot m$ = $M_{Rd.u.LT}\left(X_{check\_m}\right) = 26979\ kN \cdot m$

SS-EN 1993-1-5 D.2.1 Equation D.1

5.2.5 Check of lateral torsional buckling

Check of the buckling capacity of the girders is performed.

$M_{Rd.u.LT}(X)$	Capacity of the girder against LT- buckling during casting
$M_{d\_ULS}(X)$	Load effects during casting, Appendix X



$$\eta_{max} := max\left(\frac{M_{d\_ULS}(X)}{M_{Rd.u.LT}(X)}\right) = 76\%$$

Maximum utilization rate of the steel cross-section during casting

5.3 Stresses in steel cross-section

The stresses in the top and bottom flange is calculated for the loading senario to be able to superposition them with the other loadcases to determinte the ultimate capacity of the composite section.

Upper flange

Control calculation at midspan

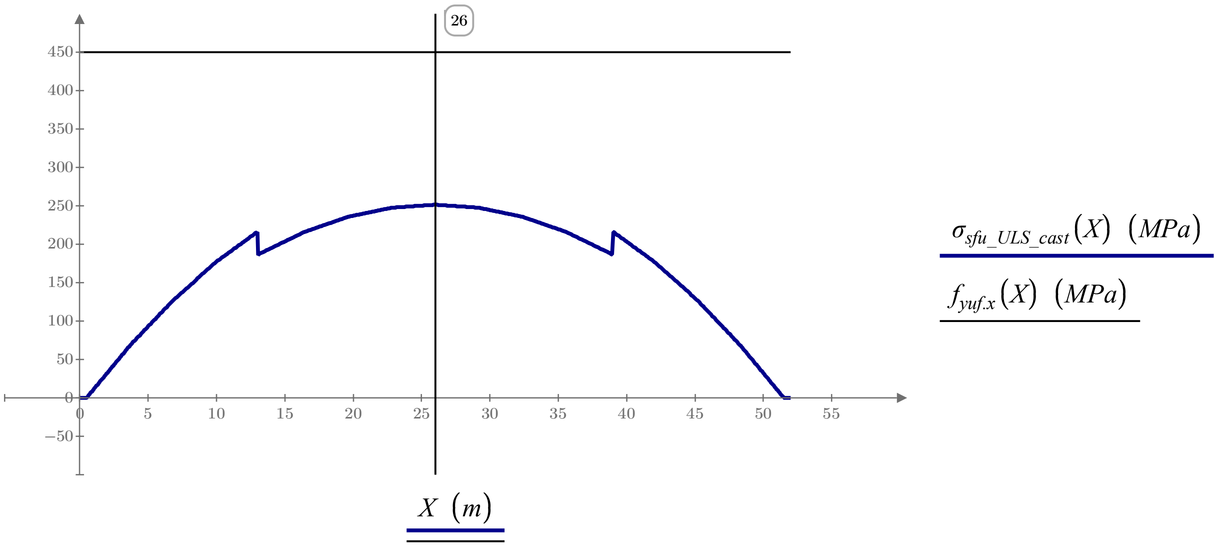
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 20383\ kN \cdot m$ Load effext at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.202\ m^4$ Moment of inertia at midspan

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 2.486\ m$ Centre of gravity at midspan maesured from the top of the beam

$\sigma := \frac{M}{I} \cdot z = 251\ MPa$ = $\sigma_{sfu\_ULS\_cast}\left(X_{check\_m}\right) = 251\ MPa$

$\sigma_{sfu\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\frac{\sigma_{sfu\_ULS\_cast}(X)}{f_{yuf}}\right) = 56\%$

Lower flange

Check calculation at midspan

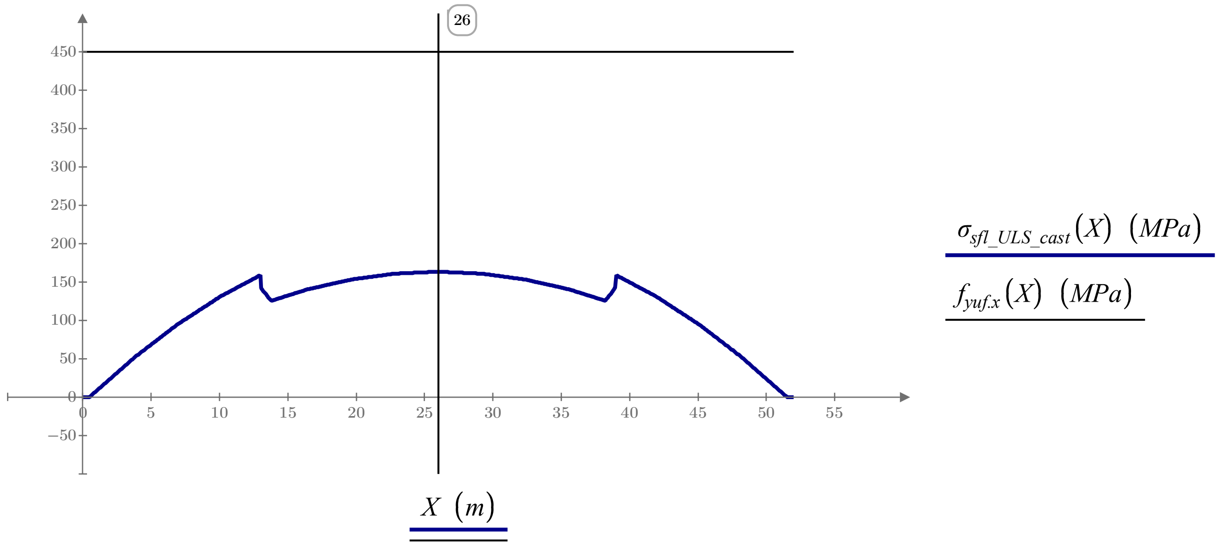
$M := M_{d\_ULS}\left(X_{check\_m}\right) = 20383\ kN \cdot m$ Load effect at midspan

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.202\ m^4$ Moment of inertia at midspan

$z := z_{tplf\_steel}\left(X_{check\_m}\right) = 1.614\ m$ Centre of gravity at midspan measured from the bottom of the beam

$\sigma := \dfrac{M}{I} \cdot z = 163\ MPa$ = $\sigma_{sfl\_ULS\_cast}\left(X_{check\_m}\right) = 163\ MPa$

$\sigma_{sfl\_ULS\_cast}(X)$ Stresses in the lower flange in the casting phase.



$\eta_{max} := max\left(\dfrac{\sigma_{sfl\_ULS\_cast}(X)}{f_{ylf}}\right) = 36\%$

6 Capacity checks - Ultimate limit state, global

The capacity checks that are to be carried out are bending moment capacity, shear capacity, web breathing and design of studs, both in ULS and due to fatigue

The checks are done using vectors and closed areas. Therefore calculations are verified using a "control-calculation" for each important check. Values relevant for the control calculations are presented in blue.

$X_{check\_m} = 26\ m$ Coordinate for control calculations - bending moment

$X_{check\_v} = 0.5\ m$ Coordinate for control calculations - shear force

6.1 Load effects in ULS

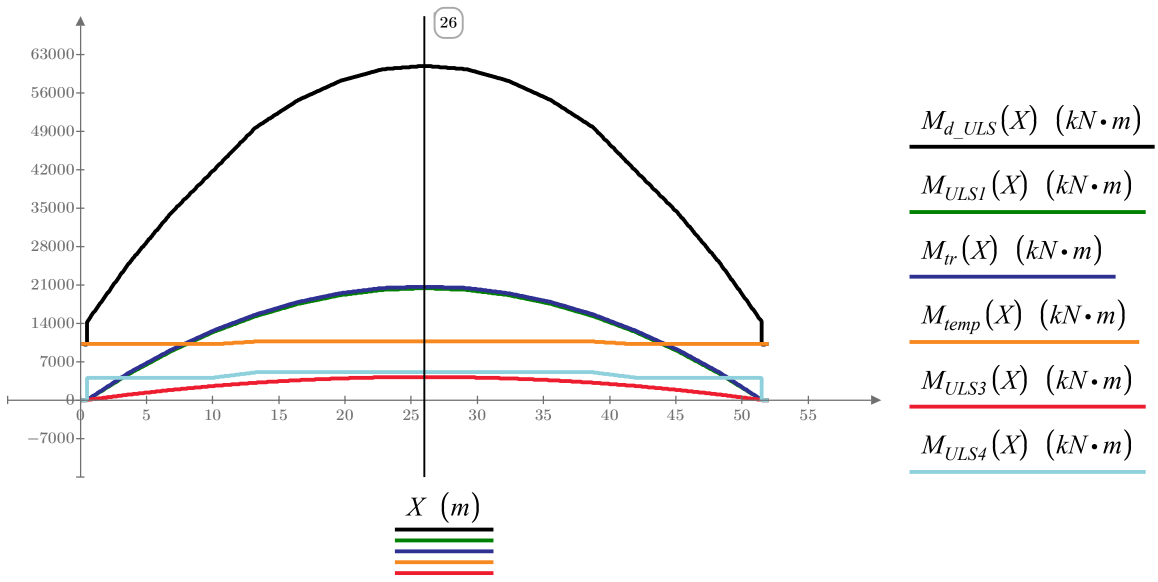
6.1.1 Bending moment with corresponding axial force

Permanent loads during casting do not contribute with any stresses in the concrete since the entire slab is casted in one step. Load effects retrieved from Strip-Step2, Appendix X.

<u>Bending moment in the ultimate limit state</u>

$$M_{d\_ULS}(x) := M_{ULS1}(x) + M_{tr}(x) + M_{temp}(x) + M_{ULS3}(x) + M_{ULS4}(x)$$

M_{d_ULS}	Design bending moment effect from all loads
$M_{ULS1}(x)$	Bending moment during casting (Appendix X)
$M_{tr}(x)$	Bending moment from multi component loads (Appendix X)
$M_{tr}(x)$	Bending moment from temperature loads (Appendix X)
$M_{ULS3}(x)$	Bending moment from additional permanent loads after construction (Appendix X)
$M_{ULS4}(x)$	Bending moment from shrinkage (Appendix X)

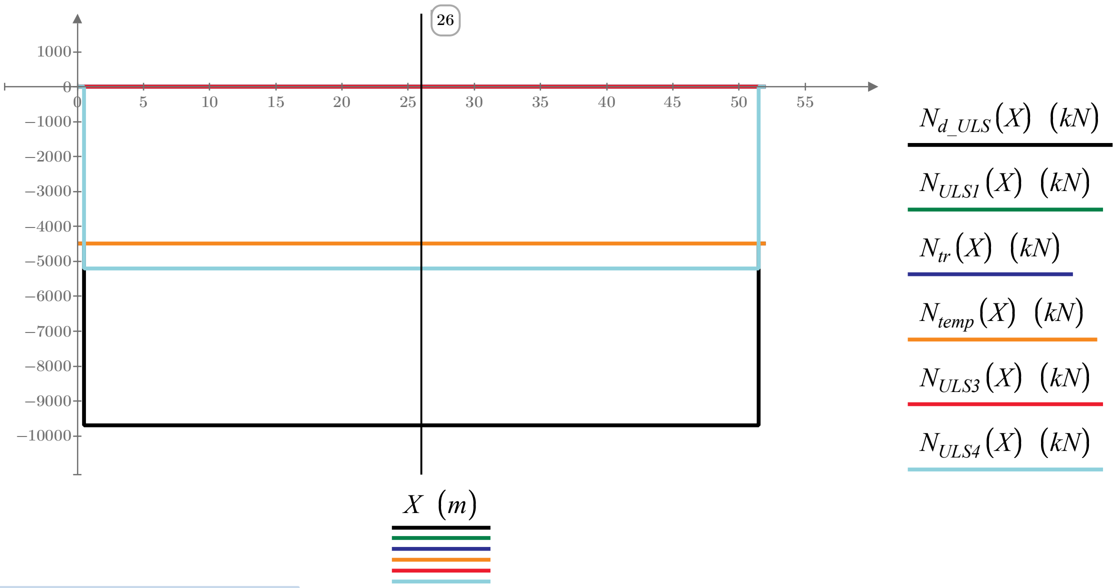


$M_{d\_ULS}(X_{check\_m}) = 60890\ kN \cdot m$ Bending moment at control point - ULS

Axial force in the ultimate limit state

$$N_{d\_ULS}(x) := N_{ULS1}(x) + N_{tr}(x) + N_{temp}(x) + N_{ULS3}(x) + N_{ULS4}(x)$$

N_{d_ULS}	Design normal force from all loads
$N_{ULS1}(x)$	Normal force during casting (Appendix X)
$N_{tr}(x)$	Normal force from multi components loads (Appendix X)
$N_{temp}(x)$	Normal force from temperature loads (Appendix X)
$N_{ULS3}(x)$	Normal force from additional permanent loads after construction (Appendix X)
$N_{ULS4}(x)$	Normal force from shrinkage (Appendix X)

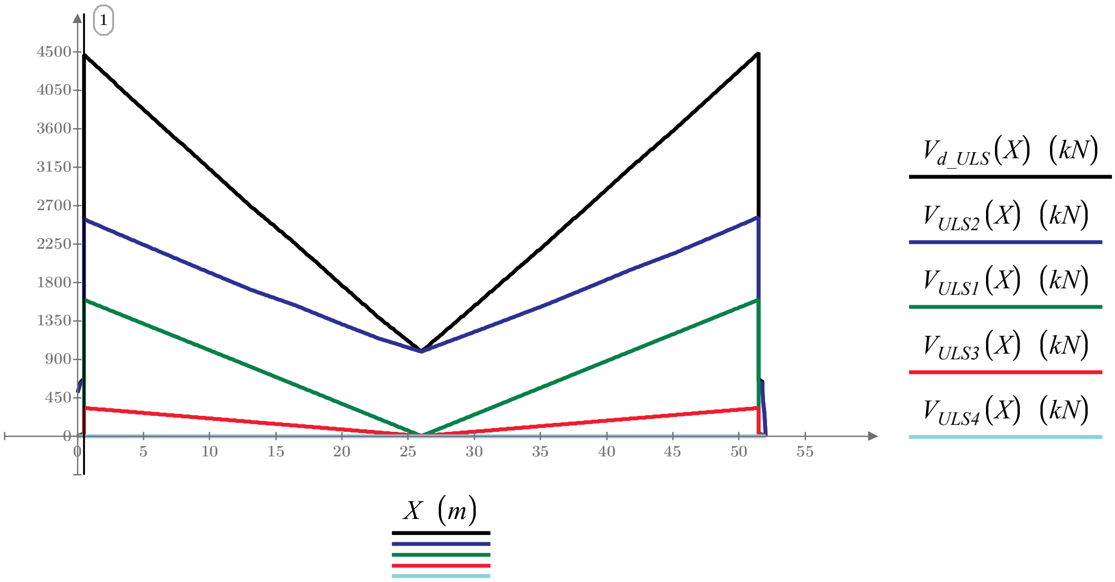


$N_{d\_ULS}(X_{check\_m}) = -9692\ kN$ Normal force at control point - ULS

6.1.2 Shear force

$$V_{d\_ULS}(x) := V_{ULS1}(x) + V_{ULS2}(x) + V_{ULS3}(x) + V_{ULS4}(x)$$

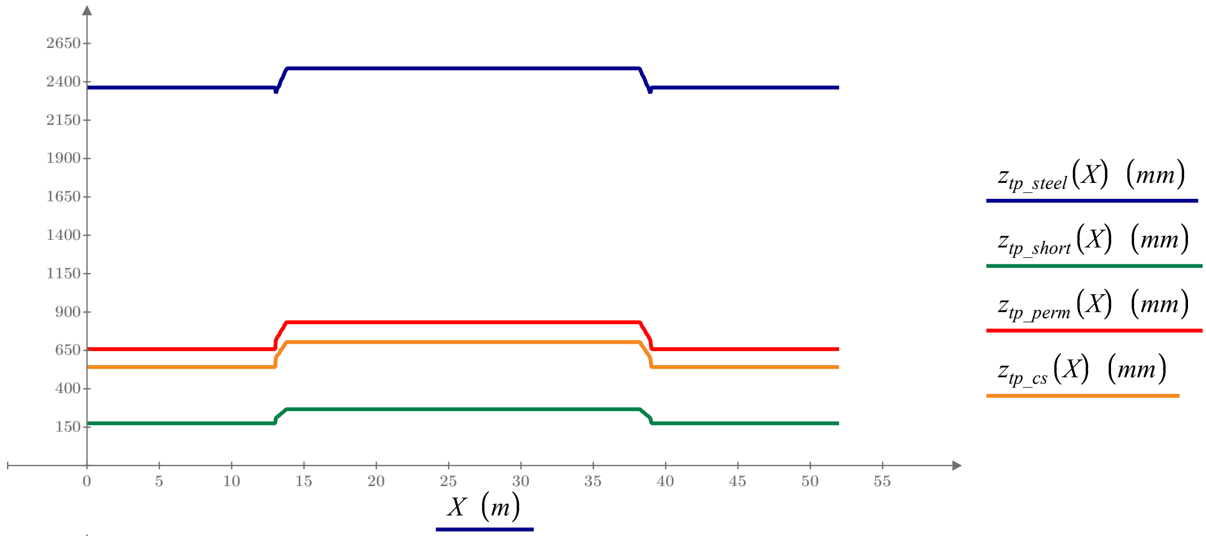
V_{d_ULS}	Shear force descending from permanent loads during construction in ultimate limit state
$V_{ULS1}(x)$	Shear force during casting (Appendix X)
$V_{ULS2}(x)$	Shear force from variable loads (both temperature and traffic) (Appendix X)
$V_{ULS3}(x)$	Shear force from additional permanent loads after construction (Appendix X)
$V_{ULS4}(x)$	Shear force from shrinkage (Appendix X)

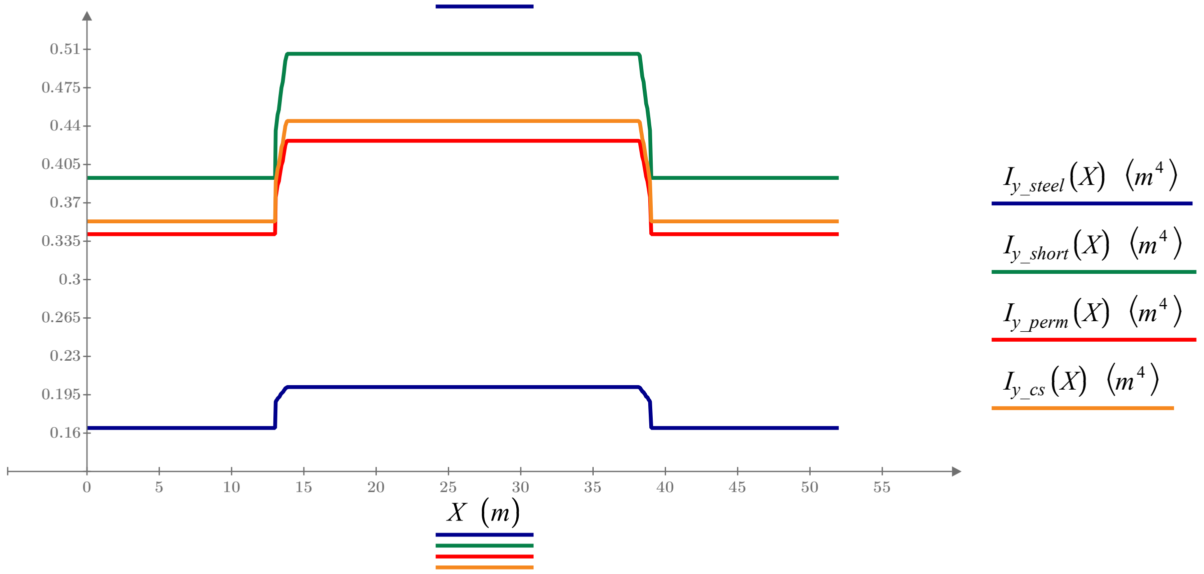


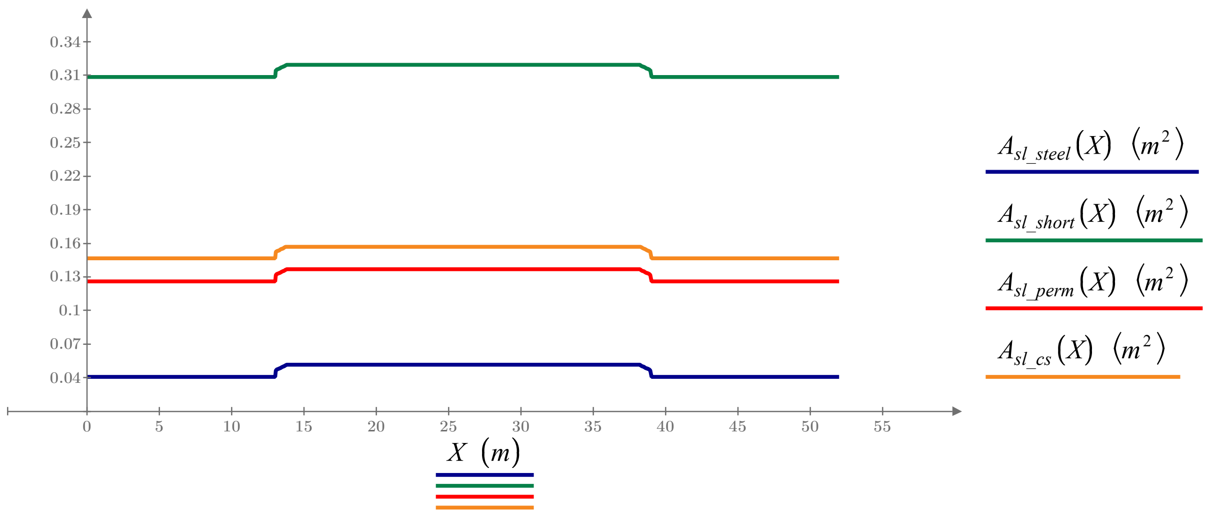
$V_{d\_ULS}(X_{check\_v}) = 4463\ kN$ Shear force at control point - ULS

6.2 Cross-sectional constants

For calculations, see chapter 5.







6.3 Stresses in steel cross-section

The stresses is calculated for each load case taking into acount load duration and creep. The stresses are then superpositioned.

6.3.1 Stresses during casting

$M := M_{ULS1}\left(X_{check\_m}\right) = 20383\ kN \cdot m$ Bending moment

$I := I_{y\_steel}\left(X_{check\_m}\right) = 0.202\ m^4$ Moment of inertia

$z := z_{tp\_steel}\left(X_{check\_m}\right) = 2486\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 4100\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot -z = -251\ MPa$ = $\sigma_{s.u.cast}\left(X_{check\_m}\right) = -251\ MPa$ Stresses in upper flange from loads durng casting

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 163\ MPa$ = $\sigma_{s.l.cast}\left(X_{check\_m}\right) = 163\ MPa$ Stresses in lower flange from loads durng casting

6.3.2 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses must be calculated for that specific section. The worst load case is determined dependent on the largest stress in each part whereas the multi-component load is the main load for the lower flange, and the temperature load is the worst load case for the upper flange.

6.3.2.1 Multi component loads (traffic)

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0.01\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.319\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.506\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) = 266\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 4100\ mm$ Height of girder

$\sigma_{s.u} := \frac{0.9\ N}{A} + \frac{0.9\ M}{I} \cdot (-z) = -10\ MPa$ = $\sigma_{s.u.tr}\left(X_{check\_m}\right) = -10\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{1.5\ N}{A} + \frac{1.5\ M}{I} \cdot (h - z) = 235\ MPa$ = $\sigma_{s.l.tr}\left(X_{check\_m}\right) = 235\ MPa$ Stresses in lower flange from short term loads

6.3.2.2 Temperature loads

$M := M_{temp}\left(X_{check\_m}\right) = 10642\ kN \cdot m$ Bending moment

$N := N_{temp}\left(X_{check\_m}\right) = -4486\ kN$ Normal force

$A_s := A_{sl\_steel}\left(X_{check\_m}\right) = 0.051\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.319\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}\left(X_{check\_m}\right) = 0.202\ m^4$ Moment of inertia - steel section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.506\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}\left(X_{check\_m}\right) = 2486\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 266\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}\left(X_{check\_m}\right) = 4100\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 4243\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(1 - \left(\frac{A_s}{A} + \frac{A_s}{I}\left(z_s - z\right)^2\right)\right) = -1518\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{1.5\ N_s}{A_s} + \frac{1.5\ M_s}{I_s} \cdot \left(-z_s\right) = -123\ MPa$ = $\sigma_{s.u.temp}\left(X_{check\_m}\right) = -123\ MPa$ Stresses in upper flange from short term loads

$\sigma_{s.l} := \frac{0.9\ N_s}{A_s} + \frac{0.9\ M_s}{I_s} \cdot \left(h - z_s\right) = 4\ MPa$ = $\sigma_{s.l.temp}\left(X_{check\_m}\right) = 4\ MPa$ Stresses in lower flange from short term loads

6.3.3 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.427\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) = 835\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 4100\ mm$ Height of girder

$\sigma_{s.u} := \frac{M}{I} \cdot (-z) = -8\ MPa$ = $\sigma_{s.u.perm}(X_{check\_m}) = -8\ MPa$ Stresses in upper flange from additional permanent loads

$\sigma_{s.l} := \frac{M}{I} \cdot (h - z) = 32\ MPa$ = $\sigma_{s.l.perm}(X_{check\_m}) = 32\ MPa$ Stresses in lower flange from additional permanent loads

6.3.4 Stresses due to shrinkage

In the same manner as the temperature load must be calculated for the steel section, the shrinkage which acts on the concrete section must be converted for the steel section.

$M := M_{ULS4}(X_{check\_m}) = 5026\ kN \cdot m$ Bending moment

$N := N_{ULS4}(X_{check\_m}) = -5206\ kN$ Normal force

$A_s := A_{sl\_steel}(X_{check\_m}) = 0.051\ m^2$ Cross-sectional area - steel section

$A := A_{sl\_cs}(X_{check\_m}) = 0.157\ m^2$ Cross-sectional area - composite section

$I_s := I_{y\_steel}(X_{check\_m}) = 0.202\ m^4$ Moment of inertia - steel section

$I := I_{y\_cs}(X_{check\_m}) = 0.444\ m^4$ Moment of inertia - composite section

$z_s := z_{tp\_steel}(X_{check\_m}) = 2486\ mm$ Center of gravity from top flange - steel section

$z := z_{tp\_cs}(X_{check\_m}) = 705\ mm$ Center of gravity from top flange - composite section

$h := h_{beam}(X_{check\_m}) = 4100\ mm$ Height of girder

$M_s := \frac{I_s}{I} \cdot M = 2279\ kN \cdot m$ Bending moment imposed on steel section

$N_s := N \cdot \left(\frac{A_s}{A} - \frac{A_s}{I} \cdot \left(z - \frac{h_{m.slab}}{2}\right) \cdot \left(z_s + \frac{h_{m.slab}}{2} - \left(z - \frac{h_{m.slab}}{2}\right)\right)\right) = -1013\ kN$ Normal force imposed on steel section

$\sigma_{s.u} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (-z_s) = -48\ MPa$ = $\sigma_{s.u.shrink}(X_{check\_m}) = -48\ MPa$ Stresses in upper flange due to shrinkage

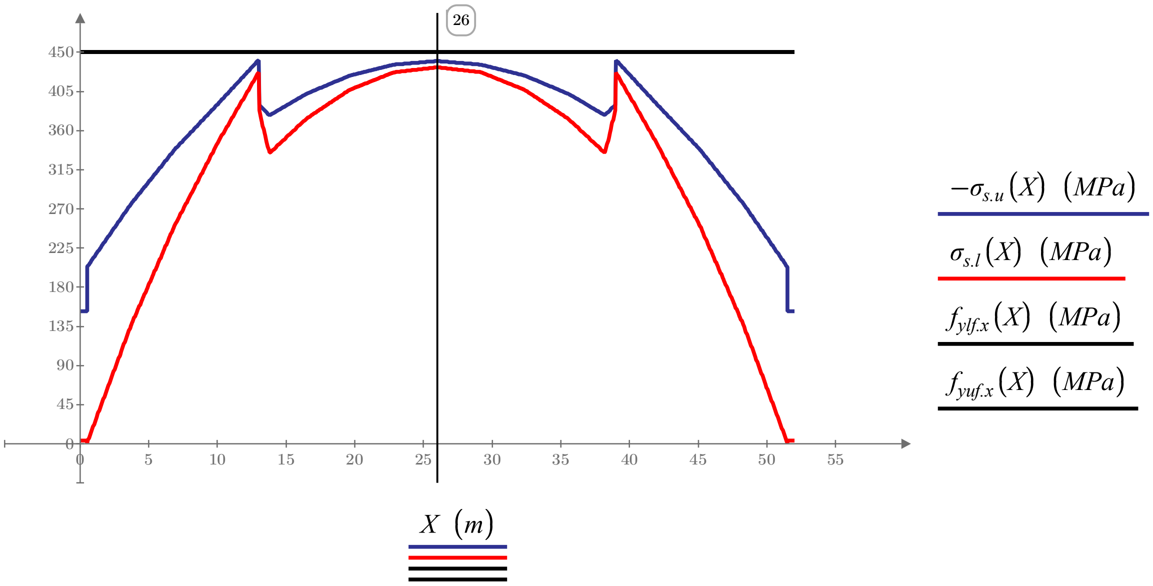
$\sigma_{s.l} := \frac{N_s}{A_s} + \frac{M_s}{I_s} \cdot (h - z_s) = -1\ MPa$ = $\sigma_{s.l.shrink}(X_{check\_m}) = -1\ MPa$ Stresses in lower flange due to shrinkage

6.3.5 Summary of stresses

The stresses from the different phases are summarised accordingly.

$$\sigma_{s.u}(x) := \sigma_{s.u.cast}(x) + \sigma_{s.u.shrink}(x) + \sigma_{s.u.perm}(x) + \sigma_{s.u.tr}(x) + \sigma_{s.u.temp}(x)$$ Stresses in upper flange

$$\sigma_{s.l}(x) := \sigma_{s.l.cast}(x) + \sigma_{s.l.shrink}(x) + \sigma_{s.l.perm}(x) + \sigma_{s.l.tr}(x) + \sigma_{s.l.temp}(x)$$ Stresses in lower flange



$$\eta_{\sigma.u.max} := max\left(\frac{-\sigma_{s.u}(X)}{f_{yuf.x}(X)}\right) = 98\%$$ Maximum utilization rate - upper flange

$$\eta_{\sigma.l.max} := max\left(\frac{\sigma_{s.l}(X)}{f_{ylf.x}(X)}\right) = 96\%$$ Maximum utilization rate - lower flange

6.4 Calculation of stresses in concrete

The stresses in the concrete is calculated with the same principle as the steel taking creep into account.

6.4.1 Stresses due to variable (short term) loads

The variable loads are divided into two load cases, one for the traffic load where the stress can be calculated for the composite section directly as the force is acting there. The other one is the temperature load where the load is acting on the steel section and so stresses for the concrete must be calculated separately.

6.4.1.1 Traffic and wind loads

$M := M_{tr}\left(X_{check\_m}\right) = 20647\ kN \cdot m$ Bending moment

$N := N_{tr}\left(X_{check\_m}\right) = 0\ kN$ Normal force

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.319\ m^2$ Cross-sectional area

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.506\ m^4$ Moment of inertia

$z := z_{tp\_short}\left(X_{check\_m}\right) + h_{m.slab} = 586\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 4100\ mm$ Height of girder

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$\sigma_c := \left(\frac{N}{A} + \frac{-M \cdot z}{I}\right) \cdot \frac{1}{n_\Gamma} = -4\ MPa$ = $\sigma_{c.short}\left(X_{check\_m}\right) = -4\ MPa$ Stresses in concrete from short-term loads

6.4.1.2 Temperature load

$M := M_{temp}\left(X_{check\_m}\right) = 10642\ kN \cdot m$

$N := N_{temp}\left(X_{check\_m}\right) = -4486\ kN$

$n_\Gamma := n_{L_2} = 5.88$ Modular ratio

$A := A_{sl\_short}\left(X_{check\_m}\right) = 0.319\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \frac{1}{n_\Gamma} = 0.268\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_short}\left(X_{check\_m}\right) = 0.506\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \frac{h_{m.slab}^{3}}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \frac{b_{eff}\left(X_{check\_m}\right)}{n_\Gamma} \cdot \frac{h_{m.slab}^{3}}{12} = 0.002\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_short}\left(X_{check\_m}\right) = 266\ mm$ Center of gravity - composite section

$z_c := \frac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

$M_c := \frac{I_{c.eff}}{I} \cdot M = 48\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(\frac{A_{c.eff}}{A} - \frac{A_{c.eff}}{I} \cdot \left(z_s - z\right) \cdot \left(z + \frac{h_{m.slab}}{2}\right)\right) = -1518\ kN$ Normal force imposed on concrete slab

$\sigma_c := \left(\frac{-N_c}{A_c} + \frac{M_c \cdot -z_c}{I_c}\right) = 0.39\ MPa$ = $\sigma_{c.temp}\left(X_{check\_m}\right) = 0.39\ MPa$ Stresses in concrete from temperature loads

6.4.2 Stresses due to additional permanent loads

$M := M_{ULS3}\left(X_{check\_m}\right) = 4192\ kN \cdot m$ Bending moment

$I := I_{y\_perm}\left(X_{check\_m}\right) = 0.427\ m^4$ Moment of inertia

$z := z_{tp\_perm}\left(X_{check\_m}\right) + h_{m.slab} = 1154\ mm$ Center of gravity from top flange

$h := h_{beam}\left(X_{check\_m}\right) = 4100\ mm$ Height of girder

$n_\Gamma := n_{L_1} = 18.48$ Creep

$\sigma_c := \frac{-M \cdot z}{I} \cdot \frac{1}{n_\Gamma} = -0.6\ MPa$ = $\sigma_{c.perm}\left(X_{check\_m}\right) = -0.6\ MPa$ Stresses in concrete from permanent loads

6.4.3 Stresses due to shrinkage

$M := M_{ULS4}\left(X_{check\_m}\right) = 5026\ kN \cdot m$ Bending moment

$N := N_{ULS4}\left(X_{check\_m}\right) = -5206\ kN$ Normal force

$n_\Gamma := n_{L_3} = 14.91$ Modular ratio considering creep

$A := A_{sl\_cs}\left(X_{check\_m}\right) = 0.157\ m^2$ Cross-section area - composite section

$A_{c.eff} := A_{slab.fic} \cdot \dfrac{1}{n_\Gamma} = 0.106\ m^2$ Cross-section area - effective concrete section

$A_c := A_{slab.fic} = 1.576\ m^2$ Cross-section area - concrete section

$I := I_{y\_cs}\left(X_{check\_m}\right) = 0.444\ m^4$ Moment of inertia - composite section

$I_c := b_{eff}\left(X_{check\_m}\right) \cdot \dfrac{{h_{m.slab}}^3}{12} = 0.013\ m^4$ Moment of inertia - concrete section

$I_{c.eff} := \dfrac{b_{eff}\left(X_{check\_m}\right)}{n_\Gamma} \cdot \dfrac{{h_{m.slab}}^3}{12} = 0.001\ m^4$ Moment of inertia - effective concrete section

$z := z_{tp\_cs}\left(X_{check\_m}\right) = 705\ mm$ Center of gravity - composite section

$z_c := \dfrac{h_{m.slab}}{2} = 160\ mm$ Center of gravity - concrete section

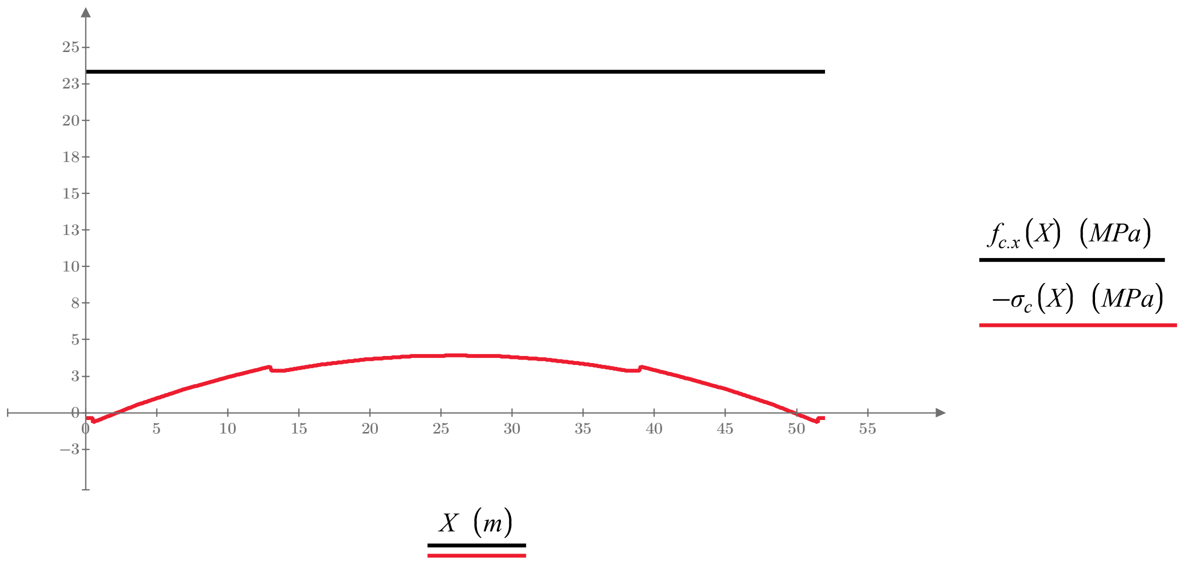
$M_c := \dfrac{I_{c.eff}}{I} \cdot M = 10\ kN \cdot m$ Bending moment imposed on concrete slab

$N_c := N \cdot \left(1 - \dfrac{A_{c.eff}}{A} - \dfrac{A_{c.eff}}{I} \cdot \left(z + \dfrac{h_{m.slab}}{2}\right)^2\right) = -775\ kN$ Normal force imposed on concrete slab

$\sigma_c := \dfrac{-N_c}{A_c} + \dfrac{M_c \cdot -z_c}{I_c} = 0.4\ MPa$ = $\sigma_{c.shrink}\left(X_{check\_m}\right) = 0.4\ MPa$ Stresses in concrete from shrinkage

6.4.4 Summary of stresses

$\sigma_c(x) := \sigma_{c.short}(x) + \sigma_{c.temp}(x) + \sigma_{c.perm}(x) + \sigma_{c.shrink}(x)$ Stresses in concrete



$$\eta_{\sigma.c.max} := max\left(\frac{-\sigma_c(X)}{f_{c.x}(X)}\right) = 17\%$$

Maximum utilization rate - upper flange

6.5 Shear capacity

The shear capacity of the corrugated steel girder is calculated accoridng to SS-EN 1993-1-5, Appendix D.
The local- and global buckling factor are calculated according to SS-EN 1993-1-5, Appendix D.

Local buckling factor

$a_{c1} = 215\ mm \qquad a_{c2} = 213\ mm$ — Corrugation geometries

$a_{cmax} := max\left(a_{c1}, a_{c2}\right) = 215\ mm$ — SS-EN 1993-1-5 D.2.2.(2)

$tw := t_w\left(X_{check\_v}\right) = 6\ mm$ — Web thickness

$\tau_{cr} := 4.83 \cdot E_s \cdot \left(\frac{tw}{a_{cmax}}\right)^2 = 752\ MPa$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.7

$\lambda_c := \sqrt{\frac{f_{yw}}{\tau_{cr} \cdot \sqrt{3}}} = 0.638$ — SS-EN 1993-1-5 D.2.2.(2) Equation D.6

$\chi_l := min\left(\frac{1.15}{0.9 + \lambda_c}, 1\right) = 0.75 \quad = \quad \chi_{c.l}\left(X_{check\_v}\right) = 0.75$ — Reduction factor local buckling - SS-EN 1993-1-5 D.2.2.(2) Equation D.5

Global buckling factor

$D_X := \frac{E_s \cdot tw^3}{12 \cdot \left(1 - \nu^2\right)} \cdot \frac{w_c}{s_c} = 4\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$D_Z := \frac{E_s \cdot tw \cdot a_{c3}^2}{12} \cdot \frac{\left(3\ a_{c1} + a_{c2}\right)}{w_c} = 3462\ kN \cdot m$ — SS-EN 1993-1-5 D.2.2.(3)

$hw := h_w\left(X_{check\_v}\right) = 4025\ mm$ — Web height

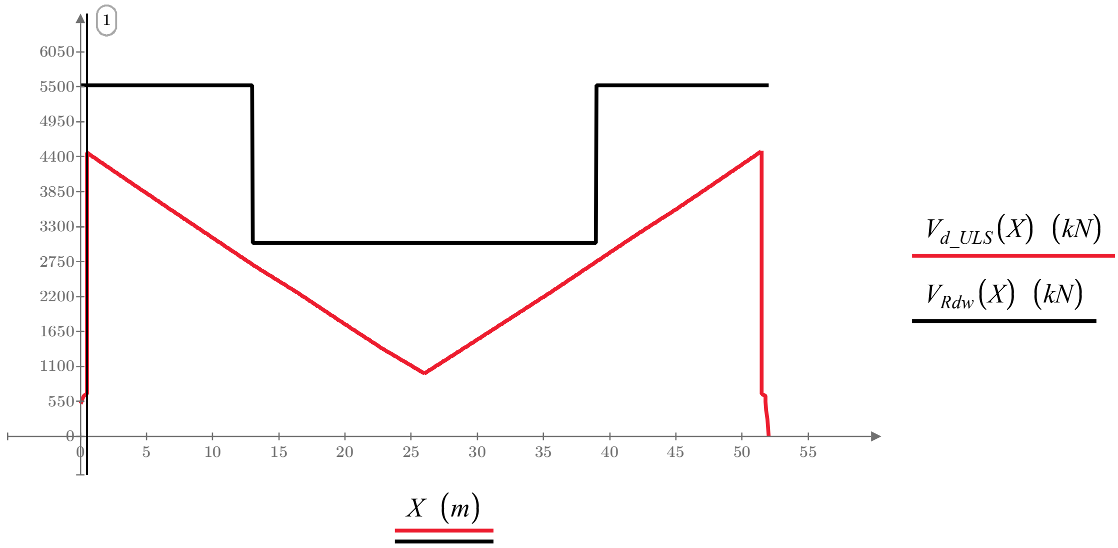
$\tau := \frac{32.4}{tw \cdot hw^2} \cdot \sqrt[4]{\frac{D_X}{N \cdot m} \cdot \frac{D_Z^3}{\left(N \cdot m\right)^3}}\ N \cdot m = 207\ MPa$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.10

$\lambda := \sqrt{\frac{f_{yw}}{\tau \cdot \sqrt{3}}} = 1.22$ — SS-EN 1993-1-5 D.2.2.(3) Equation D.9

$\chi_g := min\left(\frac{1.5}{0.5 + \lambda^2}, 1\right) = 0.76 \quad = \quad \chi_{c.g}\left(X_{check\_v}\right) = 0.76$ — Global buckling factor SS-EN 1993-1-5 D.2.2.(3) Equation D.8

$\chi_C := min\left(\chi_g, \chi_l\right) = 0.75 \quad = \quad \chi_c\left(X_{check\_v}\right) = 0.75$ — SS-EN 1993-1-5 D.2.2

$V_{Rd} := \chi_C \cdot \frac{f_{yw}}{\gamma_{M1} \cdot \sqrt{3}} \cdot hw \cdot tw = 5526\ kN \quad = \quad V_{Rdw}\left(X_{check\_v}\right) = 5526\ kN$ — SS-EN 1993-1-5 D.2.2.(1) Eq.D.4



$\eta_{V.max} = 90\%$ Maximum utilization rate

6.7 Studs

6.7.1 Ultimate limit state

The capacity of the studs in the ultimate limit state are calculated according to SS-EN 1994-2 6.6.3

$d_{stud} \equiv 22\ mm$ Diameter of stud

$h_{stud} \equiv 200\ mm$ Length of stud

$f_{ub} = 800\ MPa$ Characteristic strength of studs

$f_{u\_stud} := min\left(f_{ub}, 500\ MPa\right) = 500\ MPa$ Ultimate strength shear stud - SS-EN 1994-2 6.6.3.1 (1)

$\dfrac{h_{stud}}{d_{stud}} = 9$ Ratio height- diameter

$$\alpha_{stud} := \left\| \begin{array}{l} \text{if } 3 \le \dfrac{h_{stud}}{d_{stud}} \le 4 \\ \quad \left\| a \leftarrow 0.2 \cdot \left(1 + \dfrac{h_{stud}}{d_{stud}}\right) \right. \\ \text{else if } 4 < \dfrac{h_{stud}}{d_{stud}} \\ \quad \left\| a \leftarrow 1 \right. \\ a \end{array} \right. = 1.0$$

Correction factor for length to diameter ratio shear stud SS-EN 1994-2 6.6.3.1 (1)

$$P_{rd} := min\left(\left\| \begin{array}{c} \dfrac{0.8 \cdot f_{u\_stud} \cdot \pi \cdot d_{stud}^{2}}{4 \cdot \gamma_V} \\ \dfrac{0.29 \cdot \alpha_{stud} \cdot d_{stud}^{2} \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \end{array} \right\| \right) = 122\ kN$$

Capacity of one shear stud SS-EN 1994-2 6.6.3.1 Eq: 6.18,6.19

$S_{uf\_short}\left(X_{check\_v}\right) = \left(89 \cdot 10^{-3}\right)\ m^3$ Second moment of area

$V_{ULS2}\left(X_{check\_v}\right) = 2538\ kN$ Shear force

$I_{y\_short}\left(X_{check\_v}\right) = \left(393 \cdot 10^{-3}\right)\ m^4$ Moment of inertia

$$\tau_{sh} := \frac{S_{uf\_short}\left(X_{check\_v}\right) \cdot V_{ULS2}\left(X_{check\_v}\right)}{I_{y\_short}\left(X_{check\_v}\right)} = 578\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for short term loads

$S_{uf\_perm}\left(X_{check\_v}\right) = \left(70 \cdot 10^{-3}\right)\ m^3$ Second moment of area

$V_{ULS3}\left(X_{check\_v}\right) = 329\ kN$ — Shear force

$I_{y\_perm}\left(X_{check\_v}\right) = \left(341 \cdot 10^{-3}\right)\ m^4$ — Moment of inertia

$$\tau_{pe} := \frac{S_{uf\_perm}\left(X_{check\_v}\right) \cdot V_{ULS3}\left(X_{check\_v}\right)}{I_{y\_perm}\left(X_{check\_v}\right)} = 67\ \frac{kN}{m}$$

Shear force per meter between concrete and top flange for permanent loads

$$\tau := \left|\tau_{sh} + \tau_{pe}\right| = 646\ \frac{kN}{m}$$

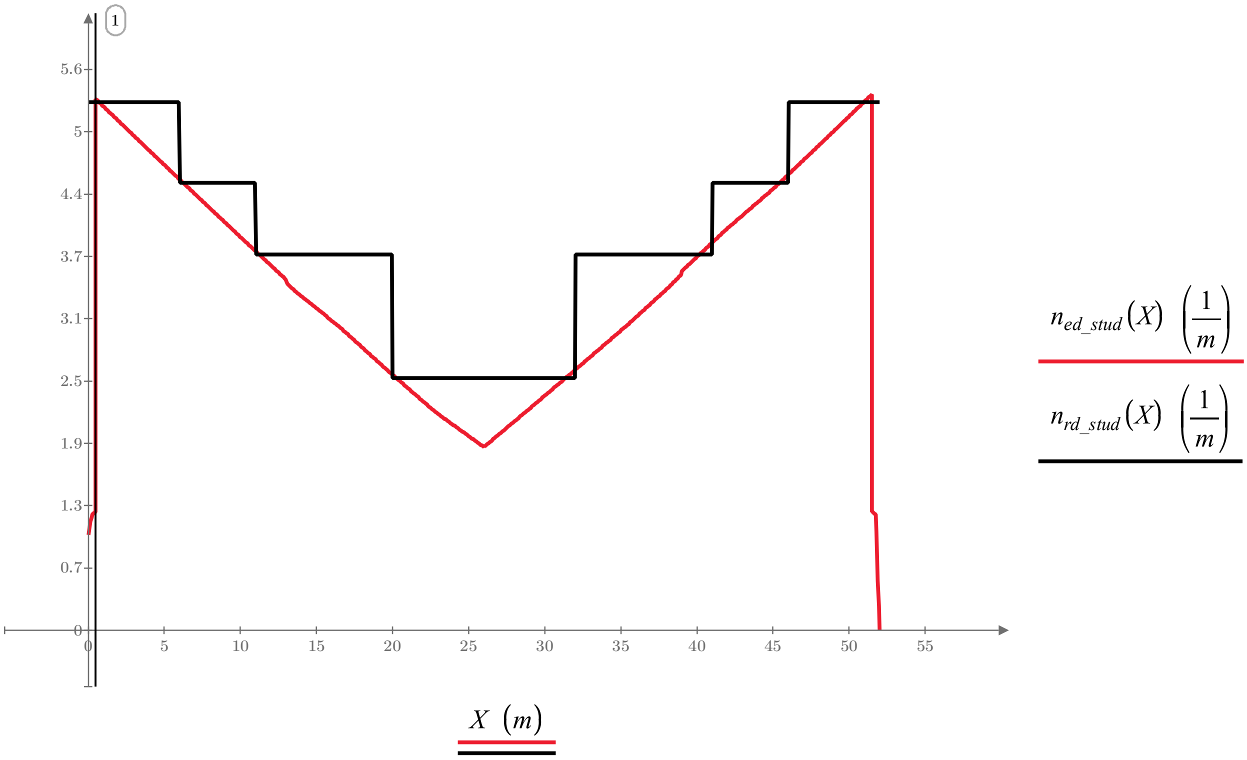
Total shear force per meter between concrete and top flange

$$n_{ed} := \frac{\tau + 0.9\ max\left(q_{1k\_br}, q_{tr\_br}\right)}{P_{rd}} = 5.3\ \frac{1}{m} = \quad n_{ed\_stud}\left(X_{check\_v}\right) = 5.3\ \frac{1}{m}$$

Number of studs needed per meter

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



6.7.1.1 Additional studs for full anchorage

Case 1: Temperature and shrinkage causes the slab to contract and therefore they are working in the opposite direction as the shear flow from ULS- loads during bending.

The anchorage length is calculated according to SS-EN 1994-2, 6.9 (3).

$B_{out} = 2.525\ m$ Distance from centre web to outer part of edge beam

$B_{in} = 2.8\ m$ Distance from web to centerline bridge

$b := max\left(B_{out}, B_{in}\right) = 2.8\ m$

$l_{anch} := 1.5 \cdot b = 4.2\ m$ Anchorage length - SS-EN 1994-2, 6.9 (3)

$N_{cs\_stud} = 687\ kN$ Shrinkage force imposed on studs - calculated in chapter 4

$\left|N_{temp\_stud_1}\right| = 1538\ kN$ Temperature force imposed on studs causing contraction - calculated in chapter 4

$$n_{ed\_cs} := \frac{1.0\ N_{cs\_stud}}{l_{anch} \cdot P_{rd}} = 1.3\ \frac{1}{m}$$

$$n_{ed\_temp} := \frac{1.5\ \left|N_{temp\_stud_1}\right|}{l_{anch} \cdot P_{rd}} = 4.5\ \frac{1}{m}$$

$$n_{ed\_anch.1} := n_{ed\_cs} + n_{ed\_temp} = 5.8\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 5.3\ \frac{1}{m}$$

$$check_1 := \textbf{if}\left(n_{ed\_anch.1} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_1 = \text{“Extra studs are needed”}$$

Case 2: Temperature causes the slab to expand and therefore working in the same direction as the shear flow from ULS- loads during bending. Shrinkage causes the slab to contract, i.e. working in the opposite direction.

$$-n_{ed\_cs} = -1.3\ \frac{1}{m}$$

$$N_{temp\_stud_2} = 1245\ kN$$

Temperature force imposed on studs causing expansion - calculated in chapter 4

$$n_{ed\_temp} := \frac{1.5 \cdot 0.6\ N_{temp\_stud_2}}{l_{anch} \cdot P_{rd}} = 2.2\ \frac{1}{m}$$

$$n_{ed\_stud}(0\ m) = 1.0\ \frac{1}{m}$$

$$n_{ed\_anch.2} := n_{ed\_stud}(0\ m) + n_{ed\_temp} - n_{ed\_cs} = 1.8\ \frac{1}{m}$$

$$n_{rd\_stud}(0\ m) = 5.3\ \frac{1}{m}$$

$$check_2 := \textbf{if}\left(n_{ed\_anch.2} \le n_{rd\_stud}(0\ m), \text{“No extra studs are needed”}, \text{“Extra studs are needed”}\right)$$

$$check_2 = \text{“No extra studs are needed”}$$

Extra studs needed due to shrinkage and temperature

$n_{rd\_stud.adj}(X)$ Needed amount of studs with regards to extra anchorage due to temperature and shrinkage

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section (ULS- loads)

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section

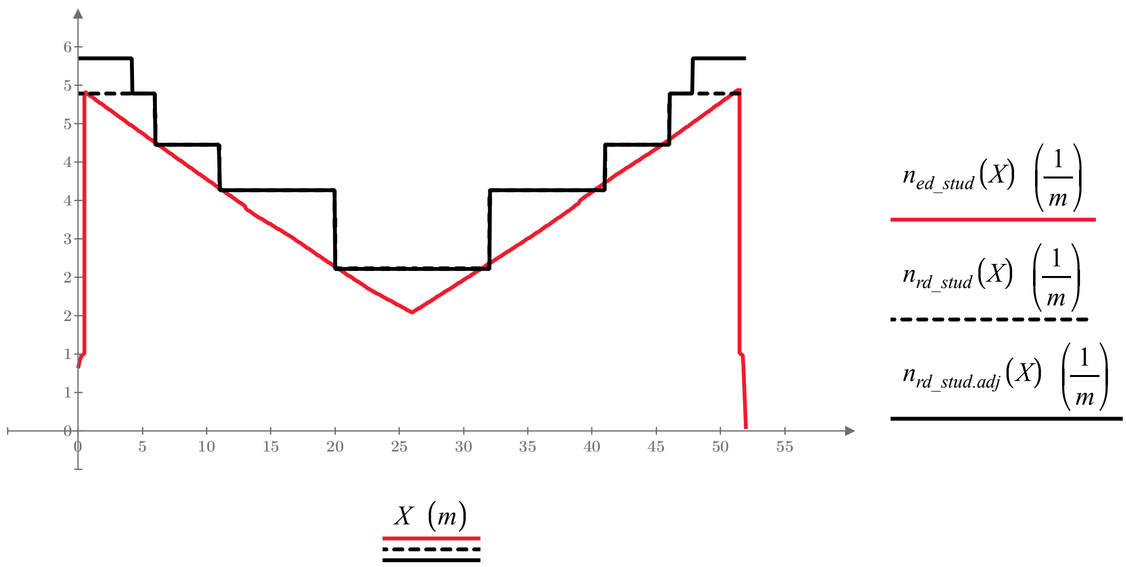


Table. Showing the adjusted need for studs near supports

X	n_{rd}	$n_{rd.adj}$
0	5.3	**5.8**
0.5	5.3	**5.8**
4.2	5.3	**5.3**
6	4.7	**4.7**
11	4.0	**4.0**
20	2.8	**2.8**

6.7.2 Fatigue limit state

6.7.2.1 Capacity

The fatigue capacity is calculated according to SS-EN 1993-1-9, Table 8.5 and SS-EN 1994-2, 6.8.3.

$\Delta\tau_c := 90\ MPa$ SS-EN 1993-1-9 - Table 8.5 (10)

$\Delta\tau_{E2} = \lambda_v \cdot \Delta\tau_c$

$\lambda_{v.1} := 1.55$ Bridge length less than 100m - SS-EN 1994-2 6.8.6.2 (4)

$Q_{mi} := 410\ kN$ Mean weight of large vehicles in the slow lane

$Q_0 := 480\ kN$

$N_{obs} := 0.05 \cdot 10^6$

$N_0 := 0.5 \cdot 10^6$

$\lambda_{v.2} := \frac{Q_{mi}}{Q_0} \cdot \left(\frac{N_{obs}}{N_0}\right)^{\frac{1}{8}} = 0.64$ SS-EN 1994-2 6.8.6.2 (4) and SS-EN 1993-2 Eq. 9.10

$t_{Ld} := 120$ Expected service life [years]

$\lambda_{v.3} := \left(\frac{120}{100}\right)^{\frac{1}{8}} = 1.02$

$\lambda_{v.4} := 1.0$ TSFS 2018:57 - 27 ch. 3 §

$\lambda_v := \lambda_{v.1} \cdot \lambda_{v.2} \cdot \lambda_{v.3} \cdot \lambda_{v.4} = 1.02$

$\gamma_{Ff} := 1.0$ SS-EN 1993-2, 9.3 (1)

$\gamma_{mF} := 1.0$ SS-EN 1994-2, 2.4.1.2 (6)

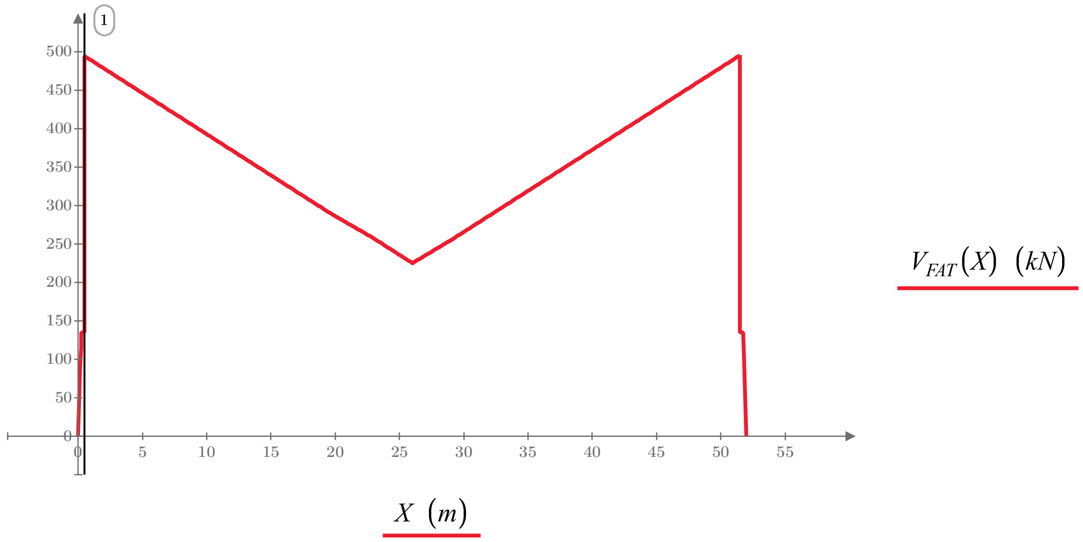
$\gamma_{Ff} \cdot \Delta\sigma_{E2} < \frac{\Delta\tau_c}{\gamma_{mF}}$

$F_{rd\_stud} := \Delta\tau_c \cdot \frac{\pi \cdot d_{stud}^{\ 2}}{4} = 34.2\ kN$ Shear fatigue capacity - one stud

$F_{rd\_stud} := \frac{F_{rd\_stud}}{\lambda_v} = 33.7\ kN$ Considering trafic load

6.7.2.2 Fatigue load

$V_{FAT}(X)$ Fatigue load from trafic - see Appendix X



$$\tau \cdot b = \frac{SV}{I} = \frac{stud\_capacity}{m}$$

$$S_{uf} := S_{uf\_short}\left(X_{check\_v}\right) = 0.089\ m^3$$

$$V := V_{FAT}\left(X_{check\_v}\right) = 494\ kN$$

$$I := I_{y\_short}\left(X_{check\_v}\right) = 0.393\ m^4$$

$$\Delta\tau := \frac{S_{uf} \cdot V}{I} = 113\ \frac{kN}{m}$$

$F_{rd\_stud} = 34\ kN$ Fatigue capacity - one stud

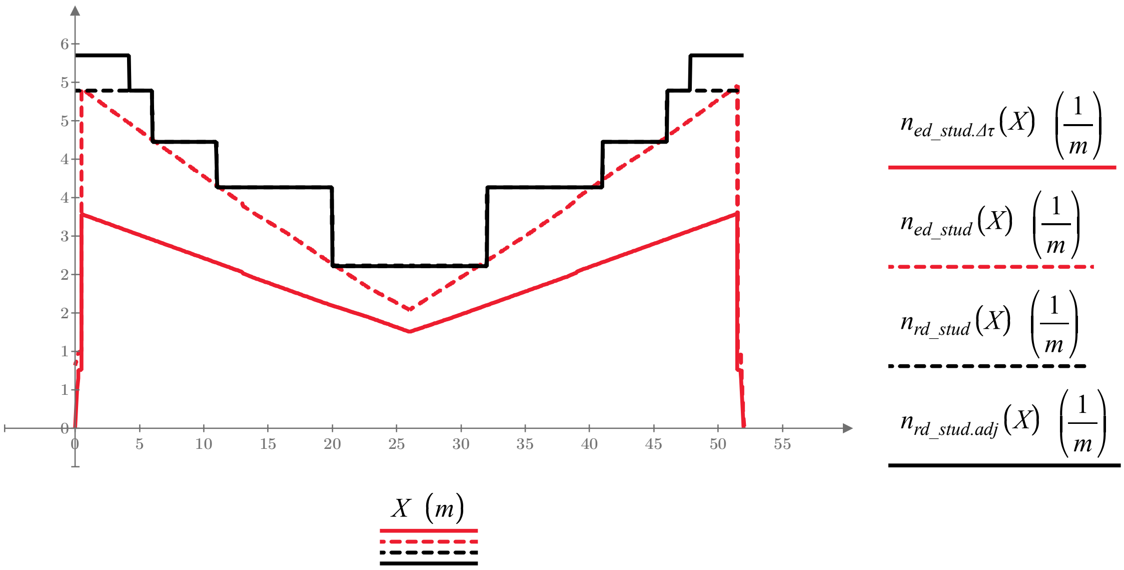
$$n_{ed\_\Delta\tau} := \frac{\Delta\tau}{F_{rd\_stud}} = 3.3\ \frac{1}{m} \quad = \quad n_{ed\_stud.\Delta\tau}\left(X_{check\_v}\right) = 3.3\ \frac{1}{m}$$

6.7.2.3 Design of studs with regards to fatigue

$n_{ed\_stud.\Delta\tau}(X)$ Needed amount of studs with regards to fatigue

$n_{rd\_stud}(X)$ Provided amount of studs in a certain section

$n_{ed\_stud}(X)$ Needed amount of studs in a certain section



Extra need of studs with regards to fatigue

$$Check := \textbf{if}\left(max\left(\frac{n_{ed\_stud.\Delta\tau}(X)}{n_{rd\_stud}(X)}\right), \text{"No extra studs are needed"}, \text{"Extra studs are needed"}\right)$$

$Check =$ "No extra studs are needed"

6.7.3 Summary - design of studs

Table. Showing the stud design for half of the span

X	$n_{stud.used}$	$n_{stud.need}$	$n_{stud.fat}$
0	**5.8**	1.0	0.0
0.5	**5.8**	5.3	3.3
4.2	**5.3**	4.8	3.1
6	**4.7**	4.5	2.9
11	**4.0**	3.8	2.6
20	**2.8**	2.6	1.9

6.8 Utilization rates

$\eta_{\sigma.u.max} = 98\%$	Stresses in top flange
$\eta_{\sigma.l.max} = 96\%$	Stresses in lower flange
$\eta_{\sigma.c.max} = 17\%$	Stresses in concrete
$\eta_{V.max} = 90\%$	Shear capacity
$\eta_{breathing} = 30\%$	Breathing

9 Material savings

The material savings that are achieved thanks to redesigning the bridge with corrugated webs and using stainless steel is compared to the old design. However it is important to note that dependent on the utilization rates the savings may always not be comparable. For this bridge the highest utilization ratios are larger than 95% (close to 99,8%) and therefore close to comparable.

9.1 Main girder

The main girder is redesigned with a corrugated web and a slightly slimmer design. Over the full bridge length this decreases the weight of the bridge. All savings are presented for the full bridge (full width). Consideration has been taken to the extra web length arising from the corrugation.

$V_{old} = 13.25\ m^3$ Steel volume - girder with flat web; original design

$V_{new} = 7.1\ m^3$ Steel volume - girder with corrugated web; new design

$m_{old\_girder} := 7850\ \frac{kg}{m^3} \cdot V_{old} = 104\ 10^3 \cdot kg$ Weight of girder - flat web; original design

$m_{new\_girder} := 7551\ \frac{kg}{m^3} \cdot V_{new} = 54\ 10^3 \cdot kg$ Weight of girder - corrugated web; new design

$\eta_{girder} := 1 - \frac{m_{new\_girder}}{m_{old\_girder}} = 48\%$ Material saving [%] - steel girder

$\Delta m_{saving\_girder} := m_{old\_girder} - m_{new\_girder} = 50\ 10^3\ kg$ Material saving [kg] - steel girder

9.2 Studs

$V_{stud\_old} = 72750\ cm^3$ Volume - studs; original design

$V_{stud\_new} = 32030\ cm^3$ Volume - studs; new design

$m_{old\_stud} := 7850\ \frac{kg}{m^3} \cdot V_{stud\_old} = 571\ kg$ Mass - studs; original design

$m_{new\_stud} := 7551\ \frac{kg}{m^3} \cdot V_{stud\_new} = 242\ kg$ Mass - studs; new design

$\eta_{stud} := 1 - \frac{m_{new\_stud}}{m_{old\_stud}} = 58\%$ Material saving [%] - studs

$\Delta m_{saving\_stud} := m_{old\_stud} - m_{new\_stud} = 329\ kg$ Material saving [kg] - studs

9.3 Cross-beams

$V_{cb}=0.59\ m^3$ Volume cross-beams old and new design

$m_{old\_cb}:=7850\ \dfrac{kg}{m^3}\cdot V_{cb}=4597\ kg$ Mass - cross-beams; original design

$m_{new\_cb}:=7551\ \dfrac{kg}{m^3}\cdot V_{cb}=4422\ kg$ Mass - cross-beams; new design

$\eta_{cb}:=1-\dfrac{m_{new\_cb}}{m_{old\_cb}}=4\%$ Material saving [%] - studs

$\Delta m_{saving\_cb}:=m_{old\_cb}-m_{new\_cb}=175\ kg$ Material saving [kg] - studs

9.4 Welds

$V_{weld\_old}=0.005\ m^3$ Volume - welds; original design

$V_{weld\_new}=0.0038\ m^3$ Volume - welds; new design

$m_{old\_weld}:=7850\ \dfrac{kg}{m^3}\cdot V_{weld\_old}=40\ kg$ Mass - welds; original design

$m_{new\_weld}:=7551\ \dfrac{kg}{m^3}\cdot V_{weld\_new}=28\ kg$ Mass - welds; new design

$\eta_{weld}:=1-\dfrac{m_{new\_weld}}{m_{old\_weld}}=28\%$ Material saving [%] - welds

$\Delta m_{saving\_weld}:=m_{old\_weld}-m_{new\_weld}=11\ kg$ Material saving [kg] - welds

9.5 Total savings

$m_{old\_bridge}:=m_{old\_girder}+m_{old\_stud}+m_{old\_cb}=109\ 10^3\ kg$ Total mass of original design

$m_{new\_bridge}:=m_{new\_girder}+m_{new\_stud}+m_{new\_cb}=58\ 10^3\ kg$ Total mass of new design

$\eta_{bridge}:=1-\dfrac{m_{new\_bridge}}{m_{old\_bridge}}=46.6\%$ Material saving [%] - full bridge

$\Delta m_{saving.tot}:=\Delta m_{saving\_girder}+\Delta m_{saving\_stud}+\Delta m_{saving\_cb}+\Delta m_{saving\_weld}=51\ 10^3\ kg$
Material saving [kg] - full bridge

DEPARTMENT ARCHITECTURE AND CIVIL ENGINERRING
CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden
www.chalmers.se

CHALMERS
UNIVERSITY OF TECHNOLOGY